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

REPLY TO
ATTENTION OF:

NEDED-E

2 November 1973

SUBJECT: Charles River Dam, Charles River Basin, Massachusetts, DM No. 5, Pumping Station and Appurtenant Structures

HQDA (DAEN-CWE-B)
WASH DC 20314

1. In accordance with ER 1110-2-1150, there is submitted for review and approval, DM No. 5, Pumping Station and Appurtenant Structures, for the Charles River Dam Project.

2. This DM is a resubmission of DM No. 5 previously submitted by NEDED letter dated 13 March 1972. Comments contained in DAEN-CWE-B 1st Indorsement thereon were incorporated in the revised DM.

FOR THE DIVISION ENGINEER:

Incl (10 cys)
as

JOHN WM. LESLIE
Chief, Engineering Division

WATER RESOURCES DEVELOPMENT PROJECT

CHARLES RIVER DAM
CHARLES RIVER BASIN
MASSACHUSETTS

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WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
CHARLES RIVER BASIN, MASSACHUSETTS

DESIGN MEMORANDUM NO. 5

PUMPING STATION
AND
APPURTENANT STRUCTURES

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

NOVEMBER 1973

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
CHARLES RIVER BASIN, MASSACHUSETTS
DESIGN MEMORANDUM NO. 5
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CHARLES RIVER DAM
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DESIGN MEMORANDUM NO. 5
PUMPING STATION
AND
APPURTENANT STRUCTURES

A. PERTINENT DATA

Purposes	Flood Control, navigation and highway transportation.
Location	
State	Massachusetts
County	Suffolk
City	Boston
River	On the Charles River 2,250 feet downstream of the present Charles River Dam.
Drainage Area	
Total Watershed	307 square miles
Lower Charles River above proposed dam	58 square miles
Surface Area	
Datum Relationship	M.D.C. Base is 105.65 feet below mean sea level (MSL) U.S.C. & G.S. Datum of 1929 (105.65 M.D.C. = 0.0 MSL U.S.C. & G.S. (1929))
Proposed Basin	705 acres
Pumping Station	
Structure	Reinforced Concrete substructure with a reinforced concrete frame superstructure and brick exterior
Size	190.0' x 86.0' Bottom Elevation 78 MDC Base Engine Room Roof 162.0 MDC Penthouse Roof 174.5 MDC
Pumps	6 - 144" diameter, 105.6 R.P.M.

Pumps, capacity each	1400 cfs @ 9 ft. static head
Engine	Diesel, 2600 B.H.P. with right angle gear unit drive
<u>MDC Police Facility</u> Structure	Reinforced concrete frame and brick exterior
Size	79.16' x 63.16' Bottom Elevation 80.0 MDC Base Roof Elevation 162.42 MDC Base
<u>Fishway</u> Structure	Reinforced Concrete
Type	Vertical slot
Size	144.8' x 41'

B. INTRODUCTION

1. Purpose. - The purpose of this design memorandum is to present the basic design criteria, computation, and the equipment snd structural layouts for the pump station and appurtenant structures.

2. Location. - The location of the Charles River Dam is approximately 2,250 feet downstream of the existing dam. The pump station is positioned north of the large navigation lock because of navigation channel alignment requirements. These requirements prevent the location and orientation of a pump station most favorable from a hydraulic standpoint, which results in a compromise in the arrangement and layout of the pumping station. (See model studies conducted by the Massachusetts Institute of Technology, Hydrodynamics Laboratory, for the Metropolitan District Commission Technical Report No. 39 Model Study of a Flood Control Pumping Station for the Charles River Basin at the Warren Avenue Bridge by D.R.F. Harleman, C.J. Huval and T.K. Chu, dated December, 1959 which determined the best arrangement and layout for the pumping station).

The fishway is located north of the pump station. This location was selected as the most optimum as it is near the point of maximum water release from sluicing operations.

The MDC harbor police facility is located adjacent to the small lock No. 1 on the Boston side of the Charles River. This location provides easy access to and from the Charles River Basin and also provides a terminus for the overhead walkway which connects the Boston Shore to Charlestown. The location of all facilities is shown on Plate 5-1.

3. Description. -

a. Pump Station. - The superstructure is 190 feet long by 86 feet wide and houses the main engine room at elevation 122.17 MDC. There are three floors along the entire length of the east elevation. The first floor, at the same elevation as the main engine room floor, has space for equipment and boiler rooms, storage and workshop areas and the personnel office, toilet and locker facilities for operation and maintenance personnel for this facility as well as the adjacent navigation locks. The second floor (lower mezzanine) provides and observation area for visitors as well as toilet facilities and also provides a lock masters office and pump control room. The third floor (upper mezzanine) is the air intake area. A penthouse, located above the main engine room, provides space for the exhaust silencers and exhaust fans. Access to the different floors is by means of stairtowers located at each end of the pump station. An elevator has been provided in the north stairtower for handicapped people. See plates 5-2 and 5-3.

The substructure foundation slab, 190.0' x 144.3' has a bottom elevation of 78 feet MDC, is an independent monolith and will be poured with construction joints. The forebay and afterbay walls will be poured in four lifts. See Plate 5-4.

The function and operation of the pump station is discussed in Design Memorandum No. 1 "Hydrology and Tidal Hydraulics". The pumping station will control the elevation of the lower Charles River Basin during periods of severe storm runoff to prevent flooding of low-lying urban areas adjacent to the basin. The pumping station will provide a continuous discharge of the lower basin to tide water, especially during periods of high tide in Boston Harbor when gravity sluicing would not be possible. The operation of the pumping station and pre-lowering of the lower basin is predicated on the advanced warning of a severe storm or hurricane. The rain gage, as mentioned in Design Memorandum No. 1, should be used as a storm warning indicator to pre-lower the lower basin.

b. MDC Police Facility - The superstructure for this facility is 79'-2" long by 63'-2" wide. It houses two (2) boat slips on the harbor (east) side of the structure. On the west side of the structure, the main floor at elevation 118.46 MDC provides space for workshop, a boiler room and stairways. Office, locker and toilet facilities for the harbor police are provided on the mezzanine, at elevation 139.0. A penthouse located above the mezzanine, provides space for HVAC equipment for both this structure and the control tower walkway located over the navigation locks. See Plate 5-5.

The substructure foundation slab, 63.16' by 82.16' has a bottom elevation of 80 MDC. The slab, which supports the foundation walls and interior columns, is 3 feet thick.

This facility which was requested by and will be funded by the using agency (the Metropolitan District Commission) has the function of providing office space for police personnel and equipment and boat storage and workshop areas. The police personnel are used to patrol and regulate recreational boat traffic in the Charles River Basin.

c. Fish Passage Facility - The fishway is a vertical slot-type reinforced concrete structure 144.83 feet long by 41.0 feet wide and has a top elevation of 122.17 MDC. It is provided with 24-1 foot thick concrete baffles with a 10.83 foot space between baffles. A false weir has been provided for the purpose of attracting fish and providing a controllable jump for their entrance to the Charles River Basin. Attraction water for the fish is provided by a 150 horsepower 30-inch propeller pump. See Plate 5-6.

C. FOUNDATIONS

4. Characteristics of Foundation Soils - The subsurface investigations and characteristics of foundation soils are discussed in Design Memorandum No. 4, "Embankments and Foundations". At the location of the structures, the subsurface consists of a 5 to 15-foot surface layer of soft river bottom black organic silt. Underlying the organic silt is a 20 to 40-foot

deposit of very dense till which consists of a heterogeneous mixture of sand, gravel and clayey silt with occasional cobbles and boulders. Sandwiched between the organic silt and till strata is a discontinuous layer of loose silty sand. This layer varies in thickness to a maximum of 10 feet. The till layer overlies the gray argillite bedrock formation.

5. Type of Foundations - The major portion of the pumping station and fishway will bear directly on the dense till stratum. At those locations where the organic silt or silty sand is encountered at the founding elevation, it will be excavated down to the level of the till and replaced with lean concrete. The maximum thickness of lean concrete will occur under the northeast corner of the fish passage facility. The limits and thickness of lean concrete are shown on Plate 5-7. The lean concrete fill will be placed in layers not exceeding 5 feet in thickness. Because of boat draft requirements, the MDC police facility will be founded directly on the dense till strata.

6. Foundation Treatment - The surface of the till may become very soft from water, frost, traffic and expansion. This softening results in loss of bearing capacity. In order to ensure against the developing of such a condition during construction, it will be required that the final 2 feet of excavation be made immediately prior to the placement of a working mud slab. The mud slab will consist of a 6-inch thick layer of lean concrete. The structural concrete will then be built over this mud slab.

7. Bearing Capacity - Foundations have been designed for a maximum allowable bearing capacity of 4 TSF. The reasons for adoption of this bearing capacity are discussed in Design Memorandum No. 4, "Embankments and Foundations". The maximum computed bearing pressure is around 2 TSF.

8. Scour Protection - The scour protection beyond the pump station (including the concrete apron on the downstream side) will consist of a 2-foot layer of 5 to 300-pound stones on a 1-foot gravel bed extending 40 feet beyond the concrete structure on the upstream and 70 feet on the downstream side.

The additional 30 feet on the downstream side has been provided to prevent scour at the location of the discharge outlets for the Boston marginal conduit which discharges just below the pump station. Scour protection requirements are based on EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Structures. The flow velocity upstream is 2.0 fps while that immediately downstream of the concrete apron is 2.2 fps. Based on these criteria, the required stone sizes are 0.05 pounds and 0.06 pounds upstream and downstream respectively. Beyond this stone protection, the maximum computed flow velocities are less than 4 feet per second. The existing till deposit, by itself, is considered to be adequate against scour for flow velocities up to 5 fps. Hence, stone protection will not be provided beyond the above noted limit except at the locations of the existing John Fitzgerald Expressway bridge and the Charlestown bridge piers. At these piers the protection against the eddy current scour will consist of a 2-foot layer of 5 to 300-pound dumped stone. This layer will extend 8 feet beyond the perimeters of the pier footings.

D. CONCRETE MATERIALS

9. Concrete Materials - Several nearby commercial sources of aggregate have been tested and found acceptable. See Design Memorandum No. 3, "Concrete Materials".

E. ARCHITECTURAL TREATMENT

10. General - The pump station and the MDC Police Facility features brick as the dominant exterior material. Brick was selected because of its warm character, its durability, its compatability with the adjacent structures in Boston and Charlestown, and its historic use as a building material in the Boston area.

The brick will be similar to a waterstruck brick, rich in deep red tones with mortar tinted to approximate the color of the brick. Arches have been introduced over the pump discharge openings on the downstream elevation. These arches reflect the character of the Charles River with its many arched bridges. Above the arched pump discharge bays is the enclosed walkway which connects the free standing stair towers and the pump station. The walkway exterior is porcelain enameled steel, which will be dark in color to match the visual sense of the brick, and a gray tinted glass. Above the walkway is a horizontal band of louvers and clerestory windows. The windows provide natural light for office space on the mezzanine level and the louvers provide air for the pump engine air intakes. Above this band is a sloping metal roof of the same material as the walkway. Above the sloping roof is another horizontal band of louvers which provide cooling air for the pump engine exhaust silencers. These horizontal bands create a strong integrated relationship between the various elements composing the east elevation.

The dominant feature of the North and South elevations is a fifty (50) foot high window that extends from the pump room floor to the underside of skylights which form the west elevation of the penthouse structure. These windows dramatically express the interior volume of the pump room and display the various internal functions of the structure.

The west elevation consists of arched openings which contain windows and louvers. The louvers supply fresh air to the pump room. The windows serve to define the pump room volume and to provide a visual link between the pedestrian access area and the pump room equipment including pumps, engines, and associated piping. The arches continue the theme of the structure as expressed in the downstream discharge bays.

The walkway containing the lock control modules extends over the locks and is connected to the locks with circular stairs that are also sheathed in brick. The south terminus of the walkway is formed by a structure containing facilities for a Metropolitan District Commission police harbor patrol. This facility was added to the original Charles River Dam program during the course of the design reviews. The Metropolitan District Commission recognized the necessity of a structure of this type and realized that this was the ideal location with ready access to Boston Harbor and the Charles River Basin. This structure contains two slips for patrol boats, repair facilities, police offices, and crew facilities.

The boat slips and repair facilities are located on the same level as the locks and the offices and crew facilities are located on the overhead walkway level. A mechanical penthouse forms a third level which contains air conditioning and heating equipment for the structure. The design is consistent with the design vocabulary of the pump station using brick as the dominant exterior material. The downstream or east elevation consists of a large rectangular opening for access to the boat slips. Above the opening is a horizontal band of louvers in a recessed opening.

The large opening reflects the volume of the interior boat slip area and relates to the scale of the structure and the complex. The horizontal louver indicates the function of the mechanical space it encloses and the clean horizontal lines reflect the character of the project design. The north and south elevations contain horizontal windows similar in proportion to the walkway and pump station windows. The west elevation consists of brick expanses with two horizontal bands of windows which reflect the extent of the offices and workshops located in this area.

The designs (Plates 5-8 thru 5-11) were developed through a series of conferences with an Architect Review Board convened with the approval of OCE. This board was composed of:

Pietro Belluschi - Dean Emeritus, School of Architecture and Planning, Massachusetts Institute of Technology.

John C. Harkness - Principal, the Architects Collaborative Inc., 46 Brattle Street, Cambridge, Mass.

Hugh Stubbins - President and Principal Architect, Hugh Stubbins and Associates, 1033 Massachusetts Avenue, Cambridge, Mass.

The review board concluded that the design as finally developed would be an aesthetic asset to the community, would establish a standard of quality for design, and would be compatible with the adjacent historic sections of Boston and Charlestown.

F. STRUCTURAL DESIGN

11. General. This section presents the design criteria, basic data and assumptions used in design of the pump station and appurtenant structures. Plates showing the layout, typical sections, details, and stability diagrams have been included. Appendix A includes complete stability calculations and typical design caluculations of the major structural components.

12. Design Criteria.

a. General. The design conforms to the Corps of Engineers criteria. Accepted engineering practice has been employed in cases where Corps criteria is not available.

b. Concrete. The allowable working stresses in accordance with EM 1110-1-2101 have been used throughout. Concrete with an ultimate compressive strength of 3000 psi has been used except for the concrete in the superstructure frame which is 4000 psi. Nonstructural lean concrete with a compressive strength of 2000 psi is used as fill under portions of the pump station and fishway to replace unsuitable foundation materials. The design of superstructure has been based on ultimate strength design.

c. Reinforcement.

(1) Grade and Working Stresses. All reinforcement in the structure subjected to hydraulic design requirements are designed for a working stress of 20,000 psi in flexural tension. The reinforcement specified is ASTM A615 (Billet Steel), Grade 40. All other reinforcement is designed for ultimate strength using ASTM A615 (Billet Steel), Grade 60.

(2) Minimum Cover for Main Reinforcement. Requirements for minimum cover conform to EM 1110-2-2103.

Interior slabs (Except Engine Room Floor)	3/4"
Interior slabs (Engine Room Floor)	1"
Interior beams	1-1/2"
Exterior slabs	2"
Walls exposed to water	4"
Base Mat	6"

d. Splices. In accordance with EM 1110-2-2103, the lengths of splices and staggering of splices will be indicated on the contract drawings. Splices in the main reinforcement at points of maximum moment have been avoided in the design.

e. Structural Steel. Structural steel conforms to ASTM A36. The allowable design working stresses conform to those specified in EM 1110-1-2101.

f. Increase in Normal Working Stresses. Normal allowable working stresses have been increased 33-1/3 percent where wind loads or where earthquake loads govern for the substructure only. Ultimate strength load factors and design is used for the superstructure.

g. Expansion joints. The pumping station has been fully separated from the lock structure and the fishway facility by full expansion joints.

13. Basic Data and Assumptions.

a. Controlling Elevations (M. D. C. Base)

(1) Pump Station.

Bottom of structural base mat	EL 78.0 and 81.0
Service area deck	EL 118.61
Pump support slabs	EL 108.00
Engine and gear room (Conc. floor)	EL 121.83
Engine and gear room (finish floor)	EL 122.17
Engine and gear room (roof)	EL 162.00
Penthouse roof	EL 174.50

(2) MDC Harbor Police Facility.

Bottom of base mat	EL 80.00
Main floor	EL 118.46
Boathouse roof	EL 150.95
Penthouse roof	EL 162.47

(3) Fishway.

Bottom of base mat	EL 85.00
Top of structure	EL 118.61 and 122.17

b. Loads.

Dead Loads - The following unit weights for materials have been used:

Concrete	150 pounds per cubic foot (pcf)
Steel	490 pcf
Earth (saturated)	125 pcf
Earth (moist)	125pcf
Brick	120 pcf

Live loads - The following live loads have been used:

Water	64.2 pcf
Roof	30 psf
Penthouse Floor	140 psf
All other floors	100 psf
Interior metal stairs	100 psf
Exterior stairs	75 psf
Loading Platforms	300 psf
Engine & gear room	200 psf plus 127 k pump thrust
Service area deck	H-20 truck with appropriate impact
Crane	20 Ton Capacity
Impact	25%
Horizontal transverse load of lifted weight and trolley applied 1/2 each runway	25%
Longitudinal force	10% of maximum wheel load.

c. Water Pressure. Full hydrostatic water pressure has been assumed acting over the entire area of the base. During construction, while cofferdam is in place, hydrostatic uplift has been neglected.

d. Earth Pressure. At-rest pressure was used for design of the fish passage facility and the retaining walls. For soil properties refer to Design Memorandum No. 4, "Embankments and Foundations".

e. Earthquake Forces. The structures have been designed for seismic zone 3 condition. The design of the superstructure has been based on the Tri-service Manual TM 5-809-10 dated April 1973. The substructure was designed using a factor of 0.10g using ETL 1110-2-109, dtd October 1970.

f. Ice Pressure. The basin is subject to freezing depending upon the severity of the winter and the relatively constant basin elevation. The Metropolitan District Commission awards an annual contract for ice breaking in order to maintain an open channel for commercial traffic. This open channel allows for sufficient expansion of the ice. However, an ice pressure analysis assuming 8 inches of ice and using criteria in EM 1110-2-2602 and EM 1110-1-2101 had no effect on existing design. Therefore, ice pressure on the Structure is considered negligible and has been disregarded in the design of structures.

g. Wind Pressure. A horizontal wind pressure of 35 pounds per square foot has been used in the design.

h. Frost Protection. The base of the structures may be subject to frost action during construction. Proper measures will be provided to guard against this condition until the height of fill is at least 4 feet above the bottom of the base.

i. Wave Pressure. Due to the secluded harbor location of the proposed structure, wave pressure is not considered critical and has not been included in the design. Refer to Design Memorandum No. 1, "Hydrology and Tidal Hydraulics".

j. Location of Resultant. The resultant falls within the middle third of the base for all loading conditions investigated.

k. Resistance to Sliding. Except for case III a (earth quake and hydrodynamic forces added instead of wind) the required minimum factors of safety against sliding as required by ETL 1110-2-22 have been provided, assuming a friction value of 0.5 and no cohesion. For case III a, a factor of safety of 2.04 has been computed. However, as this case combines earthquake force with extreme pool levels, the computed safety factor is considered satisfactory. The various factors of safety at different loading conditions are shown on Plate 5-12.

l. Allowable Foundation Bearing Pressure. As discussed in detail in Design Memorandum No. 4, "Embankment and Foundations", the allowable bearing pressure is 4 tons per square foot.

14. Preparation of Calculations. Hand calculations supplemented by the use of an IBM - 1130 computer have been used in the design.

15. Design of Structures.

a. Superstructure.

(1) Roof. The main roof for the pump station will be constructed primarily of 24 inch precast concrete double tees. The roof is supported on cast in place reinforced concrete bents which are spaced 31 feet on centers. That portion of the roof over the walkway consists of a 6 inch reinforced concrete slab supported by the steel truss framing system of the walkway. The portion of the main roof which forms the floor for the penthouse also consists of a 6 inch reinforced concrete slab. The roof over the penthouse will be 10 inch precast hollow core slab construction. The concrete frame supporting the penthouse roof is supported by the main roof framing system. Overlying the double tees and the hollow core slabs will be a 2 to 4 inch thick normal weight concrete topping which slopes from the supports to the centers of each bay. Plate 5-13 shows the roof plans for the pump station superstructure.

(2) Framing System. The framing system for the pump station will consist of reinforced concrete bents spaced 31 feet on centers. The frame is designed for ultimate strength using ACI 318-71 except Appendix A and TM 5-809-10 for wind and seismic criteria. A typical section is shown on plate 5-13.

The framing system for the MDC police facility similar to the pump station. The reinforced end bents are spaced approximately 10 feet from the interior bents with a spacing approximately 21 feet between the interior bents. Typical sections are shown on Plate 5-14.

(3) Crane. A 20 ton capacity crane, extending the length of the pump station and supported by brackets along column lines A and E has been provided. A 5-ton capacity monorail has been provided in the workshop area of the police facility.

(4) Exterior Walls. The exterior walls for the pumping station, stair towers and police facility will be constructed of 6-inch brick (Plate 5-10A) having 2 minimum compressive strength of 7000 psi. A high-bond additive will be specified for use in the brick mortar to provide the following minimum allowable strengths:

axial compressive strength - 1250 psi
flexural compressive strength - 1670 psi
tensile strength - 112 psi
shear strength - 100 psi

These allowable strengths are in accordance with the International Conference of Building Officials Report No. 2585 dated January 1972.

The use of high-bond additive in the mortar produces sufficient flexural strength with the laid-in-place curtain wall to resist lateral loads without a back-up wall. The brick wall is designed as a simple span vertically between concrete spandrels and the building foundations, with a maximum span of 13-feet. Shelf angles will be provided at each spandrel beam with appropriate brick ties above and below the shelf angle. Actual tensile and shear stresses are less than the allowable strengths tabulated above. Mortar cubes using the high-bond additive were tested in the Division Laboratory with the following results:

	3 days	90 days
tensile strength	103 psi	535 psi
compressive strength	113 psi	5850 psi

b. Substructure.

(1) Pump Station

(a) Service Area. The service area slab consists of reinforced concrete beams spanning between 4 feet thick forebay walls which support an 18 inch thick slab. Several grating covered access openings are provided in the slab. The concrete beams and grating are designed for H20 truck loading. Three of the forebay walls support the future viaduct pier columns, see Plate 5-4.

(b) Engine and Gear Room. This floor, supporting heavy equipment has been designed as a 3.83 feet thick slab spanning between walls below. The floor is pierced by many trenches, hatches, sumps and maintenance openings which are covered by either grating or checkered plate.

(c) Pump Room. The five foot thick pump room slab is supported by the longitudinal walls, spillway and transverse wall. A circular opening will be left in the floor slab to permit installation of the pumps.

The access opening will be filled with grout after the pump casing is set.

(d) Walls and Foundation Mat. A rigid frame analysis has been used for load determination. The walls have been designed for bending and axial load due to lateral and superimposed loads.

(e) Stop Logs. The stop logs are of welded frame structure with a 3/8" skin plate. A design water level of El. 114 has been used. Slots are provided in the afterbay walls for installation by the overhead crane.

(f) Trash Racks. Trash racks in the forebay wall of the pumping station has been designed for five foot differential head. The middle support is a concrete encased steel beam.

(g) Bulkheads. The removable bulkheads, located in the walls of forebay section of the pumping station, are welded three dimensional trusses with a 7/16" thick skin plate. A design water level of El. 110 with a 2 foot freeboard has been used. The bulkheads will be installed by a mobile crane.

c. Stability Analysis.

(1) Loading Conditions. The pumping station structure has been investigated for the following conditions of loading:

Case 1. Construction condition cofferdam in place, dead load of structure with wind load. No hydrostatic loading and no equipment.

Case 1a. Same as Case 1 except earthquake load is added instead of wind load.

Case 11. Cofferdam failure during construction with design water level at EL. 115. This condition considers full hydrostatic uplift and wind load.

Case 11a. Same as Case 11 with earthquake and hydrodynamic loads added instead of wind load.

Case 111. Extreme operating condition with high water at EL. 112 on basin side and low water at EL. 98 on tide side. Uplift and wind loads included.

Case IIIa. Same as Case III with earthquake and hydrodynamic loads added instead of wind load.

Case IV. Extreme operating conditions with high water at EL. 116 tide side and EL. 106 basin side. Uplift and wind loads included.

Case IVa. Same as Case IV with earthquake and hydrodynamic loads added instead of wind load.

Case IVb. Same condition as Case IVa, except earthquake and hydrodynamic force applied from the tide side.

(2) Critical Conditions. Stability investigations show that Case Ia (Construction Condition) will give the maximum bearing pressure on the base of 1.76 tons per square foot which is well within the allowable for glacial till. For the extreme operating condition (Case IIIa) a maximum bearing pressure of 1.48 tons per square foot is reached. For stability diagrams, refer to Plate 5-12.

d. Substructure Earthquake Considerations. Earthquake loading has been considered in the stability analyses and in the design of the major substructure components.

16. Fish Passage Facility. This facility will contain high and low sluiceways, and a vertical slot fishway. It will be located on the northerly side of the pumping station and will be approximately 41 feet wide separated from the pumping station by an expansion joint. The structure is a concrete monolith with nonstructural concrete added for ballast. Stability analysis (including earthquake loadings) meets all requirements of uplift, safety factors and location of resultant. See Section J "Fishway and Sluice Facilities" in this memorandum for description.

17. Gravity Retaining Walls. The retaining walls located north of the fish passage structure have been designed for the applicable conditions of water, earth pressure and seismic loadings using EM 1110-2-2200, Gravity Dam Design.

G. MECHANICAL DESIGN

18. General. Plates 5-15 and 5-16 indicated proposed pump curves, and Plate 5-17 show general arrangement of the pumps and miscellaneous equipment.

19. Pumping Station Equipment.

a. General. The pumping equipment will consist of six (6) propeller pumps driven through right angle transmission gear units by diesel engines, and miscellaneous mechanical equipment. A typical section is shown in Plate 5-18.

b. Pumps. The selection of the pump was based upon a detailed investigation of alternative types of pumping equipment reported in "Progress Report No. 3 - Report Upon Alternative Types of Pumping Equipment for Charles River Basin Elevation Control Project", December 1959, by Charles A. Maguire & Associates and Elson T. Killam Assiciates, Inc. The investigation concluded that the most advantageous type of pump comprises the open top or "bucket type" vertical propeller pump, driven by medium to high speed diesels, through right angle reduction gears. The principal factors upon which this conclusion is predicated are as follows: Lowest overall cost; minimizes the problem of cavitation and negative head control; the high discharge lip (EL. 116) provides a barrier against tide water passing upstream without dependence on 6 large discharge control gates, except for adjusted tide of record. There is a possibility of overtopping the discharge tube by the 1851 tide of record (elevation 115.7 feet MDC) adjusted to 1970 (elevation 116.6 feet MDC). Studies on reverse flow with all pumps inoperative indicate the effect on basin elevation and salt water intrusion is minimal. Gear units are provided with a reverse ratchet arrangement to prevent windmilling under this remote reverse head condition. Each pump will have a capacity of 1,400 cfs at a pool to pool head of nine (9) feet. Plates 5-15 and 5-16 indicate typical horsepower efficiency, and head-capacity curves for the proposed pumps.

Plate 5-15 indicates pump performance at 105.6 rpm which is the maximum speed for the particular pumps in accordance with the pump model test performed under contract with the Metropolitan District Commission by Fairbanks Morse. The pump model test also concluded that the discharge chamber ceiling be established at elevation 118.0 feet MDC. This configuration produces the minimum head loss between the cone crest and the ceiling. Plate 5-16 indicates performance at 101.5 rpm which is the normal operating speed. Under normal conditions, the pumps will be exposed to a corrosive sea water environment. In order to provide a maximum protection at minimum cost, the entire pump casing will be fabricated of 3/4" thick plate steel sand blasted and protected as described in Section I, "Corrosion Protection". The impeller would be cast of non corrosive nickel-alloy steel.

c. Traveling Crane. A 20-ton traveling crane with an electrically operated bridge, trolley and hoist will be installed for handling the equipment during construction and for maintenance. The crane capacity is based on handling the heaviest piece of equipment that may have to be replaced during the life of the station.

d. Trash Racks. Trash racks will be 6-inch spaced steel racks. The 6-inch spacing is consistent with spacing used in the Central and South Florida Flood Control Districts and in the City of New Orleans Storm Water Pumping Stations which are of comparable magnitude. These installations have spacings varying from 5-1/2 inches to 8 inches. The 6-inch spacing is compatible with the capacity of the pump to handle large solids.

20. Heating, Ventilation and Air Conditioning.

a. General. The pumping station will be fully heated and ventilated with certain portions air conditioned. Direct expansion air conditioning will be used for cooling, utilizing a package air conditioner adapted with an enclosed condenser with a forced flow air system ducted to the outdoors. These will be hung adjacent to the outside wall. Steam will be used for heating, utilizing No. 2 oil for fuel. In some remote instances, electric radiation will be used for heating.

b. Operation. The pump station in general will be heated by unit heaters with steam while electric radiation will be used in the office and personnel area. Air conditioning in the office and personnel area of the pump station will be provided by a ceiling hung package air conditioner. Steam will be used for air reheating. The heavy ventilation for heat dissipation in the pump station will be provided by an exhaust ventilation system with large gravity intakes which will be activitated by thermostats.

c. Temperature Control. A complete system of temperature control will be provided and will consist of the following:

(1) Unit heater control which will consist of local thermostats to start and stop the unit heater fan in each respective area of the pump station.

(2) Radiation control which will consist of a thermostat activating the radiation on a drop in space temperature.

(3) Local thermostats for the air conditioning control to activate the cooling in a rise in temperature and stop the cooling and activate the respective reheat coil on a drop in temperature.

(4) The overall ventilation for the pump station shall be controlled on a bay by bay basis with the respective thermostats in each area activating the roof fan and opening the intake air damper on a rise in temperature and closing the damper stopping the fan and activating the unit heater on a drop in temperature.

21. Dewatering. Provision for the dewatering of the bays is accomplished by the installation of bulkheads or stop logs, which would be installed by crane or hoist. The actual dewatering will be accomplished by a central system connected to the lock wet well sump. See Plate 5-19.

22. Cooling Water. The diesel engines will be cooled by potable water supplied by the City of Boston low service system with a backup connection from the Charlestown system. The heat exchangers will be specified as double pass units to reduce cooling water requirements. The use of brackish Charles River water or raw sea water was considered, but was disregarded because of the availability of city water and the fact that municipal water offers maximum reliability at a nominal cost to purchase, and less maintenance. In addition, the total quantity of potable water to be used in the system will be low because of the relatively small number of hours of operation of the station. Operation will be required only for excercising the pumping equipment, and perhaps, during spring runoff and/or the late summer hurricane season. For this reason, it would not be economical to use a raw sea water supply and attendant equipment.

H. ELECTRICAL DESIGN

23. General. - The design of the electrical installation for servicing the pumping station includes provisions for power, exterior and interior lighting, control systems, electric heating, ventilation, communication, telephone, grounding, and temporary power and light service.

24. Power Supply. - The Charles River Dam Project will be supplied by two 15 KV Boston Edison lines to provide a spot network system. These 15 KV cables will originate from the Hawkins Street Substation in Boston's Government Center. Each supply line would follow a separate route in the public streets to the junction of Causeway and Beverly Streets, one via Sudbury, North Washington and Beverly Streets and the other via Chardon, Merrimac and Causeway Streets. From this point both lines would continue in Beverly Street to the project property line by Boston Edison Company and continue underground to the project facilities via a transformer vault located in the small lock. Each Edison route is approximately 2,800 feet. From Causeway Street new concrete ducts would be installed in Beverly Street to the property line with a separate conduit for each cable, see Plate 5-20. At the Hawkins Street Substation, each of the 13,800 volt supply lines will be connected to separate busses. These busses will be connected to separate 75,000 KVA, 110/13.8 KV station transformers. Each of the project's two 13.8 KV lines would be supplied from either 75,000 KVA transformer by means of automatic switching arrangement available at the Hawkins Street Substation. The Hawkins Street Substation was completely renovated with every necessary safeguard to assure continuity of service to the important Government Center load in Boston, for which it was primarily planned eight years ago.

25. Main Power Transformers.

a. Main Power Transformers. - The two incoming 13.8 KV primary service lines will terminate in the transformer vault in the large navigation lock. Each of the 13.8 KV supply lines will feed a separate KVA network type transformer with 480 Y/277 volt secondaries. These transformers will be furnished and installed complete with 15 KV switch and secondary network protectors by the Boston Edison Company. The two transformer secondaries are connected to a common collector bus and divide the load. Each transformer is capable of servicing the total project demand load.

b. Spot Network System. - With the Spot Network System, a high degree of continuity of service is made possible because service is maintained to the load during such initial periods as primary circuit failures or removal of a transformer for maintenance. When secondary faults occur, they are cleared by limiters used at each end of a section of secondary cable.

26. Standby Emergency Generator.

a. Emergency Power. - Emergency power will be provided by a diesel-driven electric generator located in the Pumping Station. The power generator will be rated 300 KW - 375 KVA, 80 percent power factor, 60 cycles, 277/480 volts. The generator will be capable of supplying power for selected loads connected in the Main Control Center, including selected loads in the pumping station via the automatic transfer switch in the main control center. See Plate 5-21.

b. Automatic Controls. - Automatic controls will start the pre-set diesel engine when normal utility company voltage drops to a prescribed level and when the main service breaker opens. A time delay control will prevent the starting of the diesel-electric set during momentary interruptions. The transfer of load from the diesel-electric set to the normal power shall be accomplished manually.

27. Secondary Service.

a. Power. - Power from the electrical installations in the navigation locks and the pumping station is normally supplied from the two 500 KVA network transformers. Secondary service from the transformers, via the collector bus and limit lugs, to the main control center located at the navigation locks shall be 277/480 volts, 3 phase, 4 wire, 60 cycles. Power will be distributed to the entire project, including the pumping station, from the main control center.

b. Main Control Center. - The main control center will be totally enclosed, free standing, dead-front assembly with 2,000 ampere main horizontal busses consisting of 10 cubicles. The bus bar structure shall be braced for short-circuit faults of 75,000 amperes symmetrical. The main 2,000 ampere drawout air circuit breaker and all the feeder breakers in the main control center shall have a rated interrupting capacity of not less than 30,000 amperes.

c. Emergency Power. - Emergency power shall be supplied by the 300 KW diesel-electric generator standby unit to the main control center, at 277/480 volts, 3 phase, 4 wire, 60 cycles. The 2,000 ampere automatic transfer switch in the main control center will provide automatic transfer of load from the normal power source to the emergency supply in the event of normal voltage failure.

d. Secondary Metering. - Secondary metering will be provided adjacent to the main control center for the entire project.

e. 208Y/120V Requirements. - Dry-type transformers shall be utilized for transformation of the 480 volt service to the operational voltage of 120 volts, single phase for lighting service and 208 volt single or 3 phase for small power and heating loads. Dry-type transformers shall be air-cooled, 60 cycles, 3 phase rated at 480 volts primary with 120/208 volts, 3 phase, 4 wire secondary windings. In general, power load shall be supplied from the 480 volt, 3 phase service.

f. Reliability. - For greater reliability of service to the pumping station, two separate feeders will be installed from the main control center at the large lock to the pumping station control center. Each feeder will provide 800 amperes at 480 volts, 3 phase, 4 wire service to the pumping station. Only one feeder will be in service with the other feeder utilized as standby. Should the service feeder be damaged and unable to deliver power, the standby feeder will provide service to the pumping station through the automatic transfer switch in the pumping station control center. The two feeders will have separate routing between the two control centers. The main power feeder from the transformer vault and the main emergency power feeder from the pumping station will have separate routing to the lock control center.

g. Grounding. - In general, the grounding system will consist of a 500 MCM bare stranded copper cable for the main grounding loop. From this grounding loop, connections will extend to all equipment including the grounding electrodes to provide a complete grounding system which shall have a low resistance to ground of 5 ohms or less. A grounding bus of 1/4" x 2" copper bar shall be installed the full length of the main control center and the pumping station control center.

h. Auxiliary Systems. - The following auxiliary systems will be installed for the Charles River Dam Project:

(1) Paging & Reply system. Complete with hand set units, amplifiers, speakers and all the required components.

(2) Installation of a complete empty conduit system including outlet and junction and terminal boxes for the Telephone Company wiring.

(3) Navigation Lighting

(4) Fire Alarm System

I. CORROSION PROTECTION

28. Existing Conditions. - The site is approximately 3,000 feet west of Boston Inner Harbor. At the present time, the Charles River Basin is contaminated by sanitary and industrial wastes from combined sewerage overflows. However, the Metropolitan District Commission is presently engaged in a program to eliminate pollution from the Charles River Basin.

29. General Conditions. - In order to provide the pumping equipment with optimum protection from marine growth and the corrosive effects of salt water and salt water spray, for less cost, the following zones of attack are considered:

a. Atmospheric Zone. - Portion of structure that is not wetted directly by water, but is wetted by water spray.

b. Splash Zone. - Portion of structure just above highest water level exposed to wave action.

c. Tidal Zone. - Portion of structure between normal low and high water levels.

d. Permanent Immersed Zone. - Portion of structure below normal low water level.

30. Pumps. - The pumps for this project will be approximately 27 feet in height and 12 feet in diameter. The bottom of the suction bell will be set at approximately elevation 89 feet M.D.C. and the discharge cone at approximately elevation 116 feet M.D.C. The propeller housing and diffuser bowl will be fabricated from 3/4-inch thick plate steel. In order to attain maximum corrosion protection, the entire pump casing will receive a first coat of epoxy zinc-rich paint (E-303) and at least 2 coats of coal tar epoxy (C-200). The total thickness of the coatings will be at least 16 mils. The propeller will be cast of a non-corrosive nickel-alloy steel.

31. Trash Racks. - The trash racks will be fabricated of steel. Because of the fact the trash racks are to be placed in brackish water on the basin side of the pump station, they will be protected against corrosion by one coat of zinc rich primer followed by at least 14 mils of coal tar epoxy.

J. FISHWAY AND SLUICE FACILITIES

32. Purpose. - The purpose of the fishway is to restore anadromous fish, especially American Shad, to the Charles River. It is anticipated that the fishway will be operated during daylight hours between April 15

and June 15 for upstream migrating fish. Downstream fish migration during June through October will be by way of the boat locks. At present, alewife and smelt migrate by passing through the lock of the existing Charles River Dam. It is estimated that the Charles River has the potential to support approximately 200,000 adult shad.

33. Location. - The fishway and sluice facilities will be located in the new dam to the north of the proposed pumping station.

34. Description. - The fishway will consist of a modified vertical slot with a false weir. Attraction water will be provided by a pump. The fishway design shown in the memorandum is tentative. Model studies are being conducted by the North Pacific Division Hydraulic Laboratory and changes, if any, resulting from the model studies will be incorporated into the final contract plans.

35. Operation. - Charles River water is pumped by a 150-horsepower 30" propeller pump into a 30" by 30" by 30" tee. The flow is split for usage in two points in the fishway. A total of 55 cfs is pumped and of this, 20 cfs travels through a reducer to a 20" pipe system with a wafer-type butterfly valve for control. This 20" pipe system leads to the false weir at the basin end of the fishway. There, the flow of 20 cfs enters a hollow box arrangement where the flow is baffled and directed into the first fishway chamber. The purpose of the false weir is to attract the fish and provide a controlable jump for their entrance to the Charles River. Some of the 20 cfs is used to carry the fish over the top of the weir and is wasted to the Charles River. The other outlet of the 30" by 30" by 30" tees is reduced to 24" and also has a wafer-type butterfly valve for control. The 24" pipe system travels in the direction of the tide end of the fishway where it connects into the constant head tank with a flap valve at the discharge end. Approximately 35 cfs enters the constant head tank where two 18" pipes with flare fittings and one 24" pipe with a flare fitting are used as morning glory weirs. This overflow arrangement provides a constant head for the fishway pump to pump against, since the fishway level is influenced by the tide height and varies considerably. Thus, one pump running at a constant speed can do two jobs. The overflow pipes lead to the last three cells of the fishway so that more water can be introduced for attraction purposes. Diffusers are used to reduce the turbulence impact on the migrating fish. At the tide end of the fishway there is a velocity control in the form of a floating weir gate with a slot 1.5 feet wide. The weir rises and falls with the tide with the base of the slot always 4 feet below the tide level. The floating weir is effective from tides ranging from above 96.0 feet to below 111 feet. Headloss through this floating weir is approximately 1.5 feet.

36. Sluices. - Two sluices, each 8 feet wide by 10 feet high are utilized to control the basin during normal runoff. One sluice is located high in the monolith near the fishway to provide good quality attraction water for the fish in the spawning season while the other sluice is located low near the pumping station to draw off brackish water. Refer to Design Memorandum No. 1, "Hydrology and Tidal Hydraulics" for detailed discussion.

K. CONSTRUCTION SEQUENCE

37. Construction Sequence. - A continuing contract will be awarded for construction which will be accomplished in two stages, because of a navigation channel which must be open at all times. The pump station, large navigation lock, fishway and sluice facilities, and gravity walls will be constructed in stage 1 which will take approximately 30 months. This work will be done in the dry (See Design Memorandum No. 4, "Embankments and Foundations"). Stage 2 includes the construction of the 2 small navigation locks, the MDC Police facility and the earth dam. The total construction time is approximately 48 months. A separate contract was awarded to Colt Industries in January 1973 for advance fabrication of the pumps, engines and drive gear units for the pump station.

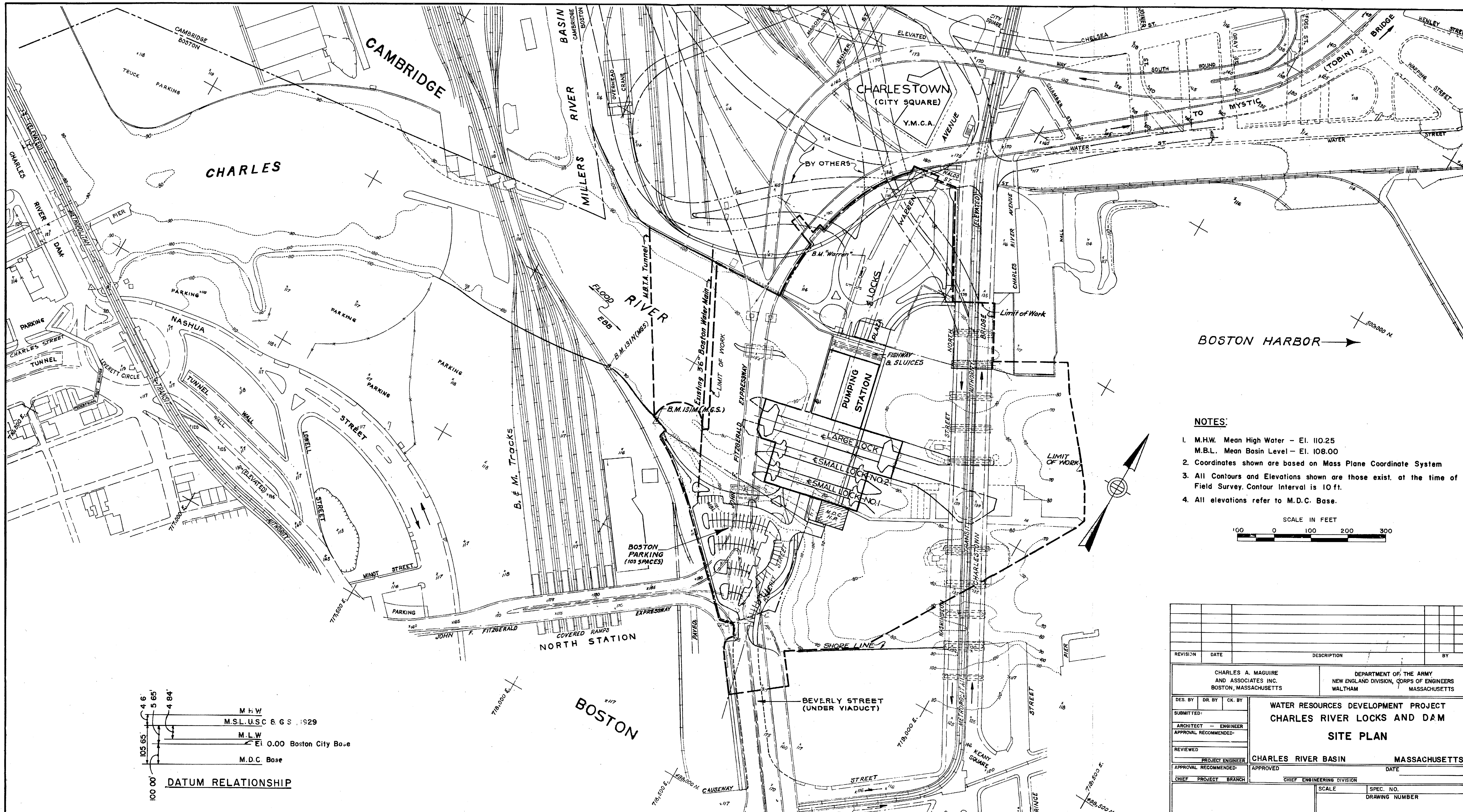
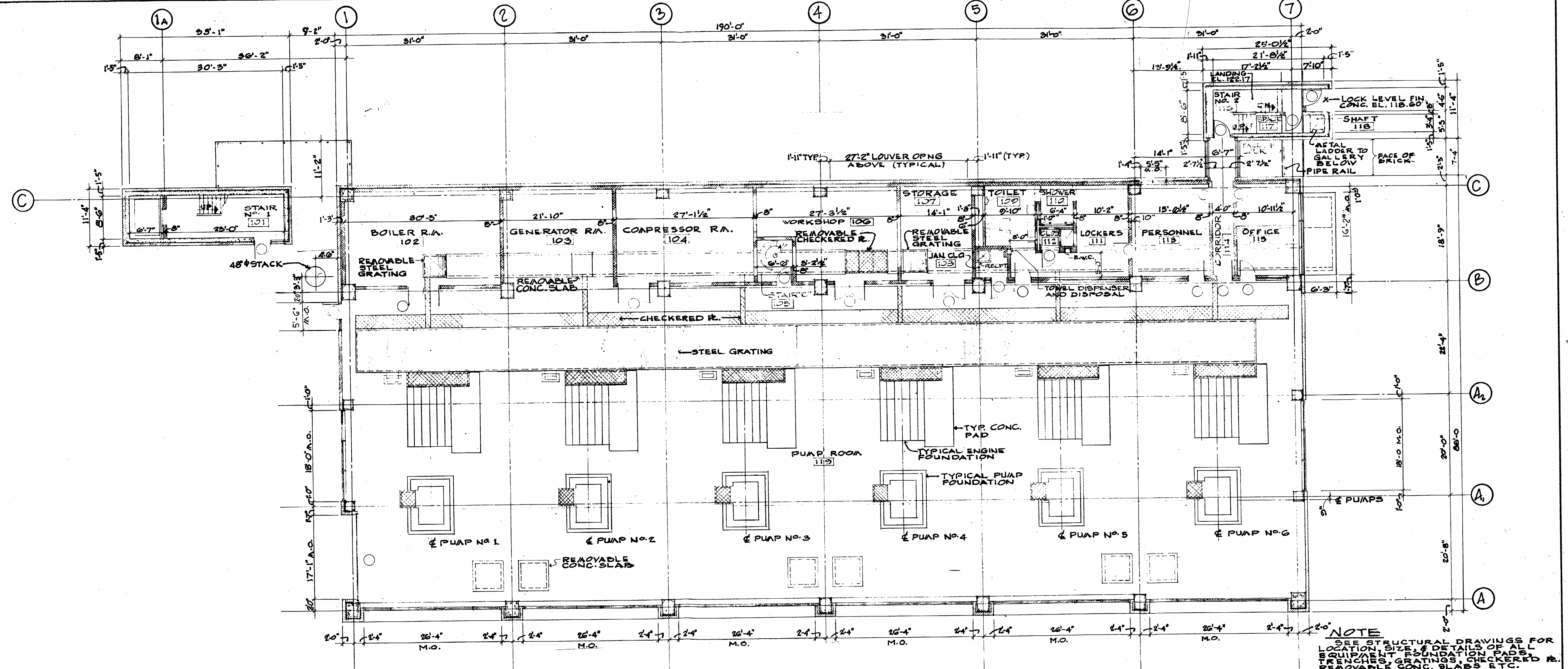

CAMBRIDGE
CHARLES
RIVER
MILLERS RIVER
BASIN
CHARLESTOWN
(CITY SQUARE)
Y.M.C.A.
BY OTHERS
TRUCK PARKING
PIER
PARKING
NASHUA STREET
CHARLES STREET
TUNNEL
LEVERETT CIRCLE
TUNNEL WALL
LOWELL STREET
MINOT STREET
B. & M. Tracks
FLOOD
EBB
M.B.T.A. Tunnel
Existing 36" Boston Water Main
LIMIT OF WORK
B.M. 131M (M.G.S.)
B.M. "Warren"
FITZGERALD EXPRESSWAY
BOSTON PARKING (103 SPACES)
LOCKS
PUMPING STATION
FISHWAY & SLUICES
LARGE LOCK
SMALL LOCK NO. 2
SMALL LOCK NO. 1
Limit of Work
LIMIT OF WORK
BOSTON HARBOR
NORTH STATION
COVERED RAMPS
JOHN F. FITZGERALD EXPRESSWAY
SHORE LINE
BEVERLY STREET (UNDER VIADUCT)
BOSTON
CAUSEWAY
KEANY SQUARE
CHELSEA
WATER ST.
TO MYSTIC (TOBIN) BRIDGE
HENLEY STREET
WAPPING STREET
NOTES:
1. M.H.W. Mean High Water - El. 110.25
M.B.L. Mean Basin Level - El. 108.00
2. Coordinates shown are based on Mass Plane Coordinate System
3. All Contours and Elevations shown are those exist. at the time of Field Survey. Contour Interval is 10 ft.
4. All elevations refer to M.D.C. Base.
SCALE IN FEET
100 0 100 200 300
M.H.W.
M.S.L. U.S.C. & G.S. 1929
M.L.W.
El. 0.00 Boston City Base
M.D.C. Base
4.6'
5.65'
4.84'
105.65'
100.00'
DATUM RELATIONSHIP
REVISION
DATE
DESCRIPTION
BY
CHARLES A. MAGUIRE
AND ASSOCIATES INC.
BOSTON, MASSACHUSETTS
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM MASSACHUSETTS
DES. BY
DR. BY
CK. BY
SUBMITTED:
ARCHITECT — ENGINEER
APPROVAL RECOMMENDED:
REVIEWED
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF PROJECT BRANCH
WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
SITE PLAN
CHARLES RIVER BASIN MASSACHUSETTS
APPROVED
DATE
CHIEF ENGINEERING DIVISION
SCALE
SPEC. NO.
DRAWING NUMBER
SHEET

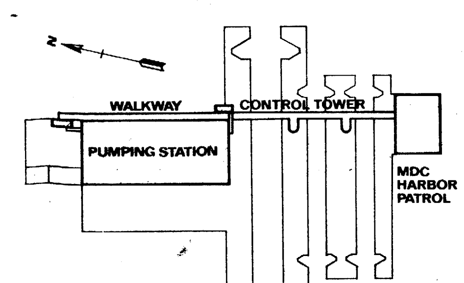
PLATE 5-1



NOTE
SEE STRUCTURAL DRAWINGS FOR LOCATION, SIZE, & DETAILS OF ALL EQUIPMENT FOUNDATION PADS, TRENCHES, GRATINGS, CHECKERED PL. REMOVABLE CONC. SLABS ETC.

MAIN FLOOR PLAN @ ELEV. 122.17

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



KEY PLAN

GRAPHIC SCALES

SAFETY PAYS

VALUE ENGINEERING PAYS DIVIDENDS

REVISION	DATE	DESCRIPTION	BY

CE MAGUIRE, INC.
WALTHAM, MASSACHUSETTS
ARCHITECTS-ENGINEERS-PLANNERS

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

DES. BY | DR. BY J.A.S. | CK. BY

SUBMITTED

ARCHITECT - ENGINEER

PROJECT ENGINEER

APPROVAL RECOMMENDED

CHIEF, TECH. ENG. BRANCH

APPROVAL RECOMMENDED

CHIEF, PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT

CHARLES RIVER DAM

PUMPING STATION MAIN FLOOR PLAN AT EL. 122.17

CHARLES RIVER BASIN — MASSACHUSETTS

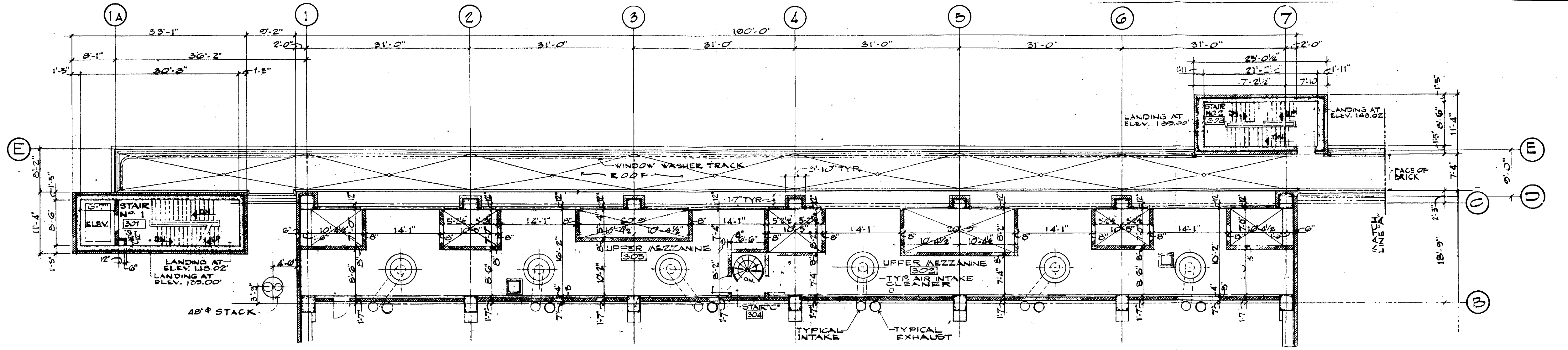
APPROVED:

CHIEF, ENGINEERING DIVISION

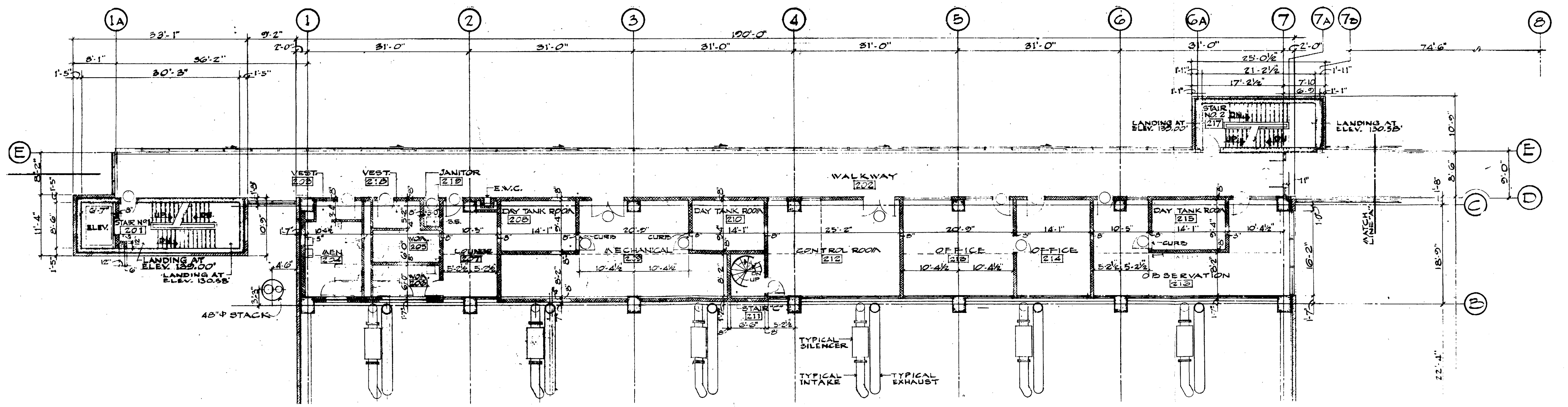
SCALE: | SPEC. NO. DACW 33-73-B-

DATE: | DWG. NO. CHA-2 | SHEET

CORPS OF ENGINEERS U. S ARMY



UPPER MEZZANINE FLOOR PLAN @ ELEV. 150.83'
SCALE 1/8" = 1'-0"



MEZZANINE FLOOR PLAN @ ELEV. 139.00
SCALE: 1/8" = 1'-0"

GRAPHIC SCALES

SAFETY PAYS

REVISION	DATE	DESCRIPTION	BY

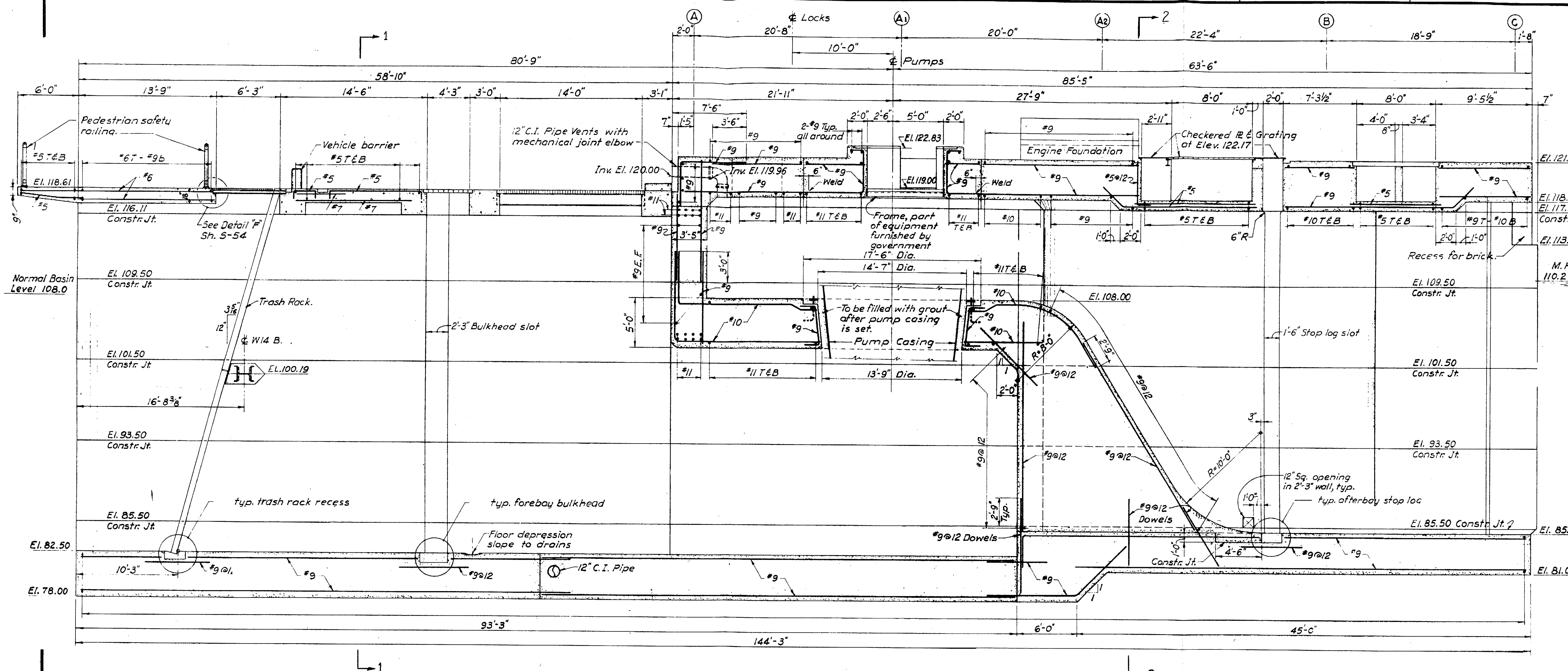
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

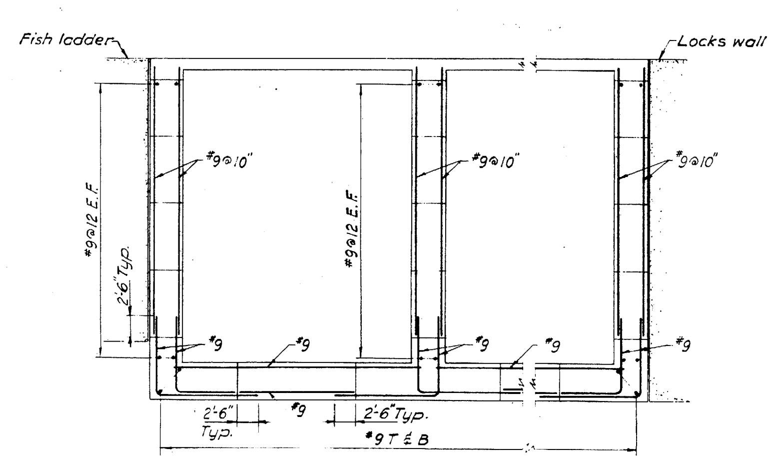
WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
MEZZANINE FLOOR
PLANS
CHARLES RIVER BASIN MASSACHUSETTS

DES. BY | DR. BY | CK. BY
SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, TECH. ENG. BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF BRANCH
APPROVED DATE
CHIEF, ENGINEERING DIVISION
SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

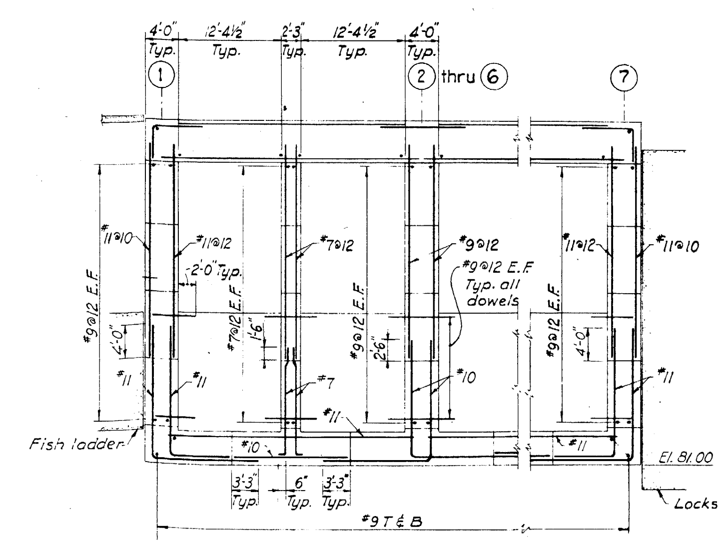
PLATE 5-3

CORPS OF ENGINEERS — U. S. ARMY





SECTION 1-1



SECTION 2-2

SAFETY PAYS

GRAPHIC SCALES

REVISION	DATE	DESCRIPTION	BY

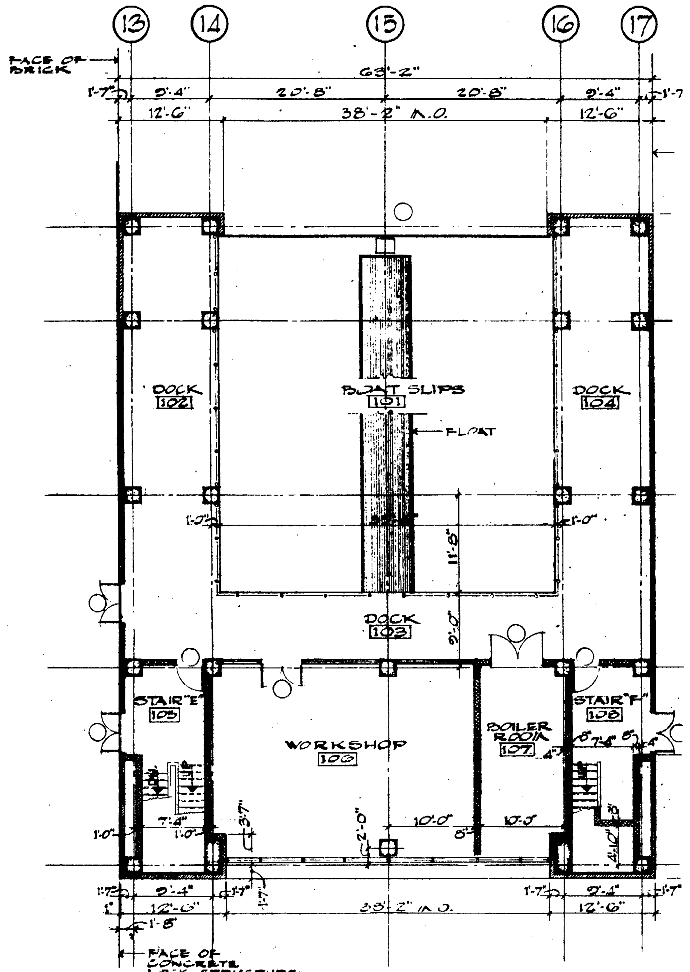
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
PUMPING STATION
GENERAL SECTIONS
CHARLES RIVER BASIN — MASSACHUSETTS

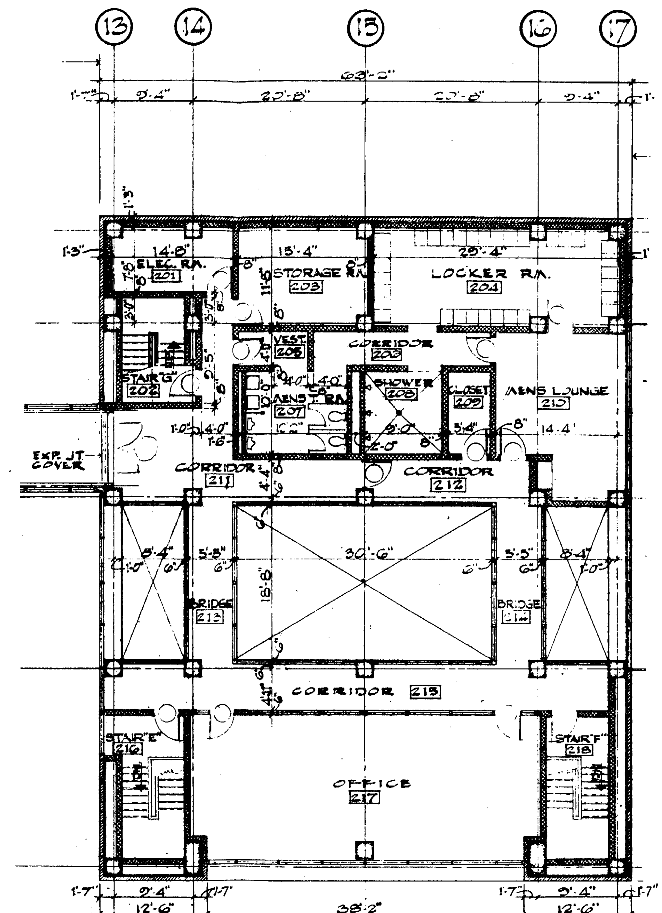
DES. BY | DR. BY | CK. BY
SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, TECH. ENG. BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF BRANCH
APPROVED — DATE
CHIEF, ENGINEERING DIVISION
SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 5-4

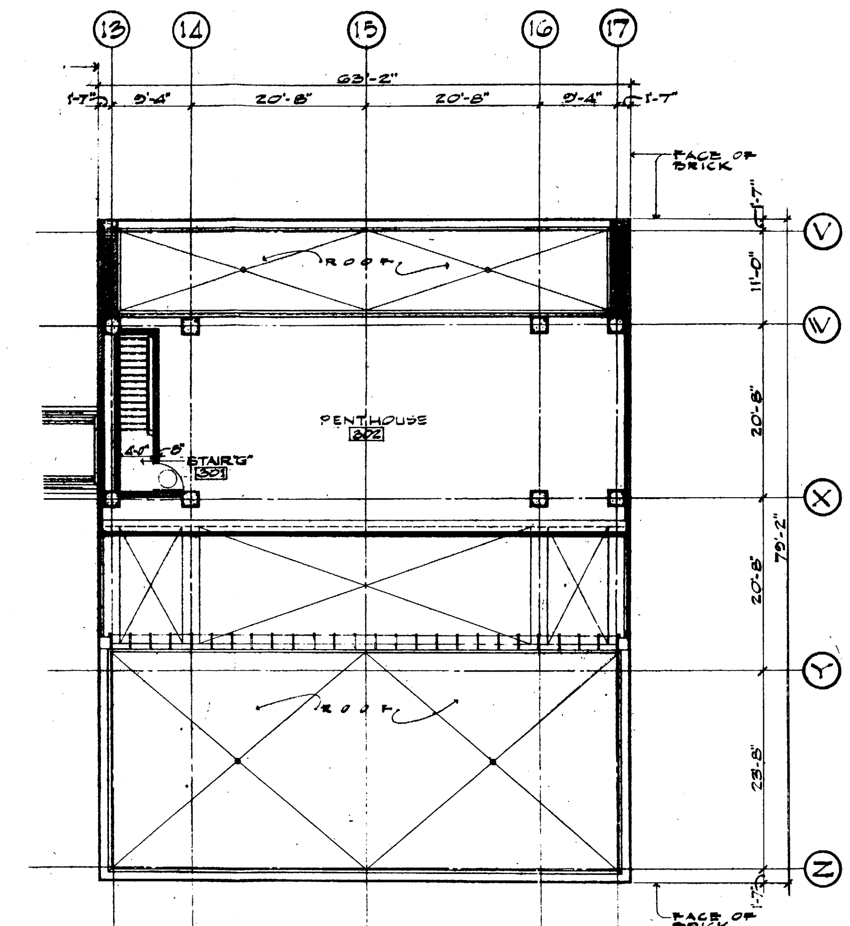
CORPS OF ENGINEERS | U. S ARMY



MAIN FLOOR PLAN
@ ELEV. 118.46'
SCALE: 1/8" = 1'-0"



MEZZANINE FLOOR PLAN
@ ELEV. 139.00'
SCALE: 1/8" = 1'-0"



PENTHOUSE FLOOR PLAN
@ ELEV. 150.62'
SCALE: 1/8" = 1'-0"

GRAPHIC SCALES

REVISION	DATE	DESCRIPTION	BY

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

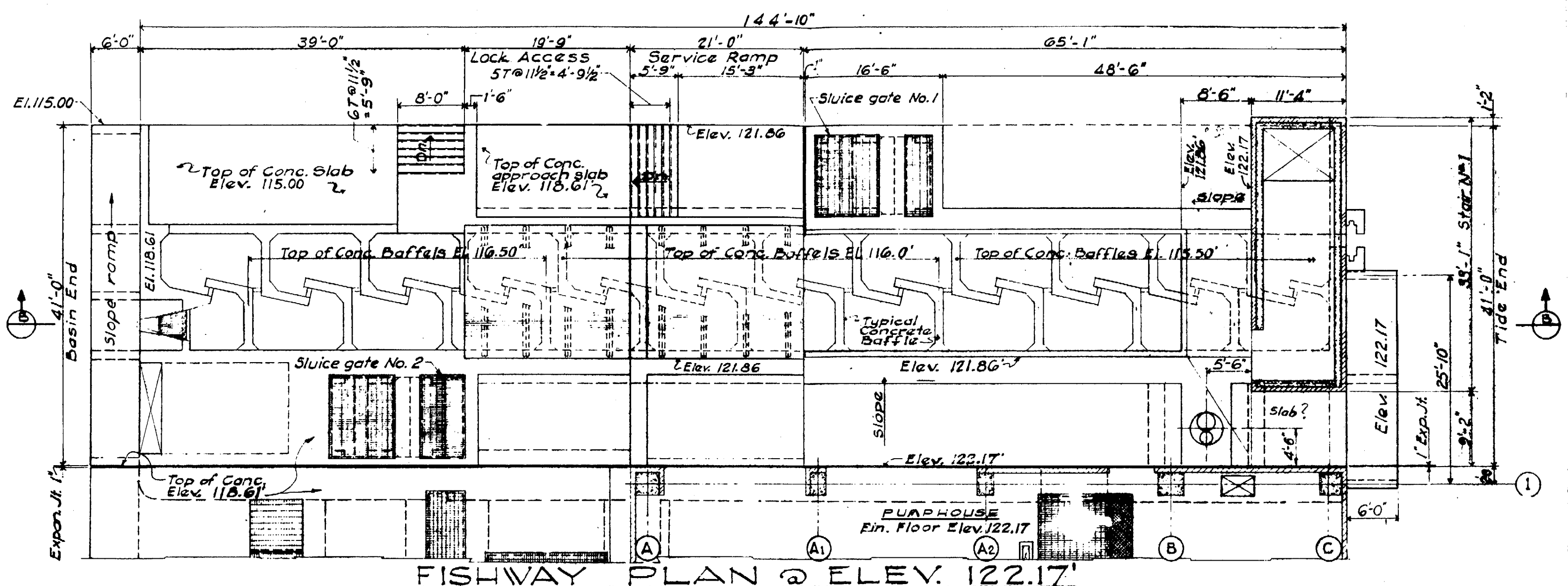
DES. BY | DR. BY | CK. BY
SUBMITTED: CHIEF, SECTION
APPROVAL RECOMMENDED: CHIEF, TECH. ENG. BRANCH
REVIEWED: PROJECT ENGINEER
APPROVAL RECOMMENDED: CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
MDC POLICE FACILITY
FLOOR PLANS
CHARLES RIVER BASIN MASSACHUSETTS

APPROVED: CHIEF, ENGINEERING DIVISION DATE
SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

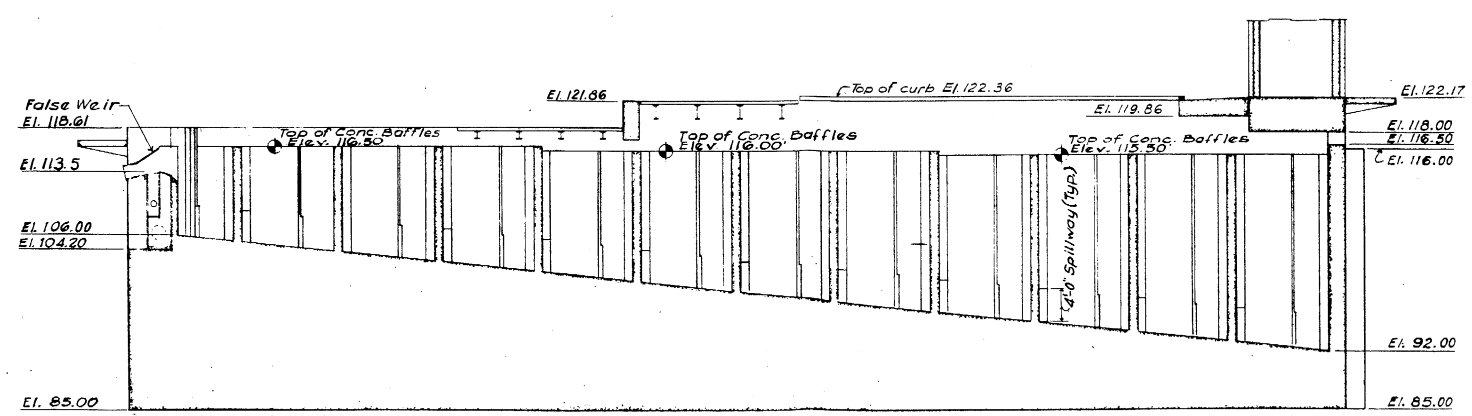
PLATE 5-5

CORPS OF ENGINEERS — U. S ARMY



FISHWAY PLAN @ ELEV. 122.17'

Scale: 1/8" = 1'-0"



SECTION B-B

Scale: 1/8" = 1'-0"

GRAPHIC SCALES

REVISION	DATE	DESCRIPTION	BY

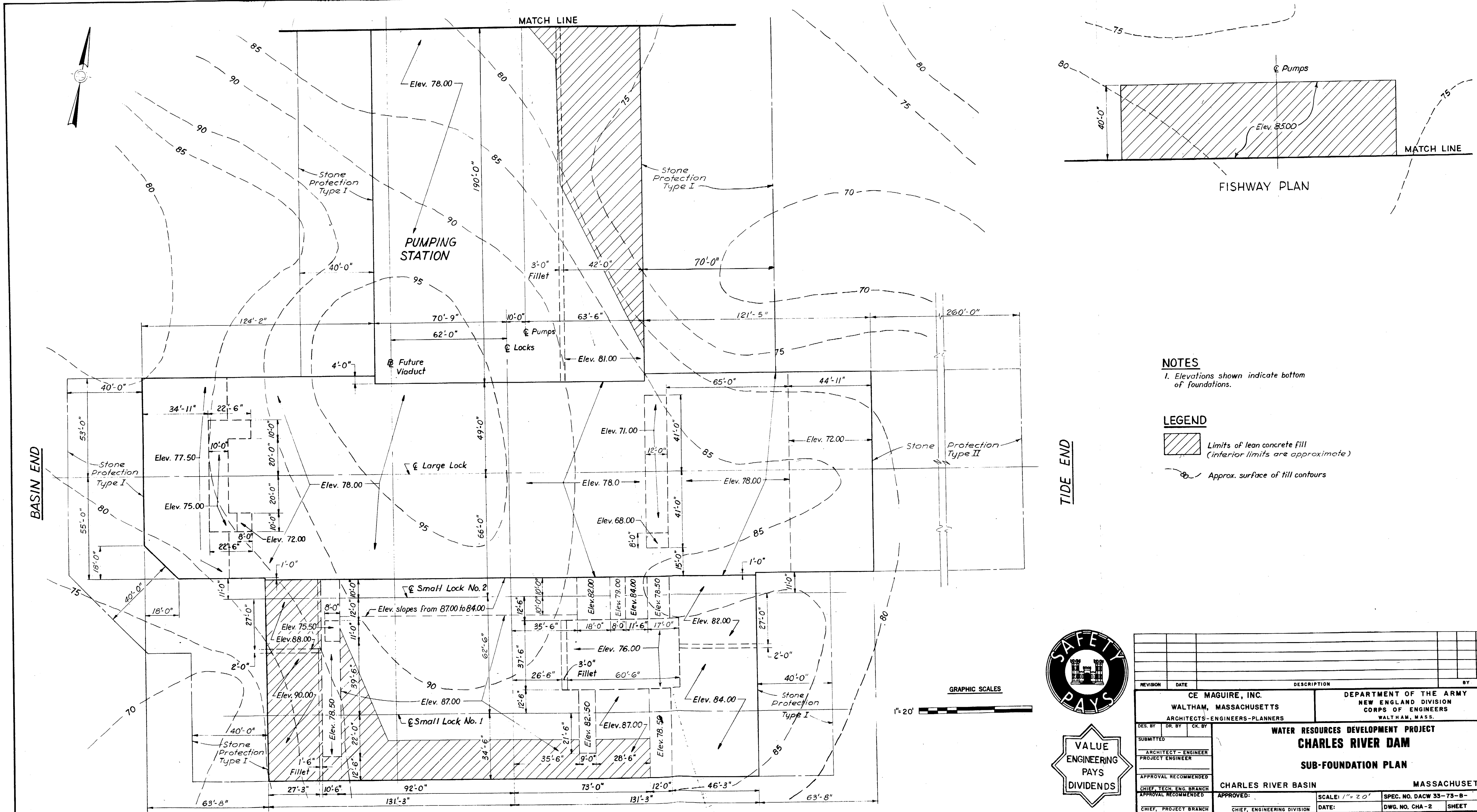
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM

FISH PASSAGE FACILITY
PLAN AND SECTION

CHARLES RIVER BASIN — MASSACHUSETTS

DES. BY | DR. BY | CK. BY
SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, TECH. ENG. BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF BRANCH
APPROVED — DATE
CHIEF, ENGINEERING DIVISION
SCALE — SPEC. NO.
DRAWING NUMBER
SHEET



NOTES
1. Elevations shown indicate bottom of foundations.

LEGEND
Limits of lean concrete fill (interior limits are approximate)
Approx. surface of fill contours

SAFETY PAYS

VALUE ENGINEERING PAYS DIVIDENDS

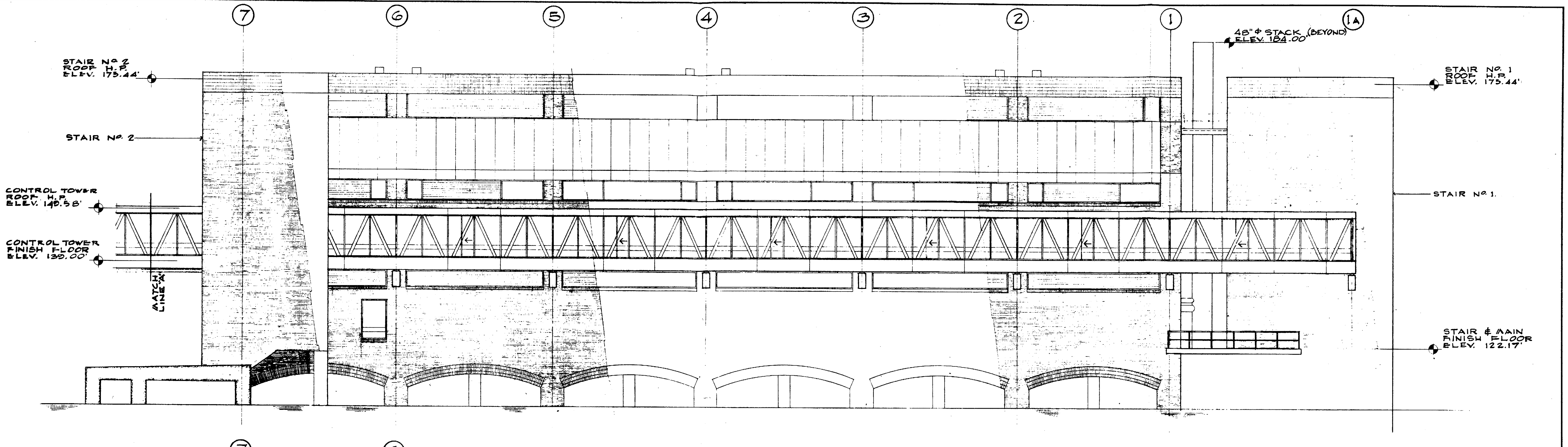
REVISION	DATE	DESCRIPTION	BY

CE MAGUIRE, INC.
WALTHAM, MASSACHUSETTS
ARCHITECTS-ENGINEERS-PLANNERS

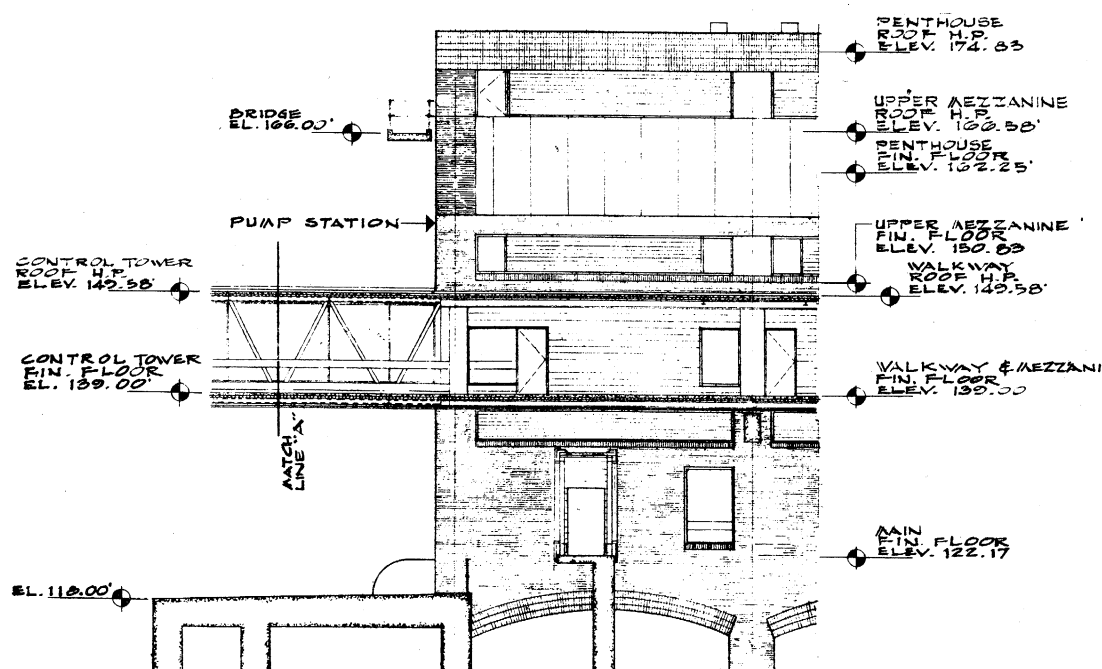
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
SUB-FOUNDATION PLAN
CHARLES RIVER BASIN — MASSACHUSETTS

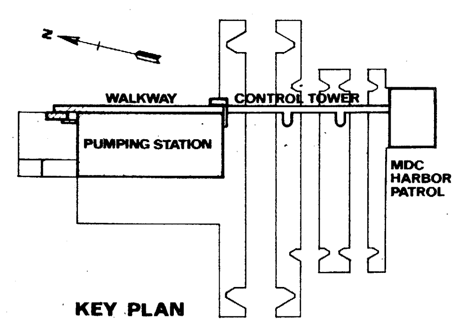
DES. BY | DR. BY | CK. BY
SUBMITTED
ARCHITECT - ENGINEER
PROJECT ENGINEER
APPROVAL RECOMMENDED
CHIEF, TECH. ENG. BRANCH
APPROVAL RECOMMENDED
CHIEF, PROJECT BRANCH
APPROVED:
CHIEF, ENGINEERING DIVISION
SCALE: 1"= 20'
DATE:
SPEC. NO. DACW 33-73-B-
DWG. NO. CHA-2
SHEET



EAST ELEVATION
SCALE: 1/8" = 1'-0"



EAST ELEVATION
SCALE: 1/8" = 1'-0"



KEY PLAN

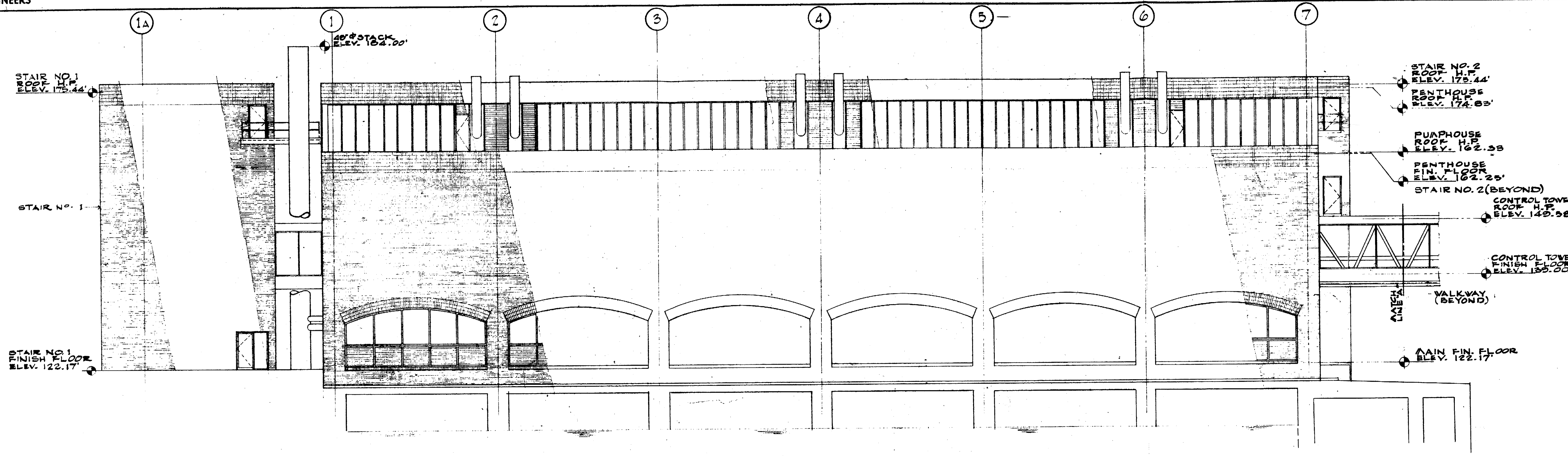
GRAPHIC SCALES

SAFETY PAYS

VALUE ENGINEERING PAYS DIVIDENDS

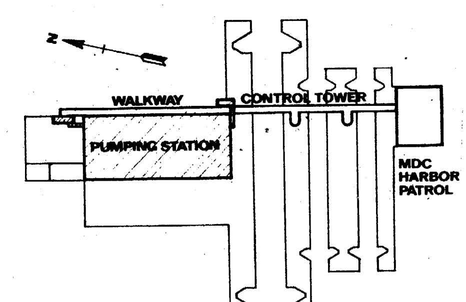
REVISION	DATE	DESCRIPTION	BY

CE MAGUIRE, INC. WALTHAM, MASSACHUSETTS ARCHITECTS-ENGINEERS-PLANNERS	DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.
DES. BY / DR. BY J.A.E. / CK. BY SUBMITTED ARCHITECT - ENGINEER PROJECT ENGINEER APPROVAL RECOMMENDED CHIEF, TECH. ENG. BRANCH APPROVAL RECOMMENDED CHIEF, PROJECT BRANCH	WATER RESOURCES DEVELOPMENT PROJECT **CHARLES RIVER DAM** PUMPING STATION EAST ELEVATION CHARLES RIVER BASIN — MASSACHUSETTS APPROVED: CHIEF, ENGINEERING DIVISION SCALE: / DATE: SPEC. NO. DACW 33-73-B- / DWG. NO. CHA-2 / SHEET



WEST ELEVATION

SCALE: 1/8" = 1'-0"



KEY PLAN

GRAPHIC SCALES

SAFETY PAYS

VALUE ENGINEERING PAYS DIVIDENDS

REVISION	DATE	DESCRIPTION	BY

CE MAGUIRE, INC.
WALTHAM, MASSACHUSETTS
ARCHITECTS-ENGINEERS-PLANNERS

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

DES. BY | DR. BY | CK. BY

SUBMITTED

ARCHITECT - ENGINEER

PROJECT ENGINEER

APPROVAL RECOMMENDED

CHIEF, TECH. ENG. BRANCH

APPROVAL RECOMMENDED

CHIEF, PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT

CHARLES RIVER DAM

PUMPING STATION
WEST ELEVATION

CHARLES RIVER BASIN — MASSACHUSETTS

APPROVED:

CHIEF, ENGINEERING DIVISION

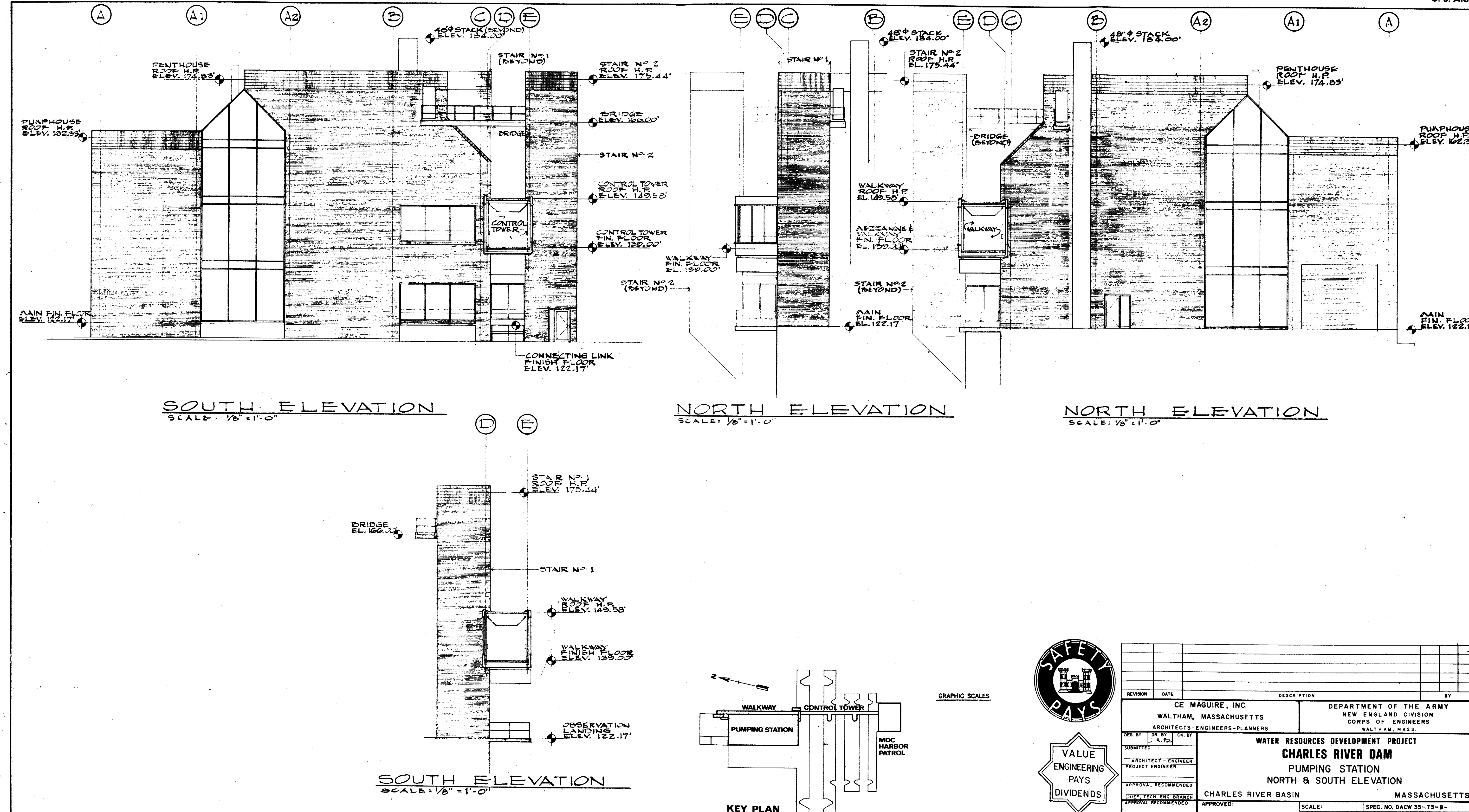
SCALE:

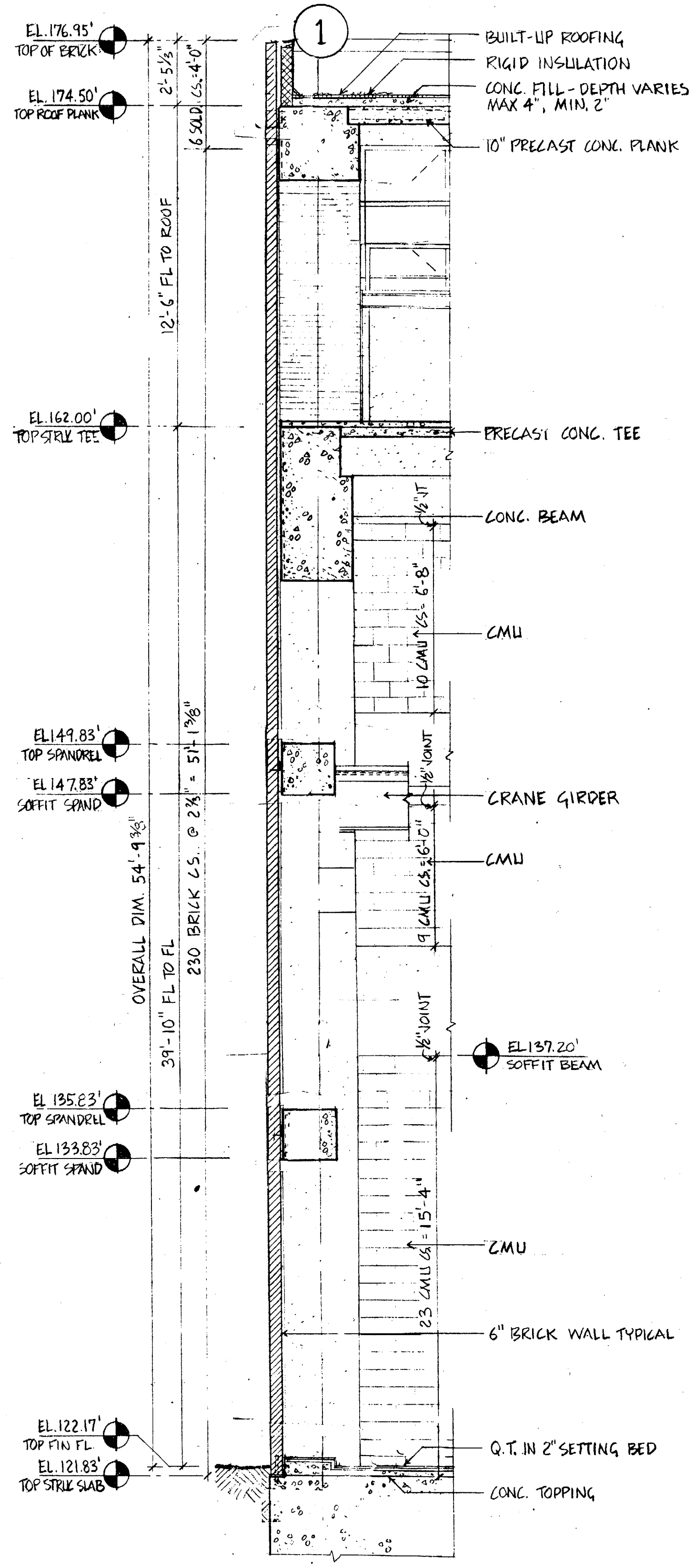
DATE:

SPEC. NO. DACW 33-73-B-

DWG. NO. CHA-2

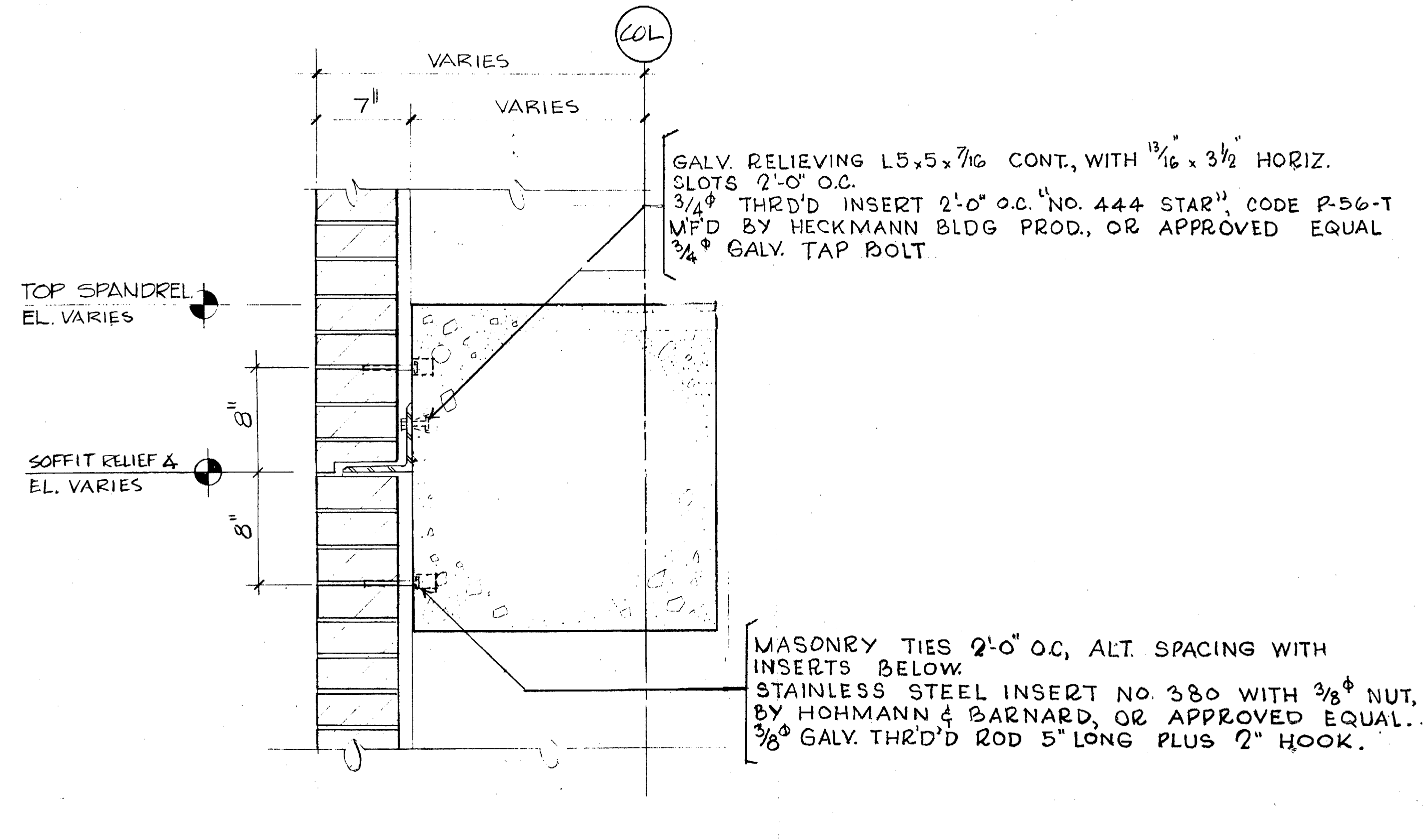
SHEET


PENTHOUSE ROOF H.P. ELEV. 174.83'
PUMPHOUSE ROOF H.P. ELEV. 162.33'
48"Φ STACK (BEYOND) ELEV. 184.00'
STAIR NO. 1 (BEYOND)
STAIR NO. 2 ROOF H.P. ELEV. 175.44'
BRIDGE ELEV. 166.00'
BRIDGE
STAIR NO. 2
CONTROL TOWER ROOF H.P. ELEV. 149.58'
CONTROL TOWER
CONTROL TOWER FIN. FLOOR ELEV. 139.00'
MAIN FIN. FLOOR ELEV. 122.17'
CONNECTING LINK FINISH FLOOR ELEV. 122.17'
SOUTH ELEVATION
SCALE: 1/8" = 1'-0"
STAIR NO. 1
48"Φ STACK ELEV. 184.00'
STAIR NO. 2 ROOF H.P. EL. 175.44'
BRIDGE (BEYOND)
WALKWAY ROOF H.P. EL. 149.58'
MEZZANINE & WALKWAY FIN. FLOOR EL. 139.00'
WALKWAY FIN. FLOOR EL. 139.00'
WALKWAY
STAIR NO. 2 (BEYOND)
MAIN FIN. FLOOR EL. 122.17'
NORTH ELEVATION
SCALE: 1/8" = 1'-0"
48"Φ STACK ELEV. 184.00'
PENTHOUSE ROOF H.P. ELEV. 174.83'
PUMPHOUSE ROOF H.P. ELEV. 162.33'
MAIN FIN. FLOOR ELEV. 122.17'
NORTH ELEVATION
SCALE: 1/8" = 1'-0"
STAIR NO. 1 ROOF H.P. ELEV. 175.44'
BRIDGE EL. 166.00'
STAIR NO. 1
WALKWAY ROOF H.P. ELEV. 149.58'
WALKWAY FINISH FLOOR ELEV. 139.00'
OBSERVATION LANDING ELEV. 122.17'
SOUTH ELEVATION
SCALE: 1/8" = 1'-0"
WALKWAY
CONTROL TOWER
PUMPING STATION
MDC HARBOR PATROL
KEY PLAN
GRAPHIC SCALES
SAFETY PAYS
VALUE ENGINEERING PAYS DIVIDENDS
REVISION
DATE
DESCRIPTION
BY
CE MAGUIRE, INC.
WALTHAM, MASSACHUSETTS
ARCHITECTS-ENGINEERS-PLANNERS
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
DES. BY
DR. BY
CK. BY
SUBMITTED
ARCHITECT - ENGINEER
PROJECT ENGINEER
APPROVAL RECOMMENDED
CHIEF, TECH. ENG. BRANCH
APPROVAL RECOMMENDED
CHIEF, PROJECT BRANCH
WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
PUMPING STATION
NORTH & SOUTH ELEVATION
CHARLES RIVER BASIN
MASSACHUSETTS
APPROVED:
CHIEF, ENGINEERING DIVISION
SCALE:
DATE:
SPEC. NO. DACW 33-73-B-
DWG. NO. CHA-2
SHEET



SECTION - NORTH WALL (BETWEEN COLS. A & B)

SCALE 1/4" = 1'-0"



SPANDREL DETAIL

SCALE 1 1/2" = 1'-0"

WATER RESOURCES DEVELOPMENT PROJECT

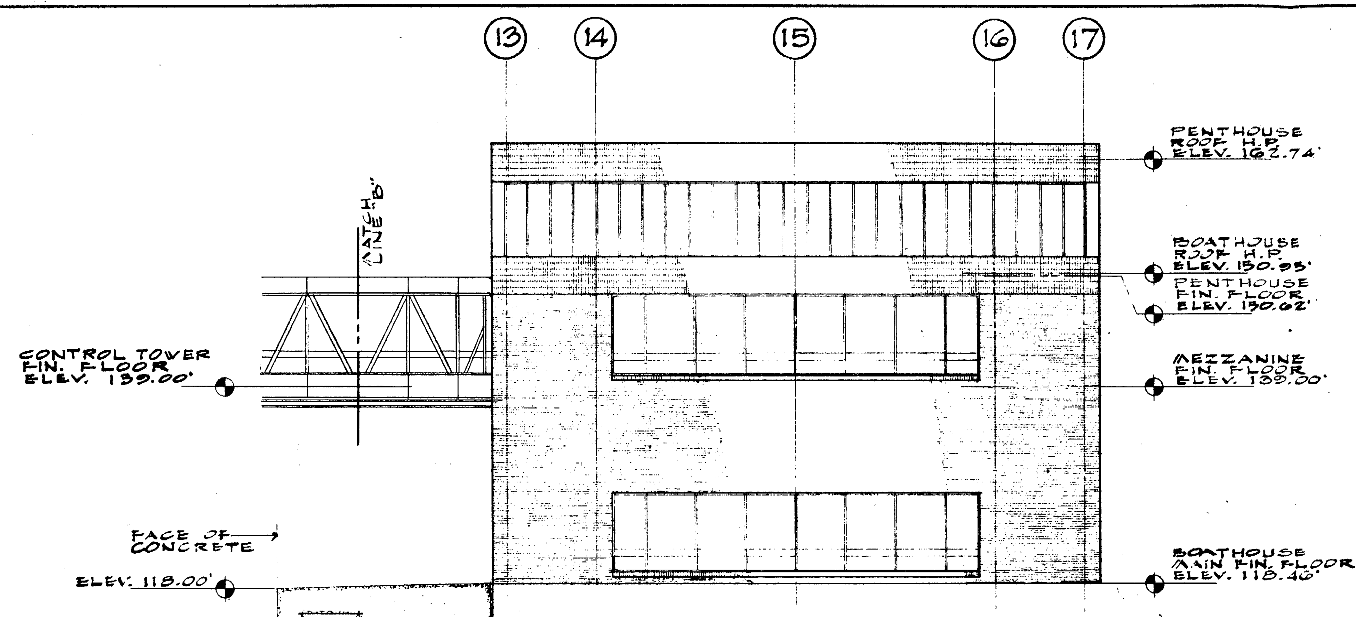
CHARLES RIVER DAM

BRICK WALL

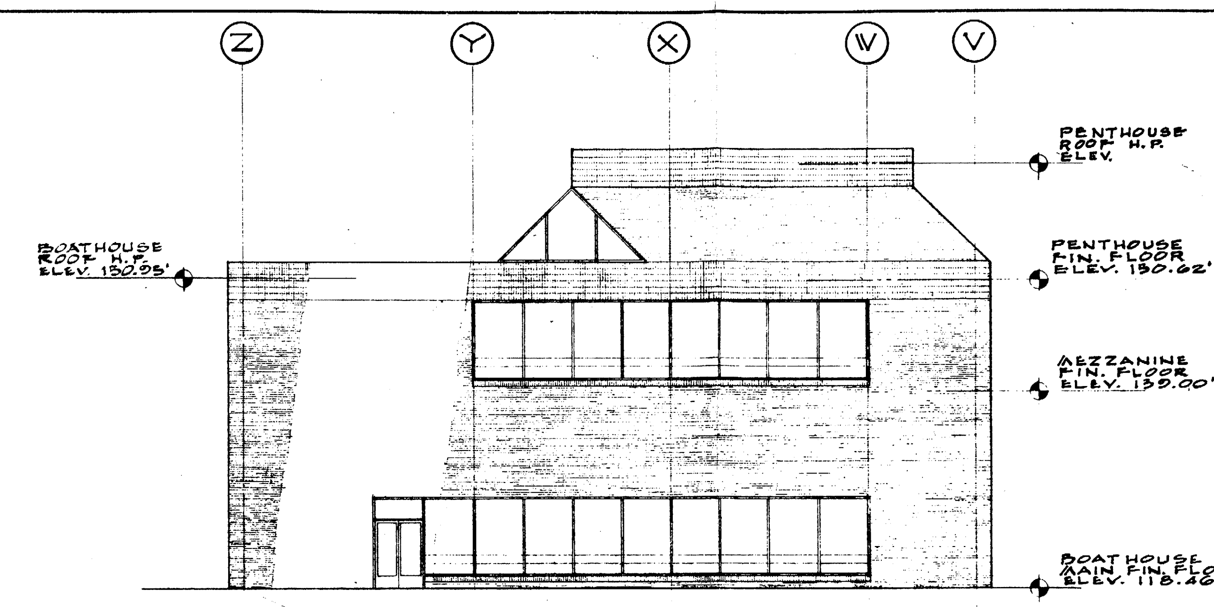
ELEVATION AND TYPICAL SPANDREL DETAIL

CHARLES RIVER BASIN — MASSACHUSETTS

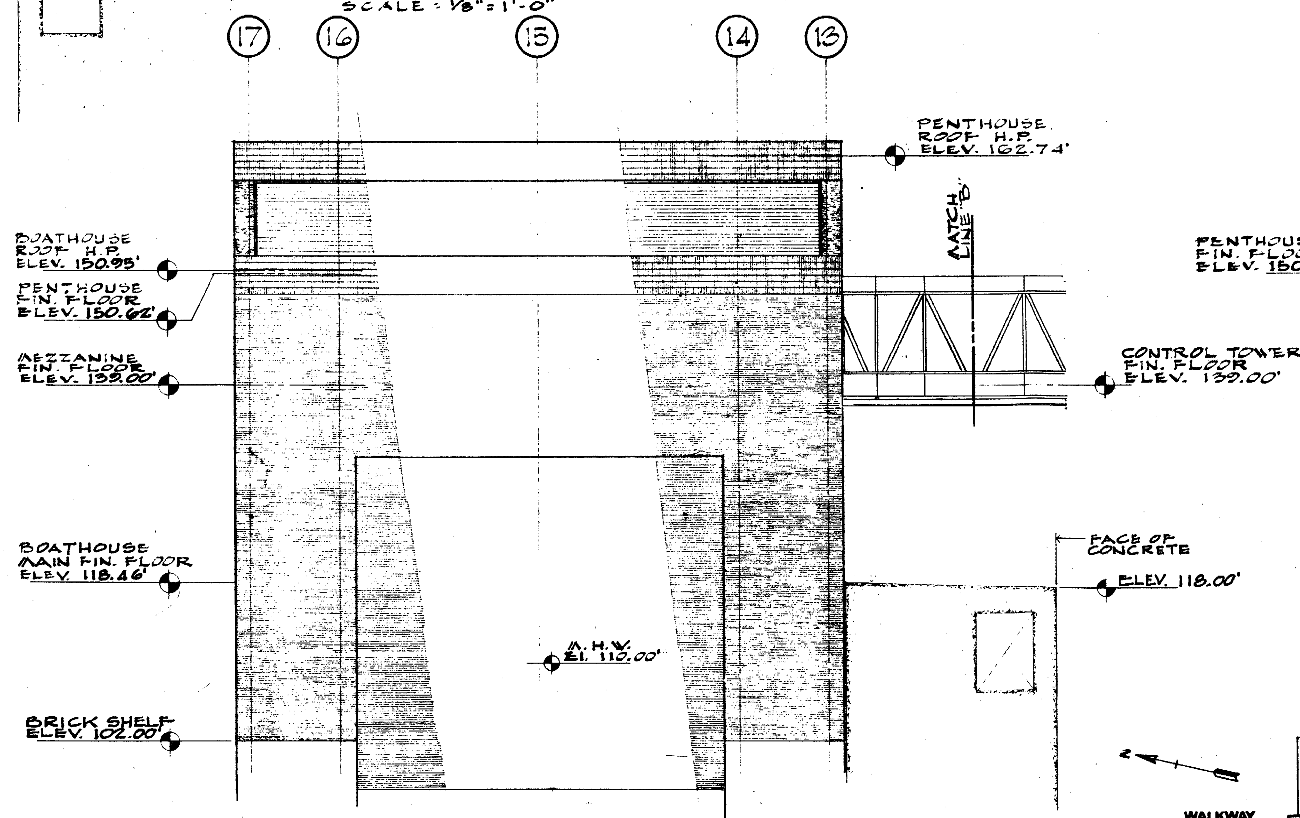
PLATE 5-10 A



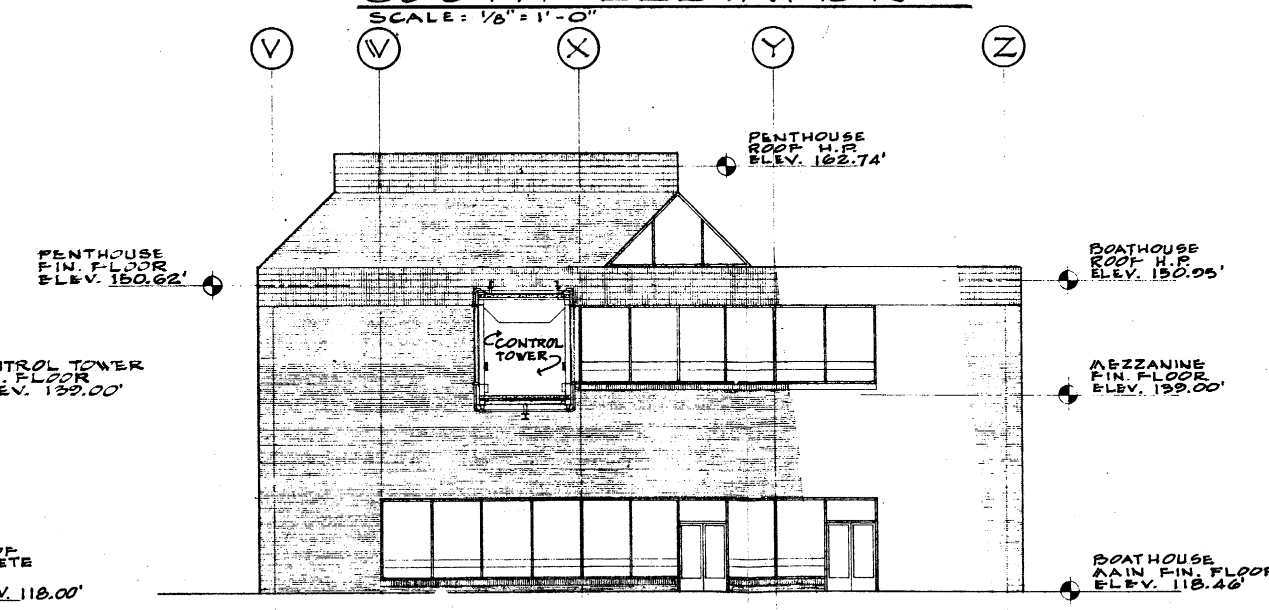
WEST ELEVATION
SCALE: 1/8"=1'-0"



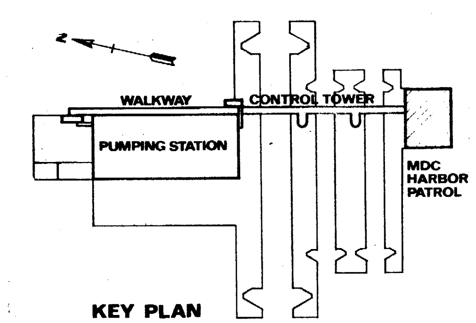
SOUTH ELEVATION
SCALE: 1/8"=1'-0"



EAST ELEVATION
SCALE: 1/8"=1'-0"



NORTH ELEVATION
SCALE: 1/8"=1'-0"



KEY PLAN

GRAPHIC SCALES

VALUE ENGINEERING PAYS DIVIDENDS

REVISION	DATE	DESCRIPTION	BY

CE MAGUIRE, INC.
WALTHAM, MASSACHUSETTS
ARCHITECTS-ENGINEERS-PLANNERS

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

DES. BY | DR. BY J. A. Z. | CK. BY

SUBMITTED
ARCHITECT - ENGINEER
PROJECT ENGINEER
APPROVAL RECOMMENDED
CHIEF, TECH. ENG. BRANCH
APPROVAL RECOMMENDED
CHIEF, PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
MDC POLICE FACILITY
ELEVATIONS

CHARLES RIVER BASIN MASSACHUSETTS

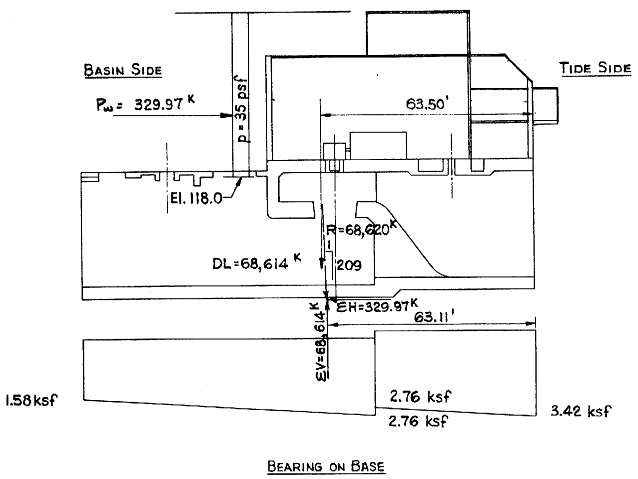
APPROVED:
CHIEF, ENGINEERING DIVISION

SCALE: | SPEC. NO. DACW 33-73-B-
DATE: | DWG. NO. CHA-2 | SHEET

CASE I

CONSTRUCTION CONDITION

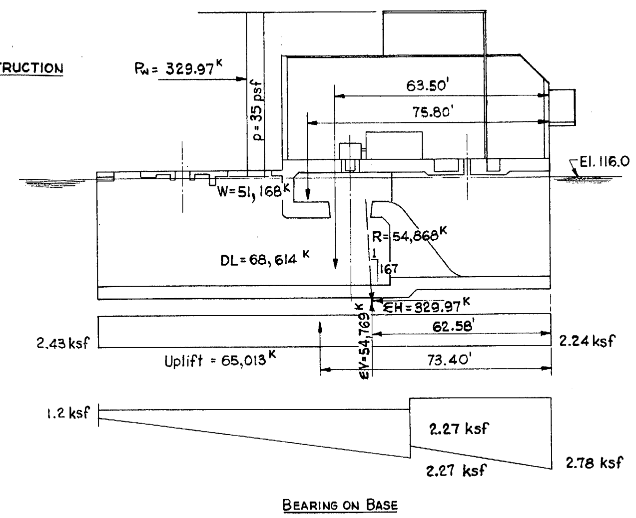
a) Cofferdam in place.
b) Dead Load of Structure.
c) No uplift.
d) Wind Load on Basin Side
e) F.S. against sliding = 105



CASE II

COFFERDAM BURST DURING CONSTRUCTION

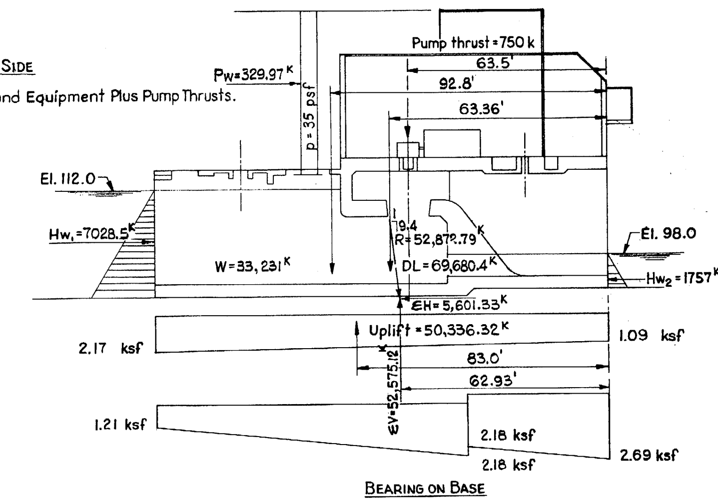
a) Dead Load of Structure
b) Water at El. 116
c) Uplift on Base
d) Wind Load on Basin Side
e) F.S. against sliding = 83.4



CASE III

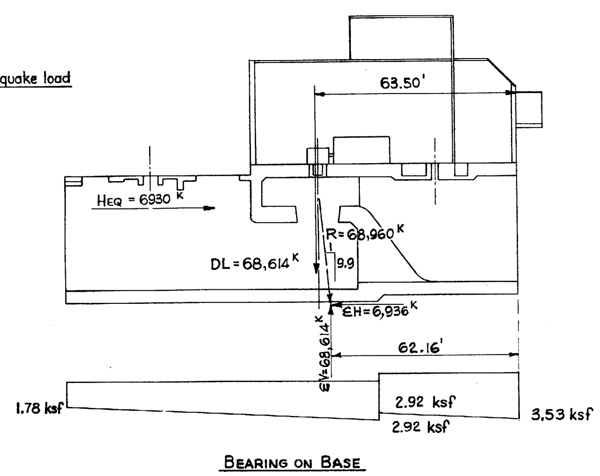
EXTREME HIGH WATER, BASIN SIDE

a) Dead Load of Structure and Equipment Plus Pump Thrusts.
b.) Water at El. 112 Basin Side
c.) Water at El. 98 Tide Side
d.) Uplift on Base
e.) Wind Load on Basin Side.
f.) F.S. against sliding = 4.75



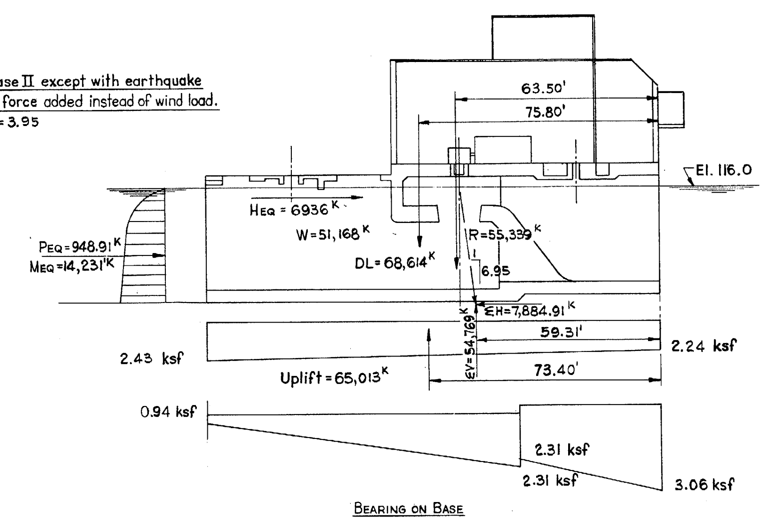
CASE Ia

Same condition as Case I except with earthquake load added instead of wind load.
F.S. against sliding = 4.95



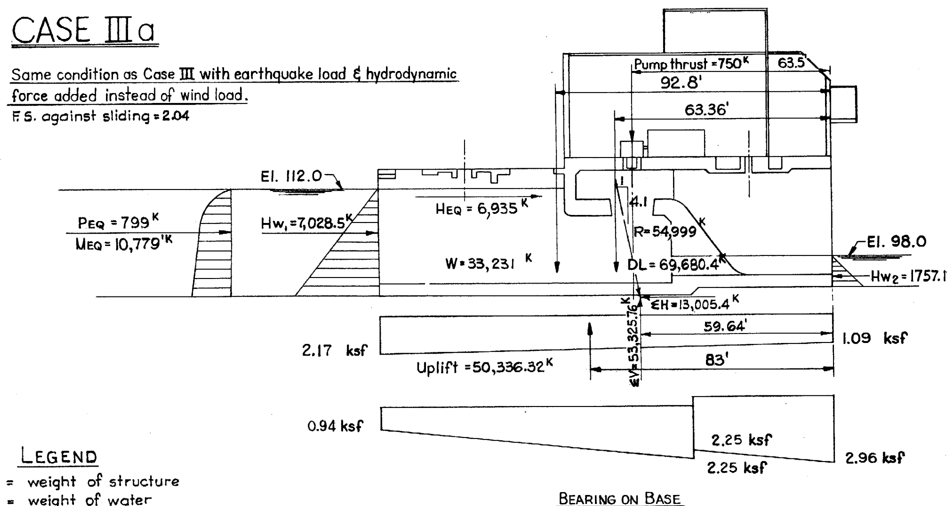
CASE IIa

Same condition as Case II except with earthquake load & hydrodynamic force added instead of wind load.
F.S. against sliding = 3.95



CASE IIIa

Same condition as Case III with earthquake load & hydrodynamic force added instead of wind load.
F.S. against sliding = 2.04



LEGEND

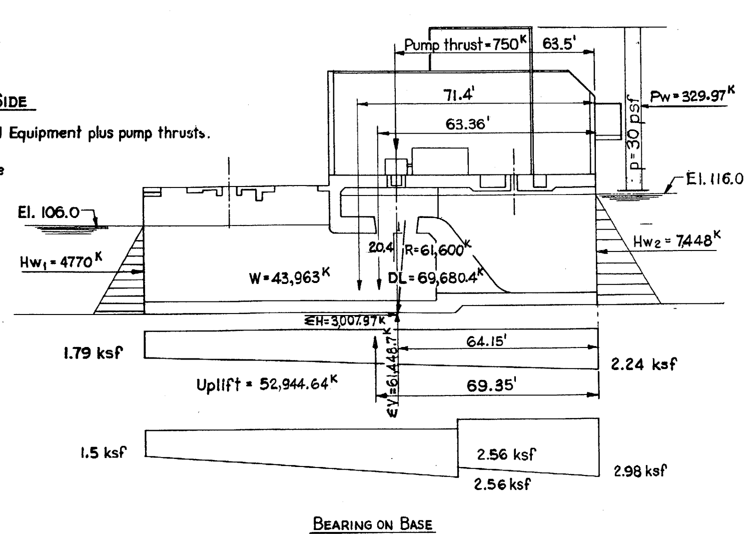
D.L. = weight of structure
W = weight of water
HEQ = earthquake force
PEQ = hydrodynamic force
MEQ = hydrodynamic moment
Hw = water pressure
Pw = wind pressure
ΣH = resultant of horizontal forces
ΣV = resultant of vertical forces
R = resultant of all forces

NOTE: 1. All elevations refer to M.D.C. base.
2. All loads shown are for the whole structure.

CASE IV

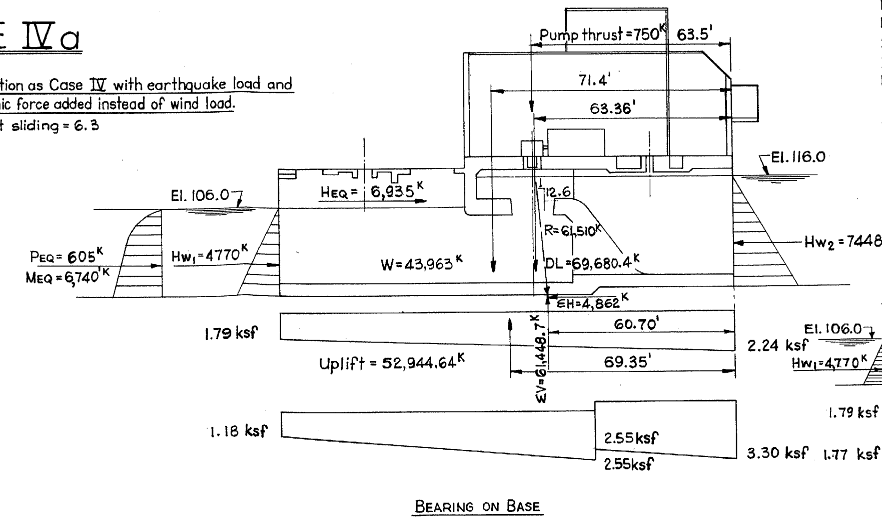
EXTREME HIGH WATER, TIDE SIDE

a) Dead Load of Structure and Equipment plus pump thrusts.
b) Water at El. 106 Basin Side
c) Water at El. 116 Tide Side
d) Uplift on Base
e) Wind load on Tide Side
f) F.S. against sliding = 7.99



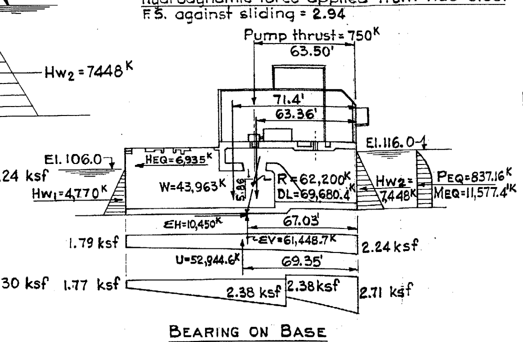
CASE IVa

Same condition as Case IV with earthquake load and hydrodynamic force added instead of wind load.
F.S. against sliding = 6.3



CASE IVb

Same conditions as Case IVa except earthquake and hydrodynamic force applied from tide side.
F.S. against sliding = 2.94



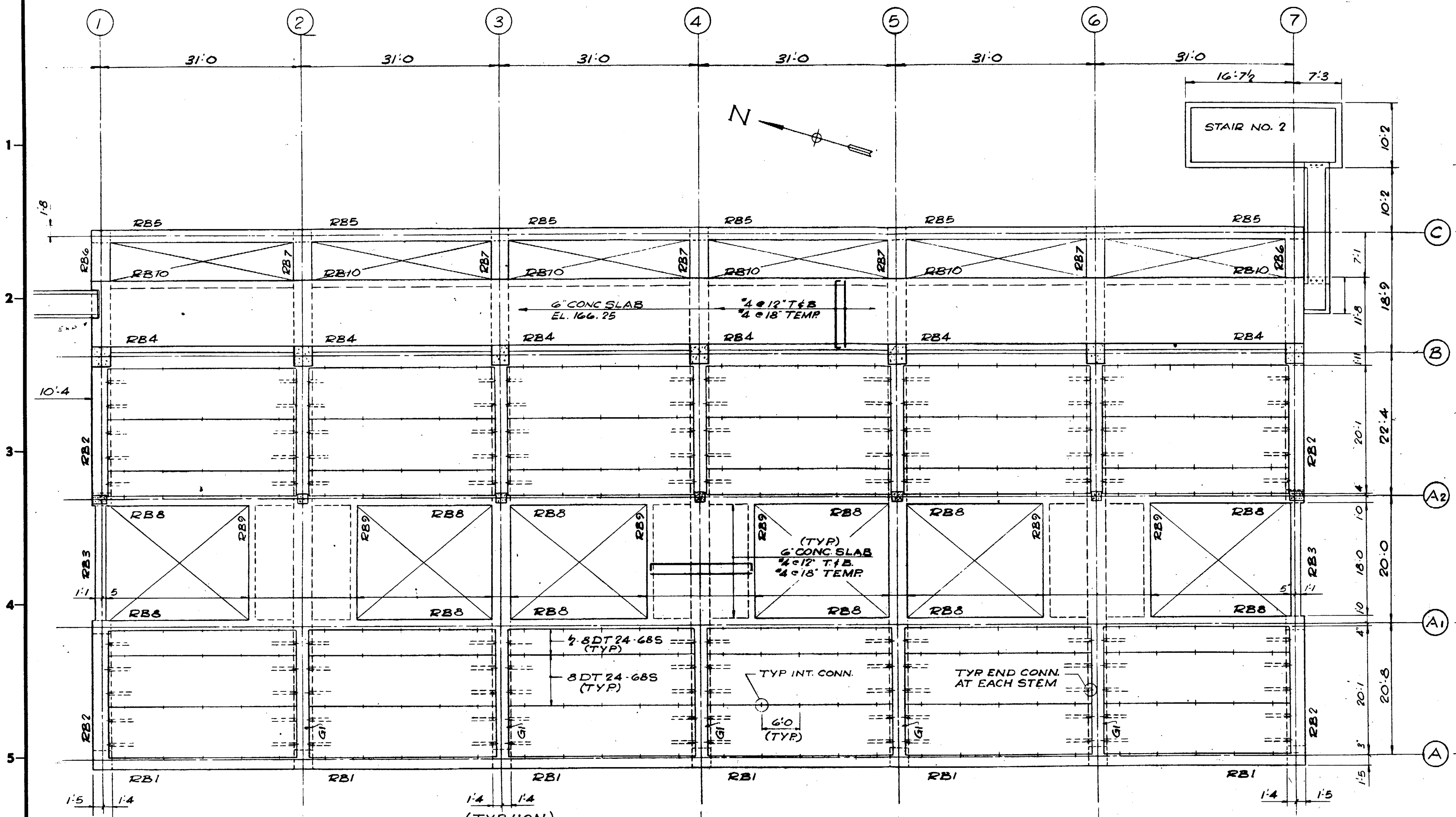
WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
CHARLES RIVER BASIN MASSACHUSETTS

STABILITY DIAGRAMS
PUMPING STATION

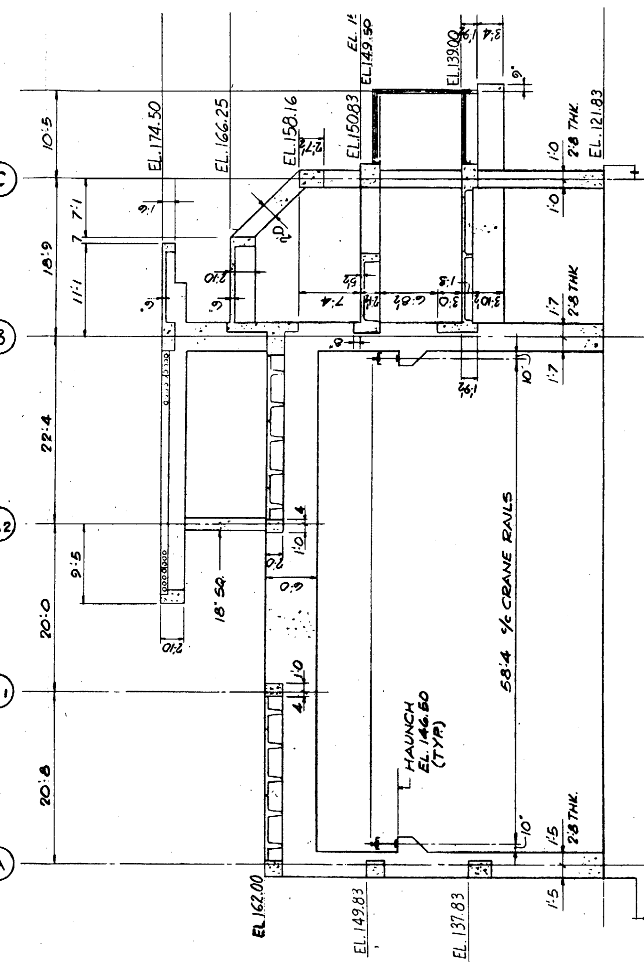
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS WALTHAM, MASS.

PLATE 5-12

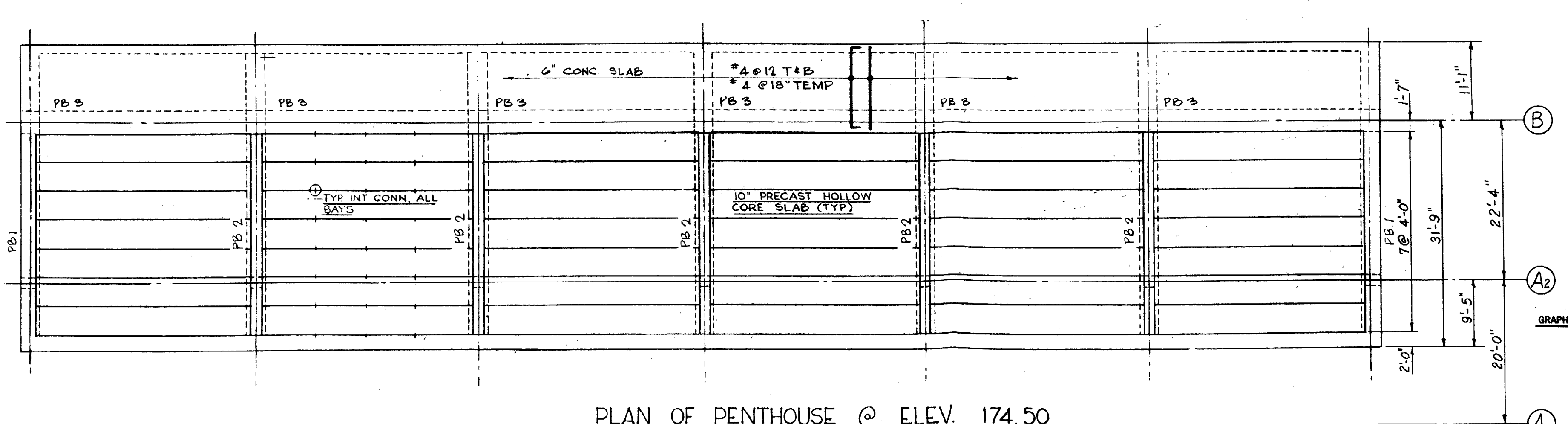
CORPS OF ENGINEERS | U. S. ARMY



PLAN AT ELEV. 162.00
MAIN ROOF & PENTHOUSE FLOOR



TYPICAL INT. SECTION



PLAN OF PENTHOUSE @ ELEV. 174.50

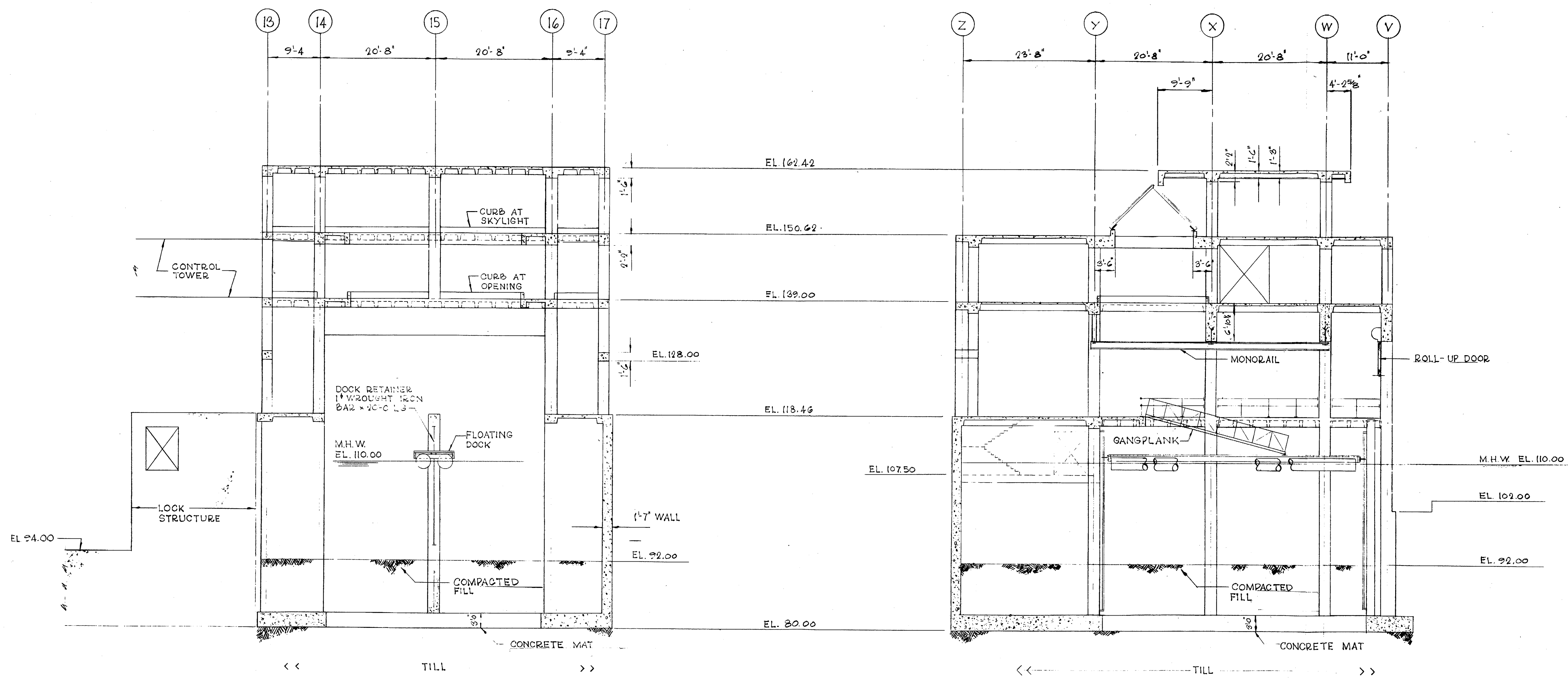
GRAPHIC SCALES

REVISION	DATE	DESCRIPTION	BY

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

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
PUMPING STATION
ROOF PLANS AND TYPICAL
INTERIOR SECTION
CHARLES RIVER BASIN MASSACHUSETTS

DES. BY | DR. BY | CK. BY
SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, TECH. ENG. BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF BRANCH
APPROVED
CHIEF, ENGINEERING DIVISION
DATE
SCALE | SPEC. NO.
DRAWING NUMBER
SHEET



TYPICAL SECTIONS

SCALE 1/8" = 1'-0"

GRAPHIC SCALES

REVISION	DATE	DESCRIPTION	BY

CE MAGUIRE, INC.
WALTHAM, MASSACHUSETTS
ARCHITECTS-ENGINEERS-PLANNERS

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

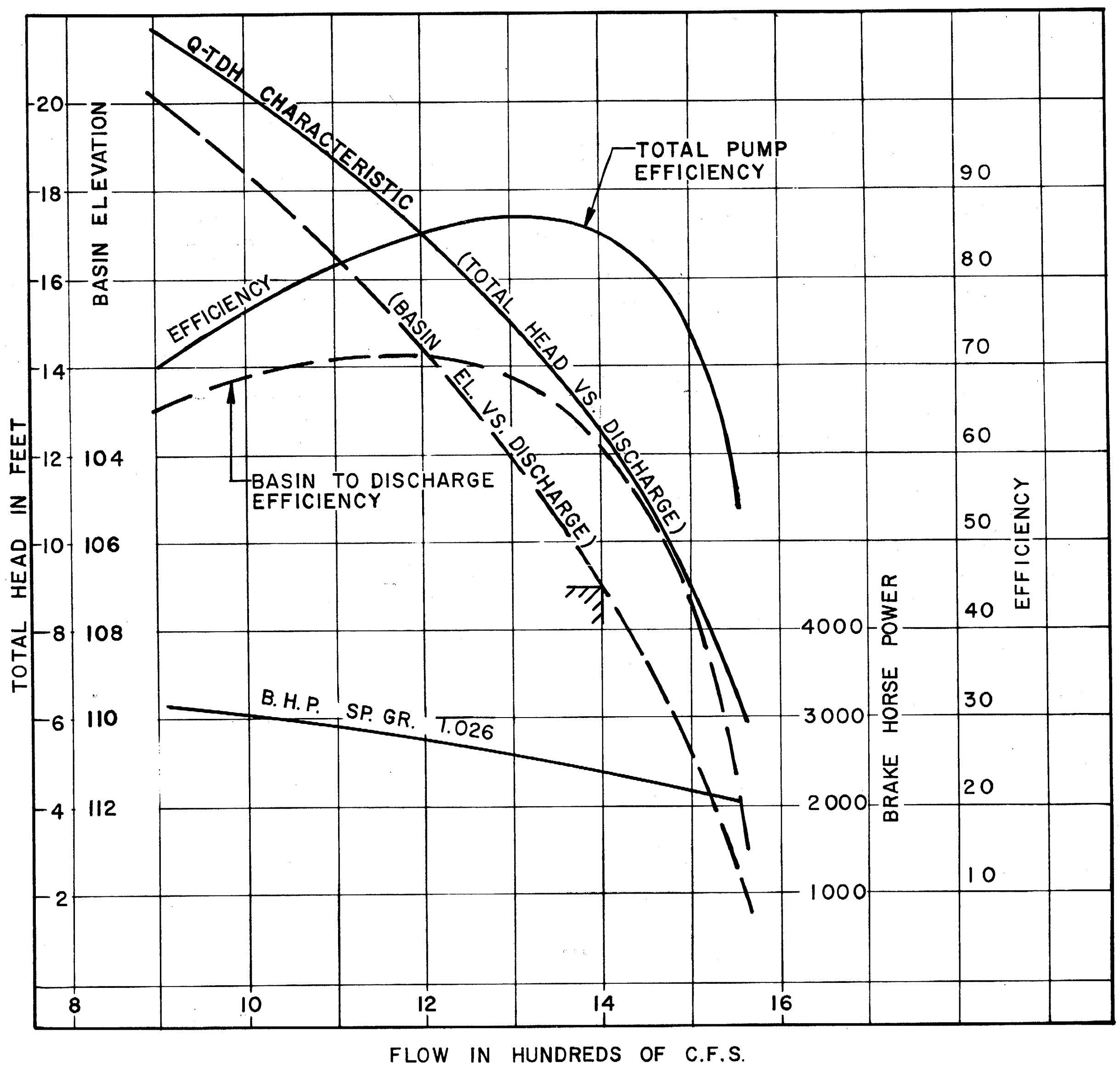
DES. BY | DR. BY J.A.F. | CK. BY

SUBMITTED
ARCHITECT - ENGINEER
PROJECT ENGINEER
APPROVAL RECOMMENDED
CHIEF, TECH. ENG. BRANCH
APPROVAL RECOMMENDED
CHIEF, PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
MDC POLICE FACILITY
TYPICAL SECTIONS
CHARLES RIVER BASIN MASSACHUSETTS

APPROVED:
CHIEF, ENGINEERING DIVISION

SCALE: | SPEC. NO. DACW 33-73-B-
DATE: | DWG. NO. CHA-2 | SHEET



DATA FROM MODEL TEST REPORT DATED 3-30-65 DONE UNDER CONTRACT #205-S FOR THE METROPOLITAN DISTRICT COMMISSION BY FAIRBANKS-MORSE

WATER RESOURCES DEVELOPMENT PROJECT

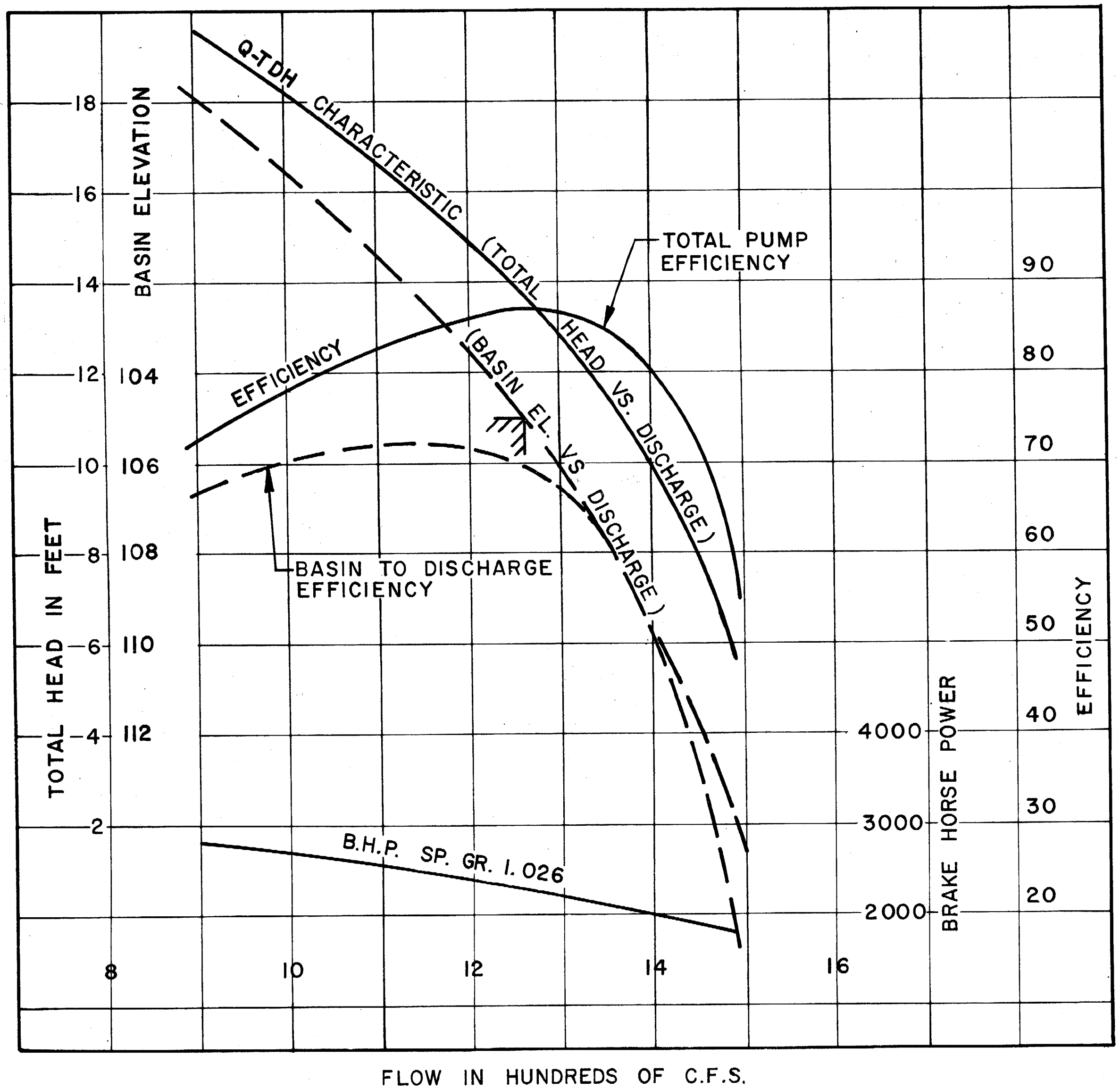
CHARLES RIVER LOCKS AND DAM

CHARLES RIVER BASIN MASSACHUSETTS

PUMP PERFORMANCE CURVES
144" PUMP AT 105.6 R.P.M.

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

PLATE 5-15



DATA FROM MODEL TEST REPORT DATED 3-30-65 DONE UNDER CONTRACT #205-S FOR THE METROPOLITAN DISTRICT COMMISSION BY FAIRBANKS-MORSE

WATER RESOURCES DEVELOPMENT PROJECT

CHARLES RIVER LOCKS AND DAM

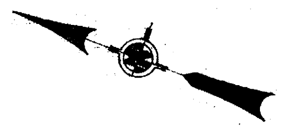
CHARLES RIVER BASIN MASSACHUSETTS

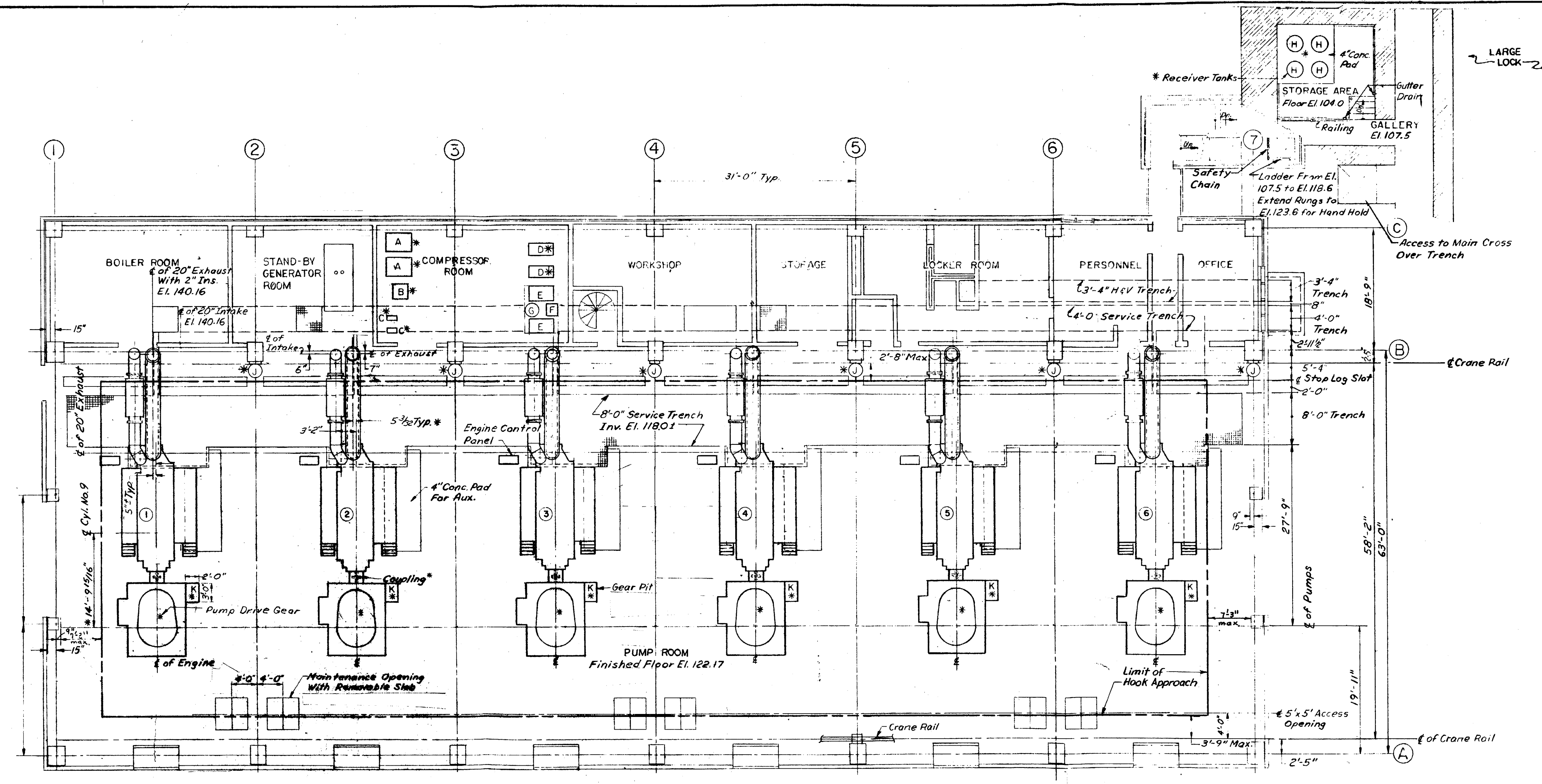
PUMP PERFORMANCE CURVES

144" PUMP AT 101.5 RPM

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

PLATE 5-16





NOTE: Asterisk Indicates Equipment Furnished by the Government to be Installed Under this Contract.

LEGEND

- * A Dirty & Clean Oil Storage Tanks
- * B Lube Oil Purifier
- * C Lube Oil Transfer Pumps
- * D Starting Air Compressors (200 PSI)
- E Instrument Air Compressors (100 PSI)
- F Air Dryer
- G Receiver Tanks (Inst Air)
- * H Receiver Tanks (Starting Air)
- * I Intake Snubber
- * J Jacket Water Expansion Tank
- * K Farval Lubricating System (Grease)

GRAPHIC SCALES
1/8"=1'-0" 8' 0 8' 16'

SAFETY PAYS

VALUE ENGINEERING PAYS DIVIDENDS

REVISION	DATE	DESCRIPTION	BY

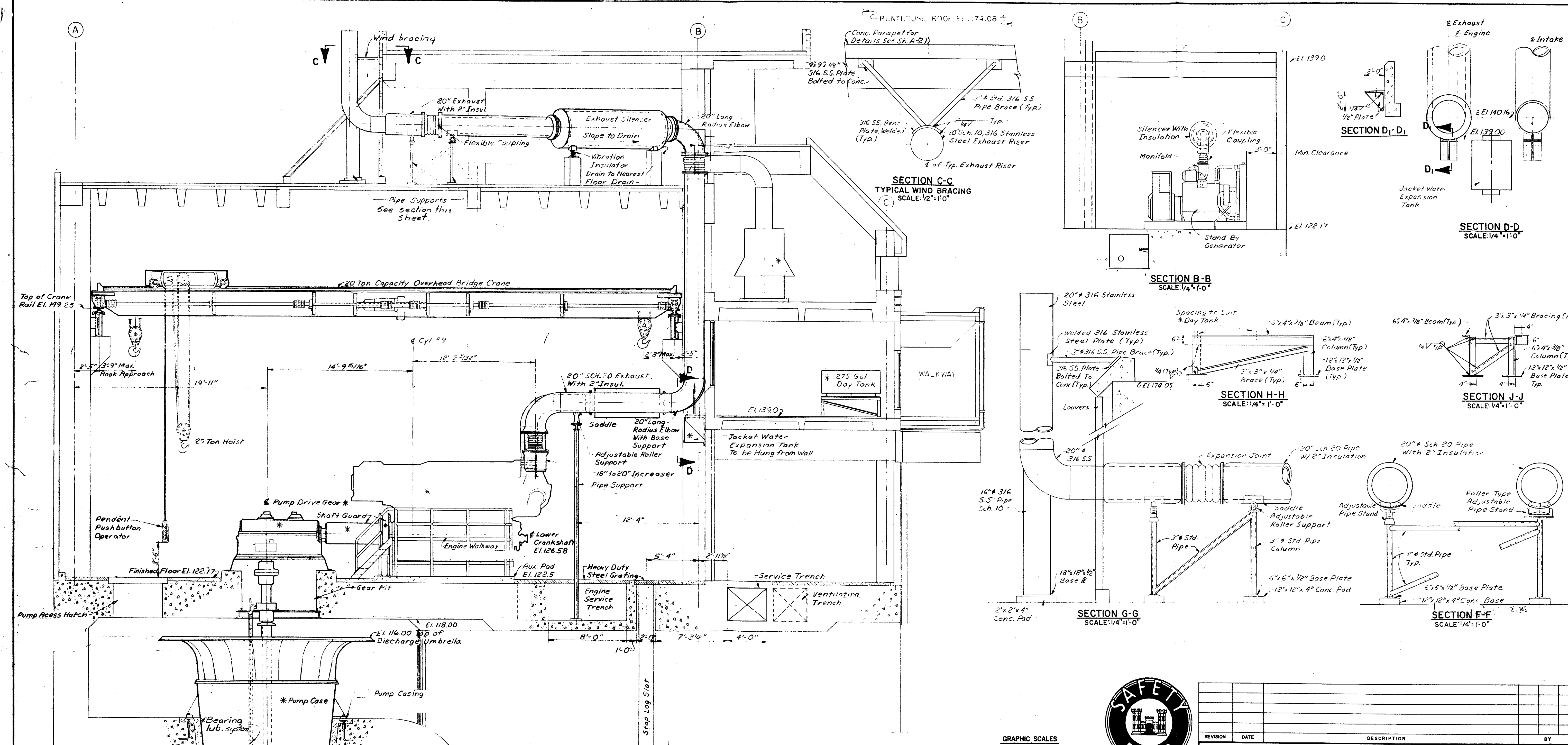
CE MAGUIRE, INC. WALTHAM, MASSACHUSETTS ARCHITECTS-ENGINEERS-PLANNERS

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

DES. BY | DR. BY | CK. BY
SUBMITTED
ARCHITECT - ENGINEER
PROJECT ENGINEER
APPROVAL RECOMMENDED
CHIEF, TECH. ENG. BRANCH
APPROVAL RECOMMENDED
CHIEF, PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
PUMPING STATION
EQUIPMENT LAYOUT - PLAN MAIN FLOOR
CHARLES RIVER BASIN MASSACHUSETTS

APPROVED: CHIEF, ENGINEERING DIVISION | SCALE: 1/8"=1'-0" | SPEC. NO. DACW 33-73-B- | DATE: | DWG. NO. CHA-2 | SHEET



REVISION	DATE	DESCRIPTION	BY

CE MAGUIRE, INC.
WALTHAM, MASSACHUSETTS
ARCHITECTS-ENGINEERS-PLANNERS

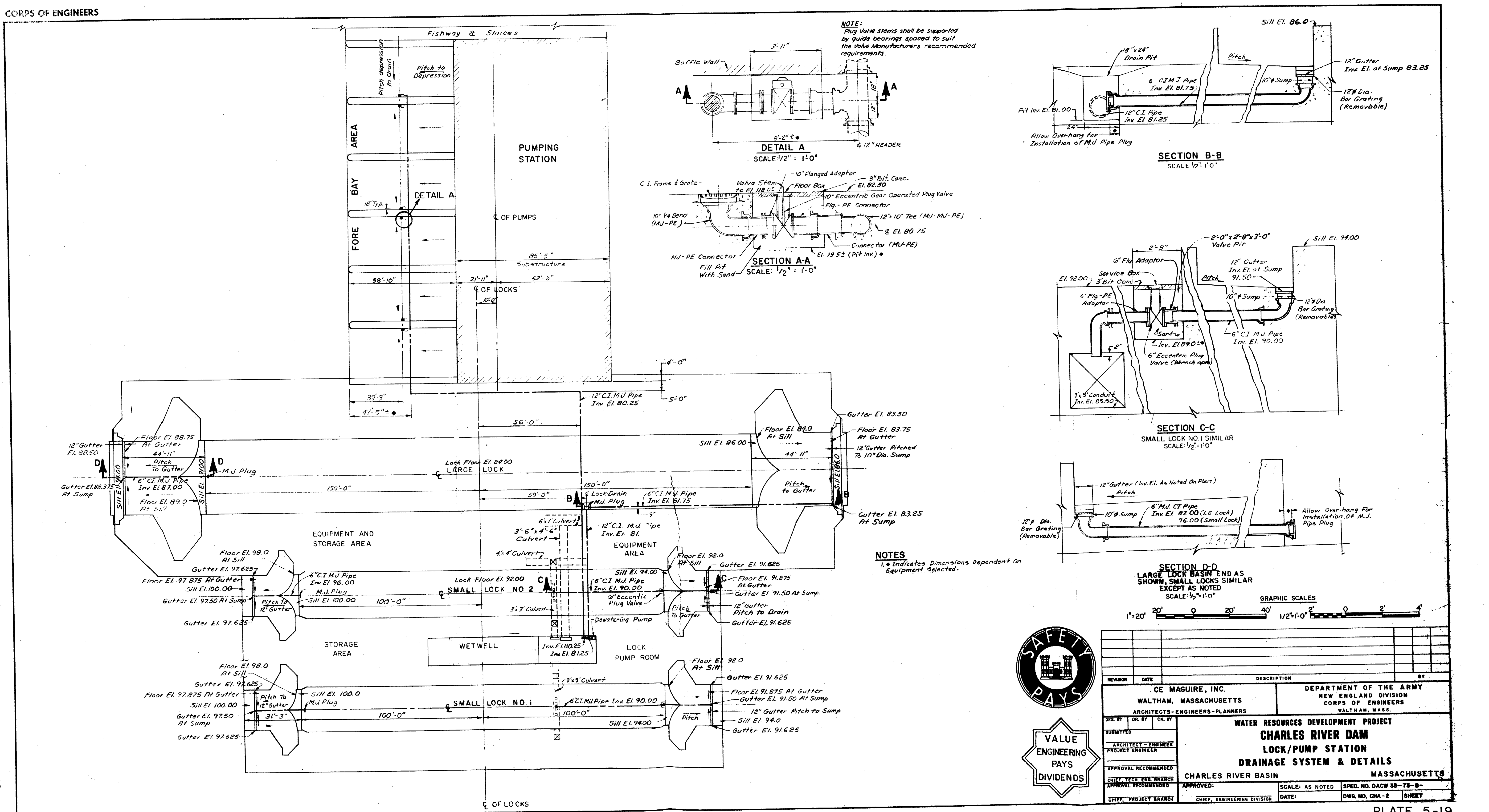
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

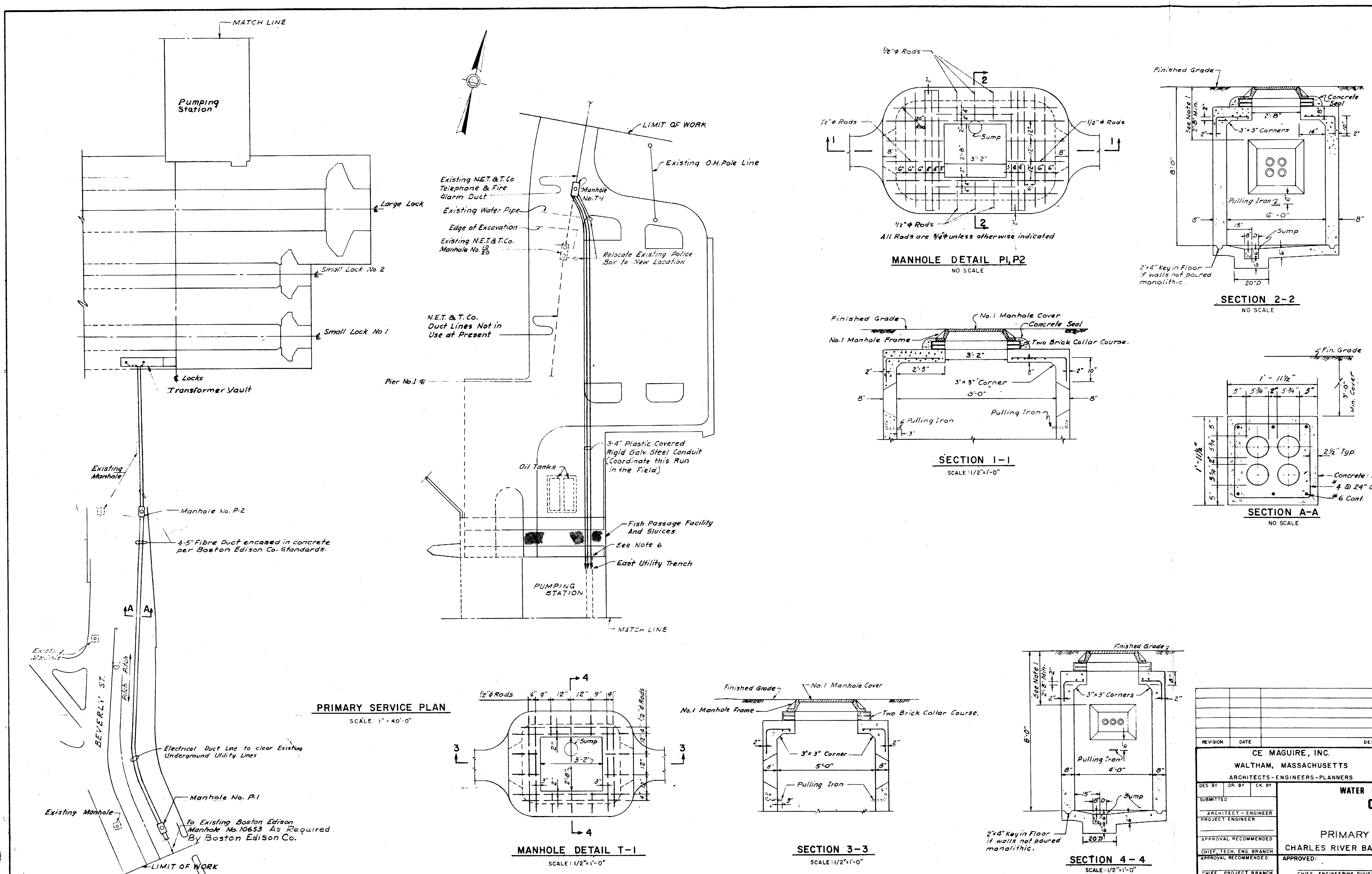
DES. BY | DR. BY | CK. BY
SUBMITTED
ARCHITECT-ENGINEER
PROJECT ENGINEER
APPROVAL RECOMMENDED
CHIEF, TECH. ENG. BRANCH
APPROVAL RECOMMENDED
CHIEF, PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
PUMPING STATION
TYPICAL SECTION AND DETAILS

CHARLES RIVER BASIN — MASSACHUSETTS

APPROVED:
CHIEF, ENGINEERING DIVISION
SCALE: AS NOTED | SPEC. NO. DACW 33-73-B-
DATE: | DWG. NO. CHA-2 | SHEET


Fishway & Sluices
PUMPING STATION
FORE BAY AREA
DETAIL A
℄ OF PUMPS
85'-5"
Substructure
58'-10"
21'-11"
63'-8"
℄ OF LOCKS
39'-3"
47'-5"±◆
56'-0"
12"C.I. MJ Pipe Inv. El. 80.25
LARGE LOCK
Lock Floor El. 84.00
150'-0"
59'-0"
EQUIPMENT AND STORAGE AREA
EQUIPMENT AREA
SMALL LOCK NO 2
Lock Floor El. 92.00
100'-0"
STORAGE AREA
WETWELL
LOCK PUMP ROOM
SMALL LOCK NO. 1
Dewatering Pump
℄ OF LOCKS
NOTE:
Plug Valve stems shall be supported by guide bearings spaced to suit the Valve Manufacturers recommended requirements.
DETAIL A
SCALE 1/2" = 1'-0"
SECTION A-A
SCALE: 1/2" = 1'-0"
SECTION B-B
SCALE 1/2"=1'0"
SECTION C-C
SMALL LOCK NO. I SIMILAR
SCALE: 1/2"=1'-0"
SECTION D-D
LARGE LOCK BASIN END AS SHOWN, SMALL LOCKS SIMILAR EXCEPT AS NOTED
SCALE: 1/2"=1'-0"
NOTES
1. ◆ Indicates Dimensions Dependent On Equipment Selected.
GRAPHIC SCALES
1"=20'
1/2"=1'-0"
SAFETY PAYS
VALUE ENGINEERING PAYS DIVIDENDS
REVISION DATE DESCRIPTION BY
CE MAGUIRE, INC.
WALTHAM, MASSACHUSETTS
ARCHITECTS-ENGINEERS-PLANNERS
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
LOCK/PUMP STATION
DRAINAGE SYSTEM & DETAILS
CHARLES RIVER BASIN
MASSACHUSETTS
SCALE: AS NOTED
SPEC. NO. DACW 33-73-B-
DWG. NO. CHA-2



NOTES:

1. Duct Lines shall enter the manhole with a minimum cover of 36".
2. For existing installations, duct locations shall be determined in the Field.
3. Use concrete chairs and wire ties to space reinforcing rods from forms.
4. Deleted
5. Deleted
6. Telephone and/or Fire Alarm Underground Runs shall Terminate in East Trench at Junction Boxes Under Control Room. From This Point on the NET & T Will Wire as Required
7. Deleted
8. All Construction Expansion Joints Shall Have Expansion and/or Deflection Fittings on all Raceways.
9. Deleted

REVISION	DATE	DESCRIPTION	BY

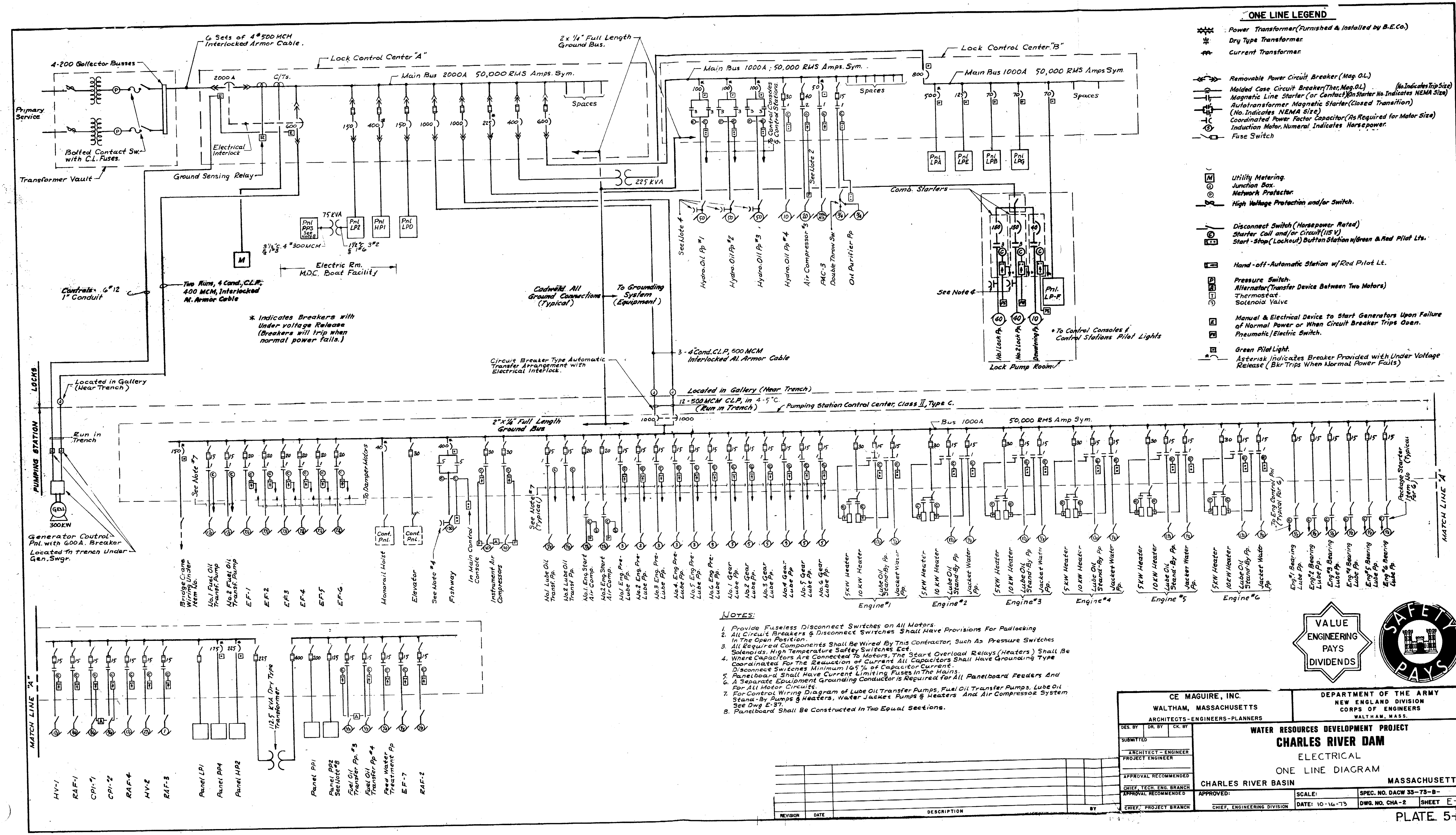
CE MAGUIRE, INC.
WALTHAM, MASSACHUSETTS
ARCHITECTS-ENGINEERS-PLANNERS

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

DES. BY | DR. BY | CK. BY
SUBMITTED
ARCHITECT - ENGINEER
PROJECT ENGINEER
APPROVAL RECOMMENDED
CHIEF, TECH. ENG. BRANCH
APPROVAL RECOMMENDED
CHIEF, PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
ELECTRICAL
PRIMARY SERVICE - PLAN & DETAILS
CHARLES RIVER BASIN — MASSACHUSETTS

APPROVED: CHIEF, ENGINEERING DIVISION | SCALE: | DATE: | SPEC. NO. DACW 33-73-B- | DWG. NO. CHA-2 | SHEET



NOTES:

1. Provide Fuseless Disconnect Switches on All Motors.
2. All Circuit Breakers & Disconnect Switches Shall Have Provisions For Padlocking In The Open Position.
3. All Required Components Shall Be Wired By This Contractor, Such As Pressure Switches Solenoids, High Temperature Saftey Switches Ect.
4. Where Capacitors Are Connected To Motors, The Start Overload Relays (Heaters) Shall Be Coordinated For The Reduction of Current. All Capacitors Shall Have Grounding Type Disconnect Switches Minimum 165% of Capacitor Current.
5. Panelboard Shall Have Current Limiting Fuses In The Mains.
6. A Separate Equipment Grounding Conductor is Required for All Panelboard Feeders And For All Motor Circuits.
7. For Control Wiring Diagram of Lube Oil Transfer Pumps, Fuel Oil Transfer Pumps, Lube Oil Stand-By Pumps & Heaters, Water Jacket Pumps & Heaters And Air Compressor System See Dwg E-37.
8. Panelboard Shall Be Constructed In Two Equal Sections.

VALUE ENGINEERING PAYS DIVIDENDS

SAFETY PAYS

REVISION	DATE	DESCRIPTION	BY

CE MAGUIRE, INC.
WALTHAM, MASSACHUSETTS
ARCHITECTS-ENGINEERS-PLANNERS

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

DES. BY | DR. BY | CK. BY
SUBMITTED
ARCHITECT - ENGINEER
PROJECT ENGINEER
APPROVAL RECOMMENDED
CHIEF, TECH. ENG. BRANCH
APPROVAL RECOMMENDED
CHIEF, PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
ELECTRICAL
ONE LINE DIAGRAM

CHARLES RIVER BASIN — MASSACHUSETTS

APPROVED:
CHIEF, ENGINEERING DIVISION

SCALE: | SPEC. NO. DACW 33-73-B-
DATE: 10-16-73 | DWG. NO. CHA-2 | SHEET E-5

APPENDIX A

STRUCTURAL DESIGN COMPUTATIONS

NED FORM 223
27 Sept 49

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

PAGE

SUBJECT

COMPUTATION

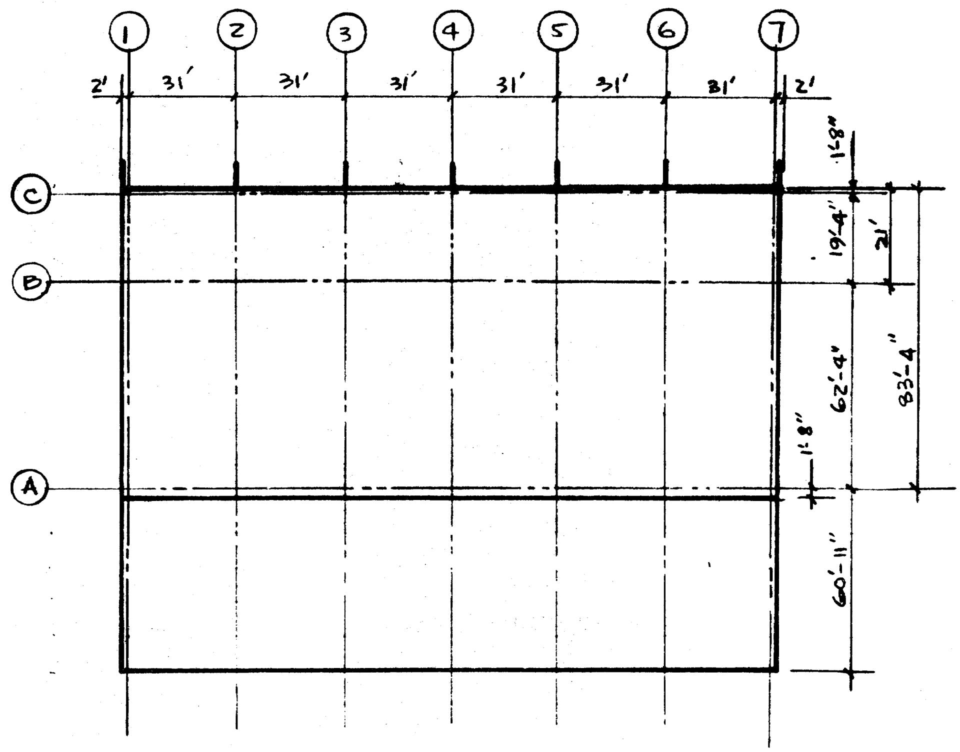
COMPUTED BY CHECKED BY DATE

APPENDIX A

STRUCTURAL DESIGN COMPUTATIONS

PROJECT	CHARLES RIVER BASIN	ACC. NO.	1856130
SUBJECT	PUMPING STATION - STABILITY ANALYSIS	SHEET NO.	1 OF
		DATE	9.15 1972
COMP.	TR	CHECK	CONT. NO.

REACTION FROM SUPERSTRUCTURE



FOR A TYP 31'-0" BAY (BY K.S)

	DEAD LOAD	LIVE LOAD	HORIZONTAL LOAD
LINE (A)	360 K	30 K	44 K
LINE (B)	510 K	150 K	44 K
LINE (C)	160 K	75 K	—

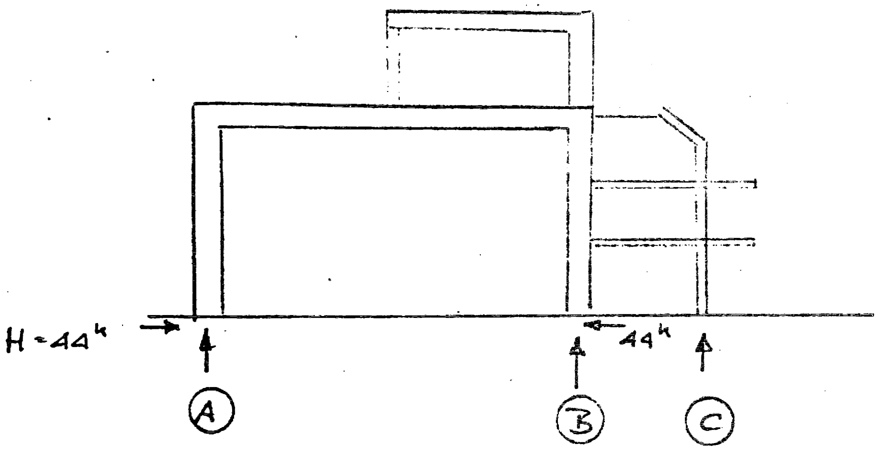
WIND LOAD NOT INCLUDED.

USE 35 #/□' OF WIND PRESSURE FOR ANALYSIS.

total load of building = Load x 6.

A-1

PROJECT Charles River Dam ACC. NO.
SUBJECT Reactions for Steering Cases SHEET NO. 16 OF
DATE 9-11-72 19
COMP. KS CHECK CONT. NO.

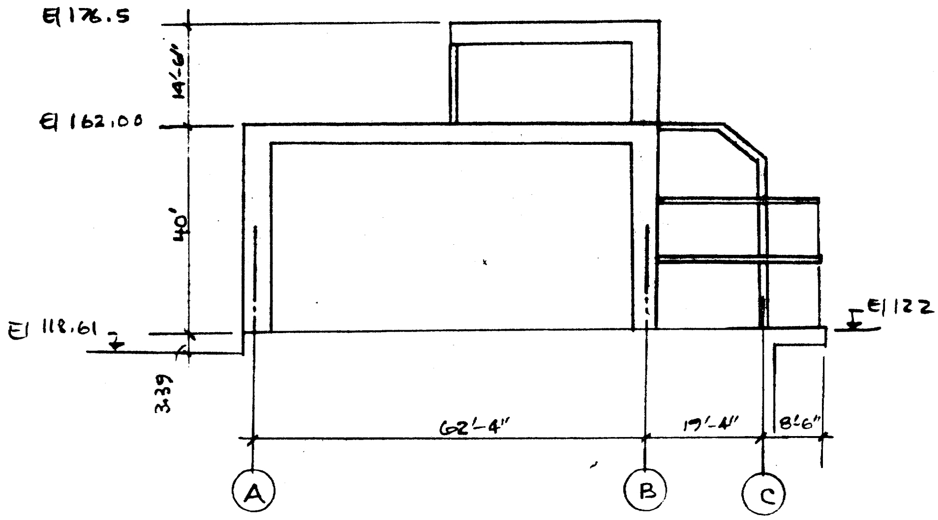


	A	B	C
DL	360k	510k	160k
LL	30k	150k	75k

} FOR TYP. 31'-0" BAY

WIND LOADS NOT INCLUDED

PROJECT	CHARLES RIVER BASIN	ACC. NO.	1858.30
SUBJECT	PUMPING STATION – STABILITY ANALYSIS	SHEET NO.	2 OF
		DATE	9.15 197_
COMP.	T.L. CHECK	CONT. NO.	



total super structure load on building lines:

	DEAD	LIVE	Moment arm
LINE (A)	2,160,000#	180,000#	83.25'
LINE (B)	3,060,000#	900,000#	21.0'
LINE (C)	960,000#	450,000#	1.67'
	Σ = 6,180,000#		X_c = 41' (approximately)

WIND LOAD –

Neglect the sub structure wind resistance since this portion is open – consider only from El 118.61 to top of roof.

Building length 190' –

From El 118.61 to El 162 :

F_{w_1} = 43.39 × 190 × 35 = 247,323 #

Y_c = 37.61 + 21.69 = 59.30'

A-3

From El. 162 to El. 176.50

$$F_{W_2} = 14.5 \times 190 \times 30 = 82,650^{\#}$$

$$Y_c = 37.61 + 43.39 + 7.25 = 88.25'$$

EARTH QUAKE

Assumed the center of gravity for the superstructure is $\frac{54.5}{2} = 27.25$ Above El 122.

take $F_{EQ} = .1 \times 6,180,000^{\#} = 618,000^{\#} \rightarrow$

act horizontally at $Y_c = 27.25 + 41 = 68.25'$ from base

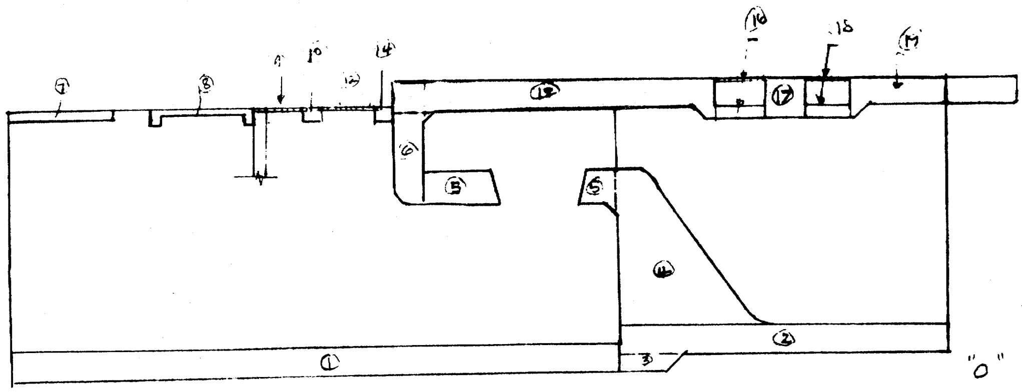
PROJECT CHARLES RIVER BASIN ACC. NO. 1856.30

SUBJECT PUMPING STATION - STABILITY ANALYSIS SHEET NO. 4 OF

DATE 9.13 1978

COMP. T.L CHECK CONT. NO.

SUB STRUCTURES —

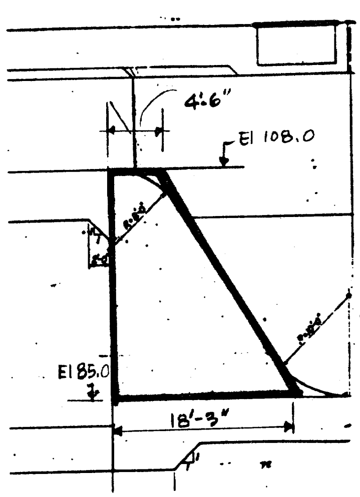


BASE SLAB —

CENTER OF GRAVITY FROM RHS "O"

1	4.5 x 93.25 x 190 x 150	11990000 #	$\frac{93.25}{2}+51$	= 97.63'
2	4.0 x 51.00 x 190 x 150	5820000 #	$\frac{51}{2}$	= 25.5'
3	3.0 x 7.5 x 190 x 150	642000 #	$\frac{7.5}{2}+51$	= 54.8'

SPILLWAY —



4

top El 108.00
Bott El 85.00
23.00

Approximately of the cross section as trapezoidal — $b = 4.5'$, $B = 18.25'$, $h = 23'$

$$A = (B+b)\frac{h}{2} = (18.25+4.5)\frac{23}{2}$$

$$A = 262\ ft^2$$

Weight $W = 148.5 \times 262 \times 150 = 5,520,000$ #

$X_c = 51 - 5 = 46'$

A-5

CHARLES A. MAGUIRE & ASSOCIATES INC. ENGINEERS

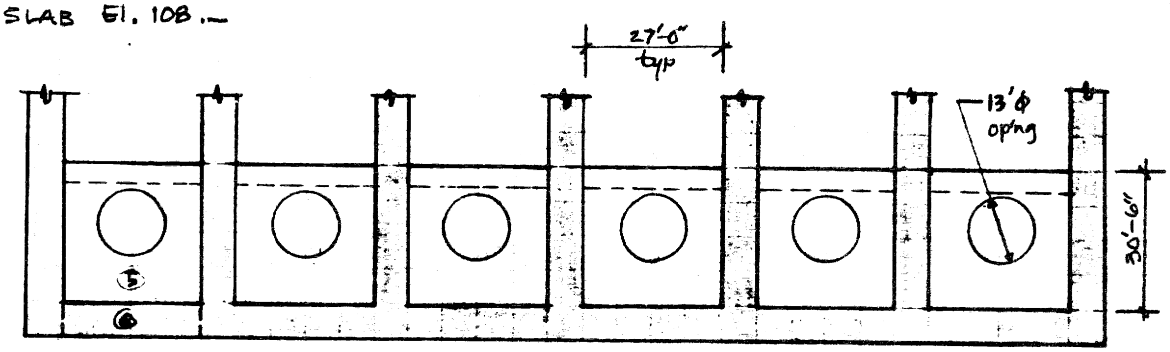
PROJECT CHARLES RIVER BASIN ACC. NO. 1856.30

SUBJECT PUMPING STATION - STABILITY STUDY SHEET NO. 5 OF

DATE 9.13 1972

COMP. CHECK CONT. NO.

SLAB El. 108.—



ONE SLAB —

⑤ $A = 30.5 \times 27 = 823$

less $3.14 \times \overline{6.5}^2 = 133$

$690\ ft^2 \times 5 = 3450\ ft^3$

$W = 3450\ ft^3 \times 150 = 517{,}500\ \#$

total weight $= 6 \times 517{,}500\# = 3{,}110{,}000\ \#$

$X_c = 51 + \frac{30.5}{2} = 66.25'$

PARTIAL WALL.—

⑥ $W = 3.5 \times 10 \times 27 \times 150 = 14{,}180\#$

total Weight $= 6 \times 14{,}180 = 85{,}080\ \#$

$X_c = \frac{3.5}{2} + 30.5 + 51 = 83.25\ ft.$

⑦ $W = 1.5 \times 13.75 \times 190 \times 150 = 587{,}800\#$

$X_c = 137.38\ ft$

⑧ Beams.— 30" x 30"

Slab 14"

$W\ \text{Beams} = 2\{2.5 \times 2.5 \times 190 \times 150\} = 357{,}000\ \#$

A-6

W_{slab} = 1.17 × 9.5 × 190 × 150 = 316,000 #

X_c = 117 ft W_t = 316,000 + 357,000 = 673,000 #

(9) CONCRETE FILLED GRATING. — 4" Say Avg of 200 #/FT³

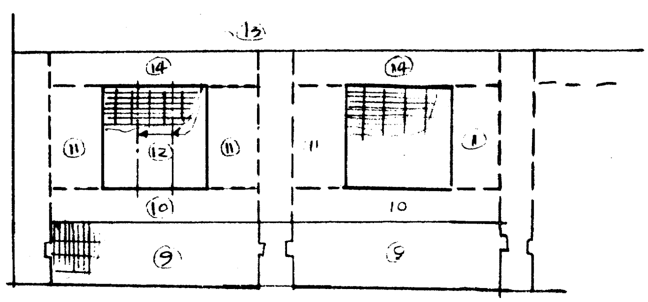
W = 4/12 × 4.25 × 190 × 200 = 5390 #

X_c = 108 ±

(10) BEAM — 3'-3" × 2'-6"

W = 3.25 × 2.5 × 190 × 150 = 232,000 #

X_c = 104.0 '



(11) 18" slab -

Each W = 1.5 × 6.5 × 14 × 150 = 20,500 #

total W = 12 × 20,500 = 246,000 #

X_c = 95.25'

(12) 14'×14" conc filled grating (assumed 70#/FT²)

W_{Each} = 14×14 × 70 = 13,720 #

PROJECT CHARLES RIVER BASIN ACC. NO. 1856.30
SUBJECT PUMPING STATION – STABILITY ANALYSIS SHEET NO. 7 OF
DATE 9-13 1972
COMP. TW CHECK CONT. NO.

total $W = 6 \times 13,720 = 82,400\#$

$X_c = 95.25$

(13) STEEL BEAMS – GRATING SUPPORT : W12x36

Each $W = 14 \times 36 = 196\#$

total $W = 12 \times 196 = 2350\#$

$X_c = 95.25'$

(14) BEAM 3'-3" x 2'-6"

$W = 3.25 \times 2.5 \times 190 \times 150 = 232,000\#$

$X_c = 86.6'$

SLAB El 122.00

(15) 4' slab

$W = 4 \times 49 \times 190 \times 150 = 5,590,000\#$

$X_c = \frac{49}{2} + 36 = 60.5'$

(16) 4" conc filled grating : 70 #/FT²
12" slab 150 #/FT²
220 #/FT²

$W = 8 \times 190 \times 220 = 334,400\#$

$X_c = 32'$

A 8

PROJECT: CHARLES RIVER BASIN | ACC. NO. 1856.30
SUBJECT: PUMPING STATION - STABILITY ANALYSIS | SHEET NO. 8 OF
DATE 9.13 1972
COMP. | CHECK | CONT. NO.

(17) 4'- slab -

$W = 4 \times 8.5 \times 190 \times 150 = 970{,}000^{\#}$

$X_c = 23.75'$

(18) 4" conc. filled grating = 70#/FT²

12 slab = 150#/FT²

220#/FT²

$W = 8 \times 190 \times 220 = 334{,}400^{\#}$

$X_c = 15.5'$

(19) 4' slab -

$W = 4 \times 11.5 \times 190 \times 150 = 1311{,}000^{\#}$

$X_c = 5.75'$

A-9

PROJECT CHARLES RIVER BASIN ACC. NO. 1856.30
SUBJECT PUMPING STATION – STABILITY ANALYSIS SHEET NO. 9 OF
DATE 9.13 1972
COMP. [initials] CHECK CONT. NO.

WALLS –

a) FORE BAY WALLS (4'-0" thick)

top of wall El 118.61
Bott of wall El 82.50
H = 36.11

W(each) = 4 x 36.11 x 59.25 x 150 = 1,284,000 #

total W = 7 x 1,284,000 = 8,986,000 #

X_c = 114.63'

b) AFTERBAY WALLS

Left portion –
top of wall 118.00
bottom of wall 82.50
35.50'

W(each) = 4' x 35.5 x 34 x 150 = 725,000 #

W total = 7 x 725,000 = 5,070,000 #

X_c = 68'

Right portion –

a) 4'-0" thick walls –

top of wall El 118.00
bot of wall El 85.00
33.00'

W(each) = 4 x 33 x 51 x 150 = 1,010,000 #

total W = 7 x 1,010,000 = 7,070,000 #

PROJECT CHARLES RIVER BASIN ACC. NO. 1856.30
SUBJECT PUMPING STATION – STABILITY ANALYSIS SHEET NO. 10 OF
DATE 9.13 1972
COMP. [illegible] CHECK CONT. NO.

$X_c = \frac{51}{2} = 25.5'$

b) 2'-3" thick walls – $L = 48.5'$
$H = 33.00'$

$W_{(each)} = 2.25 \times 33 \times 48.5 \times 150 = 540,200$ #

total $W = 6 \times 540,200 = 3,241,200$ #

$X_c = \frac{48.5}{2} = 24.25$ ft

FIXED EQUIPMENT LOADS.–

ENGINE : $W = 35,700$ # $X_c = 85 - 40 = 45'$

GEAR : $W = 20,000$ # $X_c = 85 - 21.5 = 63.5'$

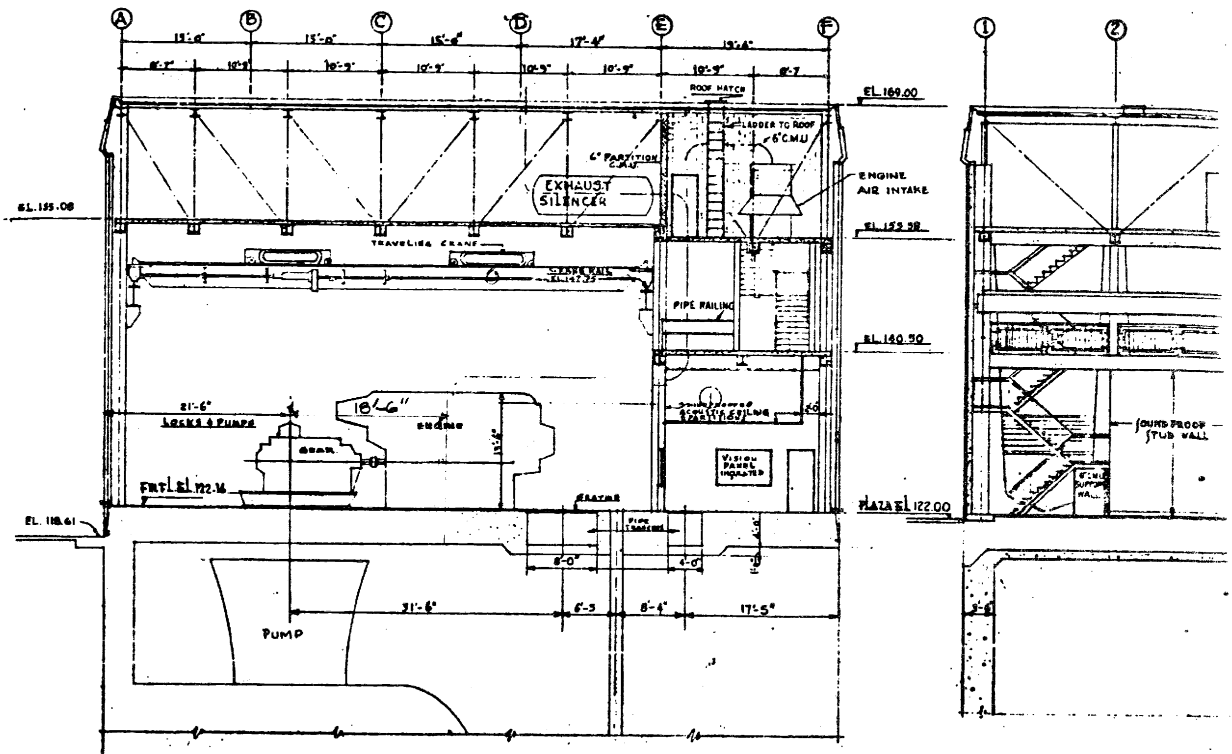
THRUST = 125,000 #

PUMP : $W = 122,000$ # + (550,000 # water max) $X_c = 63.5'$

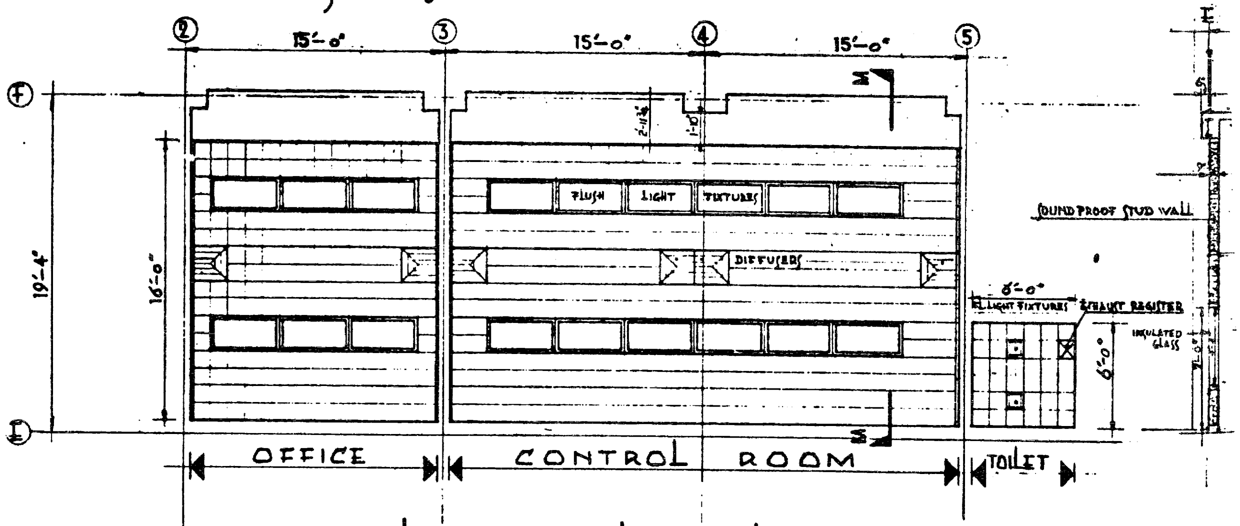
Total EQUIPMENT LOAD (without thrust)

$W = 6(35,700 + 20,000 + 122,000) = 1,066,200$ #

A-11



CROSS SECTION
SCALE 1/8"=1'-0"



REFLECTED CEILING PLAN
SCALE 1/4"=1'-0"

COMPLETE D.W.

↓

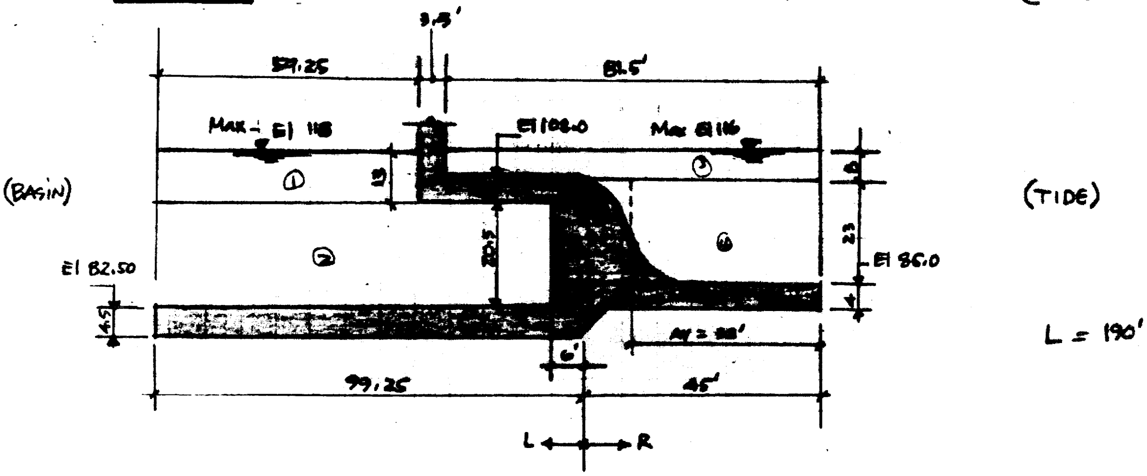
I ENGINE TOT. 35.7 K

II GEAR TOT 20.0 K + 125 K THRUST

III PUMP TOT 122.0 K + (550 K WATER MAN)

PROJECT	CHARLES RIVER BASIN		ACC. NO. 1853.30
SUBJECT	PUMPING STATION – STABILITY ANALYSIS		SHEET NO. 12 OF ___
			DATE 9.14 1976
COMP.	TL	CHECK ___	CONT. NO. ___

COND. ONE max condition – water to El. 116 both side (Extreme case)



LEFT PORTION –

$H_w = 116 - 78 = 38'$

$p_u = 64 \times 38 = 2,432$ #/ft

$A_L = 99.25 \times 190 = 18,857.50\ ft^2$

$P_U = 18,857.50 \times 2,432 = 45,861,440$ # ↑

$X_c = 94.62$

water inside:

$W_1 = 13 \times 59.25 \times 190 \times 64 = 9,366,240$ # ↓

$X_c = 114.62$

$W_2 = 20.5 \times 93.25 \times 190 \times 64 = 23,245,421$ # ↓

$X_c = 97.62'$

RIGHT PORTION –– $H_w = 116 - 81 = 35'$

$p_u = 64 \times 35 = 2,240$ #/ft

$P_U = 45 \times 190 \times 2,240 = 19,152,000$ # ↑

$X_c = 22.5$

PROJECT CHARLES RIVER BASIN ACC. NO. 1856.30
SUBJECT PUMPING STATION - STABILITY ANALYSIS SHEET NO. 13 OF
DATE 9.14 1972
COMP. [illegible] CHECK CONT. NO.

water inside, -

$W_3 = 81.5 \times 8 \times 190 \times 64 = 7{,}928{,}320$ # ↓

$X_c = 40.75'$

$W_4 = 38 \times 23 \times 190 \times 64 = 10{,}627{,}840$ # ↓

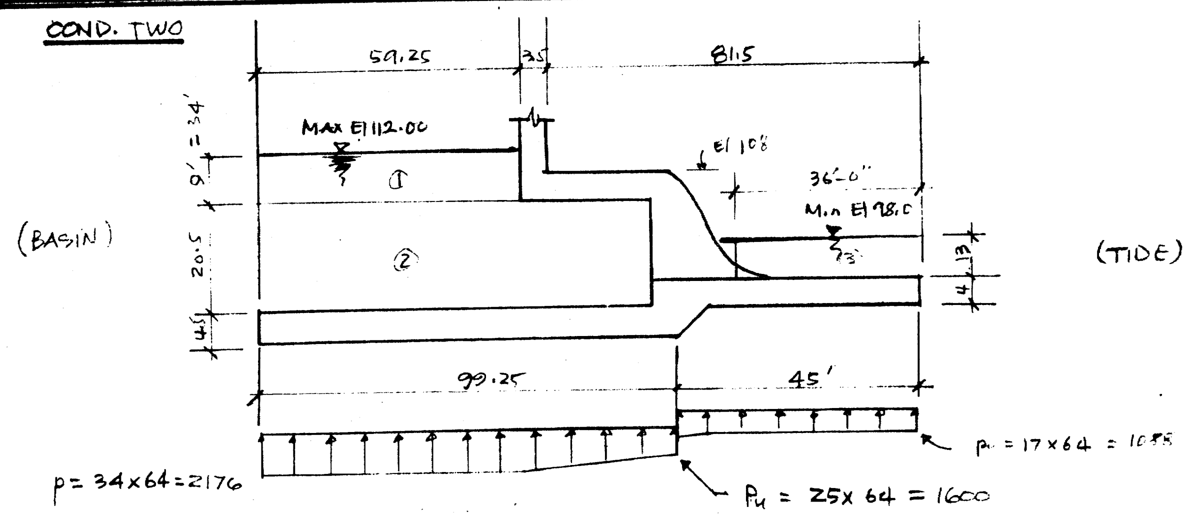
$X_c = 19'$

total downward force = 51,167,821 # ↓

total uplift force = 65,013,440 # ↑

ΣP_u (net) = 13,845,619 # ↑

COND. TWO



take max value for analysis -

LEFT PORTION $P_U = 2176 \times 99.25 \times 190 = 41{,}033{,}920$ # ↑

$X_c = 94.62'$

PROJECT CHARLES RIVER BASIN ACC. NO. 1856.30
SUBJECT PUMPING STATION - STABILITY ANALYSIS SHEET NO. 14 OF
DATE 9.14 1972
COMP. Td CHECK CONT. NO.

downward force due to water inside -

$W_1 = 9 \times 59.25 \times 190 \times 64 = 6,484,320^{\#}$

$X_c = 114.62'$

$W_2 = 20.5 \times 93.25 \times 190 \times 64 = 23,245,421^{\#}$

$X_c = 97.62'$

Horizontal $p_H = 2176 \times \frac{34}{2} = 36,992 \ \#/'$

$P_H = 36,992 \times 190 = 7,028,480^{\#} \rightarrow$

$Y_c = \frac{34}{3} = 11.33'$

RIGHT PORTION.-

upward force: $H_w = 98 - 81 = 17'$

$p_u = 64 \times 17 = 1088 \ \#/\square'$

$P_u = 45 \times 190 \times 64 \times 17 = 9,302,400^{\#}$

$X_c = 22.5'$

downward force $W_3 = 36 \times 8 \times 190 \times 64 = 3,502,080^{\#}$

$X_c = 18'$

Horizontal $p_H = 1088 \times \frac{17}{2} = 9,248^{\#}$

$P_H = 9248 \times 190 = 1,757,120^{\#} \leftarrow$

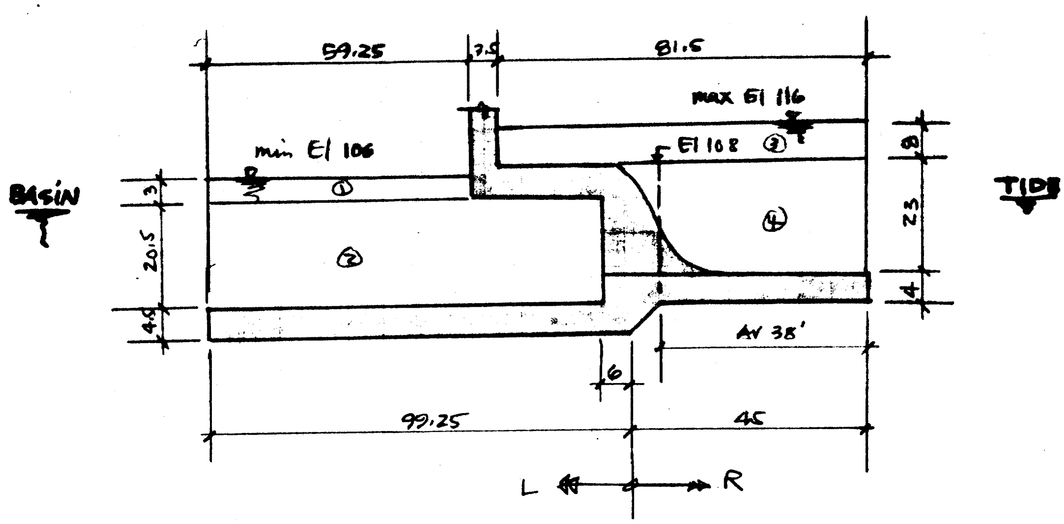
$Y_c = \frac{17}{3} = 5.66'$

$\Delta H = 7,028,480^{\#} - 1,757,120^{\#} = 5,271,360^{\#} \rightarrow$

$\Sigma P_U = 41,033,920^{\#} + 9,302,400^{\#} = 50,336,320^{\#} \uparrow$

$\Sigma P_D = 6,484,320 + 23,245,421 + 3,502,080 = 33,231,821^{\#} \downarrow$

COND. THREE



LEFT PORTION -

$H_w = 28'$

$p_u = 28 \times 64 = 1792 \ \#/□$

↑ $P_u = 1792 \times 99.25 \times 190 = 33,792,640^{\#}$ ↑

$X_c = 94.62'$

Downward $W_1 = 3 \times 59.25 \times 190 \times 64 = 2,161,440.^{\#}$ ↓

$X_c = 114.62'$

$W_2 = 20.5 \times 93.25 \times 190 \times 64 = 23,245,421^{\#}$ ↓

$X_c = 97.62$

Horizontal $p_H = 1792 \times \frac{28}{2} = 25,100 \ \#/1$

$P_H = 25,100 \times 190 = 4,779,000^{\#}$ →

$Y_c = \frac{28}{3} = 9.33$

RIGHT PORTION : $p_u = 35 \times 64 = 2,240 \ \#/□$

PROJECT	CHARLES RIVER BASIN	ACC. NO.	1856.30
SUBJECT	PUMPING STATION - STABILITY ANALYSIS	SHEET NO.	16 OF
		DATE	9.14 1972
COMP.	TL	CHECK	CONT. NO.

$P_u = 2240 \times 190 \times 45 = 19,152,000^{\#}$

$X_c = 22.5'$

downward $W_3 = 8 \times 81.5 \times 190 \times 64 = 7,928,320^{\#}$

$X_c = \frac{81.5}{2} = 40.75$

$W_4 = 23 \times 38 \times 190 \times 64 = 10,627,840^{\#}$

$X_c = \frac{38}{2} = 19$

Horizontal $p_H = 35 \times \frac{35}{2} \times 64 = 39,200^{\#}/_1$ ←

$P_H = 39,200 \times 190 = 7,448,000^{\#}$ ←

$Y_c = \frac{35}{3} = 11.66'$

$\Delta H = 7,448,000^{\#} - 4,470,000 = 2,978,000^{\#}$ ←

$\Sigma P_U = 33,792,640 + 19,152,000 = 52,944,640^{\#}$ ↑

$\Sigma P_D = 2161,440 + 23,245,421 + 7,928,320 + 10,627,840 = 43,963,021.0$ ↓

PROJECT CHARLES RIVER BASIN ACC. NO. 1962-10

SUBJECT PUMPING STATION - STABILITY ANALYSIS SHEET NO. 17 OF

DATE 2 15 1972

COMP. [illegible] CHECK CONT. NO.

TOTAL DEAD LOAD (NO EQUIPMENTS)

Superstructure }
Sub structure } W_t = 68,614,240.#

Water load W = 33,231,821.# case II (min)

ΣW_D = 101,846,061 # ↓

Max P_u = 65,013,440 # ↑ CASE I (max)

$$S.F = \frac{101,846,061}{65,013,440} = 1.565$$

Total Dead load + Equipments -

$$\Sigma W_D = 101,846,061 + 1,066,200 = 102,912,261 \ \#$$

Safty factor against floating :

$$SF = \frac{102,912,261}{65,013,440} = 1.58$$

EARTH QUAKE LOAD ON SUB STRUCTURES:-

total W_D = 62,109,395# + 1,066,200 = 63,175,595# ↓

F_{EQ} = .1 x 63,175,595 = 6,317,559# ⇆

y_c = 21' (approximately !)

EARTH QUAKE LOAD ON INPOUND WATER BODY :-

Westergaard's formula

$$P_e = \frac{2}{3} C_e \alpha y \sqrt{hy}$$

$$M_e = \frac{4}{15} C_e \alpha y^2 \sqrt{hy}$$

where : P_e = earth quake force cause by water pressure at depth y

M_e = moment cause by P_e at depth y

α = 0.1

g = 32 ft/sec²

h = total height of dam

$$C_e = \frac{51}{\sqrt{1 - 0.72\left(\frac{h}{1000\, t_e}\right)^2}}$$

for acceleration = .1g t_e = 1 sec

Say top of lock 118.00

bottom base 78.00

h = 40.00

$$C_e = \frac{51}{\sqrt{1 - .72\left(\frac{40}{1000}\right)^2}} \simeq 51.00$$

PROJECT: CHARLES RIVER BASIN — ACC. NO. 1856.30
SUBJECT: PUMP STATION - STABILITY ANALYSIS — SHEET NO. 19 OF
DATE 9.15 1972
COMP. — CHECK — CONT. NO.

FOR COND ONE.— max Water El 116.00

$y = 116 - 78 = 38$

total Earth quake load $P_e = \frac{2}{3} C_e \alpha y \sqrt{hy}$

$P_e = 190 \times \frac{2}{3} \times 51 \times .1 \times 38 \sqrt{40 \times 38} = 948\,909$ #

$24,331 \sqrt{1520}$ 39

$M_e = 190 \times \frac{4}{15} \times 51 \times .1 \times 38^2 \sqrt{40 \times 38} = 14,231,022$ #'

$364,898 \sqrt{1520}$ 39

FOR COND TWO.— max water El 112.00 $y = 34$

$P_e = 190 \times \frac{2}{3} \times 51 \times .1 \times 34 \sqrt{40 \times 34} = 799,032$ #

$21.654 \sqrt{1360}$ 36.9

$M_e = 190 \times \frac{4}{15} \times 51 \times .1 \times 34^2 \sqrt{40 \times 34} = 10,779,265$ #'

$292,124 \sqrt{1360}$ 36.9

FOR CON. THREE.— max water El 116

R.H.S Bott Base El 81 ⟶ $y = 35$

$P_e = 190 \times \frac{2}{3} \times 51 \times .1 \times 35 \sqrt{40 \times 35} = 837,161$ #

$22,384 \sqrt{1400}$ 37.4

$M_e = 190 \times \frac{4}{15} \times 51 \times .1 \times 35^2 \sqrt{40 \times 35} = 11,577,431$ #'

$= 309,557 \sqrt{1400}$

L.H.S max water El 116

bott base 78

28 — $y = 28'$

PROJECT	CHARLES RIVER BASIN		ACC. NO. 1856.30
SUBJECT	PUMP STATION - STABILITY ANALYSIS		SHEET NO. 19a OF
			DATE 9.19 1972
COMP.	TL	CHECK	CONT. NO.

$$Pe = 190 \times \frac{2}{3} \times 51 \times .1 \times 28 \sqrt{40 \times 28} = 605{,}000\ \#$$

18,0[illegible] × 33.4

$$Me = 190 \times \frac{4}{15} \times 51 \times .1 \times 28^2 \sqrt{40 \times 28} = 6{,}740{,}000\ \#\cdot\text{ft}$$

202.585 $\sqrt{1102}$

[illegible]

PROJECT CHARLES RIVER BASIN — ACC. NO. 1863.30
SUBJECT PUMP STATION – STABILITY ANALYSIS — SHEET NO. 20 OF
DATE 9.15 1972
COMP. TL — CHECK — CONT. NO.

MOMENTS AT POINT "O" DUE TO DEAD LOADS

REF. SH. NO.	DESCRIPTION	LOADS – #	Arm (')	Mom. (K-ft)
2	Super structure	6180,000.	41.00	253,380.
4	Sub structure – Base slab	11,990,000.	97.63	1,170,583.
4	Base slab	5,820,000.	25.50	148,410.
4	Base slab	642,000.	54.80	35,181.
4	Spillway	5,820,000.	46.00	267,720.
5	PUMP'S SLAB	3,110,000.	66.25	206,037.
5	PUMP'S WALL	85,080.	83.25	7,076.
4,5	BRIDGE PIER SLAB	587,800.	137.38	80,751.
4,5,6	BEAMS + SLAB El 118.61	673,000.	117.00	89,271.
6	component (9)	5,390.	108.00	582.
6	component (10)	232,000	104.00	24,128.
6	component (11)	246,000.	95.25	23,431.
6,7	component (12)	82,400.	95.25	7,848.
7	component (13)	2,350.	95.25	223.
7	component (14)	232,000.	86.6	20,091.
7	4'- slab – machine room	5,590,000.	60.5	338,195.
7	component (16)	334,000.	32.0	10,688.
8	4'- slab -	970,000.	23.75	23,037.
8	component (18)	334,000.	15.5	5,177.
8	component (19)	1,311,000.	5.75	7,538.
9	Forebay walls	8,986,000.	114.63	1,029,795.
9	Wall left of spill way	5,070,000.	68.0	344,760.
9	4'wall right of spill way	7,070,000.	25.5	180,285.
		Σ = 65,373,020.		Σ = 4,274,187.

PROJECT CHARLES RIVER BASIN ACC. NO. 1856.30
SUBJECT PUMPING STATION - STABILITY ANALYSIS SHEET NO. 21 OF
DATE 9.15 1972
COMP. TL CHECK CONT. NO.

REF. SH. NO	DESCRIPTION	LOADS - lbs	Arm (FT)	Mon. (K-Ft)
	total on sheet 20	65,373,020.		4,274,187.0
10	2'-3" wall -	3,241,220.	24.25'	78,594.0
		Σ= 68,614,240.#		Σ= 4,352,781.
12	COND. ONE - UPLIFT - LEFT	45,861,440.# ↑	94.62	4,339,409. ↻
12	W_1 (L)	9,366,240. ↓	114.62	1,073,558. ↺
12	W_2 (L)	23,245,241. ↓	97.62	2,269,200. ↺
12	UPLIFT - RIGHT	19,152,000. ↑	22.5	430,920. ↻
13	W_3 (R)	7,928,320.	40.75	323,079 ↺
13	W_4 (R)	10,627,840.	19.0	201,929 ↺
13	COND. TWO - UPLIFT - LEFT	41,033,920.# ↑	94.62	3,882,627. ↻
14	W_1 (L)	6,484,320.#	114.62	743,196. ↺
14	W_2 (L)	23,245,241.#	97.62	2,269,176. ↺
14	F (Horizontal)	7,025,480.# →	11.33	79,627. ↻
14	UPLIFT - RIGHT	9,302,400.#	22.5	209,317. ↻
14	W_3 (R)	3,502,080 #	18.	63,036. ↺
14	F (Horizontal)	1,757,120.# ←	5.66	9,944. ↺
15	COND - THREE * UPLIFT - LEFT	33,792,640.#	94.62	3,197,399. ↻
15	W_1 (L)	2,161,440.#	114.62	247,650. ↺
15	W_2 (L)	23,245,421.#	97.62	2,269,176. ↺
15	F (horizontal)	4,770,000.# →	9.33	44,504. ↻
16	UPLIFT - RIGHT	19,152,000.#	22.5	430,920. ↻
16	W_3 (R)	7,928,320.#	40.75	323,066. ↺
16	W_4	10,627,840.#	19.0	201,913. ↺
16	F (horizontal)	7,448,000.# ←	11.66	86,843. ↺

PROJECT	CHARLES RIVER BASIN	ACC. NO.	1856.30
SUBJECT	PUMPING STATION - STABILITY ANALYSIS -	SHEET NO.	2? OF
		DATE	9.18 1972
COMP.	T.L. CHECK	CONT. NO.	

SUMMARY OF LOADINGS -

COND. ONE :

P_{up} = 45,861,440 + 19,152,000 = 65,013,440# = 65,013,440.#

P_{down} = 9,366,240 + 23,245,241 + 7,928,320 + 10,627,840 = 51,167,641.#

M (↷) = 4,339,409. + 430,920 = 4,770,329. K-ft

M (↶) = 1,073,558 + 2,269,200 + 323,079 + 201,929 = 3,867,766. K-ft

Me = (pg 19) = 14,231. K-ft

Heq = = 948,909#

COND. TWO :

P_{up} = 41,033,920 + 9,302,400 = 50,336,320#

P_{down} = 6,484,320 + 23,245,241 , 3,502,080 = 33,231,641.#

H (→) = 7,028,480 − 1,757,120 = 5,271,360#

Heq = (see pg 19) = 799,032#

M_{up} (↷) = 3,882,627 + 79,627 + 209,317 = 4,171,571 K-ft

M_{w} (↶) = 743,176 + 2,269,176 + 63,036 + 9,944 = 3,085,352. K-ft

Me = (see pg 19) = 10,779. K-ft

COND. THREE :

P_{up} = 33,792,640 + 19,152,000 = 52,944,640 #

P_{down} = 2,161,440 + 23,245,421 + 7,928,320 + 10,627,840 = 43,963,021 #

H (←) = 7,448,000 − 4,770,000 = 2,678,000.

Heq = (see pg 19) = 837,161 #

M_{uplift} (↷) = 3,197,399 + 44,504 + 430,920 = 3,672,823 K-ft

M_{water} (↶) = 247,650 + 2,269,176 + 323,066 + 201,913 + 86,843 = 3,128,648

Me = (see pg 19) = 11,577 K-ft

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WIND - SUPER STRUCTURE : (Ref Pg 23)

$F_{W_1} = 247,323^{\#}$

$F_{W_2} = 82,650^{\#}$

$\overset{\curvearrowleft}{M_W} = 247.323 \times 59.3 + 82.65 \times 88.25 = 21,959$ K-ft

14,666 7,293

EARTH QUAKE – SUPER STRUCTURE :

$F_{EQ} = 618,000^{\#}$

$\overset{\curvearrowright}{M_{EQ}} = 618.00 \times 68.25 = 42,178$ K-ft

EARTH QUAKE – SUB STRUCTURE : (REF. pg 18)

$F_{EQ} = 6,317,559^{\#} \rightleftarrows$

$\overset{\curvearrowleft}{M} = 6,317.559 \times 23 = 145,304$ K-ft

EQUIPMENT LOAD – (Pg 10)

ENGINE	=	35,700 × 6	=	214,200 #
GEAR	=	20,000 × 6	=	120,000 #
PUMP	=	122,000 × 6	=	732,000 #
THRUST	=	125000 × 6	=	750,000 # ↓

$\overset{\curvearrowleft}{M_{EQ}} = 214 \times 45 + (120 + 732)\ 63.5 = 63,732$ K ft

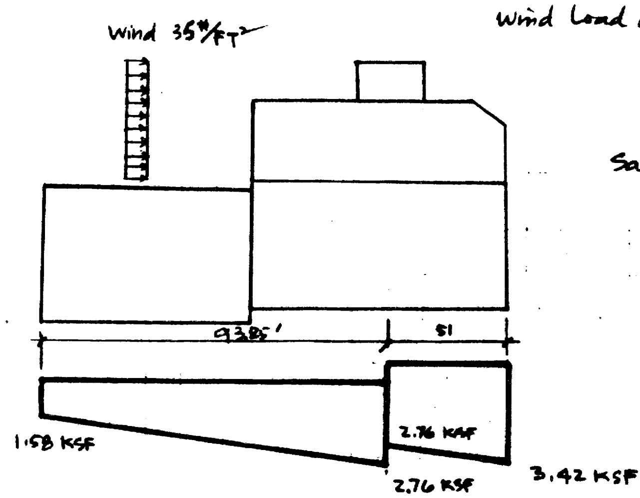
$M_{THRUST} = 750 \times 63.5 = 47,625$ Kft.

PROJECT CHARLES RIVER BASIN — ACC. NO. 1852x38
SUBJECT PUMPING STATION - STABILITY ANALYSIS — SHEET NO. 44 OF
DATE 9.18 1972
COMP. T.L. — CHECK — CONT. NO.

CASE I - construction condition - coffer dam in place

considering only DL of structure

wind load on basin side



Safety against sliding :

$$F.S = \frac{.5 \times 68,614}{329.97} = 105$$

M_{wind} = 21,959 K-ft (pg 23) — P_{wind} = 82.65 + 247.32 = 329.97 K

M_{DL} = 4,352,781 K-ft (pg 21)

ΣM = 4,330,822 K-ft at point "O"

ΣP = 68,614.24 Kips. (pg 21)

$$\text{Resultant arm} = \frac{4,330,822}{68,614.24} = 63.11$$

$$e = \frac{144.25}{2} - 63.11 = 9.01'$$

Soil pressure

$$q = \frac{V}{BL}\left(1 \pm \frac{6e}{L}\right)$$

$$= \frac{68,614.24}{144.25 \times 190}\left(1 \pm \frac{6 \times 9.01}{144.25}\right) = 2.50 \begin{pmatrix} 1.37 \\ .63 \end{pmatrix} = \begin{matrix} 3.42 \text{ KSF} \\ 1.58 \text{ KSF} \end{matrix}$$

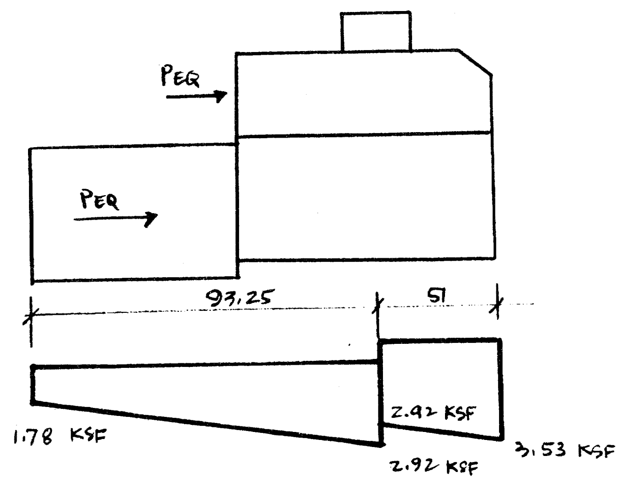
1.84

$$\frac{1.84}{144.25} \times 51 = .66$$

pressure in between

$$q = 3.42 - .66 = 2.76 \text{ KSF}$$

Case Ia.- construction condition - No wind, considering Earth quake



↶ M_{DL} = 4,352,781 K-ft = 4,352,781 k-ft (Pg 21)

↷ M_{EQ} = 42,178 + 145,304 = 187,482 K-ft (pg 23)

ΣM = 4,265,299 K-ft ↶

ΣP = 68,614 Kip (Pg 21)

$$\text{Resultant arm} = \frac{4,265,299}{68,614} = 62.16$$

$$\text{eccentricity } e = \frac{144.25}{2} - 62.16 = 9.96'$$

$$\text{Soil pressure } q = \frac{68,614}{144.25 \times 190}\left(1 \pm \frac{6 \times 9.96}{144.25}\right)$$

$$= 2.503\begin{pmatrix}1.41\\ .59\end{pmatrix} = \begin{matrix}3.53 \text{ KSF} \leftarrow \\ 1.78 \text{ KSF} \leftarrow\end{matrix}$$

$$\frac{1.75}{144.25} \times 51 = .61$$

pressure at 51' from "0"

$$q = 3.53 - .71 = 2.92 \text{ KSF} \leftarrow$$

Safety against sliding

$$SF = \frac{.5 \times 68,614}{6,317 + 618} = \frac{34,307}{6,935} = 4.95 \gg 1 \quad ok$$

PROJECT CHARLES RIVER BASIN — ACC. NO. 1856.30

SUBJECT PUMPING STATION - STABILITY ANALYSIS — SHEET NO. 26 OF

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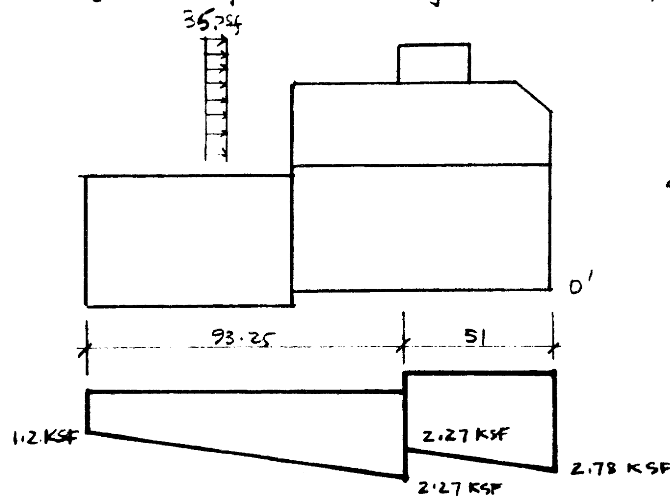
COMP. T.L. — CHECK — CONT. NO.

CASE II - coffer damp burst during construction, wind load on basin side boyancy considered



$P_{wind} = 82.65 + 247.32 = 329.97k$

Safety factor against sliding

$SF = \frac{.5 \times 54,768}{329.97}$

$= 83.4$

M_{DL}	= 4352,781 k-ft	(Pg 21)	P_{DL}	= 68,614.24 K	↓
M_{wind}	= 21,959 k-ft	(Pg 23)			
M_{uplift}	= 4,770,329 k-ft	(Pg 22)	P_{uplift}	= 65,013.44 K	↑
M_W	= 3,867,766 k-ft	(Pg 22)	P_{water}	= 51,167.64	↓
ΣM	= 3,428,259 k-ft		ΣP ↓	= 54,766.44	↓

Resultant arm $= \frac{3428,259}{54,768} = 62.58$ ft

$e = \frac{144.25}{2} - 62.58 = 9.54$ ft

Soil pressures = $q = \frac{54,768}{144.25 \times 190} \left(1 \pm \frac{6 \times 9.54}{144.25}\right)$

$= 1.99 \left(1 \pm .396\right) = 1.99 \times 1.396 = 2.78$ KSF

$= 1.99 \times .604 = 1.20$ KSF

$\Delta\Sigma = \frac{1.58}{144.25} \times 51$

$\simeq .51$

pressure at 51 from "O"

$q = 2.78 - .51 = 2.27$ KSF

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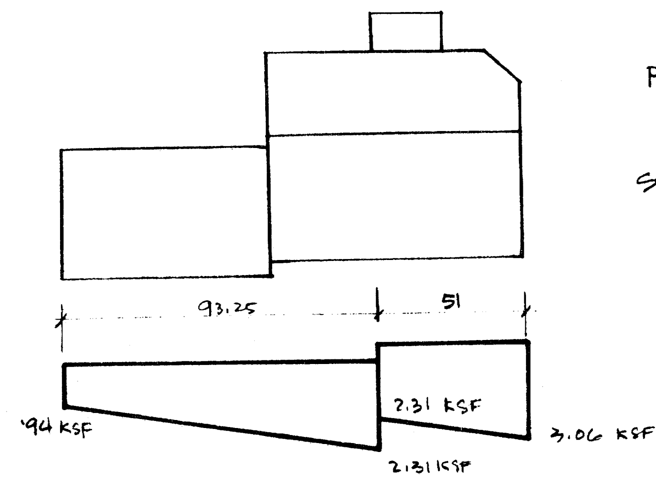
		DATE	9.18 1972
COMP.	TL	CHECK	CONT. NO.

CASE II a - same as II except No wind - Add Earthquake + hydrodynamic load -



P_{EQ} = 6317 + 618 = 6935 K →

SF = $\frac{.5 \times 54,765.44}{6935}$

= 3.95

$\overset{\curvearrowleft}{M}_{DL}$ = 4,352,751 K-ft (Pg 21)

$\overset{\curvearrowright}{M}_{UL}$ = 4,770,329 K-ft (Pg 22)

$\overset{\curvearrowleft}{M}_{WT}$ = 3,867,766 K-ft (Pg 22)

$M_{DYN.H_2O}$ = 14,231 K-ft (Pg 24)

M_{EQ} = 187,482 K-ft (Pg 25)

ΣM_i = 3,248,505 K-ft

P_{DL} = 68,614.24 K

P_{UPLIFT} = -65,013.44 K

P_{WT} = 51,167.64 K

ΣP = 54,768.44

Resultant arm = $\frac{3248,505}{54,765.44}$ = 59.313

$e = \frac{144.25}{2} - 59.31 = 12.81'$

$q = \frac{54,768.44}{144.25 \times 190} \left(1 \pm \frac{6 \times 12.81}{144.25}\right) = 1.998 \binom{1.53}{0.47}$

$q_1 = 1.998 \times 1.53 = 3.06$ KSF

$q_2 = 1.998 \times .47 = .94$ KSF

$q_3 = 3.06 - .75 = 2.31$ KSF

$\Delta q = \frac{2.121}{144.25} \times 51 = .75$

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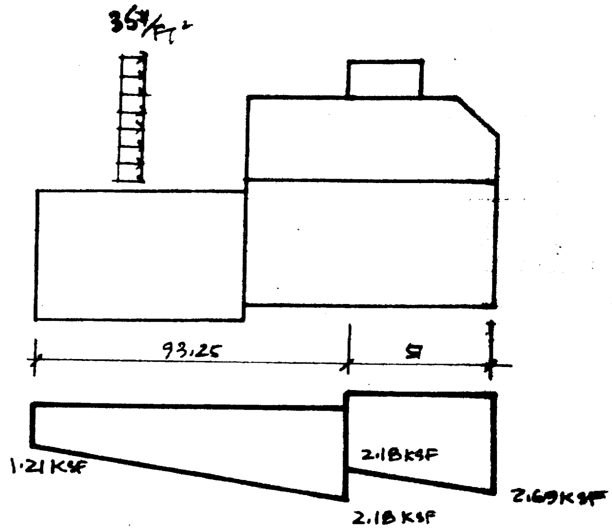
PROJECT CHARLES RIVER BASIN
SUBJECT PUMPING STATION - STABILITY ANALYSIS
COMP. T.L. CHECK
ACC. NO. [illegible] SHEET NO. 32 OF DATE 9.17 19 7[illegible] CONT. NO.

CASE III - Extreme high water, Basin side, min. water tide side.
Dead Load + Equipments + Thrust + bouyancy
Wind on basin side.



P_{WIND} = 329.97 kips →

P_{water} = 5,271.36 kips → (pg 22)

ΣP_H = 5601.33 Kip

Safety factor against sliding

$$SF = \frac{.5 \times 53325.67}{5601.33} = 4.75$$

M_{DL} = +4,352,781 + k-ft	P_{DL} = 68,614.24 K ↓	
M_{wind} = -21,959 - k-ft		
M_{uplift} = -4,171,571 - k-ft	P_{up} = 50,336.32 K ↑	(pg 23)
M_{water} = +3,085,352 + k-ft	P_W = 33,231.64 K ↓	(pg 22)
M_{Equip} = + 63,732 + k-ft	P_{eq} = 11,066.20 K ↓	
M_{thrust} = + 47,625 + k-ft	P_{THRUST} = 750.00 K ↓	
ΣM = 3,355,960 k-ft	ΣP = 53,325.67 ↓	

$$\text{Resultant arm} = \frac{3,355,960}{53,325.67} = 62.93$$

$$e = \frac{144.25}{2} - 62.93 = 9.19$$

$$q = \frac{53,325.67}{144.25 \times 190}\left(1 \pm \frac{6 \times 9.19}{144.25}\right) = 1.95\left(1 \pm .38\right)$$

q_1 = 1.95 × 1.38 = 2.69 KSF

q_2 = 1.95 × .62 = 1.21 KSF

q_3 = 2.69 - .51 = 2.18 KSF

$$\Sigma q = \frac{1.48 \times 51}{144.25} = .51$$

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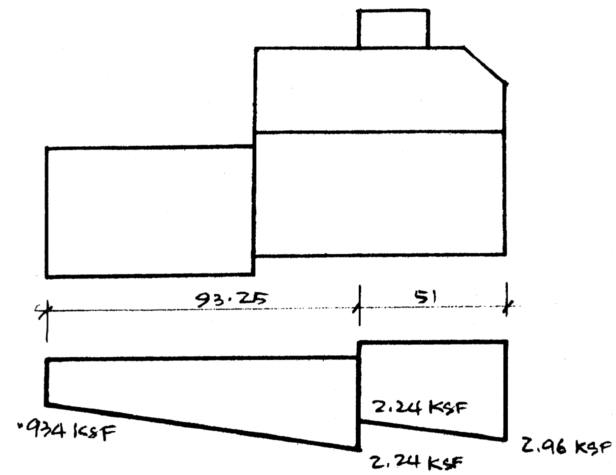
COMP.	T.L. CHECK	CONT. NO.	

<u>CASE IIIa</u> - Same as of III, added Earthquake + hydrodynamic force instead of wind



$H_{EQUAKE} = 6935$

$\vec{H} = 5,271.36$ (pg 22)

$\vec{h}_{dyn} = 799.03$

$\Sigma\vec{H} = 13,005.39$ Kips

Safety factor against sliding

$$SF = \frac{.5 \times 53,325.76}{13,005.39} = 2.04$$

$M_{DL} = +4,352,781$ K-ft

$M_{UPLIFT} = -4,171,571$ K-ft

$M_{water} = +3,085,352$ K-ft

$M_{EQUIP} = +63,732$ K-ft

$M_{THRUST} = +47,625$ K-ft

$M_{EQUAKE} = -187,482$ K-ft

$M_{hydrodyn} = -10,779$ K-ft

$\Sigma M_o = +3,179,658.$ K-ft

$P_{DL} = +68,614.24$ K

$P_{up} = -50,336.32$ K

$P_{water} = +33,231.64$ K

$P_{equip} = +1,066.20$ K

$P_{THRUST} = +750.0$ K

$\Sigma P = 53,325.76$ K ↓

$$X = \frac{3,179,658}{53,325.76} = 59.63$$

$$e = \frac{144.25}{2} - 59.63 = 12.495$$

$$q = \frac{53,325.76}{144.25 \times 190}\left(1 \pm \frac{6 \times 12.495}{144.25}\right) = 1.946\,(1 \pm .52)$$

$q_1 = 1.946 \times 1.52 = 2.96$ KSF

$q_2 = 1.946 \times .48 = .934$ KSF

$q_3 = 2.96 - .72 = 2.24$ KSF

$$\Delta q = 2.96 - .934 = \frac{2.03 \times 51}{144.25} = .72$$

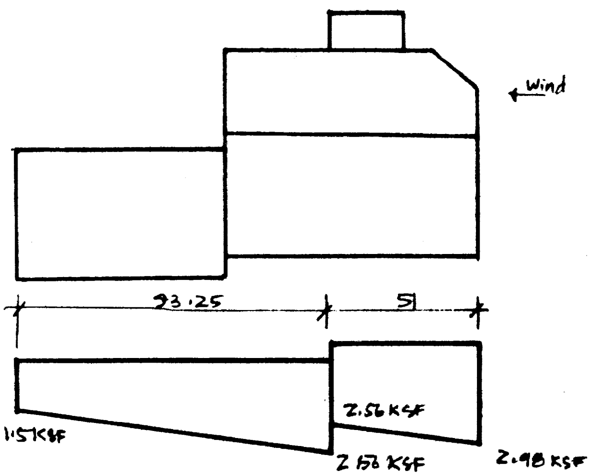
PROJECT: CHARLES RIVER BASIN — ACC. NO. 1856.30

SUBJECT: PUMPING STATION - STABILITY ANALYSIS — SHEET NO. 30 OF ____

DATE: 9.19 — 197_

COMP.: T.L. — CHECK: ____ — CONT. NO. ____

CASE IV Extreme high water tide side, wind load tide side + Equipment & pump thrust



$\overleftarrow{H} = 2,678.0$ K (Pg 22)

$\overleftarrow{h_{dyn}} = 837.16$ K (Pg 19)

$\overleftarrow{H_{wind}} = 329.97$

$\Sigma P_H = 3,845.13$ K ←

$$S.F._{sliding} = \frac{.5 \times 61,448.62}{3,845.13} = 7.99$$

$M_{DL} = +4,352,781$ K-ft

$M_{uplift} = -3,672,823$ K-ft (Pg 22)

$M_{water} = +3,128,648$ K-ft

$M_{equip} = +63,732$ K-ft

$M_{THRUST} = +47,625$ K-ft

$M_{wind} = +21,959$ K-ft

$\Sigma M = 3,941,922.$

$P_{DL} = 68,614.24$ K

$P_{up} = -52,944.64$ K

$P_{water} = +43,963.12$ K

$P_{equip} = +1,066.0$ K

$P_{Thrust} = +750.0$ K

$\Sigma P\downarrow = 61,448.62$ K ↓

$$X = \frac{3941,922}{61,448.62} = 64.15$$

$$e = \frac{144.25}{2} - 64.15 = 7.97 \text{ ft}$$

$$q = \frac{61,448.62}{144.25 \times 190}\left(1 \pm \frac{6 \times 7.97}{144.25}\right) = 2.24\,(1 \pm .33)$$

$q_1 = 2.24 \times 1.33 = 2.98$ KSF

$q_2 = 2.24 \times .67 = 1.5$ KSF

$$\Delta q = 2.98 - 1.5 = \frac{1.48 \times 51}{144.25} = .52$$

$q_3 = 2.98 - .52 = 2.56$ KSF

PROJECT CHARLES RIVER BASIN ACC. NO. 1856.30
SUBJECT PUMPING STATION - STABILITY ANALYSIS SHEET NO. 31 OF
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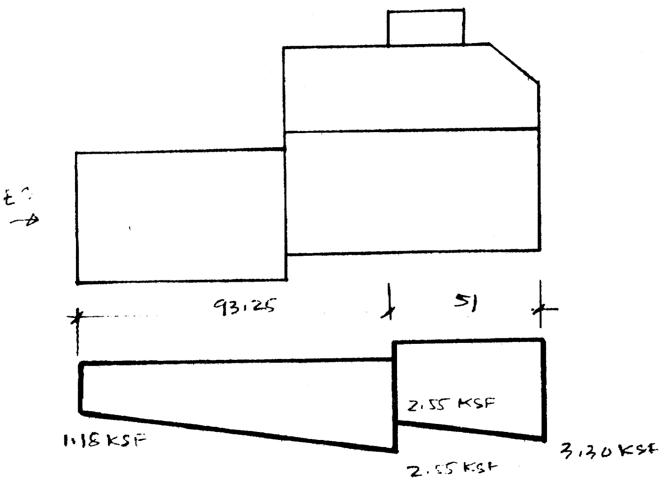
COMP. TL CHECK CONT. NO.

CASE IVa Same as IV but no wind, add EQ and hydrodynamic forces.



$\vec{P_e}$ = +605 K (pg 19a)
$\vec{H}_{quake}$ = +6935 K
$\overleftarrow{F}_{water}$ = −2,678 K
ΣP_H = 4,862. K

SF sliding = $\frac{.5 \times 61,448.62}{4,862.}$ = 6.3

M_{DL} = +4,352,751 K-ft
M_{uplift} = −3,672,823 K-ft
M_{water} = +3,128,648 K-ft
M_{equip} = +63,732 K-ft
M_{thrust} = +47,625 K-ft
M_{Quake} = −187,482 K-ft
$M_{hydrodyn}$ = −6740 K-ft (Pg 19a)
ΣM_0 = 3,725,741 K-ft

P_{DL} = 68,614.24 K ↓
P_{uplift} = −52,944.64 K ↑
P_{water} = +43,963.02 K ↓
P_{equip} = +1,066.00 K ↓
P_{thrust} = + 750.00 K ↓
ΣP = 61,448.62

X = $\frac{3,725,741}{61,448.62}$ = 60.75 ft
e = $\frac{144.25}{2}$ − 60.75 = 11.42

$q = \frac{61,448.62}{144.25 \times 190}\left(1 \pm \frac{6 \times 11.42}{144.25}\right) = 2.24 (1 \pm .475)$

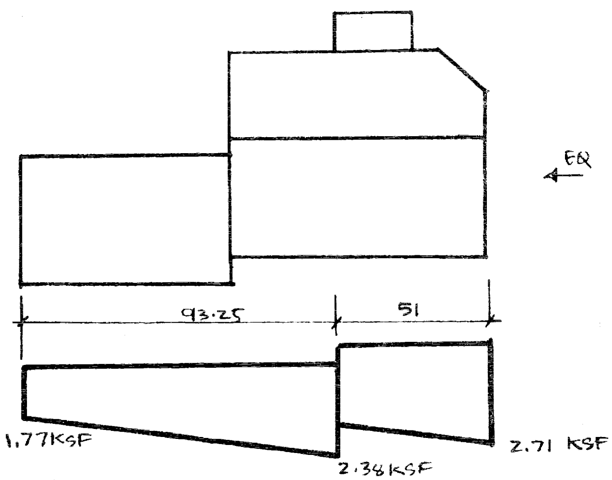
q_1 = 2.24 × 1.475 = 3.30 KSF
q_2 = 2.24 × .525 = 1.18 KSF
q_3 = 3.30 − .74 = 2.55 KSF

$\Delta = \frac{3.30 - 1.18}{144.25} \times 51 = .74$

CHARLES A. MAGUIRE & ASSOCIATES INC. ENGINEERS

PROJECT CHARLES RIVER BASIN — ACC. NO. 1856.30
SUBJECT PUMPING STATION - STABILITY ANALYSIS — SHEET NO. 32 OF
DATE 9.19 1972
COMP. TL — CHECK — CONT. NO.

CASE IV b — Same as IVa Except earthquake & hydrodynamic applied from tide side.



$\overleftarrow{P_e} = 837$ K

$\overleftarrow{H_{quake}} = 6935$ K

$\overleftarrow{P_{water}} = 2678$ K

$\Sigma P_H = 10,450$ K

S.F. sliding $= \frac{.5 \times 61,448.62}{10,450} = 2.94$

$M_{DL} = +4,352,781$ K-ft

$M_{uplift} = -3,672,823$ K-ft

$M_W = +3,128,648$ K-ft

$M_{Equip} = 63,732$ K-ft

$M_{thrust} = 47,625$ K-ft

$M_{quake} = +187,482$ K-ft

$M_{hydrodyn} = +11,577$ K-ft

$\Sigma M_o = 4,119,022$

$P_{DL} = 68,614.24$ K

$P_{up} = -52,944.64$ K

$P_{water} = +43,963.02$ K

$P_{equip} = 1,066.0$ K

$P_{thrust} = 750.0$ K

$\Sigma P = 61,448.62$ K ↓

$x = \frac{4,119,022}{61,448.62} = 67.03'$

$e = \frac{144.25}{2} - 67.03 = 5.1'$

$$q = \frac{61,448.62}{144.25 \times 190}\left(1 \pm \frac{6 \times 5.1}{144.25}\right) = 2.24(1 \pm .21)$$

$q_1 = 2.24 \times 1.21 = 2.71$ KSF

$q_2 = 2.24 \times .79 = 1.77$ KSF

$q_3 = 2.71 - .33 = 2.38$ KSF

$\Delta q = 2.71 - 1.77 = \frac{.94 \times 51}{144.25} = .33$

PROJECT CHARLES RIVER DAM
SUBJECT FISHWAY/LOCK & SLUICES STABILITY
COMP. GLC
CHECK
ACC. NO.
SHEET NO. 1 OF
DATE 29 Aug 1972
CONT. NO. 1856.32

Preliminary Weight (concrete only)

Wall 1 (28/12) x 9 x 68.25 1' x 9' x 68.25' =	614.25 (1432)
1' x 5' x 76.0 =	380.00 (28/12) = (887)
Wall 2 40/12 x 28 x 144.25 =	13449.87
Wall 3 1 x 4 x 144.25 = + 4 x 4 x 144.25	577.00 2308.00
Wall 4 32/12 x 37 x 68.25 =	6717.00
32/12 x 33 x 76.0 =	6671.28
Wall 5 2 x 12 x 144.25	3462.00
Wall 6 2 x 12 x 144.25	3462.00
Wall 7 2 x 10 x 144.25	2885.00
Wall 8 2 x 10 x 144.25	2885.00
Wall 9 8 x 144.25 (2)	2308.00
Wall 10 8 x 144.25 ((4 + 12.5)/2) [8.25]	9520.50
Wall 11 5 x 24 x 144.25	17310.00
Wall 12 6 x 144.25 ((11.5 + 15.0)/2) [13.25]	11467.87
Wall 13 4[15(10) + 90((10 + 17.0)/2) + 39.25(17)] [150, 13.5, 667.25]	8129.00
West Face: 2.5[8.5(12) + 4(16) + 4(9) + 1(5) + 6(18) + 1(5) + 8(18)] [102, 64, 36, 5, 108, 5, 144]	1160.00
East Face: 2.5[23(10) + 2.33(9) + 6(25.5) + 1(9) + 4(13) + 4(27) + 10(21)] [230, 20.97, 153, 9, 52, 108, 210]	3131.88

94,130 cf. + 3633

x .15 = 14,119ᴷ + 545

= 14,664

A-35

PROJECT: CHARLES RIVER DAM
SUBJECT: FISH WAY/LOCK & SLUICES STABILITY
COMP.: GLC
CHECK:

ACC. NO.:
SHEET NO. 2 OF
DATE: 19
CONT. NO. 1856.32

Wall	V	x	$\bar{X}$		
1	1432	x	109.1 x .15	=	23,400
	887	x	38.0 x .15	=	5,060
2	13,450	x	72.1 x .15	=	145,200
3	577	x	72.1 x .15	=	6,230
	2310	x	72.1 x .15	=	25,000
4	6720		109.1 x .15	=	111,000
	6670		38.0 x .15	=	38,000
5	3462		72.1 x .15	=	37,450
6	3462		72.1 x .15	=	37,450
7	2885		72.1 x .15	=	31,200
8	2885		72.1 x .15	=	31,200
9	2308		72.1 x .15	=	25,000
10	9520		57.9 x .15	=	82,600
11	17310		[illegible] x .15		187,100
12	11470		72.1 x .15		124,000
13	8130		[illegible] x .15		87,900
W.T.	1160		[illegible] x .15	=	220
E.F.	3130		[illegible] x .15		67,200
	97,768 x .15 = 19,665^k				1,085,410

W = 19,665

$$\bar{X}_1 = \frac{1,085,410}{19,665} = 74.1'$$

Soil

		c.f.	x	γ		W		$\bar{X}$	=	
(1)	10 x 19 x 76 =	14,420		.125	=	1,802		38	=	68,500
(2)	10 x 8.5 x 76 =	6450	x	.125	=	806		38	=	30,600
(3)	10 x 3.75 x 38 =	1510	x	.125		189		25.3	=	4,750
(4)	10 x 23 x 68 =	15,650	x	.125	=	1760		110	=	215,900
(5)	10 x 16.5 x 68 =	11,210		.125	=	1402	x	110	=	154,220
(6)	10 x 3.55 x [illegible] =	1210		.125	=	1512	x	87.3	=	131,000
						7,671^k				604,970

W = 22,346

$$\bar{X} = \frac{1,690,380}{22,346} = 75.7'$$

CE MAGUIRE, INC.
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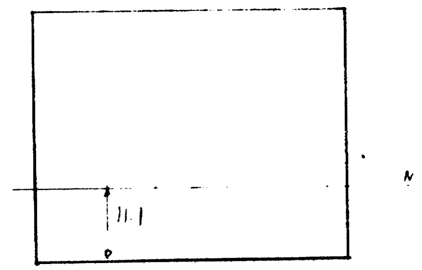
PROJECT: CHARLES RIVER DAM
SUBJECT: FISHWAY/LOCK & SLUICES STABILITY
COMP. GLC CHECK

ACC. NO.
SHEET NO. 3 OF
DATE 25 Sept 1972
CONT. NO. 1856.32

	Wall	Arm		
1)	$\frac{28}{12} \times 9 = 21.0$	32.5	=	682
2)	$\frac{40}{12} \times 28 = 93.3$	14.0	=	1305
3)	—	—	=	—
4)	$\frac{32}{12}(37) = 98.6$	18.5	=	1825
5)	2(8) = 16	13	=	208
6)	2(8) = 16	15	=	240
7)	2(14) = 28	7	=	296
8)	2(16) = 32	8	=	256
9)	2(8) = 16	1	=	16
10)	4(8) = 32	2	=	64
11)	5(24) = 120	12.	=	1440
12)	6(11.5) = 69.0	5.75	=	397
13)	4(10) = 40	5.0	=	200

$\Sigma A = 582$ $\Sigma M = 6929$

$$\bar{y} = \frac{\Sigma M}{\Sigma A} = \frac{6929}{582} = 11.1'$$



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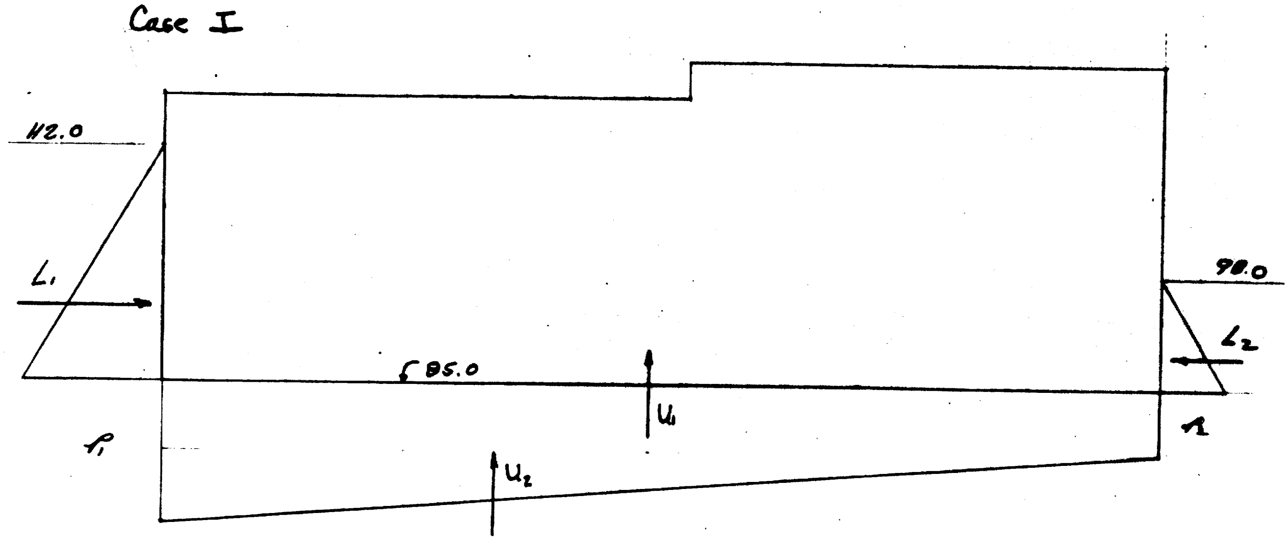
PROJECT CHARLES RIVER DAM
SUBJECT FISHWAY/LOCK & SLUICES
STABILITY
COMP. BLC CHECK
ACC. NO.
SHEET NO. 4 OF
DATE 22 Sept 1976
CONT. NO. 1856.32

Case I



$p_1 = 64(27) = 1728^{\#/\prime}$

$p_2 = 64(13) = 832^{\#/\prime}$

$U_1 = .832(144.25)(41) = 4910^{k}$

$U_2 = .896(144.25)(.5)(41) = 2650^{k}$

7560^{k}

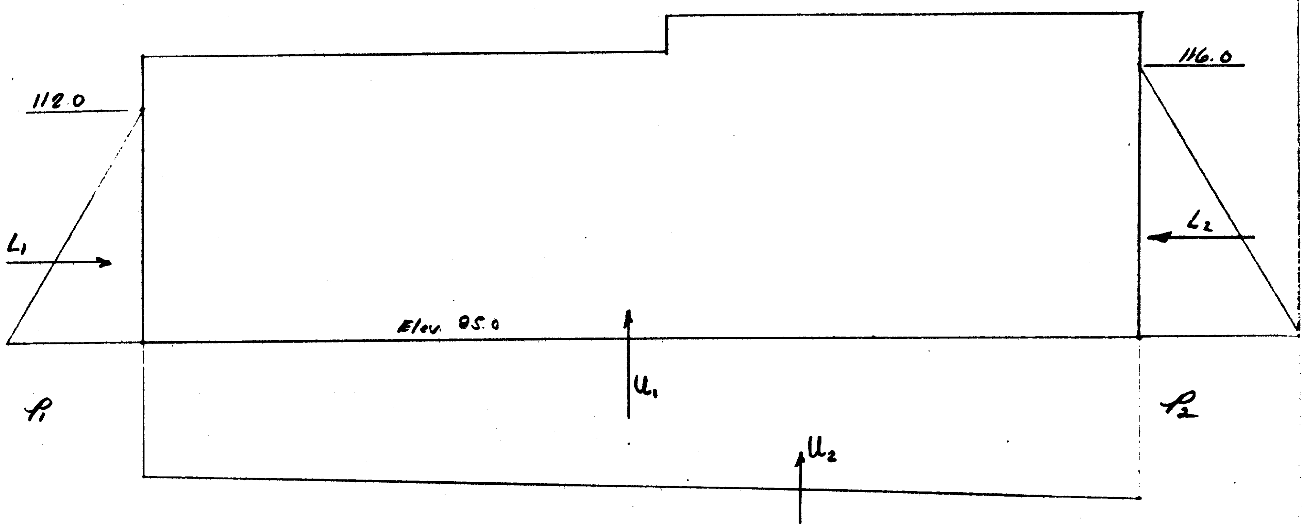
$L_1 = 1.728(27.0)(0.5)(41.0) = 956.0^{k}$

$L_2 = 0.832(13.0)(0.5)(41.0) = 221.8^{k}$

$\Delta L = \overrightarrow{734.2}$

PROJECT CHARLES RIVER DAM ACC. NO.
SUBJECT FISHWAY/LOCK & SLUICES SHEET NO. 5 OF
STABILITY DATE 22 Sept 1972
COMP. GLC CHECK CONT. NO. 1856.32

Case II



$P_1 = 64(27) = 1728^{\#}$

$P_2 = 64(31) = 1984^{\#}$

$U_1 = 1.728(144.25)(41.0) = 10{,}220^{k}$

$U_2 = 0.256(144.25)(0.5)(41.0) = \underline{758^{k}}$

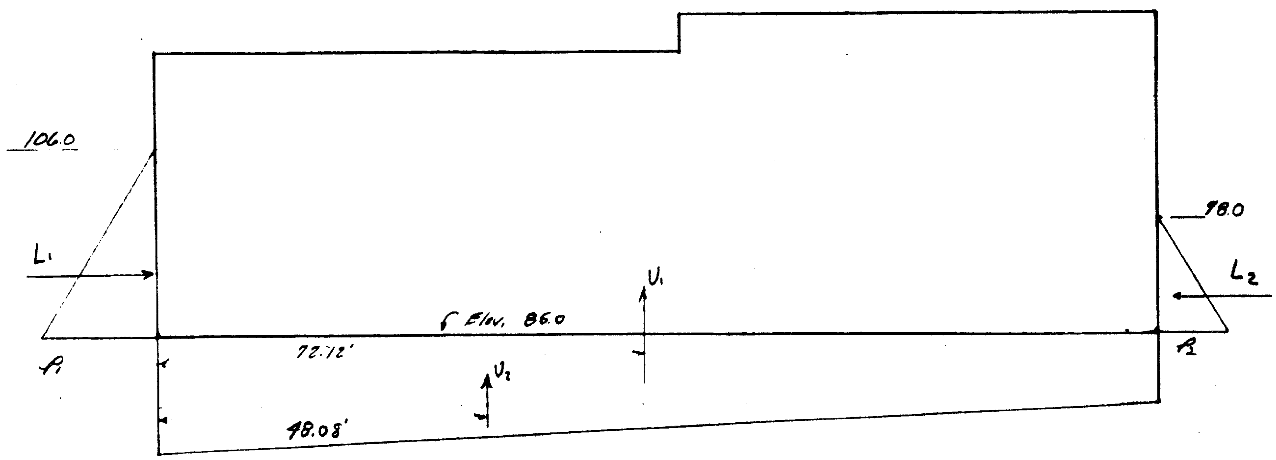
$10{,}978^{k}$

$L_1 = 1.728(27)(0.5)(41.0) = 956.0^{k}$

$L_2 = 1.984(31)(0.5)(41.0) = \underline{1260.0^{k}}$

$\leftarrow 304^{k}$

Case III



$P_1 = 64(21) = 1344$ #/'

$P_2 = 64(13) = 832$ #/'

$U_1 = .832(144.25)(41.0) = 4910^K$

$U_2 = 0.512(0.5)(144.25)(41.0) = 1512^K$

6422^K

$L_1 = 1.344(21.0)(0.5)(41.0) = 579.0^K$

$L_2 = 0.832(13.0)(0.5)(41.0) = 221.8^K$

357.2 →

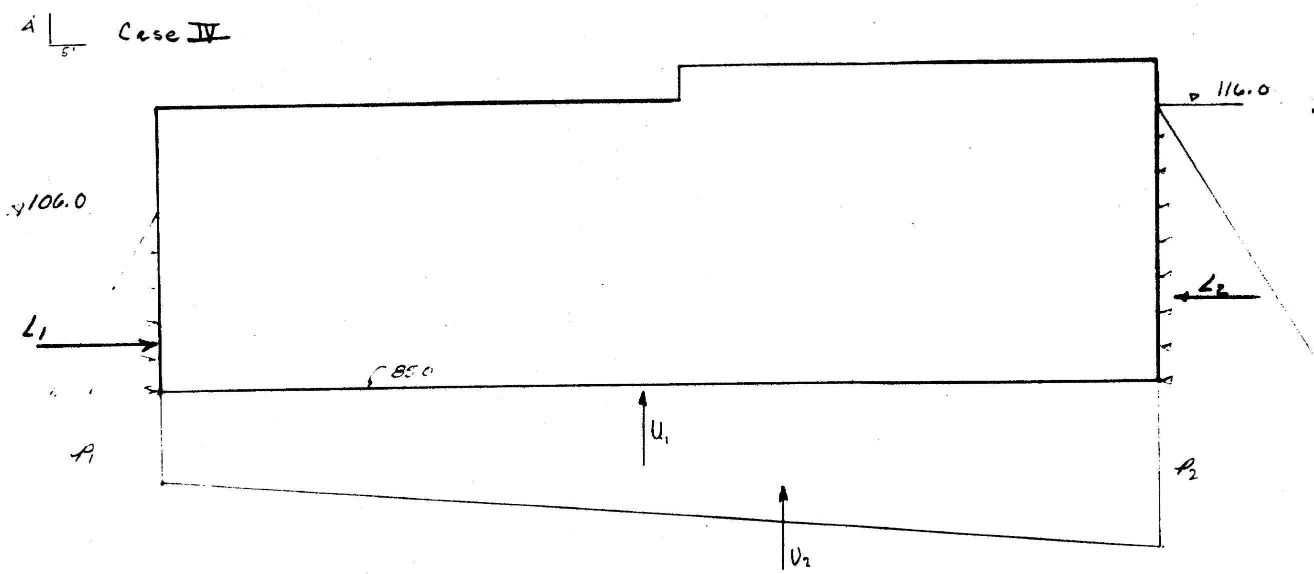
PROJECT CHARLES RIVER DAM — ACC. NO.

SUBJECT FISHWAY/LOCK & SLUICE STABILITY — SHEET NO. 7 OF

DATE 21 Sept 1972

COMP. GLC — CHECK — CONT. NO. 1856.32

Case IV



$p_1 = 64(21) = 1344\ \#/\square'$

$p_2 = 64(31) = 1984\ \#/\square'$

$U_1 = 1.344(41)(144.25) = 7950^k$

$U_2 = 0.640(41)(144.25)(0.5) = 1892$

9842.0

$L_1 = 1.344(21.0)(1/2)(41.0) = 579.0^k$

$L_2 = 1.984(31.0)(1/2)(41.0) = 1260.0^k$

← 681.0

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PROJECT CHARLES RIVER DAM | ACC. NO. ______
SUBJECT FISHWAY/LOCK & SLUICE STABILITY | SHEET NO. 8 OF ____
DATE 22 Sept 1978
COMP. GLC CHECK ______ | CONT. NO. 1856.32

Up Lift

Case I-A

$W_1 = 14,664$

$U = 7560$ $F.S. = \frac{14,664}{7560} = 1.93$

Case I-B

$W = 22,346$

$U = 7560$ $F.S. = \frac{22,346}{7560} = 2.95$

Case II-A

$W = 14,664$

$U = 10,978$ $F.S. = \frac{14,664}{10,978} = 1.33$

Case II-B

$W = 22,346$

$U = 10,978$ $F.S. = \frac{22,346}{10,978} = 2.03$

Case III-A

$W = 14,664$

$U = 6422$ $F.S. = \frac{14,664}{6422} = 2.28$

Case III-B

$W = 22,346$

$U = 6,422$ $F.S. = \frac{22,346}{6,422} = 3.48$

Case IV-A

$W = 14,664$

$U = 9842$ $F.S. = \frac{14,664}{9842} = 1.49$

Case IV-B

$W = 22,346$

$U = 9842$ $F.S. = \frac{22,346}{9842} = 2.27$

PROJECT CHARLES RIVER DAM ACC. NO. ____

SUBJECT FISHWAY/LOCK & SLUICES SHEET NO. 9 OF ____

STABILITY DATE 25 Sept 1972

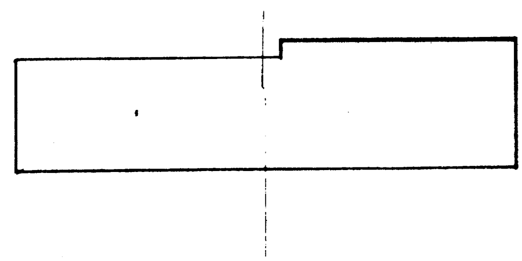
COMP. GLC CHECK ____ CONT. NO. 1856.32

CASE I-B

Soil PRESSURE

ΣM

W	22,346	+3.55	+79,100
U_1	4910	—	—
U_2	2650	−2401	+63,600
L_1	956	+9.00	+8,609
L_2	222	−4.33	−960
			151,304



A = 5,914 □'

S = 142,188 ft³

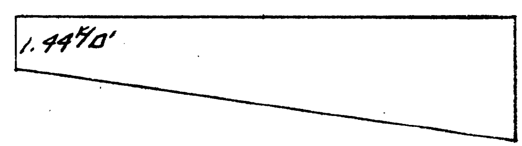
P = 14,786

$$f = \frac{P}{A} \pm \frac{M}{S}$$

$$= \frac{14,786}{5,914} \pm \frac{151,304}{142,188} = 2.5 \pm 1.06$$

= 3.56

1.44



Maximumum Concrete stress

W	22,346	3.55	79,100
L_1	956	9.00	8609
L_2	222	4.33	−960
			86,744

$$f = \frac{22,346}{5914} + \frac{86,744}{142,188} = 3.78 + 0.61 = 4.39^{k}/□'$$

f = 305#/□"

allowable bearing stress

$f_b = 0.85\,\phi\, f_c' =$

$= 0.85(0.70)(3,000)$

$= 1785$ #/□" > 305 #/□" OK

PROJECT CHARLES RIVER DAM — ACC. NO.
SUBJECT FISHWAY/LOCK & SLUICES STABILITY — SHEET NO. 10 OF
DATE 25 Sept 1972
COMP. GLC — CHECK — CONT. NO. 1856.32

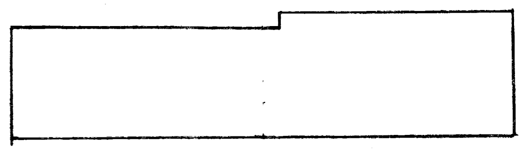
$S = 142,188 \text{ ft}^3$ $A = 5914 \ \square'$

Case II-B

Soil Pressure

Moment About ℄ ΣM

W	22,346	+3.55	+79,100
U_1	10,220	—	—
U_2	758	-24.01	-18,200
L_1	956	+9.0	+8,600
L_2	1260	-10.33	-13,000
			56,500

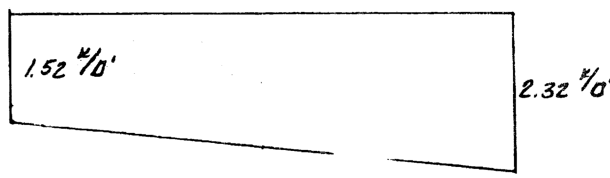


$P = W - U_1 - U_2 = 11,368$

$$f = \frac{P}{A} \pm \frac{M}{S}$$

$$= \frac{11,368}{5,914} \pm \frac{56,500}{142,188} = 1.92 \pm 0.40$$

$= 2.32$
1.52



A-44

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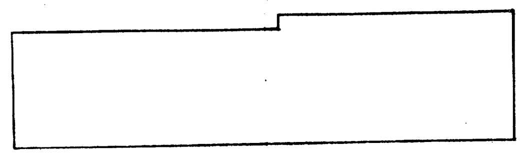
PROJECT CHARLES RIVER DAM — ACC. NO. ______
SUBJECT FISHWAY/LOCK & SLUICES STABILITY — SHEET NO. 11 OF ____
DATE 25 Sept 1972
COMP. GLC — CHECK ______ — CONT. NO. 1856.32

Soil Pressure

Case III B

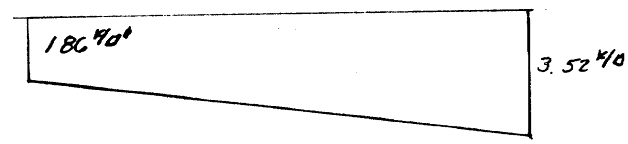
				℄M
W	22,346	x	3.55	+ 79,100
U_1	4,910	x	—	—
U_2	1512	x	24.01	+ 36,300
L_1	579	x	7.00	+ 4,050
L_2	222	x	4.33	- 962
				118,488



$A = 5914^{\square'}$

$S = 142,188 \text{ ft}^3$

$P = 15,924^{K}$



$f = \frac{P}{A} \pm \frac{M}{S}$

$= \frac{15,924}{5,914} \pm \frac{118,488}{142,188}$

$= 2.69 \pm 0.83$

$= 3.52$

1.86

A-45

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PROJECT CHARLES RIVER DAM — ACC. NO. ____

SUBJECT FISHWAY/LOCK & SLUICES — SHEET NO. 12 OF ____

STABILITY — DATE 25 Sept 1977

COMP. GLC — CHECK ____ — CONT. NO. 1856.32

CASE IV·B

W	22,346ᵏ	3.55	+79,100
U_1	7,950ᵏ	—	—
U_2	1,892ᵏ	24.01	−45,400
L_1	579	7.0	+4050
L_2	1260	10.33	−13,000
			24,750

P = 12,504

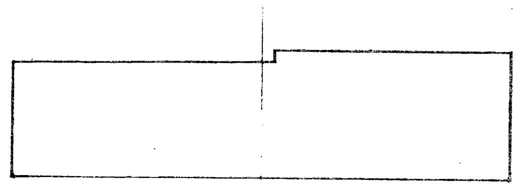
$f = \frac{12,504}{5914} \pm \frac{24,750}{142,188}$

$= 2.12 \pm 0.17$

$= 2.29$

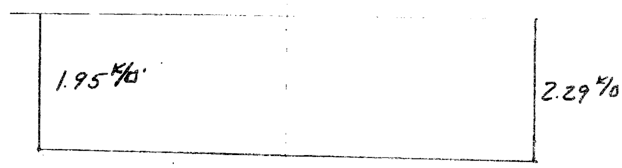
$\ 1.95$

SOIL PRESSURE



A = 5914 □'

S = 142,188 ft³



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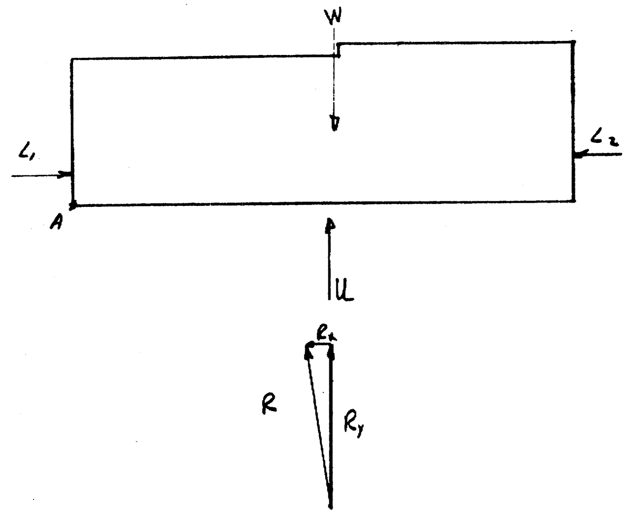
PROJECT	CHARLES RIVER DAM
SUBJECT	FISHWAY/LOCK & SLUICES STABILITY
COMP.	GLC
CHECK.	
ACC. NO.	
SHEET NO.	13 OF
DATE	26 Sept 1972
CONT. NO.	1856.32

Case I

			$\curvearrowright + \Sigma M_A = 0$
W	22,346↓	72.70	+1,624,554
U_1	4,910↑	72.07	- 353,864
U_2	2650↑	48.08	- 127,412
R_y =	14,786↓		1,143,278
L_1	956.0 →	9.0	+ 8,604
L_2	221.8 ←	4.33	- 960
R_x =	734 →		
			1,150,922

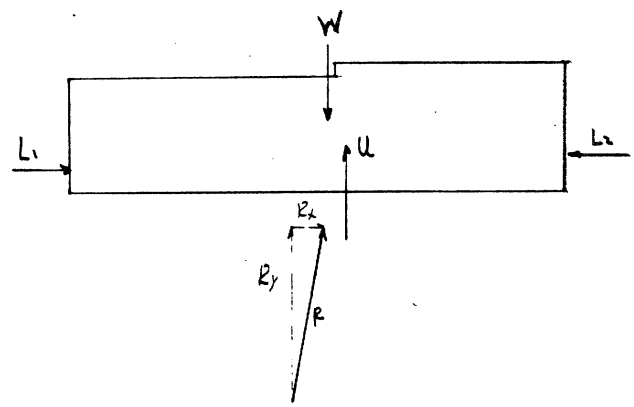
$$\bar{X} = \frac{1,150,922}{14,786} = 77.84'$$



Case II

W	22,346↓	72.7	+ 1,624,554
U_1	10,220↑	72.07	- 736,555
U_2	758↑	96.16	- 72,889
R_y =	11,368↓		815,110
L_1	956 →	9.0	+ 8,604
L_2	1260 ←	10.33	- 13,016
R_x =	304 ←		810,698

$$\bar{X} = \frac{810,698}{11,368} = 71.31'$$



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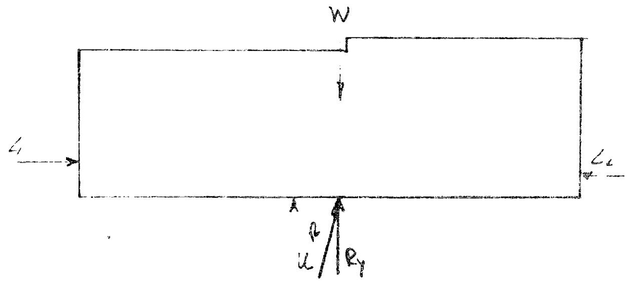
PROJECT CHARLES RIVER DAM
SUBJECT FISHWAY/LOCK & SLUICES
STABILITY
COMP. GLC
CHECK
ACC. NO.
SHEET NO. 14 OF
DATE 26 Sept 1972
CONT. NO. 1856.32

Case III-B

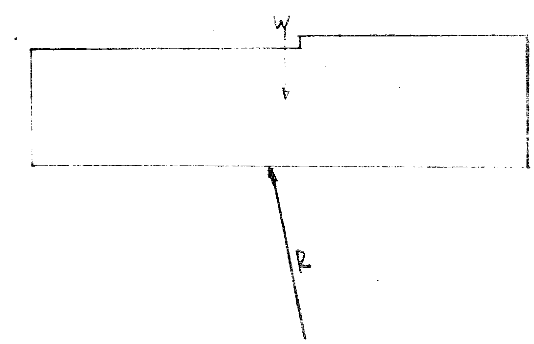
W	22,346↓	72.70	+ 1629,554
U_1	4910↑	72.12	- 353,864
U_2	1512↑	48.08	- 72,697
R_y	15,924↓		+ 1,197,993
L_1	579→	7.0	+ 4053
L_2	←221.8	4.33	- 960
R_x	357.2→		+ 1,201,086



$$\bar{X} = \frac{1,201,086}{15,924} = 75.43$$

Case IV-B

W	22,346↓	72.70	+ 1,624,554
U_1	7,950↑	72.12	- 573,354
U_2	1892↑	96.16	- 181,935
R_y	12,504		+ 869,264
L_1	579→	7.0	+ 4053
L_2	1260←	10.33	- 13,016
R_x	←681		860,301



$$\bar{X} = \frac{860,301}{12,504} = 68.80'$$

PROJECT CHARLES RIVER DAM
SUBJECT FISHWAY/LOCK & SLUICE STABILITY
COMP. GLC
CHECK
ACC. NO.
SHEET NO. 15 OF
DATE 22 Sept 1972
CONT. NO. 1856.32

Hydrodynamic Force

increase in pressure

$P_E = CK\gamma_{ZH}$

$Z = H$

$K = 0.1$

$N = 1.0$ sec

$C = \frac{51}{\sqrt{1 - 0.72\left(\frac{Z}{7000N}\right)^2}} \approx 51.0$

$\therefore P_E = 5.1H$ $\qquad P_{WE} = \frac{2H}{3}P_E$ $\qquad F_{WE} = .41\, P_{WE}$

H	P_E	P_{WE}	F_{WE}
13	66.3	587.0 #/'	24.1 K
21	107.1	1498.0 #/'	61.4 K
27	137.9	2480.0 #/'	101.8 K
31	158.1	3265.0 #/'	134.0 K

PROJECT CHARLES RIVER DAM
SUBJECT FISH WAY/LOCK & SLUICES
STABILITY
COMP. ALC
CHECK
ACC. NO.
SHEET NO. 16 OF
DATE 25 Sept 1972
CONT. NO. 1856.32

SLIDING

CASE	W	u	W-u	R	ΔL	F.S	F_{uc}	Fc	ΣF	F.S.
I-A	14,664	7560	7104	3552	734.2 →	4.83	101.8[k] →	1466[k]	2302	1.54
I-B	22,346	7560	14,786	7393	734.2 →	9.96	101.8[k] →	2235	3071	2.41
II-A	14,664	10,978	3686	1843	← 304.0	6.10	134.0[k]	1466	1904	0.99 *
II-B	22,346	10,978	11,368	5684	← 304.0	18.70	134.0[k]	2235	2673	2.12
III-A	14,664	6422	8242	4121	367.2 →	11.52	61.4[k] →	1466	1889.6	2.18
III B	22,346	6422	15,924	7962	367.2 →	22.30	61.4[k] →	2235	2653.6	3.00
IV-A	14,664	9842	4822	2411	← 681.0	3.40	← 189.0	1466	2281	1.05 *
IV-B	22,346	9842	12,504	6252	← 681.0	9.58	134.0	2235	3050	2.05

* Due to Conditions II-A, & IV A, Fish WAY/LOCK structure should be back filled prior to removal of Coffer dam

PROJECT CRD
SUBJECT [illegible] JOIST [illegible]
COMP. [illegible]   CHECK [illegible]

ACC. NO. 1551.70
SHEET NO. 2 OF ___
DATE 11 July 1973
CONT. NO. ___

JOIST

UNIFORM LOAD = (3')(275#/ᶠ) = 0.825 ᴷ/L.F (FACTORED)

SPAN = 16'2" SAY 16.5'

TRIAL; NEGLECT T-BEAM ACTION ∴ b = 6" d = 15" - 1.5" - 1" = 12.5"

LET $M_{+,-} = \frac{w l^2}{10} = \frac{(0.825^{K/LF})(16.5')^2}{10} \cong 22.5^{K-F}$

U.S.D HNDBK. FLEX 11

F = 0.073 ∴ $K_u = \frac{22.5}{0.073} = 288$ TRY p = 0.006

∴ $A_s = pbd = 0.45^{\square''}$

TRY 2#5 $A_s = 0.61^{\square''}$

$a = \frac{(0.61)(60)}{(.85)(3)(6)} = 2.392''$

∴ $M_u = (.9)(0.61)(60)(12.5 - \frac{2.392}{2}) / 12''/' = 31.03^{FT-K} > 22.5^{F-K}$ REQ'D OK

$> \frac{wl^2}{8} = 28.1^{F-K}$

USE 2#5 BOT

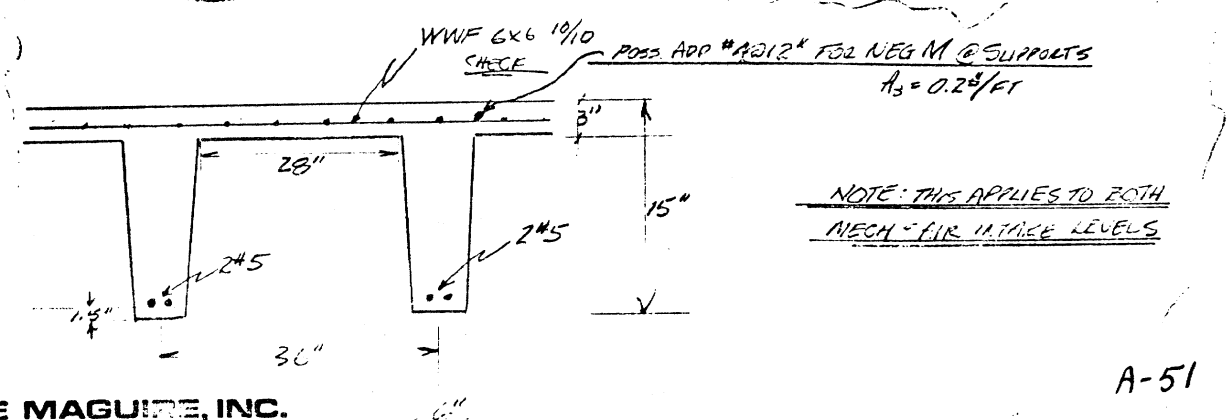
SHEAR

$V_{MAX} \cong 8.25'(.825^{K/LF}) = 6.81^K$ $v_u = \frac{6.81^K}{(6 \times 12.5)} = 0.091^{KSI}$ (CONSERVATIVE)

$v_c = 2(1.1)\sqrt{3000} = 120.49\, psi$ ∴ NO SHEAR REINF REQ'D

10% INCR FOR JOIST

$0.12^{KSI} > 0.09^{KSI}$
ALLOWB ACTUAL



NOTE: THIS APPLIES TO BOTH MECH & AIR INTAKE LEVELS

A-51

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PROJECT CRD ACC. NO. 1856.10

SUBJECT REVISION A-N E... CONC JOISTS SHEET NO. ___ OF ___

DATE 19 JULY 1973

COMP. WCS CHECK VS CONT. NO. ___

CONC. JOIST 16" RIB + 3" SLAB
SPAN 16.5' ±

LOADS:

$LL = 100^{\#}/\square' \times 1.7 = .170^{KSF}$

DL SLS RIBS $= (1.53)(150) \times 1.4 = 0.32^{KSF}$

TOTAL UNIFORM LOAD $= 3(.17) + 0.32 = 0.83^{K}/LF$

S.S. $M = \frac{w\ell^2}{8} = \frac{(.83)(16.5)^2}{8} = 28.25^{FT\text{-}K}$

NEGLECT T-BEAM ACTION

$b = 6''$ $d = 16'' - 1.5'' - 1'' = 13.5$ $\therefore F = 0.091$ $K = \frac{28.25}{0.091} = 310$

TRY $p = 0.0065$ $A_s = pbd = 0.53\square''$

TRY 2 #5 $A_s = 0.61\square''$

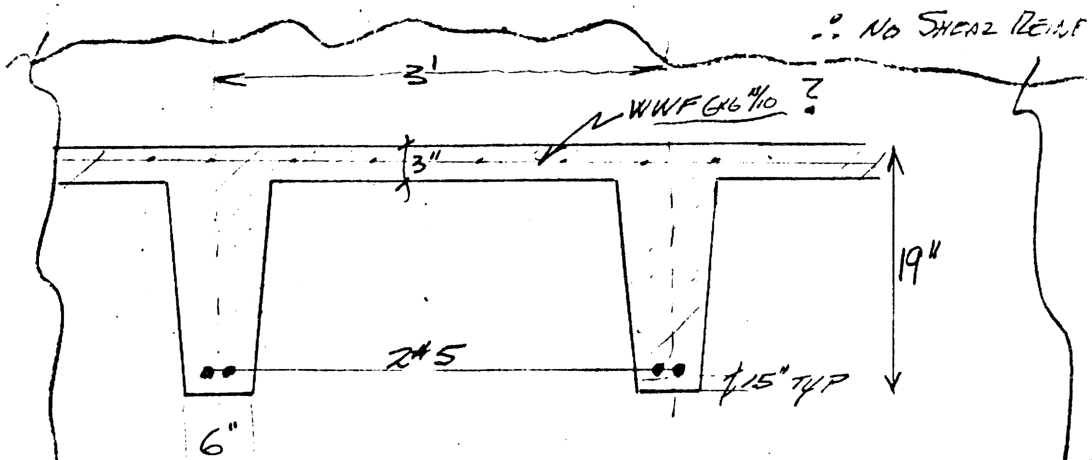
$a = \frac{(.61)(60)}{(.85)(3)(6)} = 2.4''$

$M_u = (.9)(.61)(60)(13.5 - \frac{2.4}{2}) / 12'' = 33.76^{FT\text{-}K} > 28.35^{FT\text{-}K}$ REQ'D.

SHEAR

$V_{max} = (0.83^{K}/LF)(8.5') = 7.1^{K}$ $v_u = \frac{7.1}{(6)(13.5)} = 0.087^{KSI} < 0.109^{KSI}$ ALLB

$\therefore$ NO SHEAR REINF NEC.



REQ'D A of WWF/FT = M ... d = 1.5" ... a = 2.5"

$A_{REQ'D} \approx 0.15\square''/FT$

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11-728

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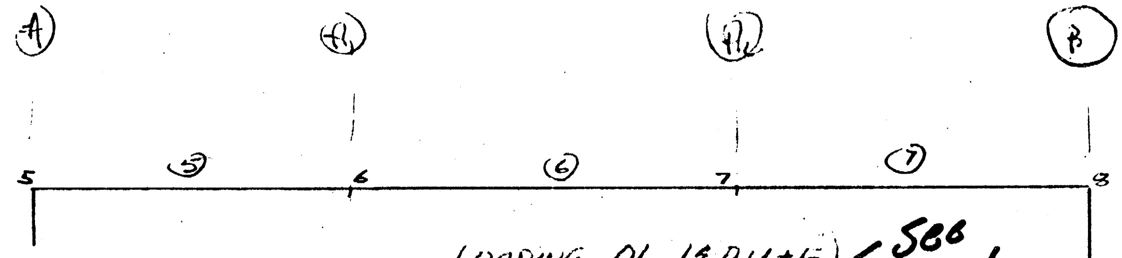
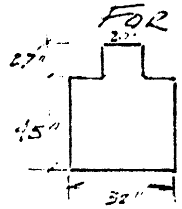


LOADING 01 (1.4D+L+E) - See Rev Forces Foll.

VOID

Axial (K)	218.546	-213.546	218.556	-213.556	267.715	-267.715
Shear (K)	911.421	-391.233	345.[illegible]71	[illegible]	-564.339	1393.89
Moment (FT-K)	3250.26	10212.26	-10,2[illegible]	13,7[illegible].49	-13,927.11	-7595.78

FOR +M; SECTION



$b = 20"$ $d = 72" - 1.5" - 2" = 68.5"$ ∴ $F = 8.05$ ∴ $K_u = \frac{13927.11}{8.05}$ = TOO LARGE

TRY DOUBLY REINF. SECTION ≈A=1980 □"

TRY $p = 0.014$ $A_s = 27.72$ □" TRY TWO ROWS 6#14S TOTAL 12#14S, $A_s = 27.00$ □"

$a = \frac{(27)(60)}{(.85)(3)(20)} = 31.76" > 27"$ - USE; 30" $d = 72" - 1.5" - 2.75" = 67.75"$

$M = (.9)(27.0)(60)(67.75 - \frac{30}{2})/12'' = 6409$ FT-K TOO SMALL

See Adj of Forces For Reduction in L.L. permitted

PROJECT C Z D ACC. NO.
SUBJECT SHEET NO. ___ OF B-96
DATE ___ 19__
COMP. CHECK KS CONT. NO.

MEM JNT	D.L. M	D.L. S	L.L. M (350/4 x1.4)	L.L. S (16/4 x1.4)	SEISMIC M	SEISMIC S	ΣM	ΣS
5	821.22	221.832	249.63	54.196	190.22	8.902		
	x1.4	x1.4	x1.7	x1.7	x1.4	x1.4		
	1149.71	310.56	[illegible]	92.12	266..	12.5	-1574.11 FT-K should be 1587 FT-K	415^K OR
(5) 6	2428.88	-72.644	605.46	-23.556	-6.21	-8.902		
	x1.4	x1.4	x1.7	x1.7				
	3400.4	[illegible]	1029.3	[illegible]			+4430	179^K LD 13
6	3201.77	79.144	-605.46	23.555	6.21	8.902		
(6) 7	3201.77	1.855	976.57	-8.555	171.82	-8.902		
	x1.4	x1.4	x1.7	x1.7				
	4482.5	2.6	1660.2	14.55			6143 FT-K (should be 6146.2 FT-K LD 13)	17.15^K
7	-3257.57	-148.535	-968.75	-15.807	-154.53	6.712		
	x1.4	x1.4	x1.4	x1.4	x1.4	x1.4		
	4561	208	1356.25	22.13	216.34	9.4		
(7) 8	-1617.42	299.597	-446.74	112.719	304.41 -288	-6.712		
	x1.4	x1.4	x1.4	x1.4	x1.4	x1.4		
	2264.4		625		403			

$M_{DES} \approx \underline{6210.2}$ FT-K

SHEAR = $\underline{594.62}$ K

LD 8

CRN LD		M	S
	5	7.91	0.245
(5)	6	-2.83	-0.245
	6	2.83	0.245
(6)	7	2.07	-0.245
	7	-1.58	0.193
(7)	8	6.04	-0.193

x1.7 +

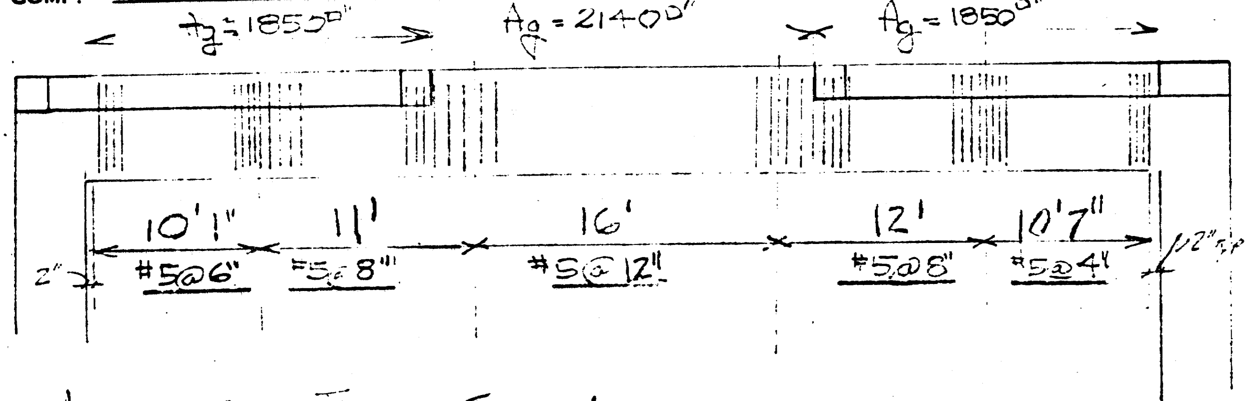
$-M \approx \underline{3300}$ FT-K

DES +M = 6210.2 FT-K
DES −M (JNTS 5,8) = ~~1587.7~~ FT-K ~~3046~~ FT-K ORIG.
MAX SHEAR = 595 K

~~1.4 DL -~~ ...

A-54

PROJECT CZD ACC. NO.
SUBJECT SHEET NO. ___ OF B-97
DATE ___ 19__
COMP. CHECK CONT. NO.



Allowable Shear Force on Sections:

$Ave\ b = \frac{32(45)+20(27)}{45+27} = 27.5''$

(I) w/ #5@4" $v_u - v_c = \frac{.62(40)}{(27.5)(4)} = 0.221$

∴ Allb v_u: $0.221 + 0.109 = 0.33^{ksi}$

∴ Allb Shear Force $= (0.33^{ksi})(1850^{\square''}) = \underline{610.5^K}$

(II) w/ #5@6" $v_u - v_c = \frac{.62(40)}{(27.5)(6)} = 0.1473^{ksi}$

∴ Allb $v_u = 0.1473 + 0.109 = 0.2563^{ksi}$

∴ Allb Shear Force $= (0.2563)(1850) = \underline{475^K}$

(III) w #5@8" $v_u - v_c = \frac{.62(40)}{(32)(8)} = 0.095^{ksi}$

∴ Allb $v_u = 0.095 + 0.109 = 0.204^{ksi}$

∴ Allb Shear Force $= 0.204(2140) = \underline{436^K}$

or Allb Shear Force $= 0.204(1850) = \underline{377^K}$

(IV) w #5@12" $v_u - v_c = \frac{.62(40)}{(27.5)(12)} = 0.0739$

$v_u = 0.0739 + 0.109 = 0.1829^{ksi}$ ∴ Allb Shear Force $= .1829(1850) = \underline{338^K}$

(V) w #5@12" $v_u - v_c = \frac{.62(40)}{(32)(12)} = 0.0476$

$v_u = 0.0476 + .109 = 0.1566^{ksi}$

∴ Allb Shear Force $= 0.1566(2140) = \underline{335^K}$

A-55

PROJECT CRD ACC. NO.

SUBJECT SHEET NO. ___ OF B-98

DATE 19

COMP. CHECK CONT. NO.

CHECK ACI A5.9

$A_v \frac{d}{s} \geq .15 A_s$ where $A_s = 16\#11 = 25^{\square''}$

$(.61)\left(\frac{67.5}{6}\right) \geq .15(25)$

6.8 > 3.75 OK

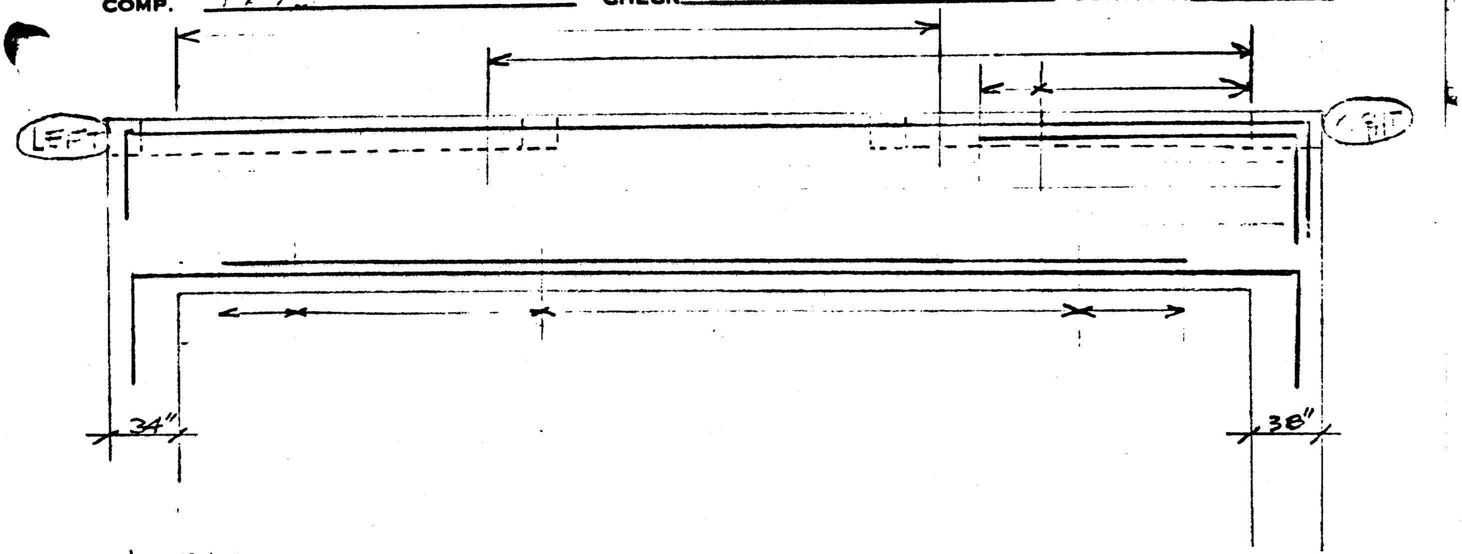
$(.61)\left(\frac{67.5}{8}\right) \geq 0.15(25)$

5.13 > 3.75 OK

(VI) w #5@12" b = 32"

ALLB SHEAR FORCE = (0.1829)(2140) = 391.4 K

PROJECT: CRD — ACC. NO. [illegible]
SUBJECT: [illegible]
SHEET NO. ___ OF B-99 — DATE 27 JULY 1973
COMP. [illegible] — CHECK KS — CONT. NO. 1



ACI CHAP. 12

BASIC DEVELOPEMENT LENGTH #11 BAR = $0.4(156)(60)/\sqrt{3} \doteq 68.57''$

TOP BARS: $1.4 \times 68.57'' = 96''$

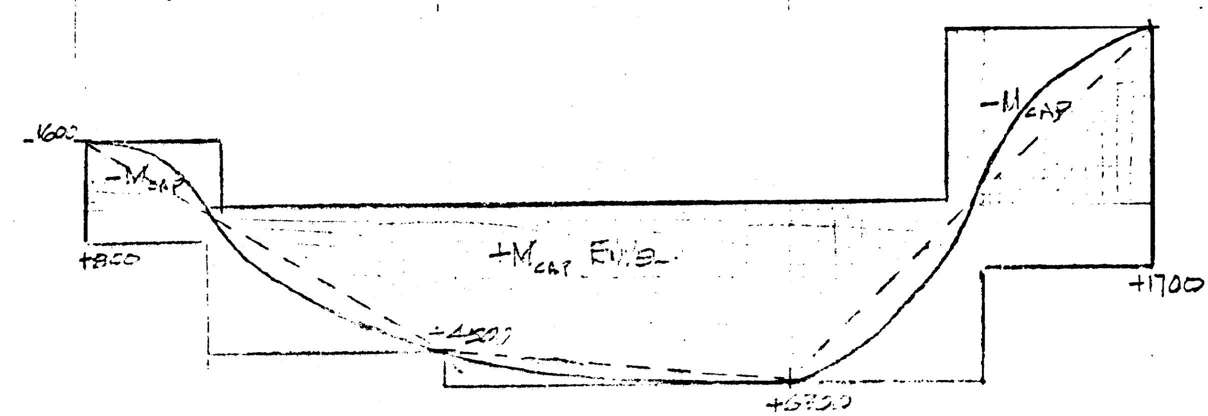
NEGLECT ANY ALLOWABLE REDUCTION IN l_d FOR A_s PROV > A_s REQ'D

COMPRESSION STEEL:

$$l_d = 0.2 f_y d_b/\sqrt{f'_c} = 0.2(60)(1.41)/\sqrt{3} = 9.8'' \text{ say } 12''$$

REQ'D MOMENT CAPACITY

−M	1600 →	4500	← 6300 ←	3300
+M	800 (MIN)			1700 (MIN)



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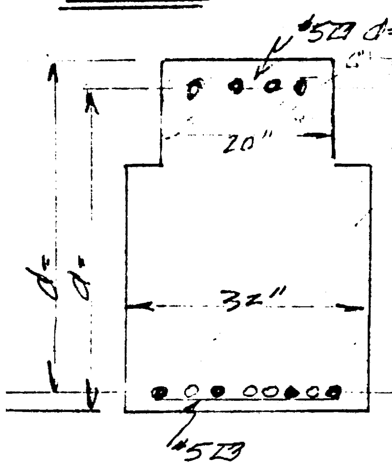
PROJECT CRD **ACC. NO.** [illegible]
SUBJECT [illegible] **SHEET NO.** ___ **OF** B-100
[illegible] **DATE** 27 [illegible] 19[illegible]
COMP. [illegible] **CHECK** KS **CONT. NO.** ___

LEFT SECTION



−M CAP as Singly Reinf.

$a = \frac{(6.25)(60)}{(.85)(3)(32)} = 4.6"$

$M_u = (.9)(6.25)(60)(69 - \frac{4.6}{2}) / 12" = \underline{1875.9}^{FT-K}$

+M CAP as Singly Reinf.

$a = \frac{(6.25)(60)}{(.85)(3)(20)} = 7.35" < 27"$ OK

$M_u = (.9)(6.25)(60)(69 - \frac{7.35}{2}) / 12" = \underline{1837.26}^{FT-K}$

[−M CAP w/ 6#11 $A_s = 9.37^{\square"}$

$d = 72" - 1.5" - .625" - 1.25(1.41) = 68"$

$a = \frac{(9.37)(60)}{(.85)(3)(32)} =$

$M_u = (.9)(9.37)(60)(68 - \frac{6.89}{2}) / 12" = \underline{2721.75}^{FT-K}$]

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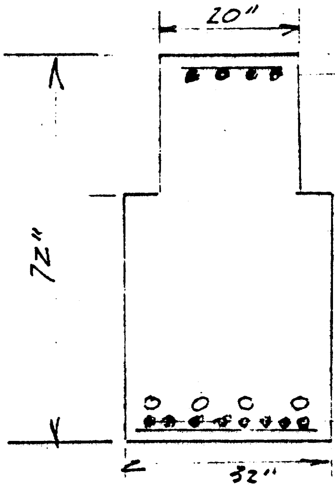
11-72B

PROJECT	CRD	ACC. NO.	[illegible]
SUBJECT	[illegible] Main [illegible]	SHEET NO. ___ OF	B-101
		DATE	[illegible] 1973
COMP.	[illegible]	CHECK	VS
		CONT. NO.	

SECTION ≈10' FROM LEFT



4#11 $A_s = 6.25^{\square''}$

$d = 72'' - 1.5'' - .625'' - .7'' \doteq 69''$

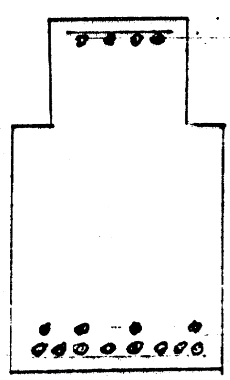
8#11 $A_s = 12.5^{\square''}$

CHK AS SINGLY REINF $+M_{CAP}$

$$a = \frac{(12.5)(60)}{(.85)(3)(20)} = 14.7'' < 27'' \text{ OK}$$

$$M_u = (.9)(12.5)(60)\left(69 - \frac{14.7}{2}\right)/12''/' = \underline{\underline{3467.8^{FT-K}}}$$

SECTION ≈20' FROM LEFT



4#11

$d = 72'' - 1.5'' - .625'' - 1.41'' - .35'' \doteq 68''$

4#11 $A_s = 6.25$
8#11 $A_s = 12.5$ > TOTAL $18.75^{\square''}$

CHK AS SINGLY REINF $+M_{CAP}$

$$a = \frac{(18.75)(60)}{(.85)(3)(20)} = 22.1'' < 27'' \text{ OK}$$

$$M_u = (.9)(18.75)(60)\left(68 - \frac{22.1}{2}\right)/12''/' = \underline{\underline{4805^{FT-K}}}$$

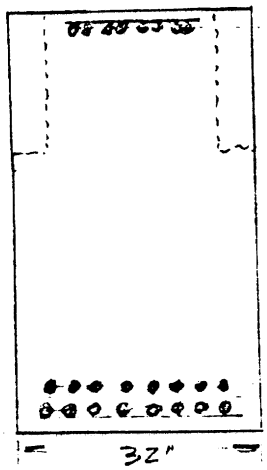
PROJECT C2D ACC. NO. [illegible]

SUBJECT [illegible] SHEET NO. ___ OF B-102

DATE [illegible] 19__

COMP. [illegible] CHECK [illegible] CONT. NO. ___

SECTION CENTER SPAN



2 SETS OF 4#11 BARS LAP SPLICED

$d = 72'' - 1.5'' - .625'' - 1.41'' - .7'' \cong 67.5''$

2 ROWS OF 8#11 TOT. 16#11 $A_s = 25.00^{\square''}$

32"

CHK AS SINGLY REINF +M CAP

$a = \frac{(25)(60)}{(.85)(3)(32)} = 18.4''$ NOTE $b = 32''$

$M_u = (.9)(25)(60)(67.5 - \frac{18.4}{2})/12''/' = 6558.75^{FT-K}$

CHK AS DOUBLY REINF w/ $A_s' = 4\#11 = 6.25^{\square''}$ $d' = 1.5'' + .625'' + .7'' = 2.825''$ SAY $3''$

+M CAP

$M_{u_1} = (.9)(6.25)(60)(67.5 - 3)/12''/' = 1814^{FT-K}$

$a = \frac{(25 - 6.25)(60)}{(.85)(3)(20)} = 22.1''$ NOTE $b = 20''$

$M_{u_2} = (.9)(25 - 6.25)(60)(67.5 - \frac{22.1}{2})/12''/' = 4762.96^{FT-K}$

$\therefore M_{u_T} = M_{u_1} + M_{u_2} = 6576^{FT-K}$

NOTE THIS APPLIES TO SECTION ≈ 20' FROM [illegible]

A-60

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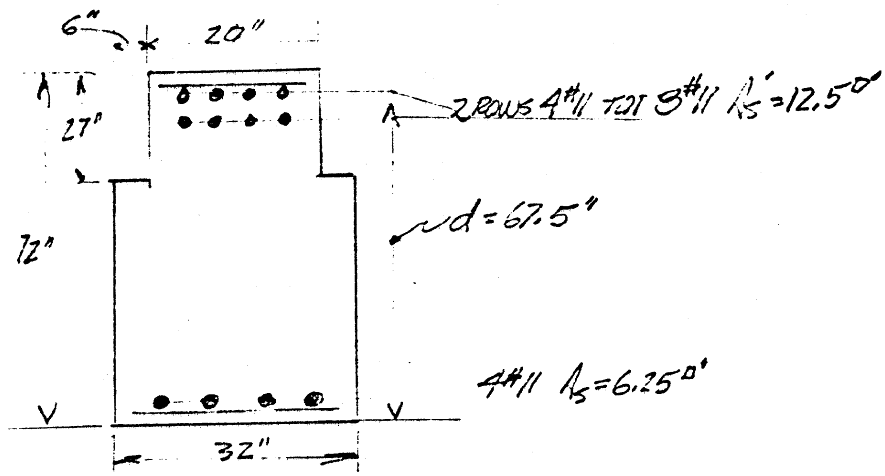
11-72B

PROJECT CRD — ACC. NO. [illegible]
SUBJECT [illegible] — SHEET NO. ___ OF ___
DATE 27 [illegible] 19[illegible]
COMP. [illegible] CHECK [illegible] CONT. NO. ___

RIGHT SECTION



$-M_{CAP}$ as Singly Reinf

$$a = \frac{(12.5)(60)}{(.85)(3)(32)} = 9.2''$$

$$M_u = (.9)(12.5)(60)\left(67.5 - \frac{9.2}{2}\right) / (12''/') = \underline{3538}^{FT-K}$$

$+M_{CAP}$

$$a = \frac{(6.25)(60)}{(.85)(3)(20)} = 7.35'' \qquad \underline{d = 69''} \quad \underline{b = 20''}$$

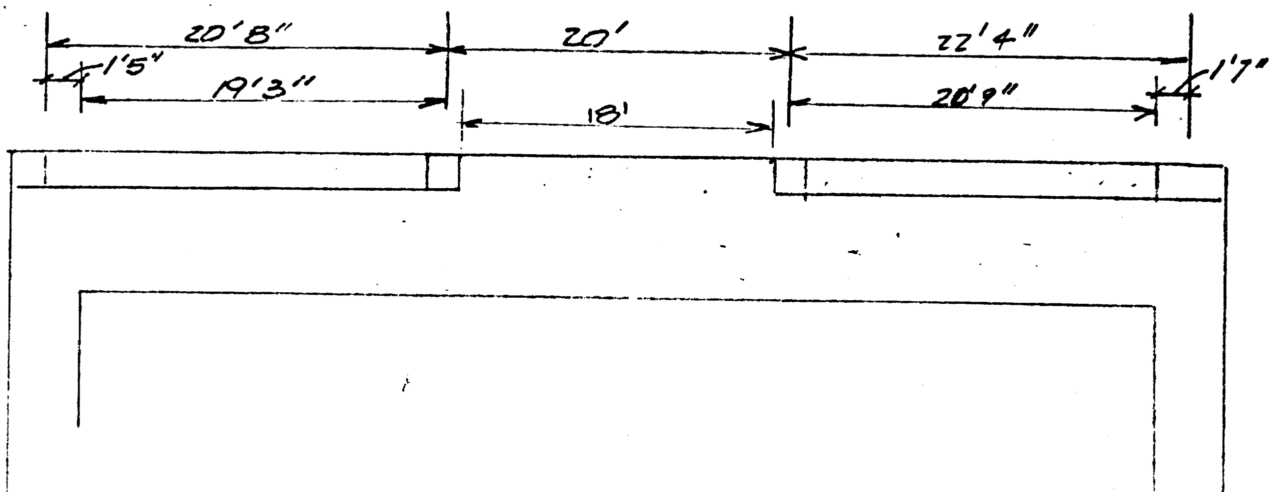
$$M_u = (.9)(6.25)(60)\left(69 - \frac{7.35}{2}\right) / (12''/') = \underline{1837.26}^{FT-K}$$

$\cong 52\% -M_{CAP}$

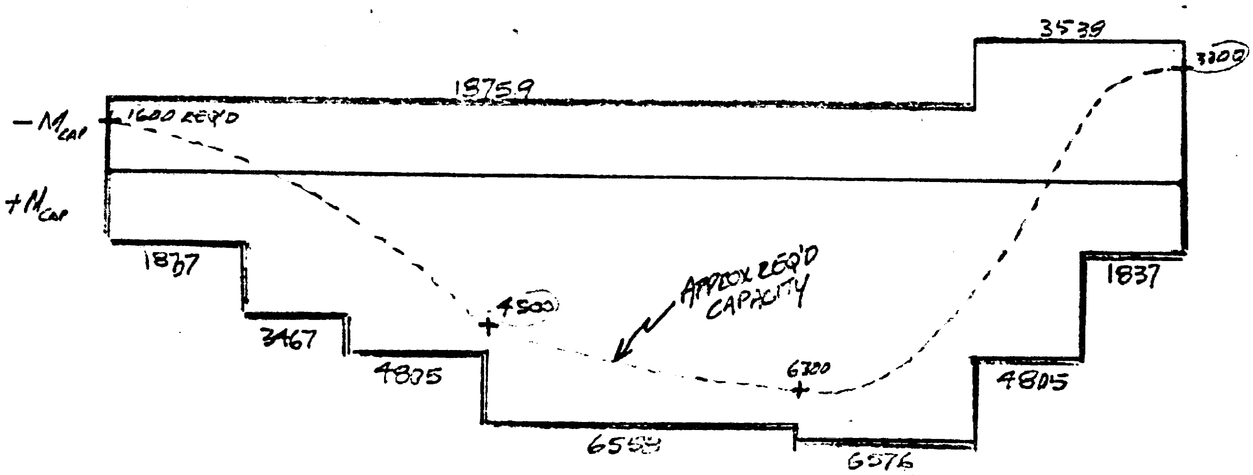
A-61

PROJECT CRD	ACC. NO. [illegible]
SUBJECT [illegible]	SHEET NO. ___ OF B-104
	DATE [illegible] 19[illegible]
COMP. [illegible] CHECK KS	CONT. NO. 1

Moment Capacity Provided in Girder +M & -M



$-M_{cap}$ FT-K	1875.9					3539	
$+M_{cap}$ FT-K	1837.3	3467.8	4805	6558.75	6576.0	4805	1837.3
	5'	5'	10'3"	18'	11'9"	5'	5'



TOP STEEL	4#11					4#11		UPPER LAYER
						+4#11		LOWER LAYER
BOT STEEL			4#11	8#11	8#11	4#11		UPPER LAYER
	4#11	8#11	2#11	4#11	8#11	8#11	4#11	LOWER LAYER

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A-62

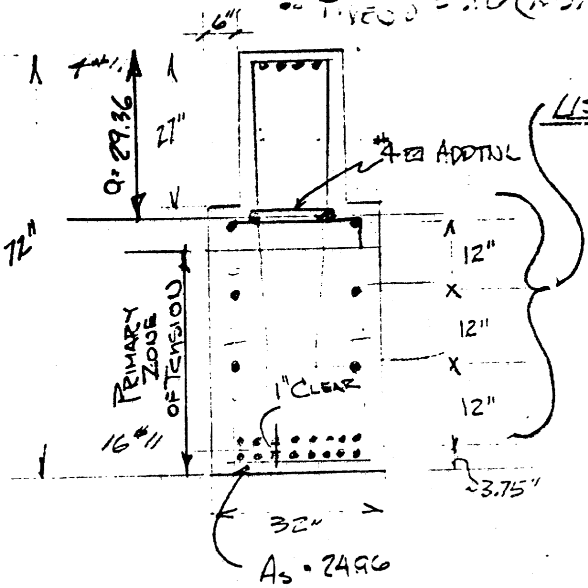
11-72B

PROJECT CRD | ACC. NO. [illegible] | SHEET NO. ___ OF B-105
SUBJECT [illegible] | DATE [illegible] 19[illegible]
COMP. [illegible] | CHECK [illegible] | CONT. NO. 1

ACI 10.6.5

LONGITUDINAL STEEL DISTRIBUTED IN WEB
TOTAL AREA = 10% OF MAIN TENSION REINF

∴ $A_{REQ'D} = .10(25.00) = 2.5$ sq in.



USE: 8#6 FOR ADDTNL WEB REINF

$A_s = 3.53 = 14\% A_s$

by WHR 9-10-73

FLEX. DIST.

⊕ SECTION

$d_c = 1.5" + .625 + 1.410 + .50$
$= 4.04$

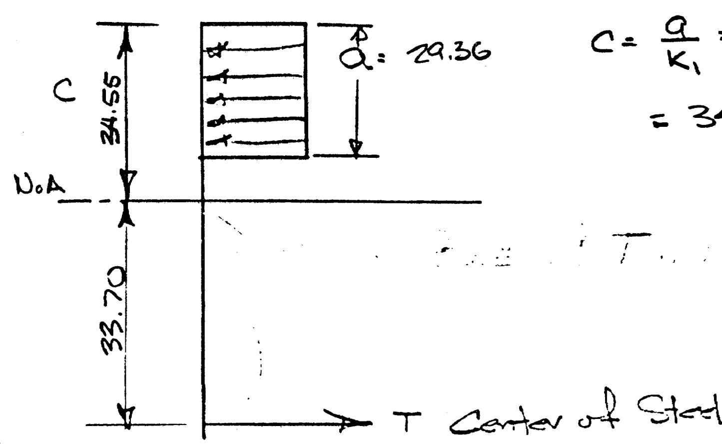
$A = \frac{8.08" \times 32"}{16} = 16.16$

$z = 36\sqrt[3]{16.16 \times 4.04} = 36 \times 4.02$
$= 145 < 175$ OK
INT. EXPOSURE

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{24.96 \times 60}{.85(3)(20)} = 29.36"$

C =

$10\% A_s = .10(24.96)$
2.496



$C = \frac{a}{k_1} = \frac{29.36}{.85}$
$= 34.55"$

A-63

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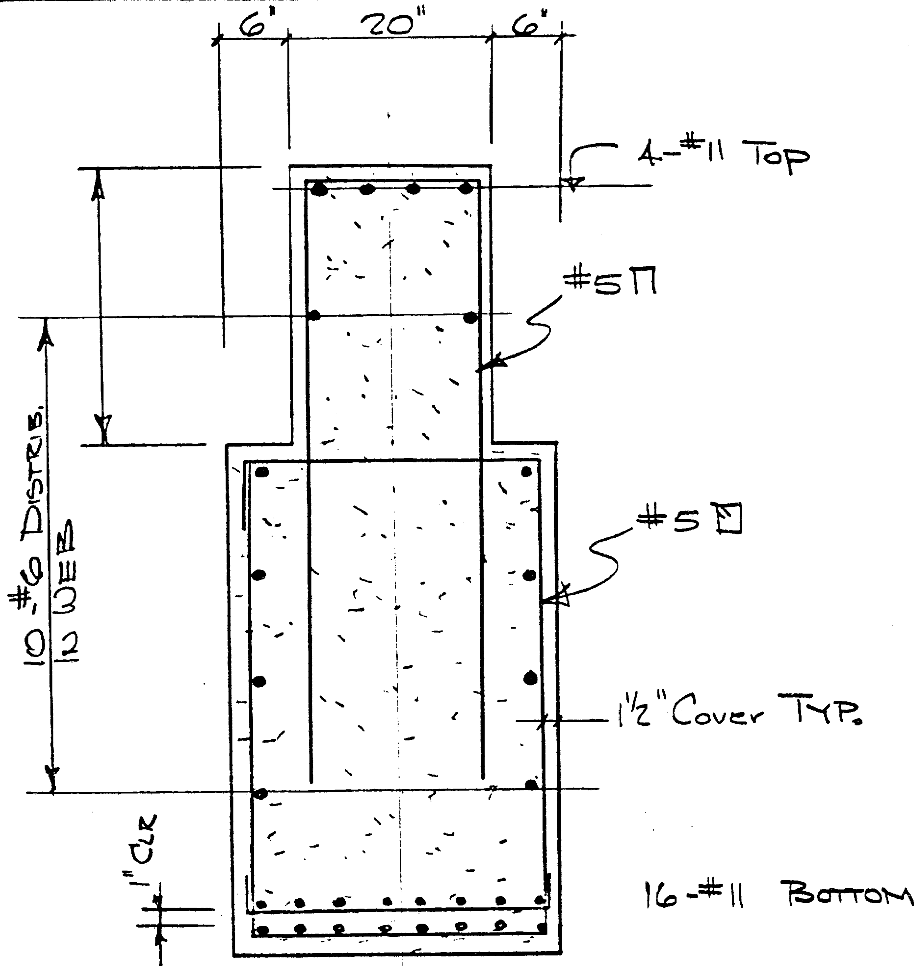
11-72B

PROJECT C.R.D ACC. NO. 1856.70

SUBJECT SHEET NO. ___ OF B-106

DATE 9-10 1973

COMP. WHR CHECK CONT. NO.



SECTION

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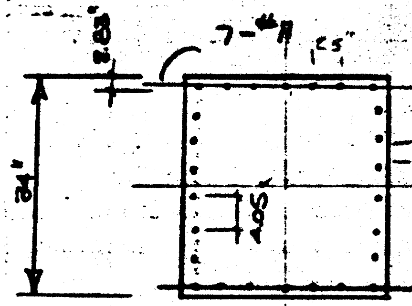
11-72B

PROJECT CRD ACC. NO. 656C67 *

SUBJECT A-Line & Typ Int Frame SHEET NO. __ OF CC-4A

DATE 10-4 1973

COMP. WHR CHECK __ CONT. NO. __



P = 445
Mx = 1777
My = 127

$E_{C\,5000} = 3605$ ✓ $A_B = 1.56$

I_{SE} =	1.56 (14) ×	14.17^2	=	4385
	1.56 (4) ×	10.12	=	639
	1.56 (4) ×	6.07	=	230
	1.56 (4) ×	2.02	=	25
	26 Bars			5279 ✓

26 Bar Arrangement

$$EI = \frac{\frac{3605(104{,}811)}{5} + 29{,}000(5279)}{1.55} = 14.75\times10^7$$ ✓

$$P_{cr} = \frac{\pi^2 EI}{(K\cdot L\cdot 12)^2} = \frac{\pi^2(14.75\times10^7)}{(1.1\times34.17\times12)^2} = 7155$$ ✓

Using P = 445k

$$\delta_m = \frac{1.0}{1-\frac{812}{.7(7155)}} = 1.19$$ ✓

→ Use Larger P To Yield higher δ_m Conservative

{ δ_m = 1.098 ✓ for P = 445
δM = 1951 ✓ Actual }

M = 1.19 (1777) = 2115 1k

$\Theta = \tan^{-1}(127/2115) = 3.44$ (3.72)

$M_{u\Theta} = (127^2 + 2115^2)^{1/2} = 2119$ 1k (1955)

K = 445 / 4×32×34 = .102 ✓   Ke/t = 2115×12 / 4×32(34)² = .172 ✓
(.159)

Biaxial #15 & #16

g	P_{tm}	
g = .80	P_{tm} = .84	.73
g = .90	P_{tm} = .73	.63
g = .833	P_{tm} = .803	.73

Uniaxial #63

g	P_{tm}	
g = .80	P_{tm} = .70	(.63)
g = .90	P_{tm} = .64	(.55)
g = .833	P_{tm} = .67	(.60)

P_{tm} = .67 + (3.44/45)(.803 − .67) = .68
(.60 + (3.72/45)(.73 − .60) = .61)

P = .68/17.65 = .0385 = 41.89/1.56 (26.85)

P = .61/17.65 = .0346 = 37.65/1.56 (24.13)

Try 26-#11 Bars

To Be Checked By PCA Col Design Program

P.C.A. - U.S.D. OF R.C. COLUMNS

CRD COLUMN DESIGN TYP INT FRAME & A-LINE 32X34

INVESTIGATION OF
TIED COLUMN

B= 32.00 T= 34.00 FC= 4.000 FY= 60.000 PHIC= 0.700 PHIB= 0.900

WITH 26 NO.11 BARS. AST = 40.55 SQ.IN. = 3.73 PCT. COVER = 2.125 IN.

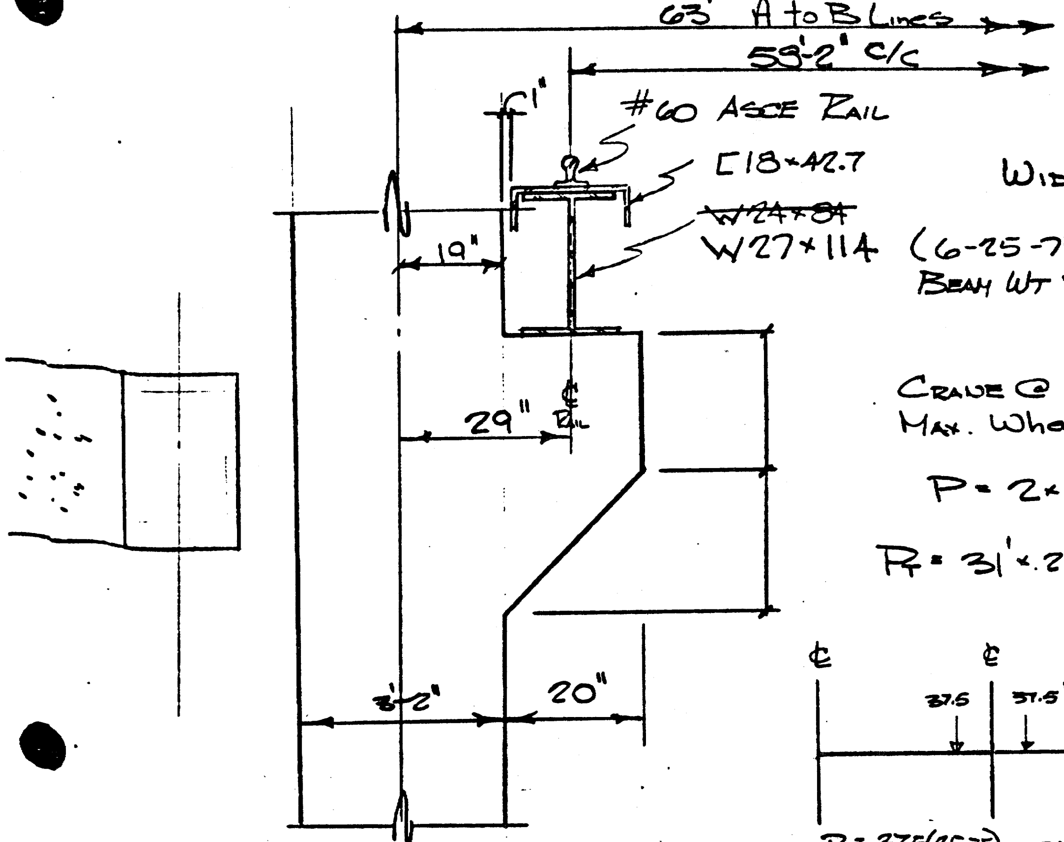
	ROW 1	ROW 2	ROW 3	ROW 4
BARS	7 NO.11	7 NO.11	6 NO.11	6 NO.11
COVER	2.125	2.125	2.125	2.125

LOAD CASE	APPLIED FORCES AP	AMX	AMY	ULTIMATE CAPACITY UP	UMX	UMY	UP/AP
1	812.	1298.	233.	1213.	1940.	348.	1.495
2	445.	1951.	127.	520.	2282.	149.	1.170
3	621.	1350.	281.	926.	2010.	418.	1.489
4	782.	1590.	275.	997.	2026.	350.	1.274

ok

REDESIGNED COLUMNS

PROJECT	C.R.D	ACC. NO.	1856.26
SUBJECT	Design of Corbel	SHEET NO. ___ OF	IC-6
		DATE	2-16 1973
COMP. WHK	CHECK KS	CONT. NO.	



d.ff = 63'-0"
4'-10"
58'-2"

Width of Bearing is 30"

Beam Wt = 90+45+60 = 195

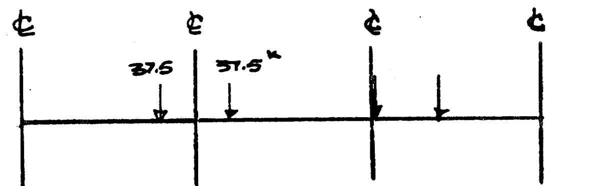
Use 200#/1

Crane @ Corbel — Impact

Max. Wheel = 30k × 1.25 = 37.5

P = 2 × 37.5 = 75k

Pf = 31' × .2k/1 = 6.2k
+ 75.0k



$P/2 = \frac{37.5(25.75)}{31} = 31.15$ $P = 37.5 + \frac{37.5(20.5)}{31}$ 24.8

P = 62.3k

P = 62.3

* Therefore Stress Runs Are Conserv ok!

Finally $P_u = 1.7(62.3 + 6.2) = 116.5^k$

& $M_u = 29''(116.5) = 3377.05''^{-k}$

Section 10.14.1 ACI

Allow Brg $.85\phi f'_c = .85(.7)(3000) = 1785^{psi}$

Area Req = 116.5/1.785 = 65.3 in²

$L_{brg} = 30''$ W < 10" Given

∴ Shear Span = 10" = a

1st Trial

Assume A/D = .25

d ≈ 40"

$N_u = 116.5^k / .85 \times 30'' \times 40 = 114.2$ from Design Aid

increase by ½ = 152.3^{psi}

Use Min of .04 (3/60) = .002 But Not < .04

A-67

PROJECT C.R.D ACC. NO. 1356.26

SUBJECT Design of Corbel SHEET NO. ___ OF IC-7

DATE 2-16 1973

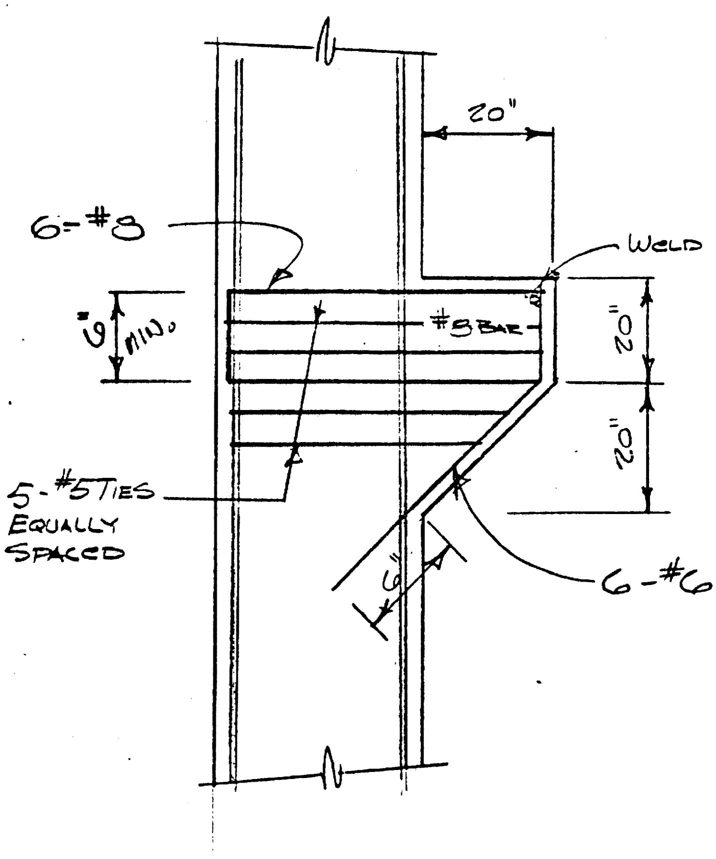
COMP. W.H.R CHECK KS CONT. NO. ___

Use $P_{min} = .004$ $A_s = .004(30'')(40'') = 4.8''$

Use #7 = 4.8/.60 = 8 - #7

#8 = 4.8/.79 = 6 - #8

#9 = 4.8/1 = 5 - #9



$$l_d = \frac{.04(0.79)(60{,}000)}{\sqrt{3000''}} = 34.6''$$

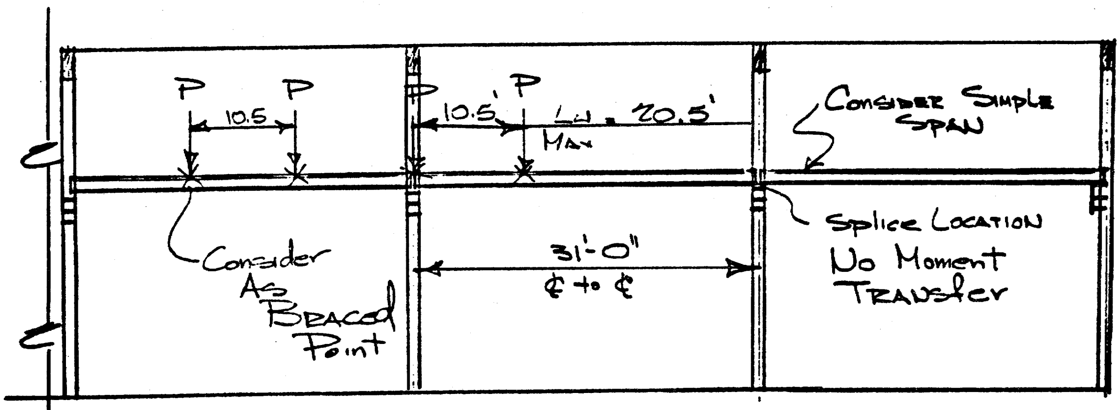
$$A_v = .002(40)(30) = 2.4 \text{ in}^2$$

PROJECT: C.R.D ACC. NO. 1856.26

SUBJECT: CRANE RAIL DESIGN SHEET NO. ___ OF CR-1

DATE 2-23 1975

COMP. WHR CHECK KS CONT. NO. ___



CRANE RAIL - LOADS 20T CRANE, W/2-Wheels on Rail

(A) Max Load/wheel = $30^k \times 1.25$ (Impact Factor) = 37.5^k

(B) Say Beam + Rail = $200^{\#/1}$

(C) Horizontal Load $30^k \cdot .2 = 6^k$/wheel

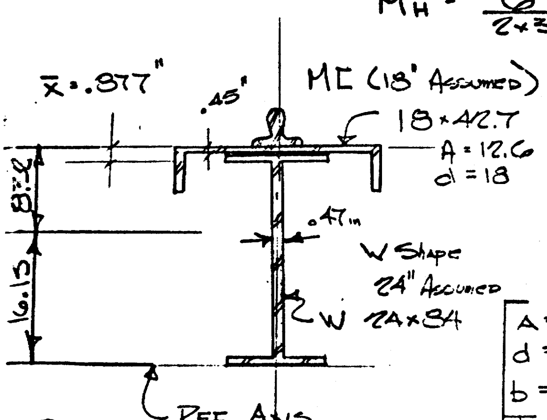
Maximum Moment

Vert $M_v = \frac{P}{2L}\left(l - \frac{a}{2}\right)^2 = \frac{37.5}{2\times31}\left(31 - \frac{10.5}{2}\right)^2 = 401^{1-k}$

$+ .20(31^2)/8$ $+ 24^{1-k}$

Say 425^{1-k}

$M_H = \frac{6}{2\times31}\left(31 - \frac{10.5}{2}\right) = 64^{1-k}$



A = 24.7 in²
d = 24"
b = 9" t_b = .77 in
I_{xx} = 2370 in⁴
I_{yy} = 94.5

Lu_{max} = 31'-0" - 10.5 = 20.5'

Shape	A	Y	AY	I_{axis}
Channel	12.6"	24.23	305.3	14.4 I_{yy}
W	24.7"	12"	296.4	2370
	37.3		601.7	2384.4

$Y_b = \frac{601.7}{37.3} = 16.13$ in.

Transfer of Channel

$I_{xx_T} = 2384.4 + (12.6 \times 8.1^2)$ (826)
$= 3211$ in⁴

$I_{yy_T} = I_{yy_w} + I_{xx}c = 94.5 + 554$
$= 648.5$

A-69

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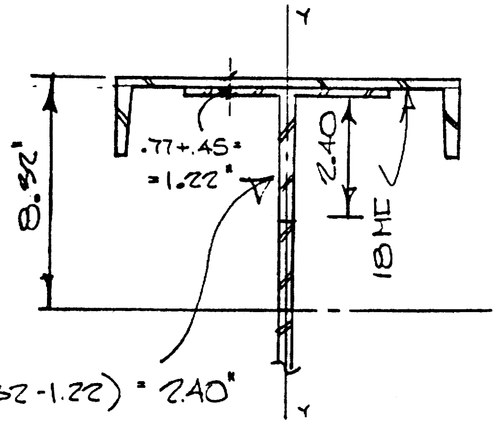
PROJECT C.R.D ACC. NO. 1956.26

SUBJECT CRANE RAIL DESIGN SHEET NO. ___ OF CD-2

DATE 2-23 1973

COMP. WHR CHECK KS CONT. NO. ___

r_t = (by AISC Def RADIUS of GYRATION of A Section Comprising the Compression flange plus 1/3 of the Compression Web Area, taken About An Axis In the plane of the Web)



$$r_T = \sqrt{\frac{I}{A}}$$

$$A_T = A_C + 9(.77) + (.47)(2.40)$$
$$= 12.6 + 6.93 + 1.13$$
$$= 20.66 \text{ in}^2$$

$$I = I_{xx}^C + 1/12(.77)(9)^3 + 1/12(2.4)(.47)^3$$
$$= 554 + 46.78 + \text{NEG}$$
$$= 600.78$$

$$r_T \therefore = \left(\frac{600.8}{20.66}\right)^{1/2} = 5.39 \text{ in}$$

(A-36 Steel)

Check $L/r_T = \frac{20.5 \times 12}{5.39} = 45.64$

& C_b = Unity = 1.0

$L/r_T < 53\sqrt{C_b} = 53$ $\therefore F_b = .60 F_y$
(Pg 5-70)

$F_{b\,comp} = 22^{ksi}$

$F_{b\,ten} = 22^{ksi}$

Check Stresses

Comp FLG $f_b = \frac{425^{1\text{-}k} \times 12 \times (8.32)}{3211} + \frac{64^{1\text{-}k} \times 12 \times 9''}{648.5}$

$= 13.21 + 10.66^{ksi} = 23.87^{ksi} > 22$ NG.

Tension Flange $f_b = \frac{425 \times 12 \times (16.13)}{3211} = 25.62^{ksi} > 22$ NG.

Note: Increased Beam Size Is Required

Also - Check I_{REQ} for deflection with Limits for $\frac{L}{720} = \frac{31 \times 12}{720} \approx .50$ Use 1/2"

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PROJECT C.R.D. ACC. NO. 1856.76

SUBJECT CRANE RAIL DESIGN SHEET NO. ___ OF CE-3

DATE 7-23 19 73

COMP. W.H.K. CHECK ____ CONT. NO. ____

Δ - Check : Determine Section REQUIRED

$$\Delta = \frac{Pa}{24EI}(3L^2 - 4a^2) \quad \text{where } \Delta = 1/2'',\ a = 10.25',\ L = 31'$$

$$.50 = \frac{37.5(10.25\times12)}{24(29000)\,I_{REQ}}\left(\overset{415152}{3(31\times12)^2} - 4\overset{60516}{(10.25\times12)^2}\right)$$

$$I_{REQ} = \frac{37.5(10.25\times12)(354636)}{24\times29000\times.5} = \underline{\underline{4700\ in^4}}$$

for L/720

Consider W27×102 & MC18×42.7 & #60 ASCE Rail

for W27×102: A = 30.0, d = 27.0, b - 10" b_f = .827", I_{xx} = 3610, I_{YY} = 139, t_w = .518

MC 18×42.7: A = 12.6, d = 18, t = .45, I_{xx} = 554, I_{YY} = 14.4

Shape	A	Y	AY	I axis
Channel	12.6	27.23	343.1	14.4
W	30.0	13.5	405	3610
	42.6		748.1	

d = 27.45"

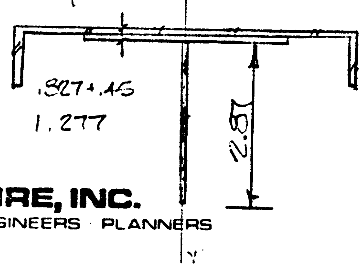
$$\bar{Y}_1 = \frac{748.1}{42.6} = 17.56 \quad \& \quad Y_2 = 9.89$$

(A) $I_{xx} = 3610 + 14.4 + 12.6\times\overset{1178}{(9.67)^2}$

$= \underline{4802\ in^4}$

(B) $I_{YY} = 139 + 554 = \underline{693\ in^4}$

Recompute R_T



Comp Web 9.89"

1/3(9.59-1.23) -

A_T = 12.6 + 10(.827) + (.518)(2.87)
12.6 + 8.27 + 1.49 = 22.36 in^2

$I = 554 + 1/12(.827)(10)^3 + 1/12(2.87)(.58)^3$
554 + 68.9 +
= 622.9

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A-71

11-72B

PROJECT C.P.D ACC. NO. 1856.26
SUBJECT CRANE RAIL DESIGN SHEET NO. ___ OF CC-4
DATE 3-1 1973
COMP. WoH-R. CHECK ___ CONT. NO. ___

$\therefore \quad r_T = \sqrt{\frac{622.9}{22.36}} = 5.27\text{ in} \quad \& \quad L/r_T = \frac{20.5 \times 12}{5.27} = 46.7 < 53$

$Fb_{comp} = 22^{ksi} \qquad Fb_{tension} = 22^{ksi}$

COMPRESSION FLANGE

$f_b = \frac{425 \times 12 \times 9.89}{4802} + \frac{64^{1\text{-}k} \times 12 \times 9''}{693}$

$10.5 + 9.97 \qquad = 20.5^{ksi} < 22^{ksi}$ OK ✓

TENSION FLANGE

$f_b = \frac{425 \times 12 \times 17.56}{4802} = 18.65^{ksi} < 22$ ALLOW OK ✓

Shear Stress $= \frac{65.4^K}{(23.88)(.518)} \qquad T = 23\frac{7}{8}''$ (from Pg 1-13 AISC)

$\tau = 5.29^{ksi} < 14.5^{ksi}$ ALLOW

$\Delta = \frac{Pa}{24EI}(3L^2 - 4a^2) = \frac{K}{I} = \frac{2350}{I}$

$\therefore \quad \Delta = \frac{2350}{4802} = .489'' \approx 1/2''$ or $L/720$

Check Stiffner REQUIREMENTS $\qquad V_{max}$

CONSIDER END PANEL $\quad A_w = (23.88)(.518) = 12.37\text{ in}^2$
$f_v = 3.28^{ksi}$

(a) h/t RATIO $= 27''/.518 = 52 < 260$

$a/h = \frac{31 \times 12}{27} = 13.78 > 1$

$k = 5.36$ → Assume $C_v > .8 \therefore C_v = \frac{190}{h/t}\sqrt{\frac{K}{F_Y}} = 1.41$

By Form 1.10-1 $F_v = \frac{F_Y}{2.89}(1.41) = 17.6$ BUT

(b) NOT > 14.

No Intermediate Stiffners REQUIRED

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PROJECT	CRD	ACC. NO.	1856.70
SUBJECT	REVISED CRANE RAIL	SHEET NO. ___ OF	CRR-1
		DATE	6-25 1973
COMP.	WHP CHECK KS	CONT. NO.	

BASED ON SURVEY of SEVERAL CRANE MANUFAC.
SEE CHRIS HIGGINS Pg 1F-9 IN COMPUTER SET-UPS

NEW CRITERION $P_v = 43^{k/\text{WHEEL}}$ $a = 10$

(A) $M_{MAX} = \frac{43}{2 \times 31}(31 - \frac{10}{2})^2 = 469 + \frac{wL^2}{8} = .2(31)^2/8 = 24$ (676; Approx)

$= \underline{493^{1\text{-}k}}$

(B) HORIZONTAL LOAD $= 34.5^k \times .2 = 6.9^k$

$M_H = \frac{6.9}{2 \times 31}(676) = \underline{75.2}^{1\text{-}k}$

CHECK STRESSES FOR EXISTING CRANE RAIL
W27×102 & MC18×42.7 & #60 ASCE RAIL

SEE CR-3 & CR-4

COMPRESSION FLANGE

$f_{b_c} = \frac{493 \times 12 \times 9.89}{4802} + \frac{75.2 \times 12 \times 9}{693}$

$12.18 + 11.72 = 23.9^{ksi}$

TENSION FLANGE

$f_{b_t} = \frac{493 \times 12 \times 17.56}{4802} = 21.63 < 22^{ksi}$

USE NEXT LARGER W27 SECTION

TRY A W27×114 & MC 18×42.7 × #60 ASCE RAIL
w = 114 + 42.7 + 85/3 = $22.3^{1\text{-}k}$
+ 469
SAY $492^{1\text{-}k}$

$T_T = 27.73''$
$27\frac{3}{4}'$

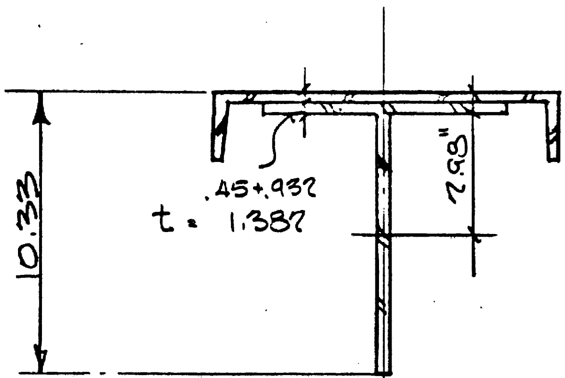
SHAPE	A	Y	AY	I
CHANNEL	12.6	27.44	345.7	
WF	33.6	13.64	453.3	
	46.2		804	

$\bar{Y}_B = 17.4$

$\bar{Y}_T = 10.33$

PROJECT C.R.D ACC. NO. 185670
SUBJECT CRANE RAIL SHEET NO. ___ OF CRR-2
DATE 6-25 1973
COMP. WR CHECK KS CONT. NO. ___

Compute R_T



$I_x = 4090 + 14.4 + (12.6)(9.453)^2$ (1126)
$5230 in^4$

$I_y = 554 + 159 = 713$

Effect Comp Web

$\frac{1}{3}(10.33 - 1.382) = 2.98 in$

$A_{TOT} = 12.6^{in^2} + (10.07)(.932) + .57(2.98)$ (Flg)
$12.6 + 9.39 + 1.70$
$= 23.69 in^2$

$I = 554 + \frac{1}{12}(.932)(10.07)^3 + \frac{1}{12}(2.98)(.57)^3$
$= 554 + 79.3 + Neg$
$= 633.3$

$R_T = \sqrt{\frac{633.3}{23.69}} = 5.17^{in}$

& $\frac{L}{R_T} = \frac{21 \times 12}{5.17} = 48.7 < 53$

$f_b = 22^{ksi}$

Therefore Stress

(A) Comp Flg

$\left[\sigma = \frac{Mc}{I}\right]$

$f_b = \frac{492 \times 12 \times 10.33}{5230} + \frac{64 \times 12 \times 9}{713}$

$= 11.66 + 9.69 = 21.35^{ksi} < 22$

(B) Tension Flg

$f_b = \frac{492 \times 12 \times 17.4}{5230} = 19.64^{ksi} < 22$

$\Delta_v = \frac{Pa}{24EI}(3L^2 - 4a^2) = \frac{K_v}{I_x} = \frac{2563^{\pm}}{5230} = .49" \approx \frac{L}{759}$

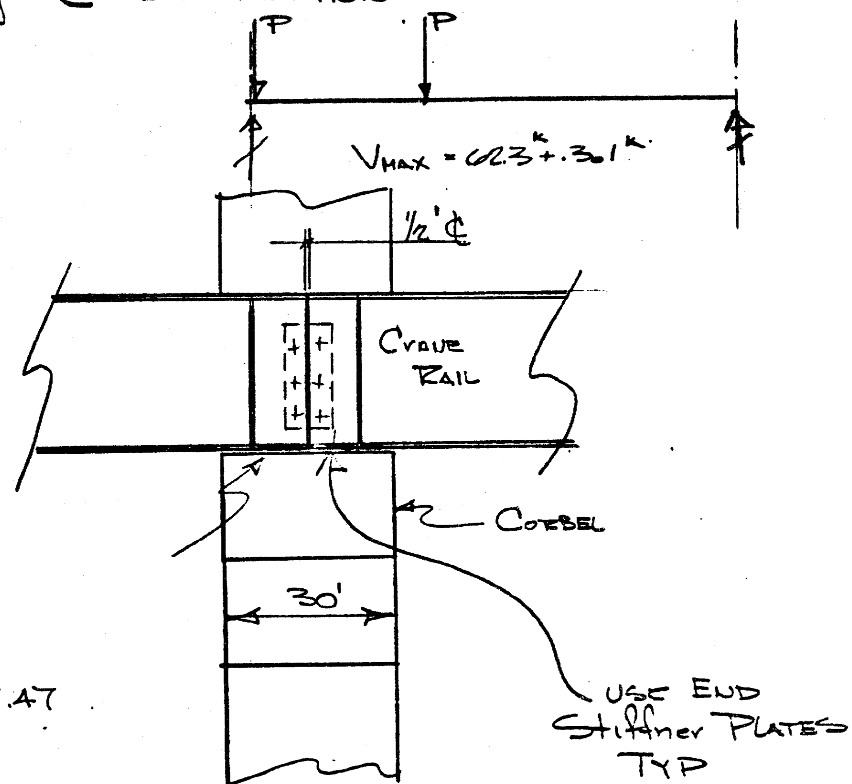
$\Delta_H = \frac{K_H}{I_y} = \frac{411}{713} = .58' = \frac{L}{640}$

USE W27×114 & MC 18×42.7 + #60 ASCE RAIL

PROJECT C.R.D ACC. NO. 1856.26
SUBJECT CRANE RAIL DESIGN SHEET NO. ___ OF CR-5
DATE 3-1 1973
COMP. W.H.R. CHECK ___ CONT. NO. ___

Check Web Crippling @ End Reaction

$R = 65.4^k$
$k = 1\frac{5}{8}"$
$N = 10"$



(A) $\frac{R}{t(N+k)} \leq .75 F_Y$

$\frac{65.4^k}{.518(10+1.625)}$

$= 10.86^{ksi} \leq 27^{ksi}$

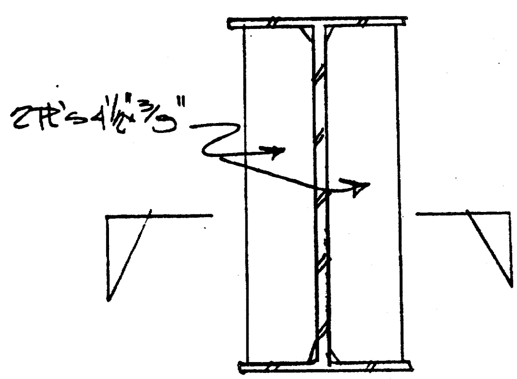
(B) $\left[2 + \frac{4}{(a/h)^2}\right] \frac{10{,}000}{(h/t)^2}$

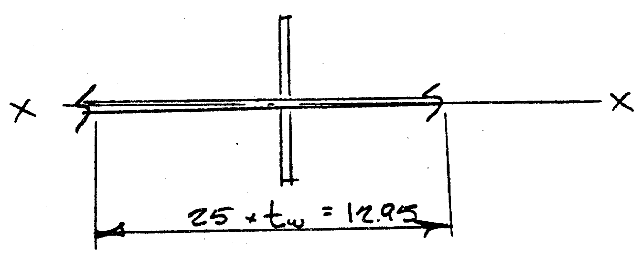
$\left[2 + \frac{4}{(13.70)^2}\right] \frac{10{,}000}{(52)^2} = 7.47$

$\frac{65.4}{.518(27)} = 4.676 < 7.47$

No Brg Stiffner Required

But Use w/t ≥ 15.8 Try 2 Pl's 4" × ⅜" @ End Nominal Stiff





$I = \frac{1}{12}(.375)(8.52)^3 = 19.33$
$A = 8(.375) = 3$
$r = \sqrt{\frac{19.33}{3}} = 2.54$

$\frac{KL}{r} = \frac{.75(27)}{2.54} \approx 8.00$ ∴ $F_A = 21.25$

A-75

$P_{ALL} = 21.25 \times 3 = 63.75 \approx 65$

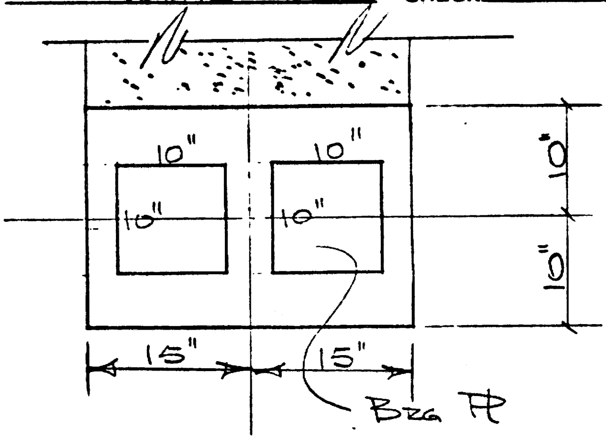
∴ Use 4¼" × ⅜" Plates

PROJECT C.R.D ACC. NO. 1?5?2?

SUBJECT Crane Rail SHEET NO. ___ OF CR-6

DATE ______ 19__

COMP. WHR CHECK RS CONT. NO. ______



½

℄ Crane Rail & Brg

Allow Bearing
= .375 f'c
= 1.125 ksi

$A_{REQ} = \frac{65.A}{1.125} = 57.$

Min 6×10" or 8"×8"
Use 10×10
∴ $f_p = .654$ ksi

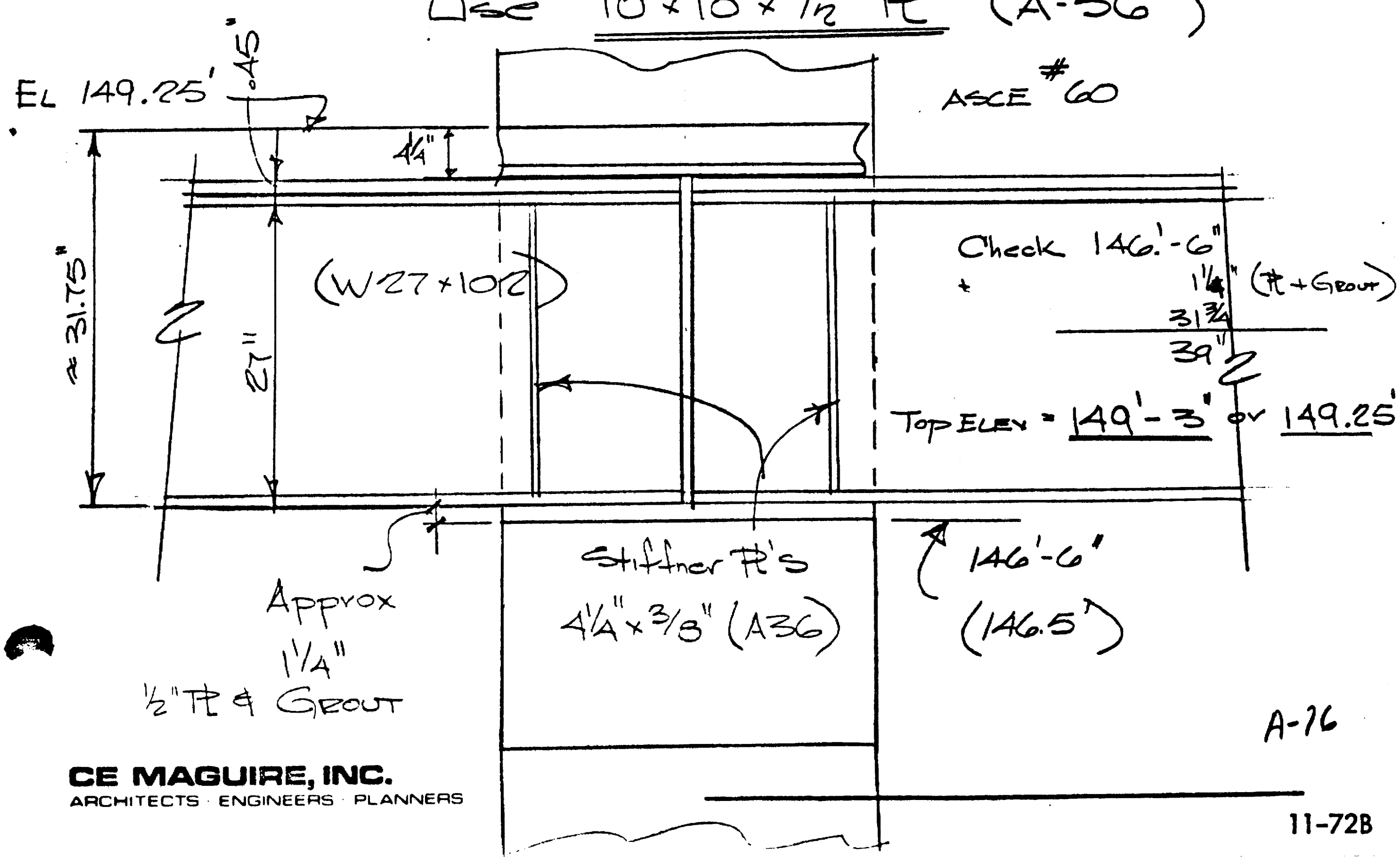
$n = 10/2 - 1.625 = 3.375$

$t = \sqrt{\frac{3 f_p n^2}{F_b}} = \sqrt{\frac{3(.654)(3.375)}{27}}$

$= .4952$

Web Crippling has Been Checked & Is Ok therefore

Use <u>10"×10"×½" PL</u> (A-36)



A-76

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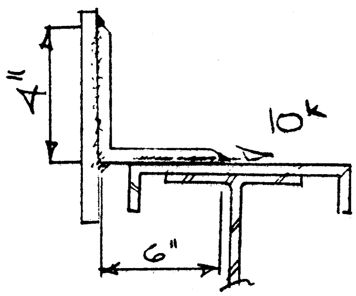
11-72B

PROJECT C.R.D ACC. NO. 12-56.86

SUBJECT Crane Tie Back SHEET NO. ___ OF CR-7

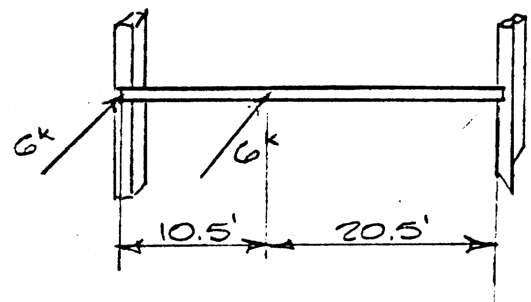
DATE 3-2 19__

COMP. W.H.R CHECK ______ CONT. NO. ______



Max Reaction

$R_H = 6^k + \frac{20.5}{31.0}(6)^k \approx 10^k$



$f_c = f_T = .6 f_y = 22$ KSI

Assume 1/4" th $A_{REQ} = \frac{10^k}{22} = .455_{in^2}$ $L_{REQ} = \frac{.455_{in^2}}{.25"} = 2"$

Use ∠ 6×4×1/4" × 4" Long

Plate In Conc. Col = 6×6×3/8"

Weld All Around

✳ Primary Purpose Of Angle Is To Brace Compression Flange of Crane Beam; Loads Are Very Small Therefore

Use 6×4×1/4" ∠ with 6×6×3/8" Plate In Column

A-77

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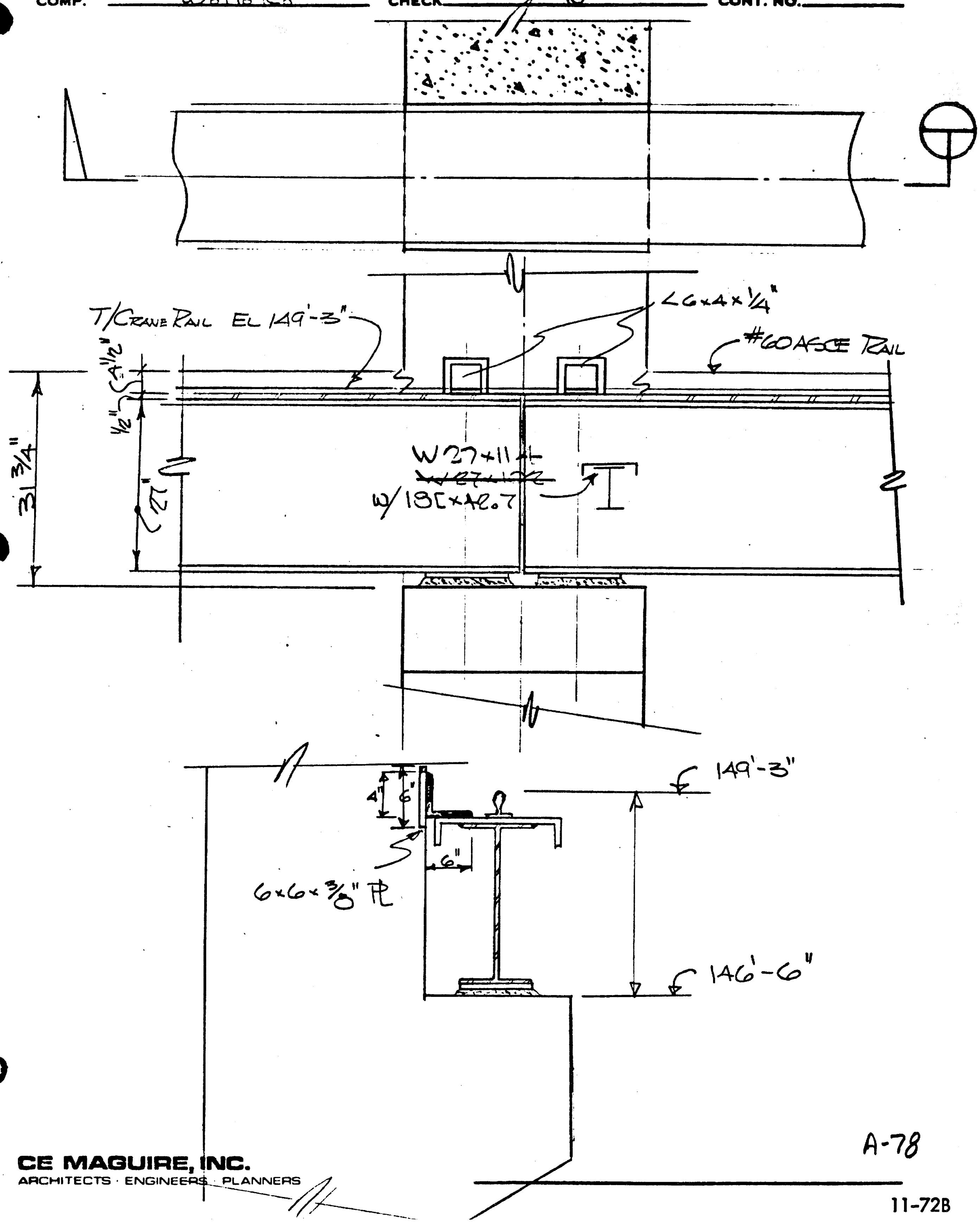
11-72B

PROJECT CoRoD
SUBJECT CRANE RAIL - TIE-BACK
COMP. W.H.R. CHECK KS
ACC. NO. 1856.26
SHEET NO. ___ OF CR-8
DATE 3-2 19 73
CONT. NO. ___



A-78

CE MAGUIRE, INC.
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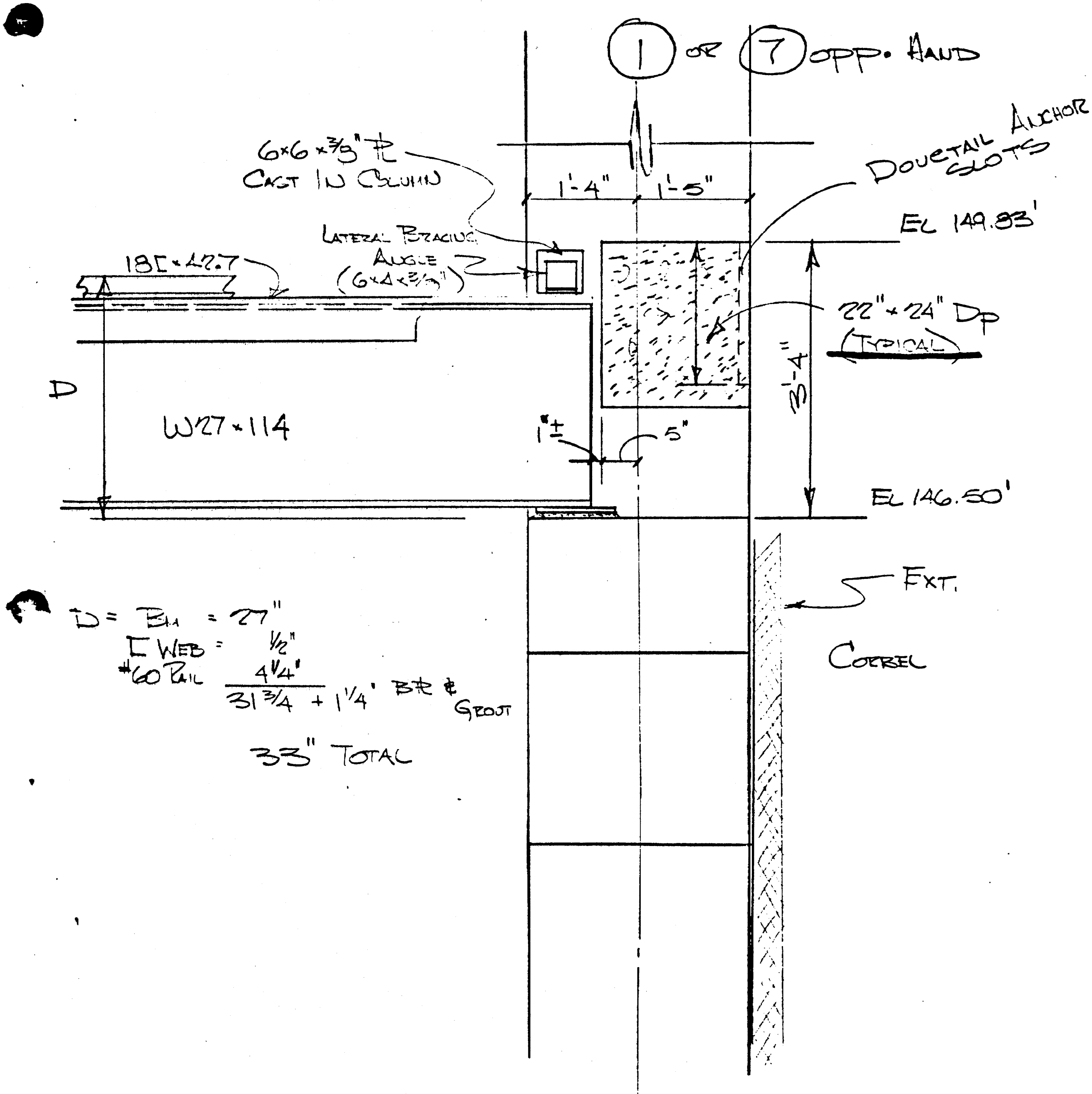
PROJECT C.R.D. | ACC. NO. 1856.70

SUBJECT Crane Rail Detail - Atrium. | SHEET NO. ___ OF B-1

DATE 5-23 19 73

COMP. WHC | CHECK KS | CONT. NO. ___



Crane Rail Detail

A-79

PROJECT CRD ACC. NO. 185670
SUBJECT Revised 6" Brick Curtain Wall SHEET NO. ___ OF RA-1
DATE 10-4 1973
COMP. WHR CHECK ___ CONT. NO. ___

Consider Wall As A Simple Span

Max Clear Span = 13'

Wind = 35 psf

Seismic = .2×60 = 12 psf

$f_t = .112(1.33) = .149$

$S = \frac{I}{C} = 63.3$

Mom = $.035(13)^2/8 = .739^{1\text{-}k}$

(A) $f_{bT} = .739 \times 12 / 63.3 = .140^{ksi} < .149$ ok for bending

$v = \frac{.2275}{12 \times 5.625} = 3^{psi}$ ok

$V = \frac{.035(13)}{2} = .2275^k$ @ 121.83'

(1) for Connections $C_p = 2$ W_{scan} 120 psf

S. Factor = 120/35 = 3.43

$V_{conn} = 3.43(.2275) = .78^{k/'}$

3/8" φ Rod A-36
A = .110
I = .000971
r = .0939

$\frac{KL}{r} = 19.17'$

For 2' Spacing

$V_u = 2 \times .78^{k/'} = 1.56^k$

$F_A = 20.66^{ksi}$
$F_T = 22^{ksi}$

$f = \frac{1.56}{.1100}$ 14.19 ksi < ↑ OK

Use 1-3/8" φ Rod @ Pumphouse Floor Level

(2) At Spandrel Levels

$V = 2 \times .2275 = .455$

$V_{conn} = 3.43(.455) = 1.56$

@ 2'-0" Spacing $V = 3.12^k$

$f = \frac{3.12}{2 \times .11} = 14.19$

2-3/8" φ Rod @ Pumphouse Spandrel Level

A-80

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11-728

PROJECT CPD — ACC. NO. 145620

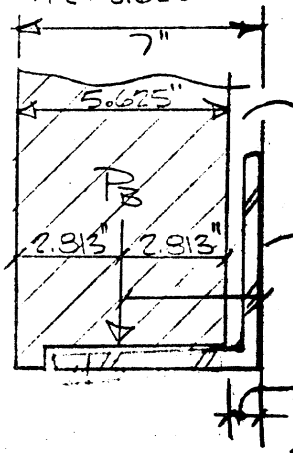
SUBJECT Revised Wall — SHEET NO. ___ OF RA-12

DATE 10-4 1973

COMP. [illegible] CHECK ___ CONT. NO. ___

$\text{Axial Comp} = .060(15^{\pm}) = .9^k$

$f_{comp} = .9/12 \times 5.625 = .013 < 1.26 \text{ Allow}$



for Lu Consider 1.5"

Galvanized $6 \times 6 \times 5/16$

$K = 13/16" = .813$

Bend & Rotation Will Occur @ End of Fillet $L' = 5.187"$

Self Angle

$M = e \times P_B = .90^k \times (1.375 + 2.813) = 3.77^{''-k}$

Try 5/16" Angle (12" Width)

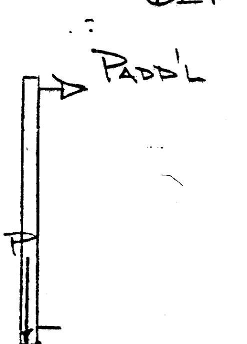
$I = \frac{1}{12}(12)(.3125)^3 = .0305 \text{ in}^4$

$S = .0305/.156 = .196$

$f_b = 3.77/.196 = 19.24^{ksi} < 22$ ok for flexure

$f_v = .90/12 \times .3125 = .24^{ksi} < 14.4^{ksi}$ ok

$\Delta_{est} = \frac{Pb^2}{6EI}(3L - b) = \frac{.9^k (4.188)^2}{6 \times 30{,}000 \times .0305}(3 \times 6 - 4.188)$ [13.812]



$= .01'' \approx 1/64''$ @ free End

Actual = .007

$\Theta_A = \frac{.01''}{6''} = .00167$

$\Delta_{wall} = .00167(15')(12''/1') = .30''$

Movement Resisted By 3/8" ϕ Rod @ Top of Wall Segment

$P_{add'l} = \frac{.9^k \times 2.813'' \times 2' \text{ (Spacing)}}{12' \times 12''/1'} = .035^K$

Negligible

3/8" ϕ Rod Is More Than Adequate To Resist $.035^K$

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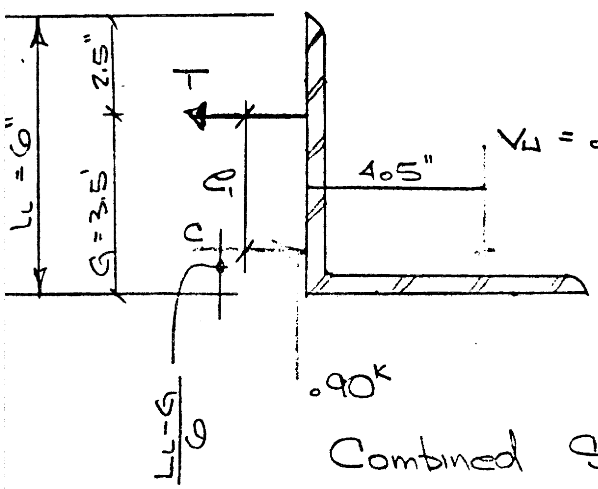
A-81

11-72

PROJECT CRD ACC. NO. 1856.20
SUBJECT Revised Wall SHEET NO. ___ OF RA-3
DATE 10-4 1973
COMP. WHL CHECK ___ CONT. NO. ___

Consider 2'-0" Spacing $G = 3.5"$ Pg A-116



$$\frac{LL-G}{6} = \frac{6-3.5}{6} = .42$$

$$e_1 = 3.5 - .42 = 3.08'$$

$$T = .90^k \times 2 \times \frac{4.5}{3.08}$$

$$= 2.63^k \text{ Tension}$$

Combined Shear and Tension

Pg 5-24 Consider 3/8" φ Bolt A 307

$$F_t = 28 - 1.6 f_v \leq 20^{ksi}$$

$$F_v = .90^k \qquad F_T = 2.63^k$$

Tensile Area = .078 in²
Shear = Root A_k = .068 in²

$$f_v = \frac{.90 \times 2}{.068} = 26.5^{ksi} > 10 \text{ (No Good)}$$

5/8" φ Rod

$A_T = .226$
$A_v = .202$

$$f_v = \frac{.90 \times 2}{.202} = 8.91^{ksi} > 10 \text{ ok}$$

$$F_T = 28 - 1.6(8.91) = 13.744 \quad (14.26)$$

$$f_T = 2.63^k / .226 = 11.64^{ksi} < 13.74^{\text{Allow}}$$

Use 5/8" φ Bolt A-307 @ 2'-0" OC

Minimum Size To Be Used
3/4" φ Bolts To Be Used

A-82

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ARCHITECTS ENGINEERS PLANNERS

11-728

PROJECT: CRD | ACC. NO. 14E6.70
SUBJECT: Revised Masonry Anchors | SHEET NO. ___ OF A-11
DATE 9-17 1973
COMP. ___ CHECK ___ | CONT. NO. ___

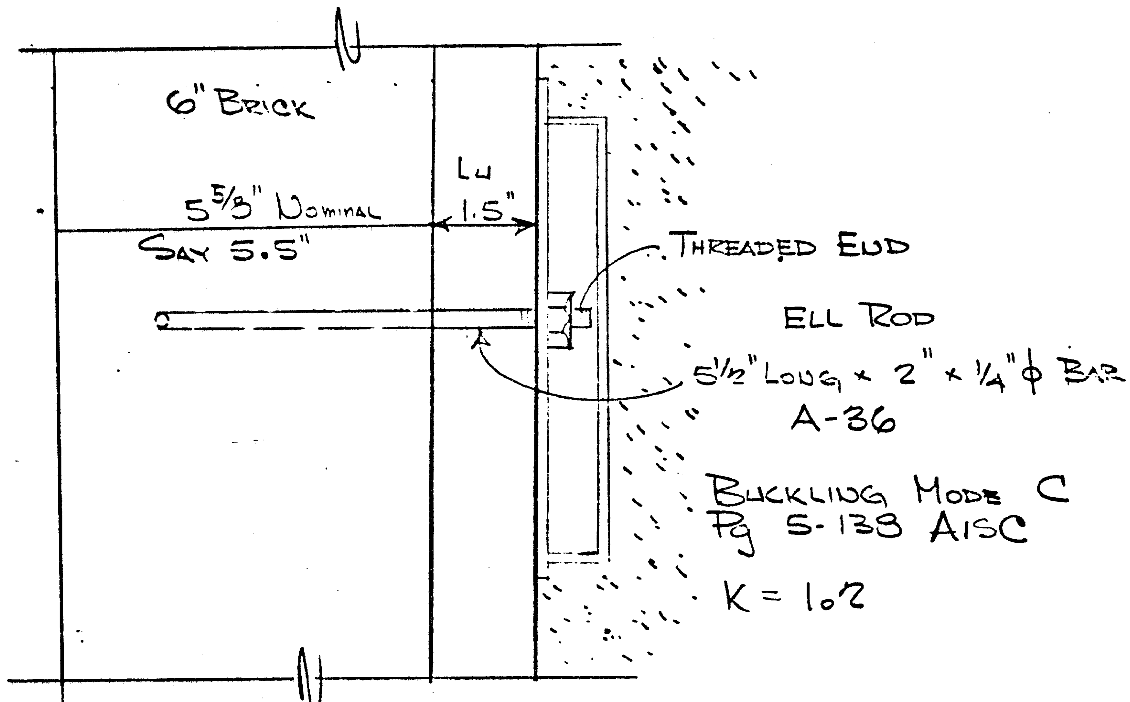
$C_p = 2.0$ For Connections

$F_p = Z \cdot C_p \cdot W_p = 1 \times 2.0 \times .06 = .120^{k/\prime}$

Load Factor = 3.43^k

For Seismic $= 3.43 \times .545 = 1.87^{k/\prime}$

For 2'-0" Spacing $= 3.74^{k/\prime}$



1/4" φ

$A = \frac{\pi d^2}{4} = .0491 \text{ in}^2$

$I = \frac{\pi d^4}{64} = .000192 \text{ in}^4$

$R = \sqrt{\frac{I}{A}} = .0625 \text{ in}$

$\frac{KL}{r} = \frac{1.2(1.5")}{.0625} = 28.8$ Say 29

$\therefore F_A = 20.01$ & $F_T = 22^{ksi}$

W/2-1/4" φ Rods $\frac{3.74^k}{2 \times .0491} = 38.086^{ksi} > 20.01$

$\frac{3.74^k}{3 \times .0491} = 25.400^{ksi} > 20.01$

A-83

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11-72B

PROJECT CRD ACC. NO. 1856.70
SUBJECT SHEET NO. ___ OF A-12
DATE ______ 19__
COMP. WHR CHECK ______ CONT. NO. ______

3/8" φ Rod Required

$A = .110\ in^2$

$I = .000971\ in^4$

$r = \sqrt{\frac{I}{A}} = .0939\ in$

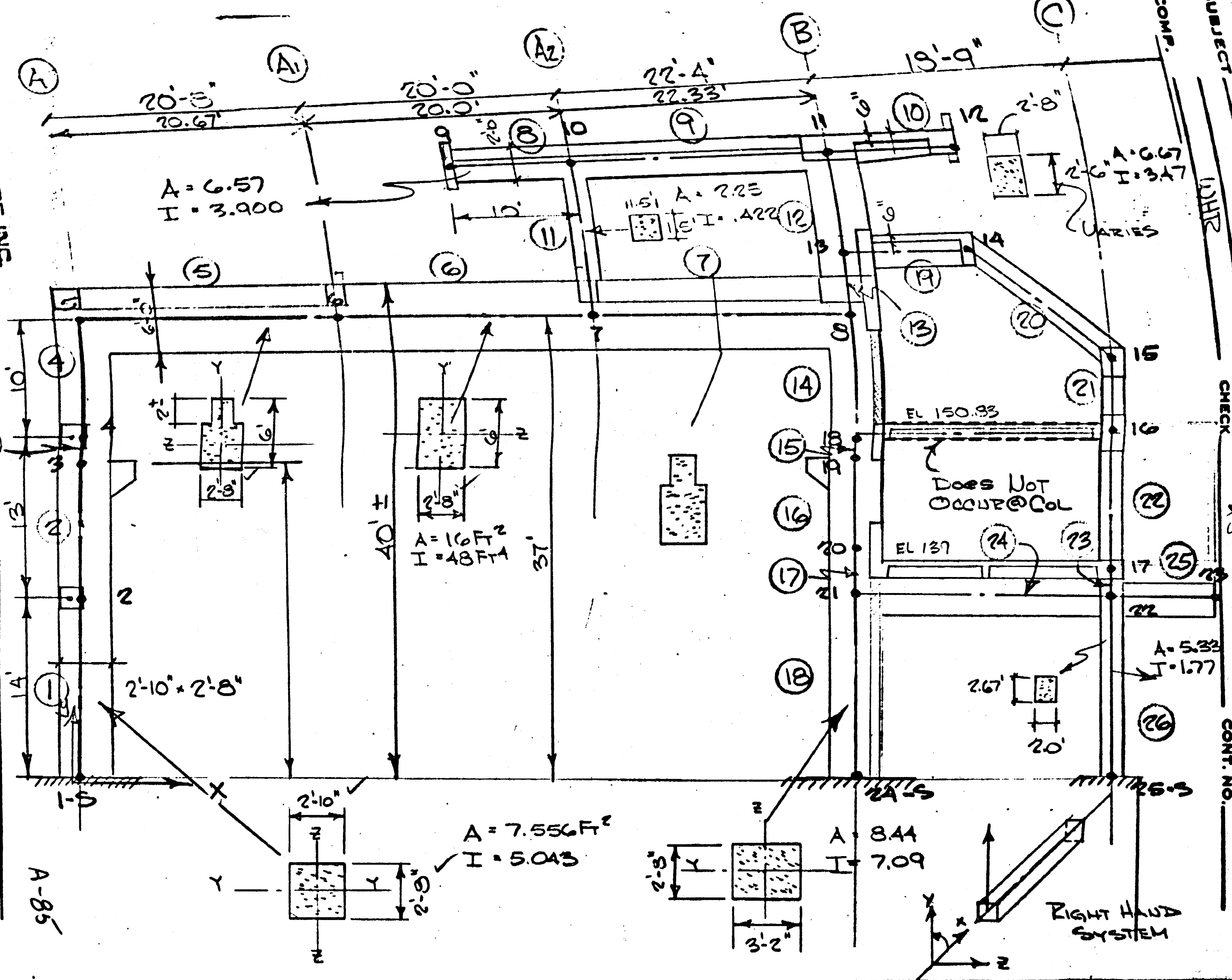
$\frac{KL}{r} = \frac{1.2(1.5)}{.0939} = 19.169$ $F_A = 20.66^{KSI}$

$F_T = 36.00^{KSI}$

$2 - 3/8" \phi = \frac{3.74^K}{2 \times .110} = 17^{KSI}$

2 - 3/8" φ Rods (A-36) @ 2'-0" Center to Center

CC MAGUIRE, INC.
ARCHITECTS ENGINEERS PLANNERS



A-85

11-72B

PROJECT
SUBJECT
COMP.
CHECK
ACC. NO. 1256.70
SHEET NO. ___ OF ___
DATE 6-18 19 73
CONT. NO.

PROJECT C.R.D ACC. NO. 1856.70
SUBJECT MEMBER PROPERTIES SHEET NO. ___ OF 1F-2
DATE ______ 19__
COMP. WHR CHECK ______ CONT. NO. ______

A-Line Col
Members 1, 2, 3, 4

$b = 2'\text{-}8'' = 2.67'$
$d = 2'\text{-}10'' = 2.83$

$A = 7.556\ Ft^2$
$I = 2.67(2.83)^3/12 = 5.043\ Ft^4$

B-Line Col
18, 17, 16, 15, 14, 13, 12

$b = 2.67'$
$d = 3.17'$

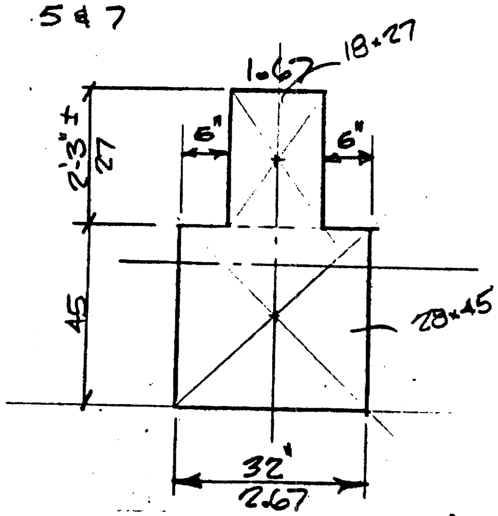
$A = 8.444\ Ft^2$
$I = 2.67(3.17)^3/12 = 7.09\ Ft^4$

C-Line Col
26, 23, 22, 21, 20

$b = 2.67'$
$d = 2.0$

$A = 5.33\ Ft^2$
$I = 2.67(2)^3/12 = 1.77\ Ft^4$

Main Girder Part 1 & 3
5 & 7



A	Y	AY
3.75	4.875	= 18.29
10.01	1.875	= 18.77
13.76		37.06

$\bar{Y} = \frac{37.06}{13.76} = 2.69'$

$I_G = \frac{1}{12}(1.67)(2.25)^3 + 3.75(2.185)^2 = 1.59 + 17.90$
$+ \frac{1}{12}(2.67)(3.75)^3 + 10.01(.815)^2 = 11.73 + 6.65$
$= 37.87\ Ft^4$

$A = 13.76\ Ft^2$
$I_G = 37.87\ Ft^4$

Center Portion
6

$b = 2.67$
$d = 6.0$

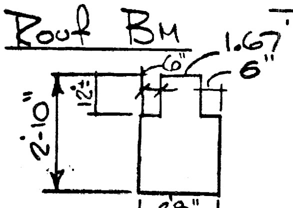
$A = 16.00\ Ft^2$
$I = 48.00\ Ft^4$

Cant. Bm @ Roof
10

$b = 32\ ?$
$d = 30$

$A = 6.67\ Ft^2$
$I = 3.47\ Ft^4$

Roof Bm
8 & 9



A	Y	AY
1.67	2.33	3.89
4.9	.917	4.50
6.57		8.39

$\bar{Y} = \frac{8.39}{6.57} = 1.27$

$I = \frac{1}{12}(1.67)(1)^3 + 1.67(1.06)^2 + \frac{1}{12}(2.67)(1.833)^3 + 4.9(.353)^2$
$= .139 + 1.87 + 1.37 + .611$
$= 3.900$

$A = 6.57$

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A-86

11-72B

PROJECT C.R.D ACC. NO. 1856.70
SUBJECT MEMBER PROPERTIES (CONT) SHEET NO. ___ OF IE-3
DATE ______ 19__
COMP. WHR CHECK ____ CONT. NO. ______

Penthouse Column
MEM. #11

b = 1.5 = 18"
d = 1.5 = 18"

A = 2.25
I = .422

BEAM @ EL 166 ± MEMBER 19

d = 3' = 36"
b = 2'-8" =

A = 8.01
I = 6.00

MEMBER 24 PART of SLAB @ 139

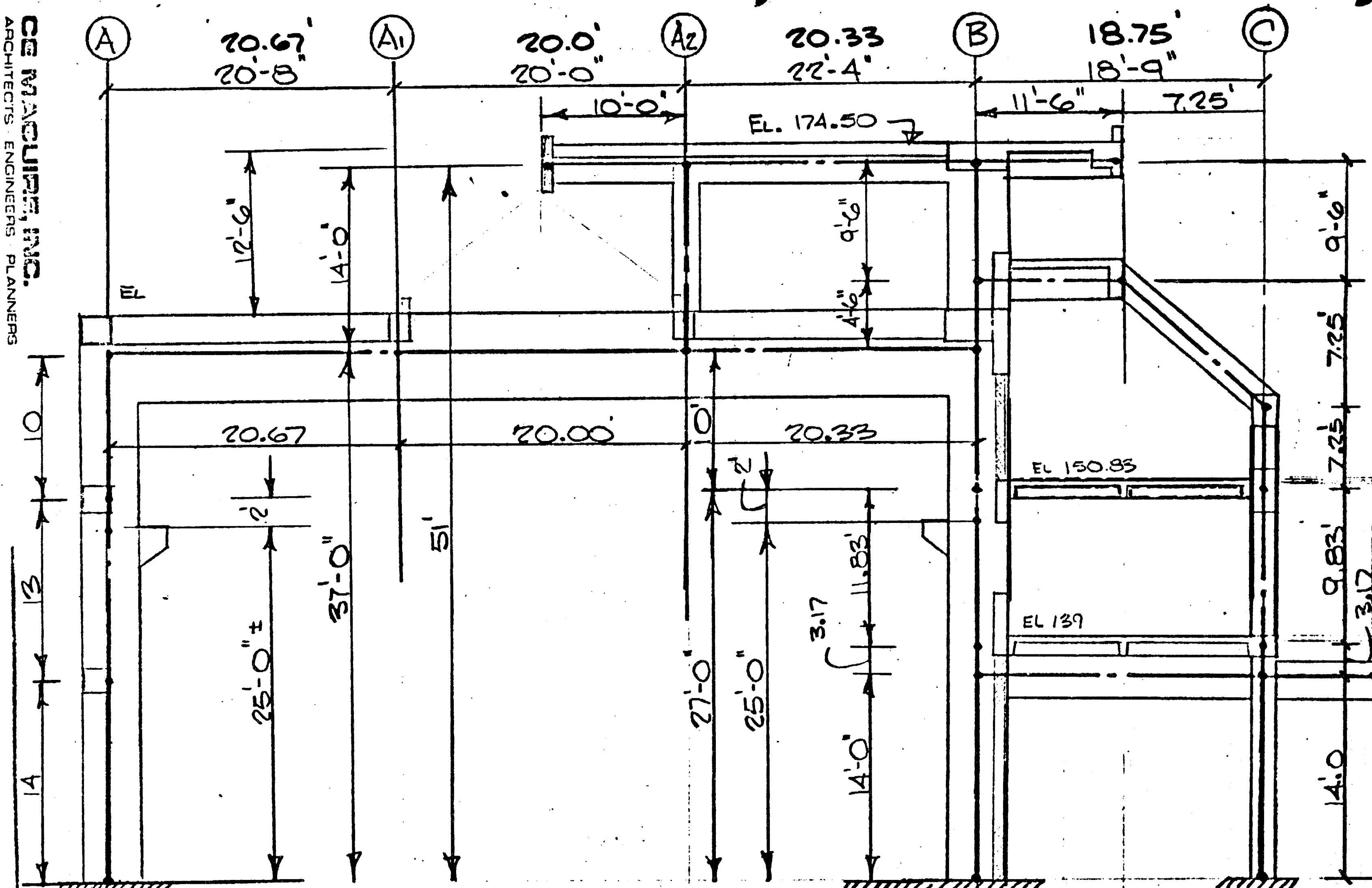
b = 1.5' = 18'
d = 5' ±

A = 7.5 FT^2
I = 15.63 FT^4

MEMBER 25 CANT. FOR WALKWAY

b = 1.5' = 18"
d = 3'-3" = 3.25'

A = 4.875 FT^2
I = 4.29 FT^4



GEOMETRY of STRUCTURE

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PROJECT C.P.D.
SUBJECT TYPICAL INTERIOR FRAME · 1,2,3,4,5,6
COMP. WHP
CHECK KS

ACC. NO. 1356.70
SHEET NO. ___ OF IE-4
DATE 6-18 19 73
CONT. NO. ___

A-88

11-72B

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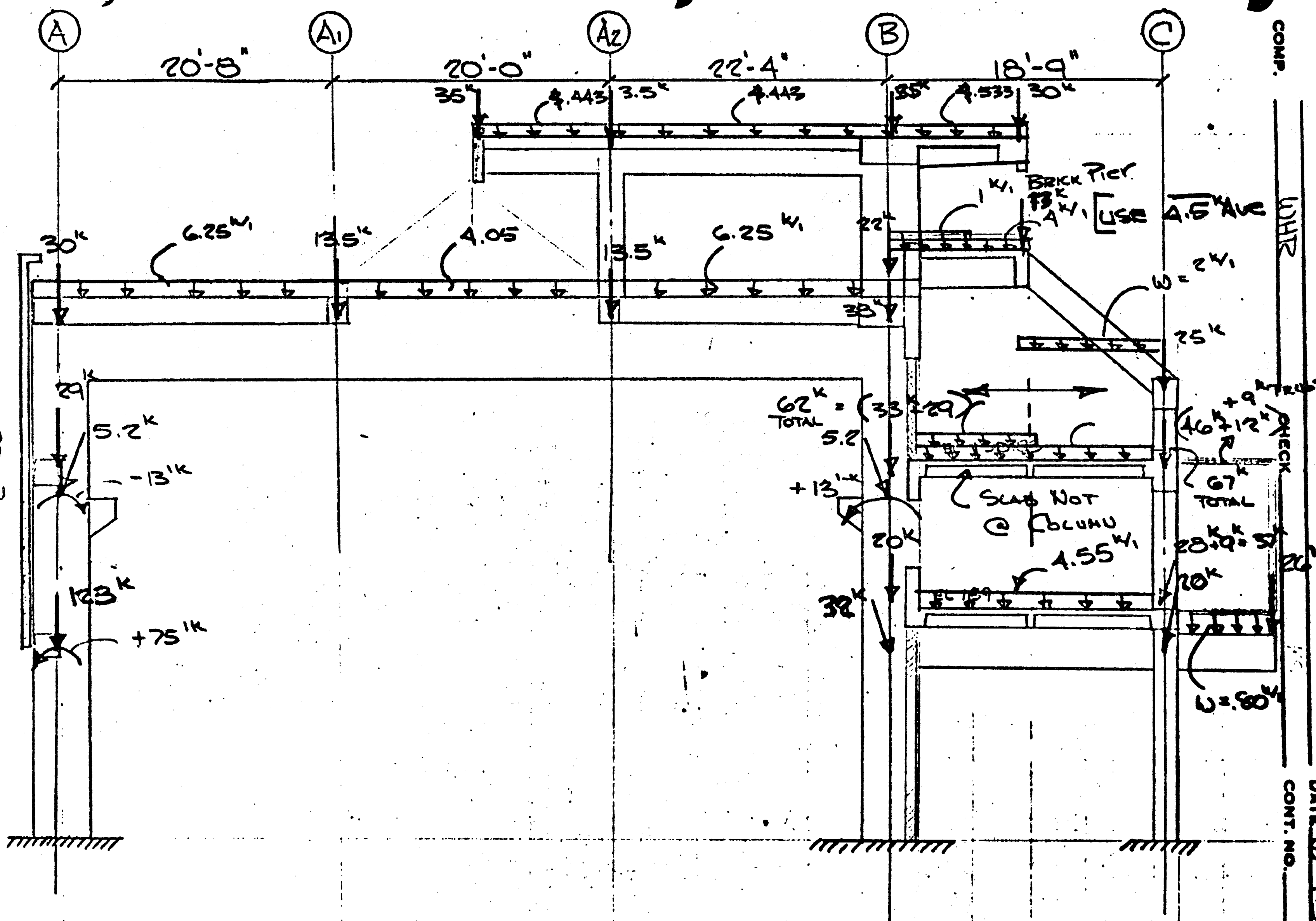
PROJECT C.P.D.
SUBJECT TYPICAL INT. FRAME · 2,3,4,5,6
COMP. WHR
ACC. NO. 1856.70
SHEET NO.
DATE 6-18 1978
CONT. NO.



DEAD LOADS ONLY

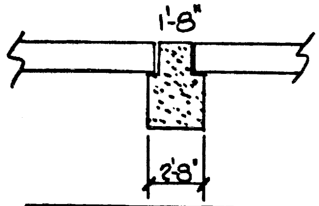
A-89
11-728

PROJECT C.R.D. ACC. NO. 185670
SUBJECT DEAD LOAD COMPS SHEET NO. ___ OF IF-6
DATE 6-19 1973
COMP. WR CHECK KS CONT. NO.

PENTHOUSE (174.5)

GENERAL	10" PLANK	= .072 × 29.33	=	2.110 k/l
	3" AVE TOP	= .038 × 31	=	1.178 k/l
	ROOF	= .005 × 31	=	.155 k/l
				3.443 k/l

(A) UNIFORM LOADS MEMBER 8 & 9



A = 6.57 FT² × .150 × 1 = .9855 SAY 1.00 k/l

$W_8 = W_9 = 4.443$ k/l

P = BRCK, MAS, BM = 35 k

(B) MEMBER 10 (CANTILEVER)

BRICK	4.5' × .060 =	.27 k/l × 31	=	8.37
MASONRY	2 × .080 =	.16 k/l × 31	=	5.00
BEAM	2 × 1.5 × .15 =	.45 k/l × 29.33	=	13.20
				26.57

SAY 30 k

W_{10} =	6" SLAB	= .075 KSF × 29.33	=	2.20
	ROOF	.005 × 31	=	.155
	3" TOPP	= .038 AVE × 31	=	1.178
				3.533

BEAM = 2.67 × 2.33 × .15 = .933 = 1.000 k

W_{10} = 4.533 k/l

JOINT LOAD @ 11 BEAM 3'-2" × 1'.5 = 3.17 × 1.5' = .713 k × 29'

SAY 21 k
13 → 34 k

MEMBER 11
COL 18×18 W = .34 k/l × 10' = 3.4 k

MEMBER 12
W = 1.27 × 10 = 12.7 k

MAIN ROOF (162.)

TEES	.095 × 30	=	2.85
TOPP	.038 × 31	=	1.18
FLR or ROOF	.005 × 31	=	.16
BEAM	13.76 × .15	=	2.06
			6.25

@ Opening

5" SLAB/12 ×	14.33'	=	.90 k/l
.038 ×	17.00	=	.65
.005 ×	17.00	=	.10
16 × .15			2.40
			4.05

JOINT LOADS
5 = .27 + .16 + 2 × 2 × .15 = 1.03 k/l × 28.33 = 29.2 k SAY 30 k

6 & 7 = SKYLIGHT .015 × 14' × 31' = 6.50 k + .10 CURB + 6.6 (DIFF BM & TEES

= 13.2

SAY 13.5 k

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11-72B

PROJECT: C.R.D. ACC. NO. 1856.70
SUBJECT: Int. Frame - Loads SHEET NO. ___ OF 1F-7
DATE 6-19 1973
COMP. WHR CHECK KS CONT. NO. ___

Joint 8 Concrete Section $9 \times 1 + 2 \times 2.5 = 14 \times .15 = 2.1^{k/'}$

$P = 2.1 \times 28' = 58.8^k$

+ Louvers $= .005 \times 7 \times 28.33 = 1.0^k$

* Use $P_8 = 22^k$
$P_{13} = 38^k$

Member 19

6" Slab $= .075^{ksf}(28.33) = 2.12^{k/'}$
Bm $= .267 \times 3 \times .15$ $1.20^{k/'}$
Light-Topp $= .020 \times 31.0 = .62$
Brick Piers $= .060 \times 8' \times 2 \times$ $.96$ → $1^{k/'}$
3.94

Use $4.00^{k/'}$ full Length & $1^{k/'}$ Half Length

$P = 3 \times 1 \times .15 \times 28.33' = 12.75^k$ Use 13^k
½ Por. Evam & Steel Frm. $.020^{ksf} \times 7' \times (31) = 4.34$ Say 18^k
(+ Half of Beam $= 2 \times 2.67 \times .15 \times 5' =$ 4^k
Include in Bm 20 Not Here) $P_{14} = 22^k$

Column Loads @ A Line Col $= 7.556^{ft^2} \times .15 = 1.13^{k/'}$

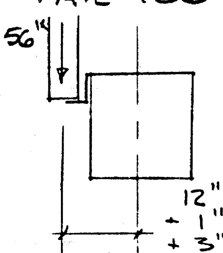
Joint 4: Col $= 10' \times 1.13^{k/'} = 11.3^k$
Bm $= .60^{k/'} \times (31 - 2.67)$ 17.0
28.3 → 29^k

Joint 3 D.L Crane Rail
(W27×102 + MC 18×42.7 + 20) $= 165^{\#/'} \times 31 = 5.2^{k/'}$
$M = 2.5' \times 5.2 = 13^{'-k}$

Joint 2

Brick $= .06 \times 30' \pm = 1.8^k \times 31' = 56^k$
(Assume 2'-8×2'-0") Beam $= 2.67 \times 2 \times .15 = .801 \times 28.33$ 22.7
Col $= 13 \times 1.13$ 14.63 (2) 30.0
Masonry Arch $= 7'$ Ave $\times .06$ Say $.50^k \times 28.33$ 14.
123.00^k
Use 123.^k

Mom



$= 56^k(1.33) = 75^{'-k}$

$\frac{16}{12} = 1.33$

A-91

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11-72B

PROJECT C.R.D — ACC. NO. 1856.70
SUBJECT INT. FRAME LOADS — SHEET NO. ___ OF IE-8
DATE 6-19 1973
COMP. WR — CHECK WS — CONT. NO. ___

Joint 15 - 2.625 × 2 × .15 × 28.33 = 22.31^k — Use 25^k

Joint 18 = Block = 7' × .038 (8" BLK) = .266^k/1 × 28.33 = 7.5
Bm 3 × .15 — .450^k/1 × 28.33 — 12.8
— 20.3
Col 8.44^Ft² × .15 × 10 — 12.65
— 32.95 → 33^k

Joint 16 Bm 2.59 × 3 × .15 × 28.33 : 33^k
Col 5.33 × .15 × 5.25' : ~ 5^k
— 38^k → 46^k
Brick & Louvers + 8^k

Slab @ EL 150.33

CRSI Book 3"/15 Total = .446 cf/sf × .15 = 67 psf

With Beams Say 85^psf × 14' = 1.019^k/1
Block : 1.00^k Ave / 14 = .076

W = 1.266
@ CrossWalk = 2.600^k/1
Use 3^k/1

Revise Jt 18 = 33^k + 3^k/1 × 9.4 = 61.2^k use 62

Jt 16 = 46 + 1.266 × 9.4 = 58^k + 1/2 Truss Roof
.290 × 31 : 9^k

@ Jt 16 = 67^k

* Use b = 2.67
Is possible b is 1.5'
Same as Cant

Slab @ 139

Bm = 2.67 × 5' × .15 = 2.^k/1
Slab = .080 × 28.33 × 1' — 2.27^k/1
Blk (Ave) — .10^k/1
Floor = .005(31) — .16^k/1

W_T = 4.53

Joint 21 = 8.44 × .15 × 25 = 32^k
Joint 22 = 5.33 × .15 × 25 = 20^k

W = 1.5 × 3.25 × .15 = .731

P = .290 + .290 + .250 = .83 × 31 ≈ 26^k

Joint 20 Glass .01 × 6 = .06^k/1
Bm 4.25 × .15 = .64^k/1

P = .70 × 28.33 = 20^k±
Joint 17 2.67 × 1.5 × .15 = .60^k/1
+ Brick & Masonry & Truss Say
P = 23^k + 9 = 37

A-92

C.E. MAGUIRE, INC.
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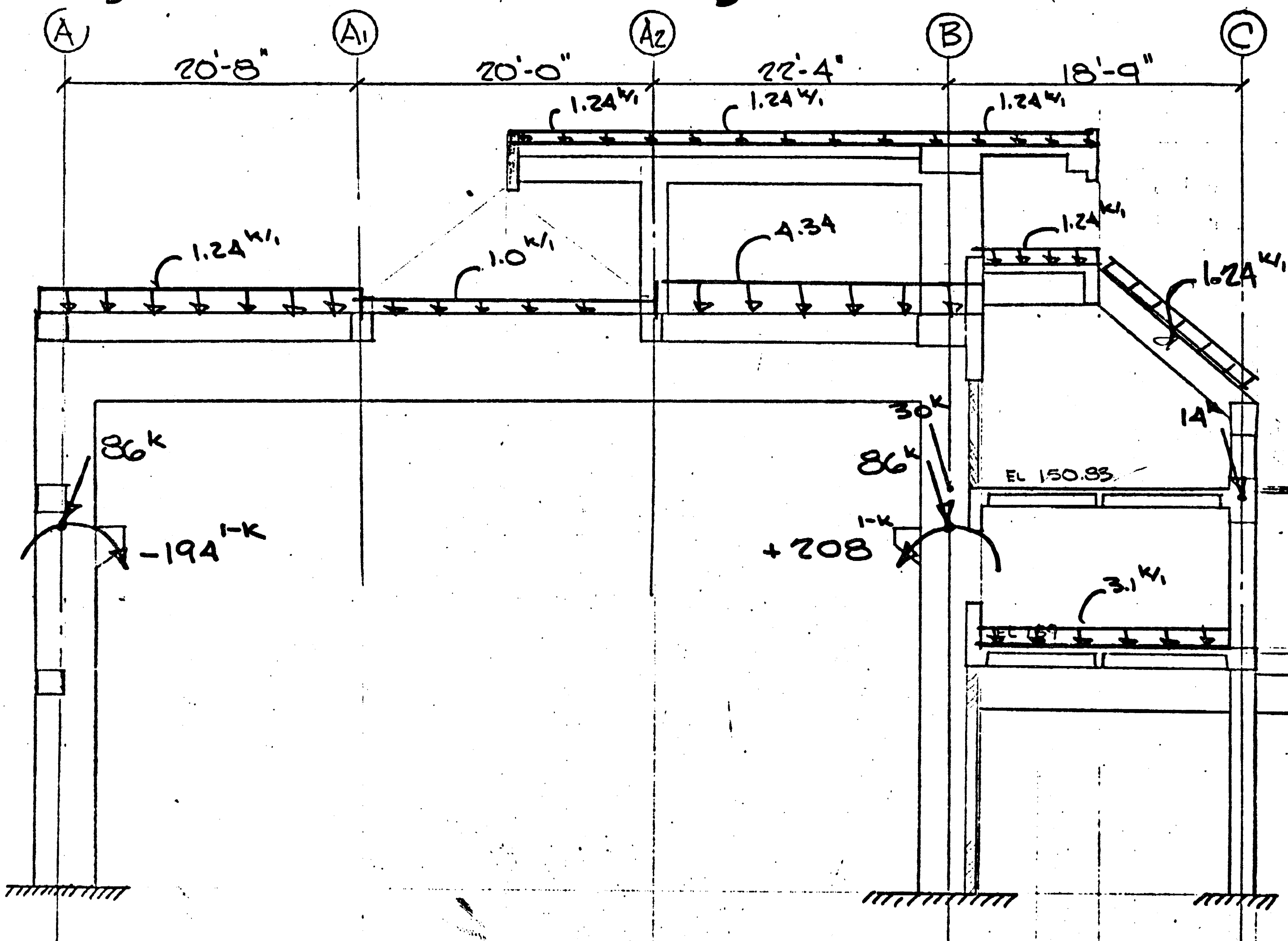
PROJECT: CRD
SUBJECT: TYPICAL INTERIOR FRAME · 4,4,4,5,6
COMP.: WHR
CHECK: KS
DATE: 6-18-73



LIVE LOAD ONLY

1856.1[illegible]

21-June-73

from CHRIS HIGGINS

Bill Rogers

Confermation of Wheel loading

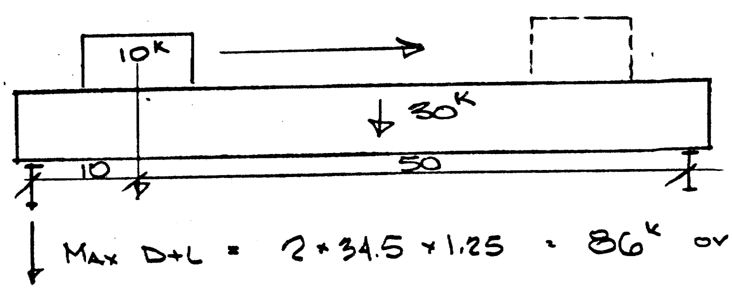
	Max Load	W or WO/impact	No wheels
Acco-Wright 969 6130	29,900	W 15% pass. over	2 1/12'
Robert Abel 939-7860	30,750 lb	WO/impact	2 wheels/10'
Dresser Crane "SHAW BOX"			
Whiting Corp. 944-7881			
Dwight Foote 666 4625	41,000 lb.	W/25%	← 2 wheels 10' wheels Base
Harnischfeger CE5-6810	34,500	WO/impact	

PROJECT	C.R.D	ACC. NO.	1856.70
SUBJECT	INT. FRAME	SHEET NO. ___ OF	IE-9
		DATE	6-19 1973
COMP.	WHR	CHECK	KS
		CONT. NO.	

BRIDGE CRANE INFO FOR PUMP HOUSE

CRANE MANUFAC.	(2 WHEELS) MAX. WHEEL LOAD	DEAD WEIGHT TOTAL
ACCO-WRIGHT	29.9^k W/O IMPACT	↑
ROBERT ABEL CO	30.8^k W/O	
DRESSER CRANE	32.3^k W/O	$37^k \pm$
WHITING CORP	31.9^k W/O	↓
DWIGHT FOOTE	41.0^k W/IMPACT (32.8) W/O	59.2^k
HARNISCHFEGER	34.5^k W/O IMPACT	$40^k \pm$

FOR DESIGN MAX LOAD 34.5^k
W/25% IMP $= 1.25(34.5)$ SAY $= 43^k$



MAX D+L $= 2 \times 34.5 \times 1.25 = 86^k$ or $43^{k/wheel}$ 10' C/C

MAX D $= 10 \times \frac{50}{60} + 15^k = 23.3^k$

PROJECT C.R.D ACC. NO. 1456.70
SUBJECT INT. FRAME SHEET NO. ___ OF IF-10
DATE 6-20 1973
COMP. WHR CHECK ___ CONT. NO. ___

(A) P.H. ROOF $31' \times .040^{ksf} = 1.24^{k/1}$

(B) MAIN ROOF $= 31 \times .04 = 1.24^{k/1}$

@ LOUVERS $= .04 \times 17' = .68^{k/1}$ USE $1.0^{k/1}$ (Some inc in LL) POSSIBLE

PENTHOUSE FLOOR $= .140 \times 31 = 4.34^{k/1}$
↑ LL + SDL

(C) SLAB @ 166' ± $= .040 (31) = 1.24^{k/1}$

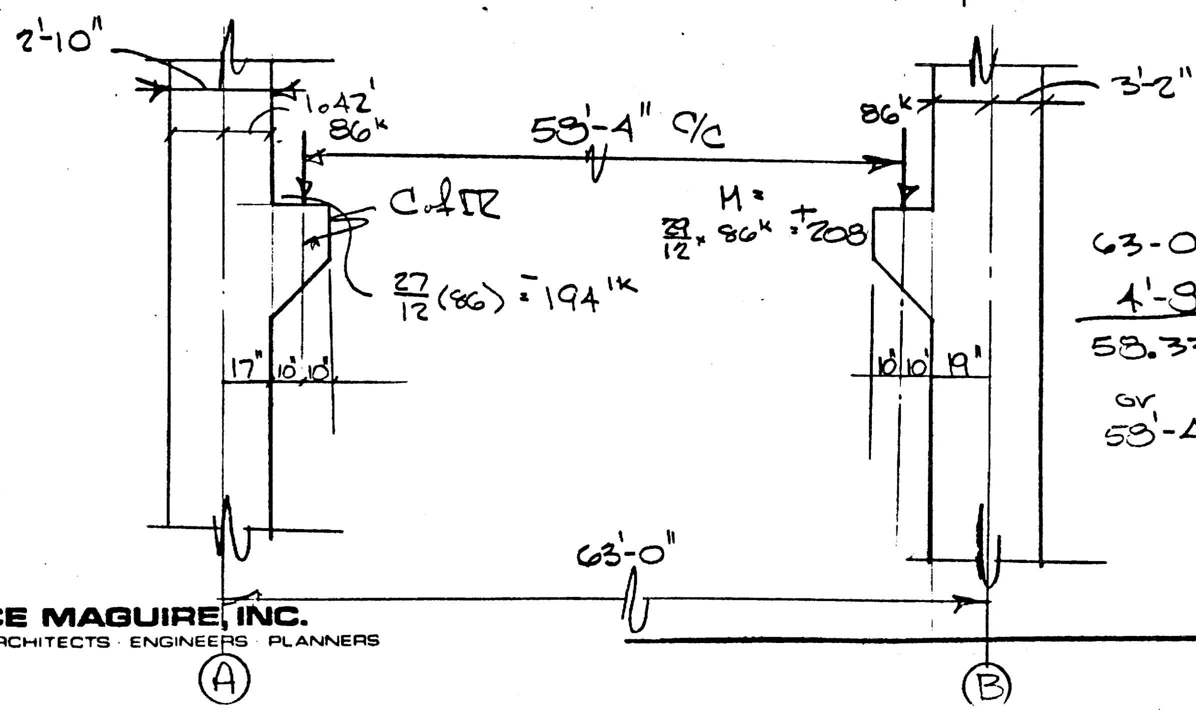
(D) SLAB @ 139' ± $= .100 (31) = 3.1^{k/1}$

LIVE LOAD @ EL 150.83

JT18 $= .100^{ksf} \times 31' \times 9.38' = 29.078^k$ SAY 30^k
JT16 $= .100^{ksf} \times 14 \times 9.38' = 13.132^k$ SAY 14^k

WALKWAY TRUSS LIVE LOAD
.1575
.4250
.5825 × 31 = 18.1^k
SAY 19^k LL

CRANE RAIL $34.5^{k/whl} \times 2\,whl = 69^k \times 1.25$ (25% IMPACT) $= 86^k$



63-0
4'-8"
58.333
or
58'-4"

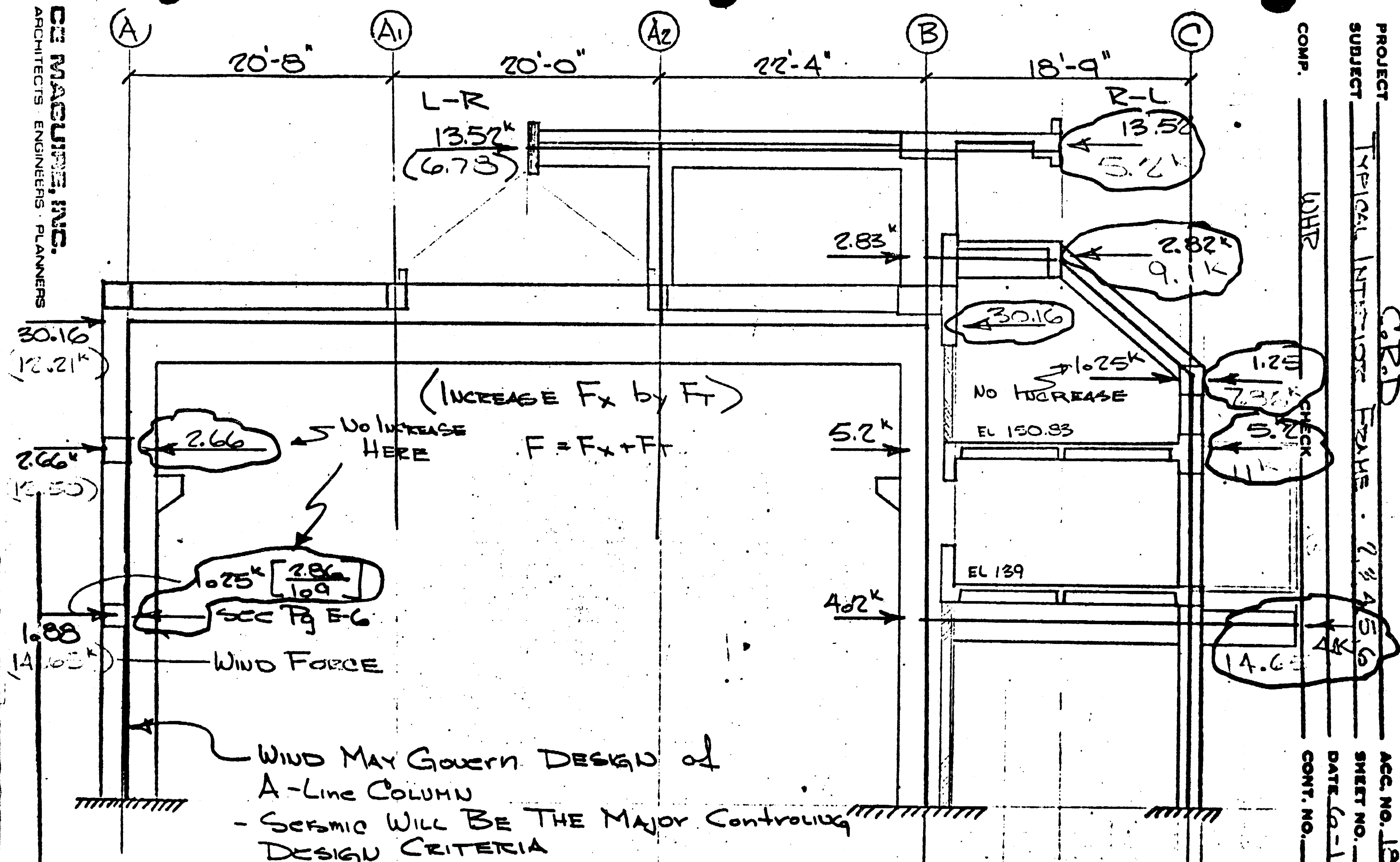
CE MAGUIRE, INC.
ARCHITECTS · ENGINEERS · PLANNERS

A-96

11-72B

CE MAGUIRE, INC.
ARCHITECTS · ENGINEERS · PLANNERS

PROJECT CRD
SUBJECT TYPICAL INTERIOR FRAME · 2 & 4 5 6
COMP. WHR
CHECK
CONT. NO.
ACC. NO. 1956.70
SHEET NO. ___ OF IE-11
DATE 6-18 1973



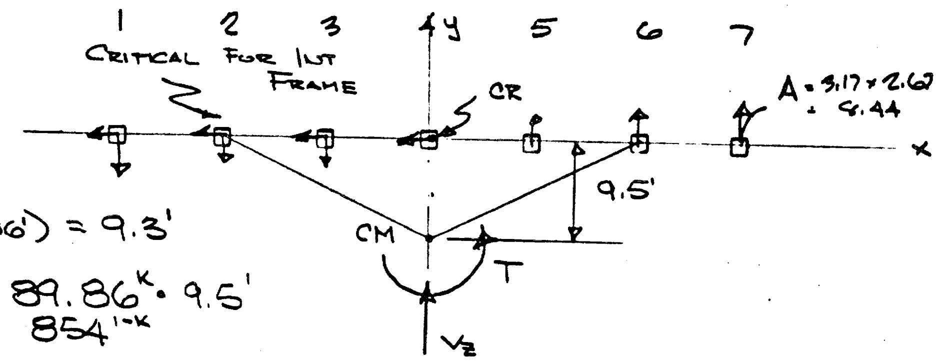
LATERAL FORCES

* BOTH WIND & SEISMIC ARE REVERSIBLE

11-72B

For Seismic Forces (Refer To Pg E-7)

(A) EL 174.5' $F_x = 13^k$ Consider F_T



$e = 0$

$e_{min} = .05(186') = 9.3'$

$T = V_z \cdot e = 89.86^k \cdot 9.5' = 854^{1\text{-}k}$

$\therefore F_{T\sigma} = \frac{TC}{J}$ where J = Polar Moment of Inertia $= A \cdot d^2$

Torsional Shear Stress !!!

$J = I_x + I_y$ & $I_x = 0$

$\therefore J = I_Y = 8.44 [2 \times (31^2 + 62^2 + 93^2)] = 227,104 \text{ Ft}^4$

$C_{INT} = (9.5^2 + 62^2)^{1/2} = 62.7$

$C_{1\&7} = (9.5^2 + 93^2) = 93.5$

$F_t = \frac{854^{1\text{-}k} \times 62.7}{227,104} = .24$ $F_{ty\sigma} = .24 \times \frac{9.5}{62.7} = .036$

$F_T = 8.44 \times (.036) = .31^k$

F_x @ 174.5 = 13^k

* J is Large
Torsion Shear Are Negligible In The Order of 2.5%
$\therefore$ Use 4% Increase To include Torsion

(B) @ 166 $F_x = 2.71^k = 2.82^k$ [Conservative]

(C) @ 162 $F_x = 29^k = 30.16^k$

For Other Forces See Sht 1F-11

Wind forces Based on $P = .035^{ksf}$

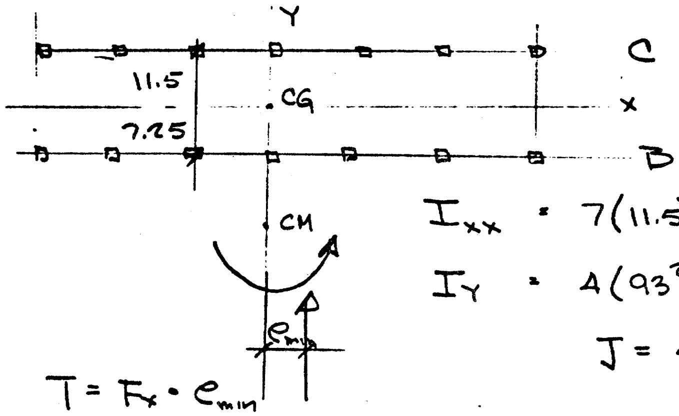
$W_p = .035 \times 31 \times h$

PROJECT C.R.D ACC. NO. 1856.70

SUBJECT TORSIONAL SHEARS SHEET NO. ___ OF 1E-13

DATE 6-22 19 73

COMP. WHR CHECK KS CONT. NO. ___



C See Pg E-15

A = 1 = Unity

$I_{xx} = 7(11.5)^2 + 7(7.25)^2 = 1294\ Ft^4$

$I_Y = 4(93^2 + 62^2 + 31^2) = 53816\ Ft^4$

$J = 55100\ (Ft^4)$ eff = Ft^2

$T = F_x \cdot e_{min}$
$= 344 \cdot 9.3 =$

F_T (6 Line) $C_c = [62^2 + 11.5^2]^{1/2} = 63.1'$
(2 Line) $C_B = [62^2 + 7.25^2]^{1/2}\ 62.4'$

$F_t = \frac{320 \cdot 63.1}{55100} \approx .37$ $F_Y = .37 \times \frac{11.5}{63.1} = .1$
2% Increas

* INCREASE of 4% IS VERY CONSERVATIVE FOR SEISMIC ACTION IN THIS SHORT DIRECTION I.E. 7 FRAMES CARRING FORCES · SMALL EFFECT

TORSIONAL SHEAR WILL BE MORE SIGNIFICANT FOR FRAMES IN LONG DIRECTION

```
STRUCTURE ---TYPICAL INTERIOR FRAME --CRD--WHR
TYPE PLANE FRAME
NUMBER OF JOINTS    25
NUMBER OF MEMBERS   26
NUMBER OF SUPPORTS   3
NUMBER OF LOADINGS   7
TABULATE ALL
JOINT COORDINATES
1    0.00   0.00 S
2    0.00  14.00
3    0.00  25.00
4    0.00  27.00
5    0.00  37.00
6   20.67  37.00
7   40.67  37.00
8   63.00  37.00
9   30.67  51.00
10  40.67  51.00
11  63.00  51.00
12  74.50  51.00
13  63.00  41.50
14  74.50  41.50
15  81.75  34.25
16  81.75  27.00
17  81.75  17.17
18  63.00  27.00
19  63.00  25.00
20  63.00  17.17
21  63.00  14.00
22  81.75  14.00
23  91.92  14.00
24  63.00   0.00 S
25  81.75   0.00 S
MEMBER INCIDENCES
1   1   2
2   2   3
3   3   4
4   4   5
5   5   6
6   6   7
7   7   8
8   9   10
9   10  11
10  11  12
11   7  10
12  11  13
13  13  8
14  8   18
15  18  19
16  19  20
17  20  21
18  21  24
19  13  14
20  14  15
21  15  16
22  16  17
23  17  22
24  21  22
    22  23
    22  25
```

Typ Int. Frame
1.4(D+L+E)

```
MEMBER PROPERTIES PRISMATIC
1  AX  7.556 IZ  5.043
   AX  7.556 IZ  5.043
   AX  7.556 IZ  5.043
4  AX  7.556 IZ  5.043
5  AX 13.760 IZ 37.870
6  AX 16.000 IZ 48.000
7  AX 13.760 IZ 37.870
8  AX  6.570 IZ  3.900
9  AX  6.570 IZ  3.900
10 AX  6.670 IZ  3.472
11 AX  2.250 IZ  0.422
12 AX  8.444 IZ  7.090
13 AX  8.444 IZ  7.090
14 AX  8.444 IZ  7.090
15 AX  8.444 IZ  7.090
16 AX  8.444 IZ  7.090
17 AX  8.444 IZ  7.090
18 AX  8.444 IZ  7.090
19 AX  8.010 IZ  6.000
20 AX  5.330 IZ  1.770
21 AX  5.330 IZ  1.770
22 AX  5.330 IZ  1.770
23 AX  5.330 IZ  1.770
24 AX  7.500 IZ 15.630
25 AX  4.875 IZ  4.290
26 AX  5.330 IZ  1.770
CONSTANTS E 449570.0 ALL
LOADING 1       FULL DL (SERVICE)
JOINT LOADS
2  FORCE Y  -123.0
2  MOMENT Z  +75.0
3  FORCE Y    -5.2
3  MOMENT Z  -13.0
4  FORCE Y   -29.0
5  FORCE Y   -30.0
6  FORCE Y   -13.5
7  FORCE Y   -13.5
8  FORCE Y   -38.0
9  FORCE Y   -35.0
10 FORCE Y    -3.5
11 FORCE Y   -35.0
12 FORCE Y   -30.0
13 FORCE Y   -22.0
14 FORCE Y   -13.0
15 FORCE Y   -25.0
16 FORCE Y   -67.0
17 FORCE Y   -37.0
18 FORCE Y   -62.0
19 FORCE Y    -5.2
19 MOMENT Z  +13.0
20 FORCE Y   -20.0
21 FORCE Y   -32.0
22 FORCE Y   -20.0
23 FORCE Y   -26.0
MEMBER LOADS
5  FORCE Y UNIFORM -6.250
6  FORCE Y UNIFORM -4.050
7  FORCE Y UNIFORM -6.250
8  FORCE Y UNIFORM -4.443
9  FORCE Y UNIFORM -4.443
```

```
10 FORCE Y UNIFORM -4.443
19 FORCE Y UNIFORM -4.500
20 FORCE Y UNIFORM -2.000
24 FORCE Y UNIFORM -4.550
25 FORCE Y UNIFORM -0.800
LOADING 2      FULL LL (SERVICE)
JOINT LOADS
16 FORCE Y  -14.00
18 FORCE Y  -30.00
23 FORCE Y  -19.0
MEMBER LOADS
5  FORCE Y UNIFORM -1.24
6  FORCE Y UNIFORM -1.00
7  FORCE Y UNIFORM -4.34
8  FORCE Y UNIFORM -1.24
9  FORCE Y UNIFORM -1.24
10 FORCE Y UNIFORM -1.24
19 FORCE Y UNIFORM -1.24
20 FORCE Y UNIFORM -1.24
24 FORCE Y UNIFORM -3.10
LOADING 3      CRANE LOAD (SERVICE)
JOINT LOADS
3  FORCE Y  -86.00
3  MOMENT Z -194.0
19 FORCE Y  -86.0
19 MOMENT Z +208.0
LOADING 4      (SEISMIC LEFT-RIGHT)
JOINT LOADS
2  FORCE X +1.880
4  FORCE X +2.660
5  FORCE X +30.16
9  FORCE X +13.52
13 FORCE X +2.830
15 FORCE X +1.250
18 FORCE X +5.200
21 FORCE X +4.200
LOADING 5   SEISMIC (RIGHT LEFT)
JOINT LOADS
12 FORCE X -13.52
14 FORCE X -2.820
8  FORCE X -30.16
15 FORCE X -1.250
16 FORCE X -5.200
23 FORCE X -4.200
4 FORCE X -2.660
2 FORCE X -1.880
LOADING 6   ULTIMATE  D+L+E      A.C.E.
COMBINE  1 1.400 2 1.400 3 1.400 4 1.400
LOADING 7   ULTIMATE  D+L+E   A.C.E.
COMBINE  1  1.400   2  1.400   3  1.400   5  1.400
SOLVE
PROBLEM CORRECTLY SPECIFIED. EXECUTION TO PROCEED.
```

STRUCTURE ---TYPICAL INTERIOR FRAME --CRD--WHR

LOADING 1 FULL DL (SERVICE)

MEMBER FORCES

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	409.030	-34.017	-499.64
1	2	-409.030	34.017	23.39
2	2	286.030	-34.017	51.60
2	3	-286.030	34.017	-425.80
3	3	280.831	-34.051	412.78
3	4	-280.831	34.051	-480.88
4	4	251.832	-34.035	480.86
4	5	-251.832	34.035	-821.22
5	5	34.036	221.832	821.22
5	6	-34.036	-92.644	2428.88
6	6	34.039	79.144	-2428.88
6	7	-34.039	1.855	3201.77
7	7	47.578	-148.535	-3257.57
7	8	-47.578	288.097	-1617.43
8	9	0.004	-35.000	0.00
8	10	-0.004	79.430	-572.15
9	10	-13.536	50.247	438.41
9	11	13.536	48.964	-424.09
10	11	0.007	81.094	638.79
10	12	-0.007	-30.000	0.00
11	7	133.180	13.538	55.80
11	10	-133.180	-13.538	133.73
12	11	165.057	-13.535	-214.69
12	13	-165.057	13.535	86.11
13	13	284.371	-12.287	-901.30
13	8	-284.371	12.287	846.01
14	8	610.467	35.292	771.40
14	18	-610.467	-35.292	-418.47
15	18	672.470	35.292	418.48
15	19	-672.470	-35.292	-347.89
16	19	677.676	35.293	360.89
16	20	-677.676	-35.293	-84.54
17	20	697.676	35.293	84.54
17	21	-697.676	-35.293	27.33
18	21	758.185	11.489	58.83
18	24	-758.185	-11.489	102.01
19	13	-1.250	97.310	815.19
19	14	1.250	-45.560	6.32
20	14	-23.908	22.137	-6.32
20	15	23.908	-1.631	128.17
21	15	6.941	-15.752	-128.17

21	16	-6.941	15.752	13.97
22	16	73.940	-15.752	-13.97
22	17	-73.940	15.752	-140.87
23	17	110.940	-15.752	140.87
23	22	-110.940	15.752	-190.81
24	21	23.803	28.507	-86.16
24	22	-23.803	56.804	-179.11
25	22	-0.000	34.135	305.79
25	23	0.000	-25.999	-0.00
26	22	221.881	8.051	64.13
26	25	-221.881	-8.051	48.58

APPLIED JOINT LOADS, FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
2	0.000	-122.999	75.00
3	0.034	-5.199	-13.01
4	-0.016	-28.999	-0.02
5	0.001	-30.000	0.00
6	0.002	-13.500	0.00
7	0.000	-13.499	0.00
8	0.002	-37.997	-0.00
9	0.004	-35.000	0.00
10	-0.002	-3.502	0.00
11	0.008	-34.998	0.00
12	-0.007	-30.000	0.00
13	-0.002	-22.004	-0.01
14	-0.001	-13.000	-0.00
15	-0.000	-25.000	-0.00
16	0.000	-66.999	-0.00
17	-0.000	-36.999	-0.00
18	-0.000	-62.003	0.00
19	0.001	-5.206	12.99
20	-0.000	-19.999	-0.00
21	-0.000	-32.001	0.00
22	0.000	-20.000	-0.00
23	0.000	-25.999	-0.00

REACTIONS,APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	34.017	409.030	-499.64
24	-11.489	758.185	102.01
25	-8.051	221.881	48.58

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
2	-0.0147	-0.0016	0.0016
3	-0.0277	-0.0026	0.0004
4	-0.0283	-0.0027	0.0000
5	-0.0158	-0.0035	-0.0028

6	-0.0159	-0.0554	-0.0015
7	-0.0160	-0.0606	0.0011
8	-0.0162	-0.0067	0.0025
9	-0.0362	-0.1127	0.0054
10	-0.0362	-0.0624	0.0040
11	-0.0361	-0.0074	0.0009
12	-0.0361	-0.0130	-0.0010
13	-0.0251	-0.0070	0.0013
14	-0.0251	-0.0035	-0.0001
15	-0.0231	-0.0017	0.0009
16	-0.0134	-0.0017	0.0015
17	-0.0001	-0.0014	0.0007
18	-0.0006	-0.0051	0.0007
19	0.0005	-0.0047	0.0004
20	0.0017	-0.0033	-0.0000
21	0.0014	-0.0027	-0.0000
22	0.0013	-0.0012	0.0001
23	0.0013	-0.0051	-0.0006

STRUCTURE ---TYPICAL INTERIOR FRAME --CRD--WHR

LOADING 2 FULL LL (SERVICE)

==

MEMBER FORCES

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	54.186	-10.982	-156.75
1	2	-54.186	10.982	2.99
2	2	54.186	-10.983	-2.99
2	3	-54.186	10.983	-117.82
3	3	54.186	-10.982	117.82
3	4	-54.186	10.982	-139.79
4	4	54.186	-10.988	139.79
4	5	-54.186	10.988	-249.68
5	5	10.989	54.186	249.68
5	6	-10.989	-28.556	605.46
6	6	10.989	28.555	-605.46
6	7	-10.989	-8.555	976.57
7	7	10.952	-15.807	-968.75
7	8	-10.952	112.719	-466.24
8	9	0.001	0.000	0.00
8	10	-0.001	12.399	-61.99
9	10	0.038	11.962	54.70
9	11	-0.038	15.726	-96.72
10	11	0.002	14.260	81.99
10	12	-0.002	-0.000	-0.00
11	7	24.363	-0.037	-7.81
11	10	-24.363	0.037	7.29
12	11	29.985	0.038	14.72
12	13	-29.985	-0.038	-14.36
13	13	59.887	-2.472	-260.28
13	8	-59.887	2.472	249.16
14	8	172.606	8.480	217.07
14	18	-172.606	-8.480	-132.27
15	18	202.608	8.477	132.27
15	19	-202.608	-8.477	-115.31
16	19	202.609	8.476	115.31
16	20	-202.609	-8.476	-48.94
17	20	202.609	8.476	48.94
17	21	-202.609	-8.476	-22.07
18	21	226.114	0.806	0.85
18	24	-226.114	-0.806	10.43
19	13	2.510	29.900	274.64
19	14	-2.510	-15.640	-12.78
20	14	-9.284	12.834	12.78
20	15	9.284	-0.120	53.62
21	15	-4.650	-6.480	-53.62
21	14	[illegible]	[illegible]	[illegible]

22	16	7.349	-6.480	-6.64
22	17	-7.349	6.480	-57.05
23	17	7.349	-6.480	57.05
23	22	-7.349	6.480	-77.59
24	21	7.670	23.504	21.21
24	22	-7.670	34.620	-125.42
25	22	-0.000	18.999	193.22
25	23	0.000	-18.999	-0.00
26	22	60.969	1.190	9.79
26	25	-60.969	-1.190	6.87

APPLIED JOINT LOADS, FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
2	0.000	0.000	-0.00
3	-0.000	0.000	0.00
4	0.005	0.000	0.00
5	0.000	-0.000	0.00
6	0.000	-0.000	-0.00
7	0.000	0.000	-0.00
8	0.000	0.000	-0.00
9	0.001	0.000	0.00
10	0.000	-0.001	0.00
11	0.002	0.000	0.00
12	-0.002	-0.000	-0.00
13	-0.000	-0.001	-0.00
14	-0.000	-0.000	0.00
15	-0.000	-0.000	0.00
16	0.000	-13.999	0.00
17	0.000	0.000	0.00
18	-0.002	-30.001	-0.00
19	-0.000	-0.001	0.00
20	-0.000	0.000	-0.00
21	0.000	-0.000	0.00
22	0.000	-0.000	-0.00
23	0.000	-18.999	-0.00

REACTIONS, APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	10.982	54.186	-156.75
24	-0.806	226.114	10.43
25	-1.190	60.969	6.87

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
2	-0.0045	-0.0002	0.0004
3	-0.0089	-0.0003	0.0002
4	-0.0093	-0.0004	0.0001
5	-0.0064	-0.0005	-0.0007
6	-0.0064	-0.0152	-0.0004

7	-0.0064	-0.0179	0.0002
8	-0.0065	-0.0019	0.0008
9	-0.0143	-0.0275	0.0009
10	-0.0143	-0.0182	0.0008
11	-0.0143	-0.0021	0.0005
12	-0.0143	0.0021	0.0003
13	-0.0095	-0.0020	0.0004
14	-0.0095	-0.0007	-0.0000
15	-0.0090	-0.0003	0.0003
16	-0.0054	-0.0003	0.0006
17	-0.0003	-0.0003	0.0002
18	-0.0010	-0.0015	0.0002
19	-0.0005	-0.0014	0.0002
20	0.0002	-0.0010	0.0000
21	0.0002	-0.0008	-0.0000
22	0.0001	-0.0003	0.0000
23	0.0001	-0.0035	-0.0004

STRUCTURE ---TYPICAL INTERIOR FRAME --CRD--WHR

LOADING 3 CRANE LOAD (SERVICE)

===

MEMBER FORCES

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	86.245	-7.533	-76.84
1	2	-86.245	7.533	-28.62
2	2	86.245	-7.533	28.62
2	3	-86.245	7.533	-111.49
3	3	0.245	-7.536	-82.50
3	4	-0.245	7.536	67.43
4	4	0.245	-7.534	-67.43
4	5	-0.245	7.534	-7.91
5	5	7.534	0.245	7.91
5	6	-7.534	-0.245	-2.83
6	6	7.535	0.245	2.83
6	7	-7.535	-0.245	2.07
7	7	7.469	0.199	-1.58
7	8	-7.469	-0.199	6.04
8	9	0.000	0.000	0.00
8	10	-0.000	-0.000	0.00
9	10	0.065	0.045	0.42
9	11	-0.065	-0.045	0.59
10	11	0.000	0.000	0.00
10	12	-0.000	-0.000	0.00
11	7	0.045	-0.065	-0.49
11	10	-0.045	0.065	-0.42
12	11	-0.045	0.065	-0.59
12	13	0.045	-0.065	1.21
13	13	0.113	0.564	2.23
13	8	-0.113	-0.564	0.30
14	8	-0.086	8.034	-6.35
14	18	0.086	-8.034	86.69
15	18	-0.085	8.032	-86.69
15	19	0.085	-8.032	102.76
16	19	85.914	8.031	105.23
16	20	-85.914	-8.031	-42.35
17	20	85.914	8.031	42.35
17	21	-85.914	-8.031	-16.89
18	21	84.341	5.658	39.42
18	24	-84.341	-5.658	39.79
19	13	-0.499	0.159	-3.45
19	14	0.499	-0.159	5.28
20	14	-0.465	-0.240	-5.28
20	15	0.465	0.240	2.82
21	15	-0.159	-0.499	-2.82
21	[illegible]	[illegible]	[illegible]	[illegible]

22	16	-0.159	-0.499	0.79
22	17	0.159	0.499	-5.70
23	17	-0.159	-0.499	5.70
23	22	0.159	0.499	-7.28
24	21	2.372	-1.572	-22.53
24	22	-2.372	1.572	-6.95
25	22	-0.000	-0.000	-0.00
25	23	0.000	0.000	-0.00
26	22	1.413	1.873	14.24
26	25	-1.413	-1.873	11.97

APPLIED JOINT LOADS, FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
2	0.000	0.000	-0.00
3	0.002	-85.999	-194.00
4	-0.001	0.000	-0.00
5	0.000	0.000	0.00
6	0.000	-0.000	0.00
7	0.000	0.000	-0.00
8	0.000	-0.000	-0.00
9	0.000	0.000	0.00
10	0.000	-0.000	0.00
11	0.000	0.000	0.00
12	-0.000	-0.000	0.00
13	-0.000	-0.000	-0.00
14	0.000	-0.000	0.00
15	-0.000	0.000	0.00
16	0.000	0.000	-0.00
17	-0.000	0.000	-0.00
18	-0.002	-0.000	-0.00
19	-0.001	-86.000	208.00
20	-0.000	0.000	-0.00
21	-0.000	-0.000	0.00
22	0.000	-0.000	-0.00
23	0.000	0.000	-0.00

REACTIONS, APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	7.533	86.245	-76.84
24	-5.658	84.341	39.79
25	-1.873	1.413	11.97

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
2	-0.0018	-0.0003	0.0001
3	-0.0019	-0.0006	-0.0001
4	-0.0016	-0.0006	-0.0001
5	-0.0013	-0.0006	0.0000
6	-0.0013	-0.0005	-0.0000

7	-0.0013	-0.0005	-0.0000
8	-0.0013	-0.0005	0.0000
9	-0.0014	-0.0006	0.0000
10	-0.0014	-0.0005	0.0000
11	-0.0014	-0.0005	0.0000
12	-0.0014	-0.0005	0.0000
13	-0.0014	-0.0005	0.0000
14	-0.0014	-0.0003	0.0000
15	-0.0010	-0.0000	0.0000
16	-0.0004	-0.0000	0.0000
17	0.0002	-0.0000	0.0000
18	-0.0008	-0.0005	0.0001
19	-0.0004	-0.0005	0.0002
20	0.0003	-0.0003	0.0000
21	0.0004	-0.0003	-0.0000
22	0.0003	-0.0000	0.0000
23	0.0003	0.0001	0.0000

STRUCTURE ---TYPICAL INTERIOR FRAME --CRD--WHR

LOADING 4 (SEISMIC LEFT-RIGHT)

MEMBER FORCES

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	-8.584	13.288	230.85
1	2	8.584	-13.288	-44.81
2	2	-8.584	11.408	44.81
2	3	8.584	-11.408	80.68
3	3	-8.584	11.416	-80.68
3	4	8.584	-11.416	103.51
4	4	-8.584	8.759	-103.51
4	5	8.584	-8.759	191.10
5	5	21.399	-8.584	-191.10
5	6	-21.399	8.584	13.66
6	6	21.398	-8.584	-13.66
6	7	-21.398	8.584	-158.03
7	7	23.446	-6.540	142.18
7	8	-23.446	6.540	-288.24
8	9	13.517	-0.000	-0.00
8	10	-13.517	0.000	0.00
9	10	11.467	-2.043	-12.84
9	11	-11.467	2.043	-32.78
10	11	-0.001	-0.000	-0.00
10	12	0.001	0.000	0.00
11	7	-2.043	2.049	15.84
11	10	2.043	-2.049	12.84
12	11	2.043	11.468	32.78
12	13	-2.043	-11.468	76.16
13	13	-15.110	4.523	87.49
13	8	15.110	-4.523	-67.13
14	8	-8.569	27.970	355.39
14	18	8.569	-27.970	-75.69
15	18	-8.570	33.230	75.75
15	19	8.570	-33.230	-9.29
16	19	-8.570	33.242	9.26
16	20	8.570	-33.242	251.02
17	20	-8.570	33.235	-251.03
17	21	8.570	-33.235	356.39
18	21	-48.462	32.302	143.71
18	24	48.462	-32.302	308.51
19	13	9.775	-17.154	-163.65
19	14	-9.775	17.154	-33.61
20	14	19.041	-5.217	33.61
20	15	-19.041	5.217	-87.11
21	15	17.152	11.025	87.11
21	16	-17.152	-11.025	-7.17

22	16	17.153	11.025	7.17
22	17	-17.153	-11.025	101.20
23	17	17.153	11.024	-101.20
23	22	-17.153	-11.024	136.15
24	21	5.137	-39.891	-500.11
24	22	-5.137	39.891	-247.85
25	22	0.000	0.000	0.00
25	23	0.000	-0.000	0.00
26	22	57.045	16.163	111.70
26	25	-57.045	-16.163	114.58

APPLIED JOINT LOADS, FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
2	1.879	0.000	0.00
3	-0.007	-0.000	-0.00
4	2.657	-0.000	0.00
5	30.158	-0.000	0.00
6	-0.001	-0.000	0.00
7	-0.001	-0.000	0.00
8	0.000	0.000	0.00
9	13.517	-0.000	-0.00
10	0.000	0.000	-0.00
11	-0.001	-0.000	-0.00
12	0.001	0.000	0.00
13	2.830	0.000	0.00
14	-0.000	0.000	-0.00
15	1.251	0.000	-0.00
16	-0.000	-0.000	0.00
17	-0.000	-0.000	-0.00
18	5.260	0.000	0.05
19	0.011	0.000	-0.02
20	-0.006	-0.000	-0.00
21	4.204	0.000	-0.00
22	0.000	-0.000	-0.00
23	0.000	-0.000	0.00

REACTIONS,APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	-13.288	-8.584	230.85
24	-32.302	-48.462	308.51
25	-16.163	57.045	114.58

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
2	0.0072	0.0000	-0.0008
3	0.0167	0.0000	-0.0007
4	0.0181	0.0000	-0.0006
5	0.0220	0.0000	-0.0000
6	0.0220	0.0010	0.0000

7	0.0219	0.0024	0.0000
8	0.0218	0.0002	-0.0002
9	0.0249	0.0033	-0.0000
10	0.0248	0.0025	-0.0000
11	0.0247	0.0002	-0.0002
12	0.0247	-0.0022	-0.0002
13	0.0228	0.0002	-0.0001
14	0.0227	0.0009	0.0001
15	0.0212	-0.0004	-0.0006
16	0.0145	-0.0004	-0.0010
17	0.0056	-0.0003	-0.0004
18	0.0151	0.0002	-0.0009
19	0.0132	0.0002	-0.0009
20	0.0065	0.0001	-0.0006
21	0.0048	0.0001	-0.0003
22	0.0048	-0.0003	-0.0000
23	0.0048	-0.0005	-0.0000

STRUCTURE ---TYPICAL INTERIOR FRAME --CRD--WHR

LOADING 5 SEISMIC (RIGHT LEFT)

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MEMBER FORCES

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	8.902	-13.201	-228.53
1	2	-8.902	13.201	43.70
2	2	8.902	-11.322	-43.70
2	3	-8.902	11.322	-80.83
3	3	8.902	-11.313	80.86
3	4	-8.902	11.313	-103.49
4	4	8.902	-8.672	103.50
4	5	-8.902	8.672	-190.22
5	5	8.673	8.902	190.22
5	6	-8.673	-8.902	-6.21
6	6	8.675	8.902	6.21
6	7	-8.675	-8.902	171.82
7	7	6.440	6.712	-154.53
7	8	-6.440	-6.712	304.41
8	9	0.001	0.000	0.00
8	10	-0.001	-0.000	-0.00
9	10	2.238	2.189	14.02
9	11	-2.238	-2.189	34.87
10	11	13.521	0.000	0.00
10	12	-13.521	-0.000	0.00
11	7	2.190	-2.236	-17.28
11	10	-2.190	2.236	-14.02
12	11	-2.189	-11.281	-34.87
12	13	2.189	11.281	-72.30
13	13	13.769	-6.917	-88.86
13	8	-13.769	6.917	57.73
14	8	7.057	-30.633	-362.16
14	18	-7.057	30.633	55.82
15	18	7.057	-30.724	-55.93
15	19	-7.057	30.724	-5.51
16	19	7.057	-30.703	5.52
16	20	-7.057	30.703	-245.94
17	20	7.057	-30.705	245.94
17	21	-7.057	30.705	-343.27
18	21	47.259	-32.767	-149.81
18	24	-47.259	32.767	-308.93
19	13	-4.366	15.958	161.14
19	14	4.366	-15.958	22.37
20	14	-16.366	6.201	-22.37
20	15	16.366	-6.201	85.95
21	15	-15.957	-8.438	-85.95
21	16	15.957	8.438	24.77

22	16	-15.957	-13.638	-24.77
22	17	15.957	13.638	-109.29
23	17	-15.957	-13.637	109.29
23	22	15.957	13.637	-152.52
24	21	2.065	40.201	493.08
24	22	-2.065	-40.201	260.69
25	22	4.200	-0.000	-0.00
25	23	-4.200	0.000	-0.00
26	22	-56.160	-15.772	-108.17
26	25	56.160	15.772	-112.64

APPLIED JOINT LOADS. FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
2	-1.879	0.000	-0.00
3	-0.008	0.000	0.02
4	-2.640	0.000	0.00
5	0.001	0.000	-0.00
6	0.001	0.000	-0.00
7	0.001	0.000	-0.00
8	-30.156	0.000	-0.01
9	0.001	0.000	0.00
10	0.001	-0.000	0.00
11	0.000	0.000	0.00
12	-13.521	-0.000	0.00
13	-0.002	-0.000	-0.01
14	-2.821	-0.001	-0.00
15	-1.250	-0.000	-0.00
16	-5.199	0.000	-0.00
17	0.000	0.000	0.00
18	-0.090	-0.000	-0.11
19	0.020	-0.000	0.01
20	-0.001	0.000	0.00
21	0.002	-0.000	-0.00
22	-0.000	0.000	0.00
23	-4.200	0.000	-0.00

REACTIONS.APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	13.201	8.902	-228.53
24	32.767	47.259	-308.93
25	15.772	-56.160	-112.64

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
2	-0.0072	-0.0000	0.0008
3	-0.0165	-0.0000	0.0007
4	-0.0179	-0.0000	0.0006
5	-0.0217	-0.0000	0.0000
6	-0.0217	-0.0012	-0.0000

7	-0.0217	-0.0027	-0.0000
8	-0.0217	-0.0002	0.0002
9	-0.0250	-0.0037	0.0000
10	-0.0250	-0.0027	0.0000
11	-0.0250	-0.0002	0.0002
12	-0.0250	0.0024	0.0002
13	-0.0228	-0.0002	0.0001
14	-0.0228	-0.0006	-0.0001
15	-0.0216	0.0004	0.0005
16	-0.0152	0.0004	0.0010
17	-0.0058	0.0003	0.0005
18	-0.0149	-0.0001	0.0009
19	-0.0130	-0.0001	0.0009
20	-0.0063	-0.0001	0.0006
21	-0.0047	-0.0001	0.0003
22	-0.0048	0.0003	0.0000
23	-0.0048	0.0007	0.0000

STRUCTURE ---TYPICAL INTERIOR FRAME --CRD--WHR

LOADING 6   ULTIMATE   D+L+E   A.C.E.
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MEMBER FORCES

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	757.228	-54.943	-703.34
1	2	-757.228	54.943	-65.87
2	2	585.028	-57.575	170.87
2	3	-585.028	57.575	-804.20
3	3	457.350	-57.616	514.38
3	4	-457.350	57.616	-629.62
4	4	416.751	-61.319	629.59
4	5	-416.751	61.319	-1242.78
5	5	103.544	374.751	1242.79
5	6	-103.544	-158.006	4263.24
6	6	103.547	139.105	-4263.24
6	7	-103.547	2.294	5631.35
7	7	125.225	-238.957	-5720.02
7	8	-125.225	570.021	-3312.21
8	9	18.932	-48.999	0.00
8	10	-18.932	128.561	-887.81
9	10	-2.749	84.297	672.96
9	11	2.749	93.364	-774.20
10	11	0.012	133.496	1009.10
10	12	-0.012	-42.000	0.00
11	7	217.763	21.579	88.67
11	10	-217.763	-21.679	214.84
12	11	275.857	-2.748	-234.89
12	13	-275.857	2.748	208.78
13	13	460.966	-13.540	-1500.60
13	8	-460.966	13.540	1439.65
14	8	1084.183	111.689	1872.53
14	18	-1084.184	-111.689	-755.64
15	18	1212.990	119.045	755.72
15	19	-1212.990	-119.045	-517.63
16	19	1340.682	119.061	826.99
16	20	-1340.682	-119.061	105.25
17	20	1368.681	119.052	-105.26
17	21	-1368.681	-119.052	482.66
18	21	1428.250	70.360	339.97
18	24	-1428.250	-70.360	645.06
19	13	14.750	154.302	1291.80
19	14	-14.750	-61.888	-48.70
20	14	-20.463	41.319	48.70
20	15	20.463	5.188	136.51
21	15	24.198	-16.388	-135.51
21	16	-24.198	16.388	17.70

22	16	137.598	-16.388	-17.70
22	17	-137.598	16.388	-143.40
23	17	189.398	-16.390	143.39
23	22	-189.398	16.390	-195.35
24	21	54.577	14.766	-822.63
24	22	-54.577	186.045	-783.09
25	22	-0.000	74.390	698.62
25	23	0.000	-62.999	-0.00
26	22	477.834	38.188	279.82
26	25	-477.834	-38.188	254.81

APPLIED JOINT LOADS. FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
2	2.632	-172.199	104.99
3	0.040	-127.678	-289.81
4	3.703	-40.598	-0.02
5	42.224	-42.000	0.01
6	0.003	-18.900	0.00
7	-0.001	-18.899	-0.00
8	0.004	-53.195	-0.01
9	18.932	-48.999	0.00
10	-0.002	-4.904	0.00
11	0.013	-48.996	0.00
12	-0.012	-42.000	0.00
13	3.958	-30.807	-0.01
14	-0.003	-18.200	-0.00
15	1.749	-35.000	-0.00
16	0.000	-113.399	0.00
17	-0.001	-51.799	-0.00
18	7.356	-128.807	0.07
19	0.016	-127.691	309.36
20	-0.009	-27.998	-0.01
21	5.885	-44.802	0.00
22	0.001	-28.000	-0.00
23	0.000	-62.999	-0.00

REACTIONS.APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	54.943	757.228	-703.34
24	-70.360	1428.250	645.06
25	-38.188	477.834	254.81

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
2	-0.0193	-0.0031	0.0019
3	-0.0307	-0.0050	-0.0003
4	-0.0294	-0.0052	-0.0009
5	-0.0020	-0.0065	-0.0050
6	-0.0024	-0.0935	-0.0027

7	-0.0027	-0.1073	0.0020
8	-0.0031	-0.0126	0.0044
9	-0.0379	-0.1924	0.0088
10	-0.0380	-0.1103	0.0067
11	-0.0379	-0.0139	0.0017
12	-0.0379	-0.0190	-0.0013
13	-0.0185	-0.0132	0.0023
14	-0.0185	-0.0052	-0.0001
15	-0.0168	-0.0036	0.0009
16	-0.0066	-0.0036	0.0016
17	0.0076	-0.0030	0.0008
18	0.0177	-0.0098	0.0003
19	0.0179	-0.0091	-0.0000
20	0.0124	-0.0064	-0.0009
21	0.0097	-0.0052	-0.0006
22	0.0094	-0.0027	0.0002
23	0.0094	-0.0127	-0.0015

STRUCTURE ---TYPICAL INTERIOR FRAME --CRD--WHR

LOADING 7   ULTIMATE D+L+E   A.C.E.

MEMBER FORCES

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	781.710	-92.030	-1346.48
1	2	-781.710	92.030	58.05
2	2	609.510	-89.399	46.94
2	3	-609.510	89.399	-1030.33
3	3	481.831	-89.438	740.55
3	4	-481.831	89.438	-919.43
4	4	441.233	-85.724	919.41
4	5	-441.233	85.724	-1776.65
5	5	85.728	399.233	1776.66
5	6	-85.728	-182.487	4235.41
6	6	85.735	163.586	-4235.41
6	7	-85.735	-22.187	6093.15
7	7	101.417	-220.402	-6135.43
7	8	-101.417	551.467	-2482.49
8	9	0.009	-48.999	0.00
8	10	-0.009	128.561	-887.81
9	10	-15.670	90.223	710.58
9	11	15.670	87.438	-679.48
10	11	18.945	133.496	1009.10
10	12	-18.945	-42.000	-0.00
11	7	223.691	15.679	42.28
11	10	-223.691	-15.679	177.23
12	11	269.930	-34.598	-329.61
12	13	-269.930	34.598	0.92
13	13	501.398	-29.558	-1747.50
13	8	-501.398	29.558	1614.48
14	8	1106.061	29.643	867.96
14	18	-1106.061	-29.643	-571.52
15	18	1234.869	29.508	571.36
15	19	-1234.869	-29.508	-512.35
16	19	1362.561	29.536	821.77
16	20	-1362.561	-29.536	-590.50
17	20	1390.560	29.534	590.50
17	21	-1390.560	-29.534	-496.87
18	21	1562.260	-20.738	-70.98
18	24	-1562.260	20.738	-219.35
19	13	-5.047	200.660	1746.52
19	14	5.047	-108.246	29.68
20	14	-70.034	57.305	-29.68
20	15	70.034	-10.797	378.81
21	15	-22.155	-43.638	-378.81
21	16	22.155	43.638	62.43

22	16	91.243	-50.918	-62.43
22	17	-91.243	50.918	-438.09
23	17	143.042	-50.917	438.10
23	22	-143.042	50.917	-599.50
24	21	50.276	126.898	567.84
24	22	-50.276	73.913	-71.12
25	22	5.880	74.390	698.62
25	23	-5.880	-62.999	-0.00
26	22	319.346	-6.521	-27.99
26	25	-319.346	6.521	-63.30

APPLIED JOINT LOADS, FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
2	-2.631	-172.199	104.99
3	0.039	-127.678	-289.78
4	-3.713	-40.598	-0.01
5	0.004	-42.000	0.00
6	0.006	-18.900	0.00
7	0.002	-18.898	-0.00
8	-42.215	-53.195	-0.04
9	0.009	-48.999	0.00
10	-0.000	-4.906	0.00
11	0.017	-48.995	0.01
12	-18.945	-42.000	-0.00
13	-0.007	-30.807	-0.05
14	-3.952	-18.203	0.00
15	-1.752	-35.001	0.00
16	-7.279	-113.399	-0.00
17	0.001	-51.799	0.00
18	-0.134	-128.807	-0.15
19	0.027	-127.691	309.42
20	-0.002	-27.998	-0.00
21	0.003	-44.802	-0.00
22	-0.000	-28.000	0.00
23	-5.880	-62.999	-0.00

REACTIONS,APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	92.030	781.710	-1346.48
24	20.738	1562.260	-219.35
25	6.521	319.346	-63.30

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
2	-0.0396	-0.0032	0.0043
3	-0.0773	-0.0051	0.0017
4	-0.0800	-0.0054	0.0009
5	-0.0634	-0.0067	-0.0049
6	-0.0636	-0.1016	-0.0030

[illegible]	-0.0636	-0.1016	-0.0030
7	-0.0639	-0.1146	0.0019
8	-0.0643	-0.0132	0.0051
9	-0.1078	-0.2024	0.0091
10	-0.1078	-0.1177	0.0069
11	-0.1077	-0.0145	0.0023
12	-0.1077	-0.0125	-0.0007
13	-0.0824	-0.0138	0.0028
14	-0.0824	-0.0075	-0.0004
15	-0.0768	-0.0023	0.0026
16	-0.0483	-0.0024	0.0046
17	-0.0084	-0.0020	0.0023
18	-0.0243	-0.0103	0.0029
19	-0.0188	-0.0097	0.0026
20	-0.0056	-0.0069	0.0008
21	-0.0037	-0.0057	0.0003
22	-0.0040	-0.0018	0.0003
23	-0.0040	-0.0109	-0.0014

STRUCTURE ---TYPICAL INTERIOR FRAME --CRD--WHR

MEMBER FORCES FOR MEMBER 1

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	409.030	-34.017	-499.64
1	2	-409.030	34.017	23.39
2	1	54.186	-10.982	-156.75
2	2	-54.186	10.982	2.99
3	1	86.245	-7.533	-76.84
3	2	-86.245	7.533	-28.62
4	1	-8.584	13.288	230.85
4	2	8.584	-13.288	-44.81
5	1	8.902	-13.201	-228.53
5	2	-8.902	13.201	43.70
6	1	757.228	-54.943	-703.34
6	2	-757.228	54.943	-65.87
7	1	781.710	-92.030	-1346.48
7	2	-781.710	92.030	58.05

MEMBER FORCES FOR MEMBER 2

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	2	286.030	-34.017	51.60
1	3	-286.030	34.017	-425.80
2	2	54.186	-10.983	-2.99
2	3	-54.186	10.983	-117.82
3	2	86.245	-7.533	28.62
3	3	-86.245	7.533	-111.49
4	2	-8.584	11.408	44.81
4	3	8.584	-11.408	80.68
5	2	8.902	-11.322	-43.70
5	3	-8.902	11.322	-80.83
6	2	585.028	-57.575	170.87
6	3	-585.028	57.575	-804.20
7	2	609.510	-89.399	46.94
7	3	-609.510	89.399	-1030.33

MEMBER FORCES FOR MEMBER 3

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	3	280.831	-34.051	412.78
1	4	-280.831	34.051	-480.88
2	3	54.186	-10.982	117.82
2	4	-54.186	10.982	-139.79
3	3	0.245	-7.536	[illegible]

3	4	-0.245	7.536	67.43
4	3	-8.584	11.416	-80.68
4	4	8.584	-11.416	103.51
5	3	8.902	-11.313	80.86
5	4	-8.902	11.313	-103.49
6	3	457.350	-57.616	514.38
6	4	-457.350	57.616	-629.62
7	3	481.831	-89.438	740.55
7	4	-481.831	89.439	-919.43

MEMBER FORCES FOR MEMBER 4

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LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	4	251.832	-34.035	480.86
1	5	-251.832	34.035	-821.22
2	4	54.186	-10.988	139.79
2	5	-54.186	10.988	-249.68
3	4	0.245	-7.534	-67.43
3	5	-0.245	7.534	-7.91
4	4	-8.584	8.759	-103.51
4	5	8.584	-8.759	191.10
5	4	8.902	-8.672	103.50
5	5	-8.902	8.672	-190.22
6	4	416.751	-61.319	629.59
6	5	-416.751	61.319	-1242.78
7	4	441.233	-85.724	919.41
7	5	-441.233	85.724	-1776.65

MEMBER FORCES FOR MEMBER 5

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LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	5	34.036	221.832	821.22
1	6	-34.036	-92.644	2428.88
2	5	10.989	54.186	249.68
2	6	-10.989	-28.556	605.46
3	5	7.534	0.245	7.91
3	6	-7.534	-0.245	-2.83
4	5	21.399	-8.584	-191.10
4	6	-21.399	8.584	13.66
5	5	8.673	8.902	190.22
5	6	-8.673	-8.902	-6.21
6	5	103.544	374.751	1242.79
6	6	-103.544	-158.006	4263.24
7	5	85.728	399.233	1776.66
7	6	-85.728	-182.487	4235.41

MEMBER FORCES FOR MEMBER 6

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LOADING JOINT AXIAL FORCE SHEAR FORCE MOMENT

1	6	34.039	79.144	-2428.88
1	7	-34.039	1.855	3201.77
2	6	10.989	28.555	-605.46
2	7	-10.989	-8.555	976.57
3	6	7.535	0.245	2.83
3	7	-7.535	-0.245	2.07
4	6	21.398	-8.584	-13.66
4	7	-21.398	8.584	-158.03
5	6	8.675	8.902	6.21
5	7	-8.675	-8.902	171.82
6	6	103.547	139.105	-4263.24
6	7	-103.547	2.294	5631.39
7	6	85.735	163.586	-4235.41
7	7	-85.735	-22.187	6093.15

MEMBER FORCES FOR MEMBER 7

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	7	47.578	-148.535	-3257.57
1	8	-47.578	288.097	-1617.43
2	7	10.952	-15.807	-968.75
2	8	-10.952	112.719	-466.24
3	7	7.469	0.199	-1.58
3	8	-7.469	-0.199	6.04
4	7	23.446	-6.540	142.18
4	8	-23.446	6.540	-288.24
5	7	6.440	6.712	-154.53
5	8	-6.440	-6.712	304.41
6	7	125.225	-238.957	-5720.02
6	8	-125.225	570.021	-3312.21
7	7	101.417	-220.402	-6135.43
7	8	-101.417	551.467	-2482.49

MEMBER FORCES FOR MEMBER 8

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	9	0.004	-35.000	0.00
1	10	-0.004	79.430	-572.15
2	9	0.001	0.000	0.00
2	10	-0.001	12.399	-61.99
3	9	0.000	0.000	0.00
3	10	-0.000	-0.000	0.00
4	9	13.517	-0.000	-0.00
4	10	-13.517	0.000	0.00
5	9	0.001	0.000	0.00
5	10	-0.001	-0.000	-0.00
6	9	18.932	-48.999	0.00
6	10	-18.932	128.561	-887.81
7	9	0.009	-48.999	0.00
7	10	-0.009	128.561	-887.81

MEMBER FORCES FOR MEMBER 9

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	10	-13.536	50.247	438.41
1	11	13.536	48.964	-424.09
2	10	0.038	11.962	54.70
2	11	-0.038	15.726	-96.72
3	10	0.065	0.045	0.42
3	11	-0.065	-0.045	0.59
4	10	11.467	-2.043	-12.84
4	11	-11.467	2.043	-32.78
5	10	2.238	2.189	14.02
5	11	-2.238	-2.189	34.87
6	10	-2.749	84.297	672.96
6	11	2.749	93.364	-774.20
7	10	-15.670	90.225	710.58
7	11	15.670	87.438	-679.48

MEMBER FORCES FOR MEMBER 10

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	11	0.007	81.094	638.79
1	12	-0.007	-30.000	0.00
2	11	0.002	14.260	81.99
2	12	-0.002	-0.000	-0.00
3	11	0.000	0.000	0.00
3	12	-0.000	-0.000	0.00
4	11	-0.001	-0.000	-0.00
4	12	0.001	0.000	0.00
5	11	13.521	0.000	0.00
5	12	-13.521	-0.000	0.00
6	11	0.012	133.496	1009.10
6	12	-0.012	-42.000	0.00
7	11	18.945	133.496	1009.10
7	12	-18.945	-42.000	-0.00

MEMBER FORCES FOR MEMBER 11

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	7	133.180	13.538	55.80
1	10	-133.180	-13.538	133.73
2	7	24.363	-0.037	-7.81
2	10	-24.363	0.037	7.29
3	7	0.045	-0.065	-0.49
3	10	-0.045	0.065	-0.42
4	7	-2.043	2.049	15.84
4	10	2.043	-2.049	12.84
5	7	2.190	-2.236	-17.28
5	10	-2.190	2.236	-14.43
6	7	217.763	21.679	88.17
6	10	-217.763	-21.679	[illegible]

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7	7	223.691	15.679	42.28
7	10	-223.691	-15.679	177.23

MEMBER FORCES FOR MEMBER 12

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	11	165.057	-13.535	-214.69
1	13	-165.057	13.535	86.11
2	11	29.985	0.038	14.72
2	13	-29.985	-0.038	-14.36
3	11	-0.045	0.065	-0.59
3	13	0.045	-0.065	1.21
4	11	2.043	11.468	32.78
4	13	-2.043	-11.468	76.16
5	11	-2.189	-11.281	-34.87
5	13	2.189	11.281	-72.30
6	11	275.857	-2.748	-234.89
6	13	-275.857	2.748	208.78
7	11	269.930	-34.598	-329.61
7	13	-269.930	34.598	0.92

MEMBER FORCES FOR MEMBER 13

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	13	284.371	-12.287	-901.30
1	8	-284.371	12.287	846.01
2	13	59.887	-2.472	-260.28
2	8	-59.887	2.472	249.16
3	13	0.113	0.564	2.23
3	8	-0.113	-0.564	0.30
4	13	-15.110	4.523	87.49
4	8	15.110	-4.523	-67.13
5	13	13.769	-6.917	-88.86
5	8	-13.769	6.917	57.73
6	13	460.966	-13.540	-1500.60
6	8	-460.966	13.540	1439.66
7	13	501.398	-29.558	-1747.58
7	8	-501.398	29.558	1614.48

MEMBER FORCES FOR MEMBER 14

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	8	610.467	35.292	771.40
1	18	-610.467	-35.292	-418.47
2	8	172.606	8.480	217.07
2	18	-172.606	-8.480	-132.27
3	8	-0.086	8.034	-6.35
3	18	0.086	-8.034	86.69
4	8	-8.569	27.970	355.39

3	18	0.086	-8.034	86.69
4	8	-8.569	27.970	35[illegible]9
4	18	8.569	-27.970	-75.69
5	8	7.057	-30.633	-362.16
5	18	-7.057	30.633	55.82
6	8	1084.183	111.689	1872.53
6	18	-1084.184	-111.689	-755.64
7	8	1106.061	29.643	867.96
7	18	-1106.061	-29.643	-571.52

MEMBER FORCES FOR MEMBER 15

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	18	672.470	35.292	418.48
1	19	-672.470	-35.292	-347.89
2	18	202.608	8.477	132.27
2	19	-202.608	-8.477	-115.31
3	18	-0.085	8.032	-86.69
3	19	0.085	-8.032	102.76
4	18	-8.570	33.230	75.75
4	19	8.570	-33.230	-9.29
5	18	7.057	-30.724	-55.93
5	19	-7.057	30.724	-5.51
6	18	1212.990	119.045	755.72
6	19	-1212.990	-119.045	-517.63
7	18	1234.869	29.508	571.36
7	19	-1234.869	-29.508	-512.35

MEMBER FORCES FOR MEMBER 16

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	19	677.676	35.293	360.89
1	20	-677.676	-35.293	-84.54
2	19	202.609	8.476	115.31
2	20	-202.609	-8.476	-48.94
3	19	85.914	8.031	105.23
3	20	-85.914	-8.031	-42.35
4	19	-8.570	33.242	9.26
4	20	8.570	-33.242	251.02
5	19	7.057	-30.703	5.52
5	20	-7.057	30.703	-245.94
6	19	1340.682	119.061	826.99
6	20	-1340.682	-119.061	105.25
7	19	1362.561	29.536	821.77
7	20	-1362.561	-29.536	-590.50

MEMBER FORCES FOR MEMBER 17

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1	21	-697.676	-35.293	27.33
2	20	202.609	8.476	48.94
2	21	-202.609	-8.476	-22.07
3	20	85.914	8.031	42.35
3	21	-85.914	-8.031	-16.89
4	20	-8.570	33.235	-251.03
4	21	8.570	-33.235	356.39
5	20	7.057	-30.705	245.94
5	21	-7.057	30.705	-343.27
6	20	1368.681	119.052	-105.26
6	21	-1368.681	-119.052	482.66
7	20	1390.560	29.534	590.50
7	21	-1390.560	-29.534	-496.87

MEMBER FORCES FOR MEMBER 18

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	21	758.185	11.489	58.83
1	24	-758.185	-11.489	102.01
2	21	226.114	0.806	0.85
2	24	-226.114	-0.806	10.43
3	21	84.341	5.658	39.42
3	24	-84.341	-5.658	39.79
4	21	-48.462	32.302	143.71
4	24	48.462	-32.302	308.51
5	21	47.259	-32.767	-149.81
5	24	-47.259	32.767	-308.93
6	21	1428.250	70.360	339.97
6	24	-1428.250	-70.360	645.06
7	21	1562.260	-20.738	-70.98
7	24	-1562.260	20.738	-219.35

MEMBER FORCES FOR MEMBER 19

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	13	-1.250	97.310	815.18
1	14	1.250	-45.560	6.32
2	13	2.510	29.900	274.64
2	14	-2.510	-15.640	-12.78
3	13	-0.499	0.159	-3.45
3	14	0.499	-0.159	5.28
4	13	9.775	-17.154	-163.65
4	14	-9.775	17.154	-33.61
5	13	-4.366	15.958	161.14
5	14	4.366	-15.958	22.37
6	13	14.750	154.302	1291.80
6	14	-14.750	-61.888	-48.70
7	13	-5.047	200.660	1746.52
7	14	5.047	-108.246	29.68

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	14	-23.908	22.137	-6.32
1	15	23.908	-1.631	128.17
2	14	-9.284	12.834	12.78
2	15	9.284	-0.120	53.62
3	14	-0.465	-0.240	-5.28
3	15	0.465	0.240	2.82
4	14	19.041	-5.217	33.61
4	15	-19.041	5.217	-87.11
5	14	-16.366	6.201	-22.37
5	15	16.366	-6.201	85.95
6	14	-20.463	41.319	48.70
6	15	20.463	5.188	136.51
7	14	-70.034	57.305	-29.68
7	15	70.034	-10.797	378.81

MEMBER FORCES FOR MEMBER 21

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	15	6.941	-15.752	-128.17
1	16	-6.941	15.752	13.97
2	15	-6.650	-6.480	-53.62
2	16	6.650	6.480	6.64
3	15	-0.159	-0.499	-2.82
3	16	0.159	0.499	-0.79
4	15	17.152	11.025	87.11
4	16	-17.152	-11.025	-7.17
5	15	-15.957	-8.438	-85.95
5	16	15.957	8.438	24.77
6	15	24.198	-16.388	-136.51
6	16	-24.198	16.388	17.70
7	15	-22.155	-43.638	-378.81
7	16	22.155	43.638	62.43

MEMBER FORCES FOR MEMBER 22

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	16	73.940	-15.752	-13.97
1	17	-73.940	15.752	-140.87
2	16	7.349	-6.480	-6.64
2	17	-7.349	6.480	-57.05
3	16	-0.159	-0.499	0.79
3	17	0.159	0.499	-5.70
4	16	17.153	11.025	7.17
4	17	-17.153	-11.025	101.20
5	16	-15.957	-13.638	-24.77
5	17	15.957	13.638	-109.29
6	16	137.598	-16.388	-17.70
6	17	-137.598	16.388	-143.40
7	16	91.243	-50.918	-62.43
7	17	-91.243	50.918	[illegible].09

7	17	-91.243	[illegible]	[illegible]

MEMBER FORCES FOR MEMBER 23

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	17	110.940	-15.752	140.87
1	22	-110.940	15.752	-190.81
2	17	7.349	-6.480	57.05
2	22	-7.349	6.480	-77.59
3	17	-0.159	-0.499	5.70
3	22	0.159	0.499	-7.28
4	17	17.153	11.024	-101.20
4	22	-17.153	-11.024	136.15
5	17	-15.957	-13.637	109.29
5	22	15.957	13.637	-152.52
6	17	189.398	-16.390	143.39
6	22	-189.398	16.390	-195.35
7	17	143.042	-50.917	438.10
7	22	-143.042	50.917	-599.50

MEMBER FORCES FOR MEMBER 24

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	21	23.803	28.507	-86.16
1	22	-23.803	56.804	-179.11
2	21	7.670	23.504	21.21
2	22	-7.670	34.620	-125.42
3	21	2.372	-1.572	-22.53
3	22	-2.372	1.572	-6.95
4	21	5.137	-39.891	-500.11
4	22	-5.137	39.891	-247.85
5	21	2.065	40.201	493.08
5	22	-2.065	-40.201	260.69
6	21	54.577	14.766	822.65
6	22	-54.577	186.045	-783.09
7	21	50.276	136.898	567.84
7	22	-50.276	73.913	-71.12

MEMBER FORCES FOR MEMBER 25

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	22	-0.000	34.135	305.79
1	23	0.000	-25.999	-0.00
2	22	-0.000	18.999	193.22
2	23	0.000	-18.999	-0.00
3	22	-0.000	-0.000	-0.00
3	23	0.000	0.000	-0.00
4	22	0.000	0.000	0.00
4	23	0.000	-0.000	0.00
5	22	[illegible]	[illegible]	[illegible]

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3	23	0.000	0.000	-0.00
4	22	0.000	0.000	0.00
4	23	0.000	-0.000	[illegible]
5	22	4.200	-0.000	-0.00
5	23	-4.200	0.000	-0.00
6	22	-0.000	74.390	698.62
6	23	0.000	-62.999	-0.00
7	22	5.880	74.390	698.62
7	23	-5.880	-62.999	-0.00

MEMBER FORCES FOR MEMBER 26
===

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	22	221.881	8.051	64.13
1	25	-221.881	-8.051	48.58
2	22	60.969	1.190	9.79
2	25	-60.969	-1.190	6.87
3	22	1.413	1.873	14.24
3	25	-1.413	-1.873	11.97
4	22	57.045	16.163	111.70
4	25	-57.045	-16.163	114.58
5	22	-56.160	-15.772	-108.17
5	25	56.160	15.772	-112.64
6	22	477.834	38.188	279.82
6	25	-477.834	-38.188	254.81
7	22	319.346	-6.521	-27.99
7	25	-319.346	6.521	-63.30

PROJECT	C.R.D.	ACC. NO.	1856.70
SUBJECT	SEISMIC REQUIREMENTS UNIFORM BUILDING CODE & TM 5-809-10	SHEET NO. ___ OF	E-1
		DATE	6-13 1973
COMP.	WHR	CHECK	KS
		CONT. NO.	

CODE REQUIREMENTS

PROJECT - CHARLES RIVER DAM - PUMPHOUSE, WALKWAY, WALKWAY SUPPORTS, & STAIRTOWER

ZONE - THREE (3) BY CORPS OF ENGINEERS
$Z = 1.0$

GENERAL: $V = ZKCW$ (14.1 UBC Pg 120)

WHERE — Z - DETERMINED BY ZONE
K - TABLE 23-H (UBC P.132) (.67)
C - .10 FOR 1 & 2 STORY BUILDINGS

or $C = \frac{0.05}{\sqrt[3]{T}}$ (14.2 UBC)

Where $T = .10N$ (14.3A UBC)
FOR 100% MOMENT RESISTANT FRAME

N - TOTAL NUMBER OF STORIES ABOVE EXTERIOR GRADE (4)

$\therefore T = .10(4) = .40$

$C = .05/\sqrt[3]{.40} = .05/.737 = \underline{.0678}$

$V = (1.0)\times(.67)\cdot(.0678)W = \underline{.0454W}$

DISTRIBUTION OF LATERAL FORCES

$F_T = .004V\left(\frac{H_n}{D_s}\right)^2$ if $\frac{H_n}{D_s} \leq 3$ $F_T = 0$

$\therefore$ FORMULA 14.5 UBC IS REDUCED TO

$$\boxed{F_x = V \cdot \frac{W_x h_x}{\Sigma W_x h_x}}$$ (14.5A Pg 121)

A-133

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11-72B

PROJECT CRD ACC. NO. 1856.70
SUBJECT Seismic Requirements SHEET NO. __ OF E-2
DATE 6-11 1973
COMP. WHR CHECK KS CONT. NO.

Lateral Forces On Portions of Structure

$$F_p = Z C_p W_p$$

Where $Z = 1.0$
C_p = Table 23-I Pg 135
W_p = Weight

Setback Requirement For Penthouse

Refer TM5-809-10

With $b = 80.0$ $t/b = 30.67/80 = .383$
$H_e = 53.0$
$H_T = 13.0$ $H_T/H_e = 13/53 = .245$
$t = 30.67$

Use Procedure "B", For Details See Pg 3-5 TM5-809-10 (Increase Shear by 40%)

Horizontal Torsional Moments

A - Increases In Shear From Torsion Due To Eccentricity Between Center of Mass & Center of Rigidity
B - Negative Shears Shall Be Neglected
C - $e_{minimum}$ = .05 × Max. Building Dimension

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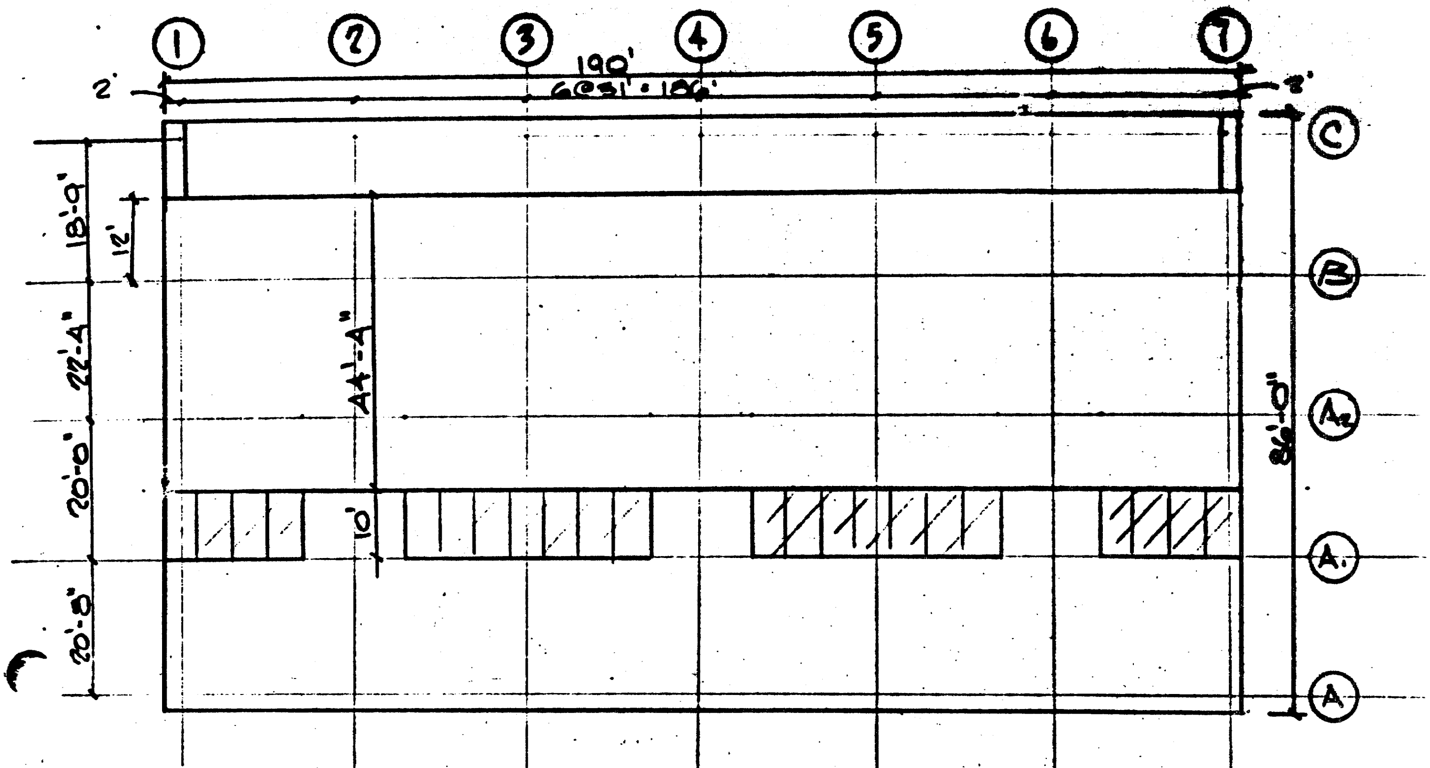
PROJECT C.R.D ACC. NO. 1856.70

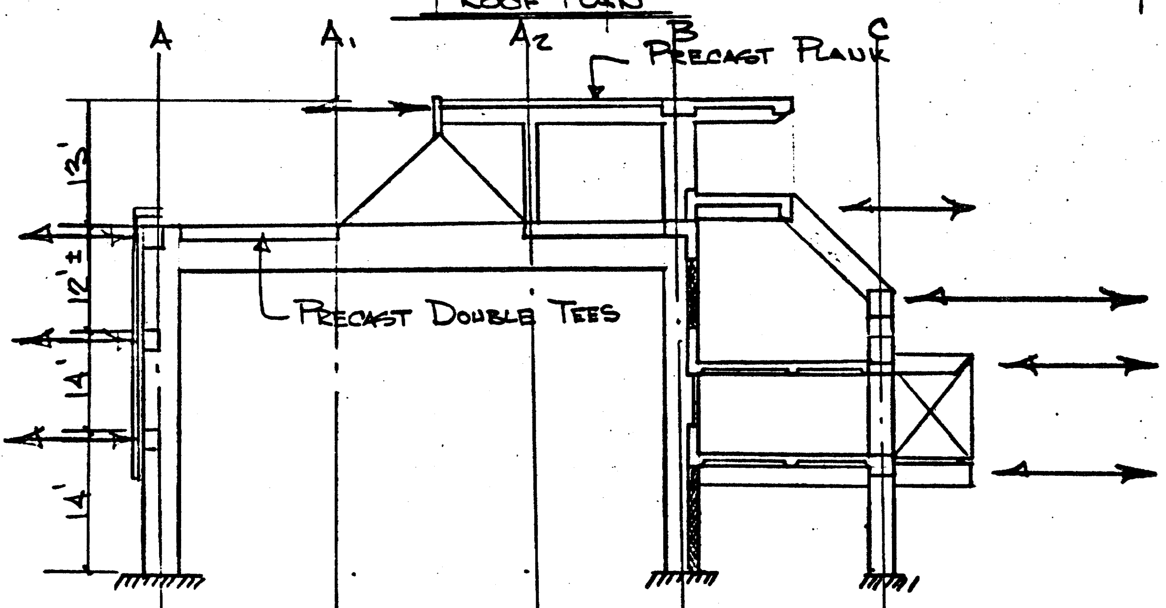
SUBJECT Seismic Requirements SHEET NO. ___ OF E-3

DATE 6-11 1973

COMP. WHR CHECK KS CONT. NO.



Roof Plan



Typical Section

A-135

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11-72B

PROJECT C.R.D ACC. NO. 1656.70
SUBJECT Compute Seismic Loads - Short Direction SHEET NO. ___ OF E-4
DATE 6-11 1973
COMP. WHR CHECK KS CONT. NO. ___

(1) Penthouse Roof

Weight of Frame Ave. (Approx)

(A)

Plank	70 Psf	1×4×.15×31	= 18.6^k
3" Topping	38 Psf	2×1.5×.15×31	= 14.0^k
Roofing	7 Psf	1.5×25.15×31	= 17.4^k
Ave Conc. Frame	50 Psf		50 / 31×32 = 50 Psf Conserv.
	165 Psf ✓		

Area = 190×32 = 6080
W = .165(6080) = 1003.2^k

(B) 10' Overhang @ B To C Lines

DL = 6" Slab	=	75 Psf
Topp		38
Roof		7 Psf
Fram		30 Psf
		150 Psf ✓

A = 12×190 = 2280
W = .150(2280) = 342^k

Therefore

Wx = 1345.2^k ✓
Hx = 53'

(2) Platform @ EL 166±

Brick Piers	(60 Psf × 8' × 20' × 2) + (60×8×10×5) = 19.2+24/190×12	= 19 Psf
6" Slab	—	= 75 Psf
Roofing	—	= 7 Psf
Frame	—	= 50 Psf
		151
		Say 150

(End)

Wx = .150(190×12) = 342^k
Hx = 44' ✓

(3) Penthouse Floor & Main Roof Level

Precast	=	95 Psf
Topping	=	38
Roof or Floor	=	7
Frame	=	75 Psf
Equip	=	30 Psf
		245 Psf

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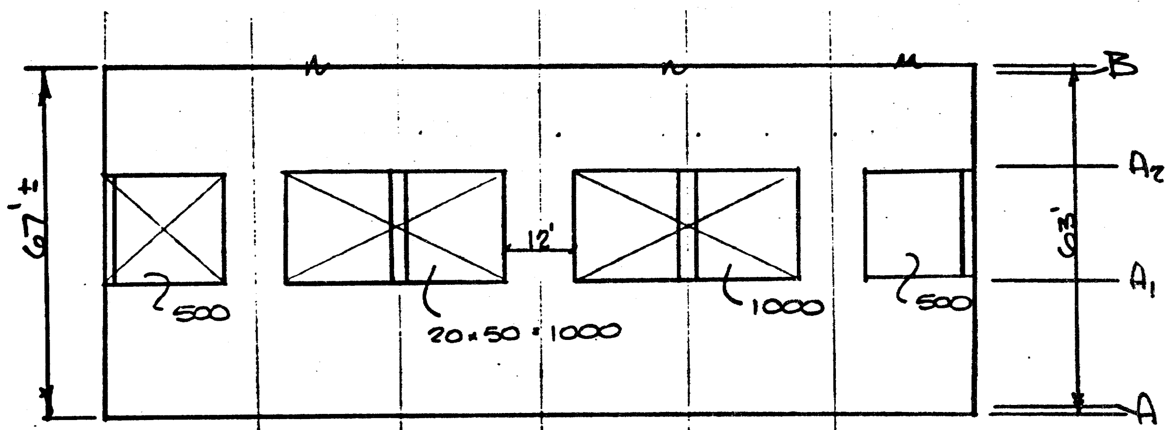
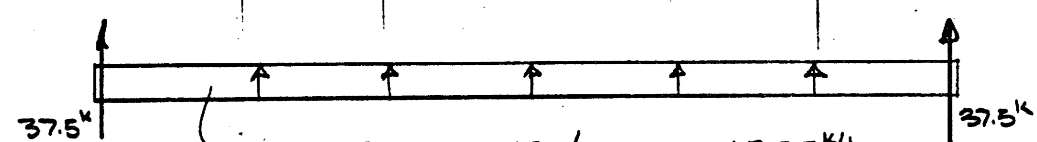
11-72B

PROJECT C.R.D. ACC. NO. 1856.70
SUBJECT Seismic Requirements SHEET NO. ___ OF E-5
DATE 6-11 1973
COMP. WHR CHECK KS CONT. NO. ___



$A_T = 67 \times 190 = 12{,}730 - 3000 = 9730$



$W_e = 9730 \times .245 / 190 = 12.55^{k/'}$ → 12.55

(10psf) L = 12.5'	Louvers	- .125 ×		.125 k/'
(20psf) L 12.5'	Glass (Skylight)	- .250 ×		.250 k/'
(10psf) L = 7'	Back Louvers	- .07 × 19		.070 k/'
6" Brick	End Walls	- 2 × 48' × 13' × 60psf = 75k or 37.5k		
	Columns	11[7(1.5)²·.15 + 7×(2.67×3.17)·.15]/190'		= .65 k/'
	Equip	- 20psf × 9730/190		1.02 k/'
				14.655

$W_x = (14.655)\,190' + 75^k = \underline{2859.45^k}$

$\underline{H_x = 40'}$

A-Line (Spandrel Level #1)

Crane 30k DL/wheel
2 wheels
$W_T = 36.5^k$
Use ½ here & ½ on B-Line

Bm	- 2' × 2' × .150 k/ft³	= .60 k/'
Brick	= 60psf × 14'	= .84 k/'
Column Weight	= .15(2.83' × 2.67') 10'/31'	= .37 k/'
		1.81
1 - Crane Rail	150#/'	.150 k/'
½ Crane Weight		→ Say 19k
		1.96

$W_x = 2.0^k(190+63) + 19^k = \underline{525^k}$

$\underline{H_x = 28'}$

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11-72B

PROJECT C.R.D ACC. NO. 185670
SUBJECT Seismic Requirements SHEET NO. ___ OF E-6
DATE 6-11 1973
COMP. WHR CHECK KS CONT. NO. ___

C-Lines - 1st Spandrel Level

Beam = (2×2) × .150 = .60 k/l
Brick None
Porcelain Panel 15' × .01 ksf = .15 k/l
Beam 2' × 2' × .15 × 10/31 = .20 k/l
.95 Say 1 k/l

∴ W_x = 190 k
H_x = 37'

Upper Mezzanine Level

15" [12 Rib + 3 Slab → .446 c.f./s.f. = 67 psf] ←
14½" [10 Rib + 4½ Slab → .530 = 80 psf]

Beam 4×1 = .60 k/l
Slab 60% Area = 70 psf (60%)(18)(190) / 190 = .756 k/l Ave (inc spandrel)
Columns = + (2×2) .15 × 9 / (3.17 × 2.67) .15 × 12/31 = .67 k/l
Masonry 8" Blk L.W (35 psf) .035(8) = .304 k/l
Equip 10 psf × 190 × 18 → 34 k
Mech 5 psf × 190 × 18 → 17 k
Brick & Int Wall (18).038(10)(12) / 190 .432 k/l
(18).060(10)(2) / 190 .114 k/l
+ .15 Crane Rail
3.03 k/l

& 19 k ½ Crane
70 k

∴ W_x = 3.03 × 190' = 575 k
+ 70 k Add'l
645 k + 55 Walkway = 700 + 75 k

Truss Roof = See Pg T-2
Roof = .290 k/l (½)
(55 k)

W_x = 775 k
H_x = 35' ±

A-Line Spandrel Level #2

Bm - 2×2×.15 = .60 k/l
Brick - 60 psf × 14 (30') .84 k/l (1.8)
Col - (Same As Level #1) .37 k/l
1.81 Use 1.9 k/l (2.86)

Revise Brick Detail

W_x = 1.9(190+63) = 481 k
H_x = 14'

PROJECT C.R.D ACC. NO. 1856.70
SUBJECT Seismic Requirements — Short Direction SHEET NO. ___ OF E-7
DATE 6-14 1973
COMP. WHR CHECK KS CONT. NO.

Lower Mezzanine & Walkway El. 139'-0"

Beams	.6 + .8 k/l	1.40 k/l
1-Way Slab	70 psf × 18'	1.26 k/l
Column	(Same As Before)	.670 k/l
Truss Weight	½ Roof + Floor + Truss	(.290 + .58 + .125 + .125) = 1.12
Cantilevered Bm	4 × 1.5 × 25 × .15/31	.730 k/l
Equipment	5 psf × 18	.09 k/l
Block (Int)	.038(10)(15)(18)/190	.54 k/l
Brick (Ext)		.114 k/l
		Say 6.00 k/l

$W_x = 6(190) = 1140 + 75^k = 1215^k$

$H_x = 18$

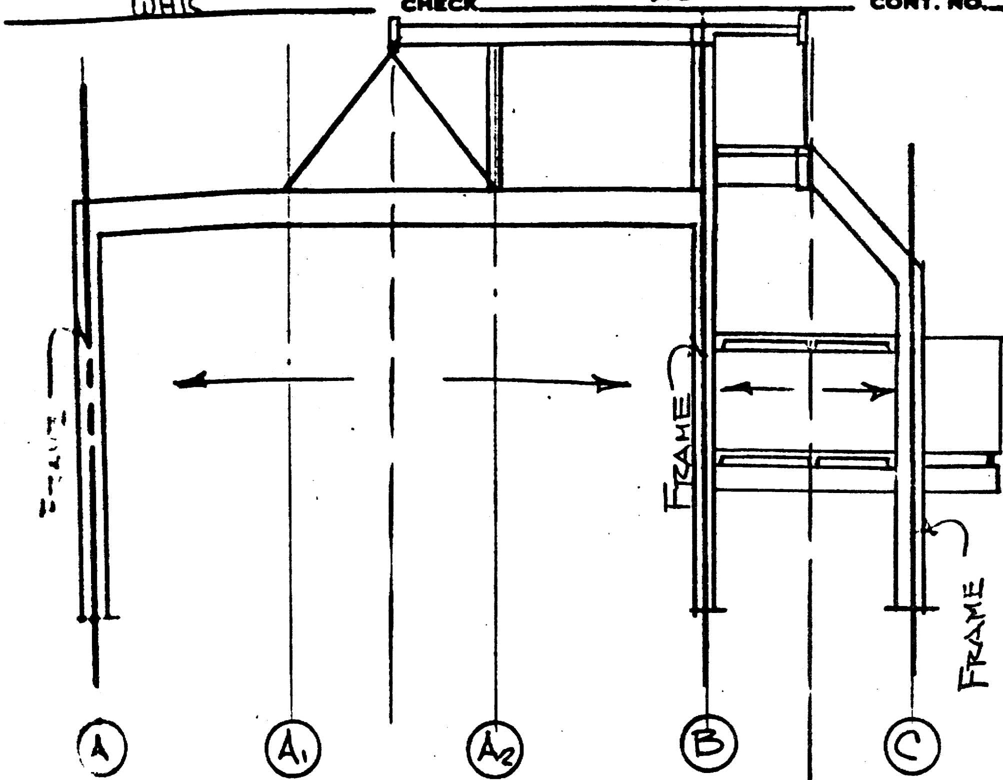
H_x	W_x	$h_x W_x$	$\frac{W_x h_x}{\Sigma Wh}$	F_x	V_x	Force Per Frame (Round off)
53	1345.2	71,296	.256	89.86	89.86	13^k
44	342	15,048	.054	18.95	108.81	2.71^k
40	2859.5	114,380	.411	*144.26	253.07	*29^k
28	525	14,700	.053	18.60	271.67	2.66^k
37	190^k	7,030	.025	8.78	280.45	1.25^k
35	775^k	27,125	.098	34.40	314.85	5.0^k
14	481	6,734	.024	8.42	323.27	1.25^k
18	1215	21,870	.079	27.73	351	4.0^k
	7732.7	278183	1.000			

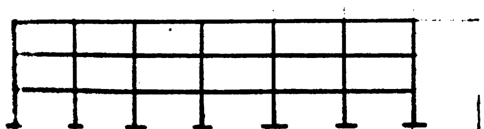
(Pg E-1) Base Shear = .0454 W = .0454 (7732.7) = 351^k

Distribution

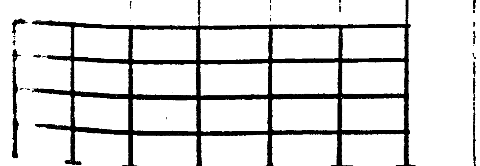
5 - Interior Frames } Distribute Evenly
2 - Exterior Frames } To All Frames

* Increase By 40% - Procedure B for Setback of Penthouse

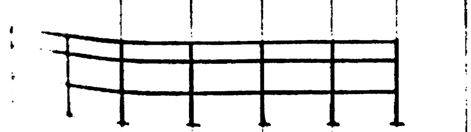




A-Line Frame
(K=.67)



B-Line Frame
(K=.67)



C-Line Frame
(K=.67)

PROJECT	C.R.D.	ACC. NO.	1956.70
SUBJECT	Seismic Requirements Long Direction	SHEET NO. __ OF	E-9
		DATE	6-13 1973
COMP. WHR	CHECK KS	CONT. NO.	

Distribution of Seismic Forces

(A) The Large Frame Will Have A Total Fx Which Will Be Divided Evenly Between Frames On A & B Lines

(B) The Penthouse Will Be Carried By The B-Line Frame

(C) Additional Positive Torsional Shears Will Be Included To Account For An Eccentricities Between The Center of Mass & Center of Rigidity At All Levels

(D) The office Area Between Line B & C Will Have A Fx Which is Carried By B & C Line Frames

A-Line Frame

(A) At Main Roof Level (Ref Pg E-5) $W_x = 2859.45^k/2$

or $W_x = 1430^k$
$H_x = 40' \pm$

(B) 1st Spandrel Level (Ref Pg E-5) $W_x = 525$
$H_x = 28'$

(C) 2nd Spandrel Level (Ref Pg E-6) $W_x = 481^k$
$H_x = 14'$

H_x	W_x	$h_x w_x$	$\frac{h_x w_x}{\Sigma W_x h}$	F_x	V_x
40	1430	57200	.727	80.7	80.7
28	525	14700	.187	20.8	101.5
14	481	6734	.086	9.5	111 (ok)
	2436	78634	1.000		

Base Shear = .0454 (2436) = 110.6 Say 111

B-Line Frame

(A) Penthouse $W_x = 1345.2$
$H_x = 53'$

(B) Plat & Main Roof $W_x = 342^k + 1430 = 1772$
$H_x = 40$

(C) Spandrel $W_x = 775/2$ = Say 388
$H_x = 28'$

1.12 × 190 ≈ 213^k 1215 − 213 = 1002

(D) 139 Level $W_x = 501$
$H_x = 18$

PROJECT C.R.D		ACC. NO. 185679
SUBJECT Seismic Requirements		SHEET NO. ___ OF E-11
		DATE 6-15 1973
COMP. WHR	CHECK KS	CONT. NO.

H_x	W_x	H_xW_x	$\frac{h_xW_x}{\Sigma hw}$	F_x	V_x
53	1345.2	71296	.441	80.3	80.3
40	1772	70880	.439	80.0	160.3
28	388	10864	.067	12.2	172.5
17	501	8517	.053	9.5	182.0
	4006.2	161557	1.000		

BASE SHEAR = .0454 (4006.2) = 182^k

C-LINE FRAME

C LINE SPANDREL — $W_x = 190^k$, $H_x = 35'$

UPPER MEZZANINE — $W_x = 388$, $H_x = 28'$

LOWER MEZZANINE — 1215-501 = $W_x = 714$, $H_x = 17'$

H_x	W_x	H_xW_x	$\frac{h_xW_x}{\Sigma hw}$	F_x	V_x
35	190	6650	.224	13.2	13.2
28	388	10864	.366	21.6	34.8
17	714	12138	.410	24.2	59.0
	1292	29652	1.000		

BASE SHEAR = .0454 (1292) SAY 59^k

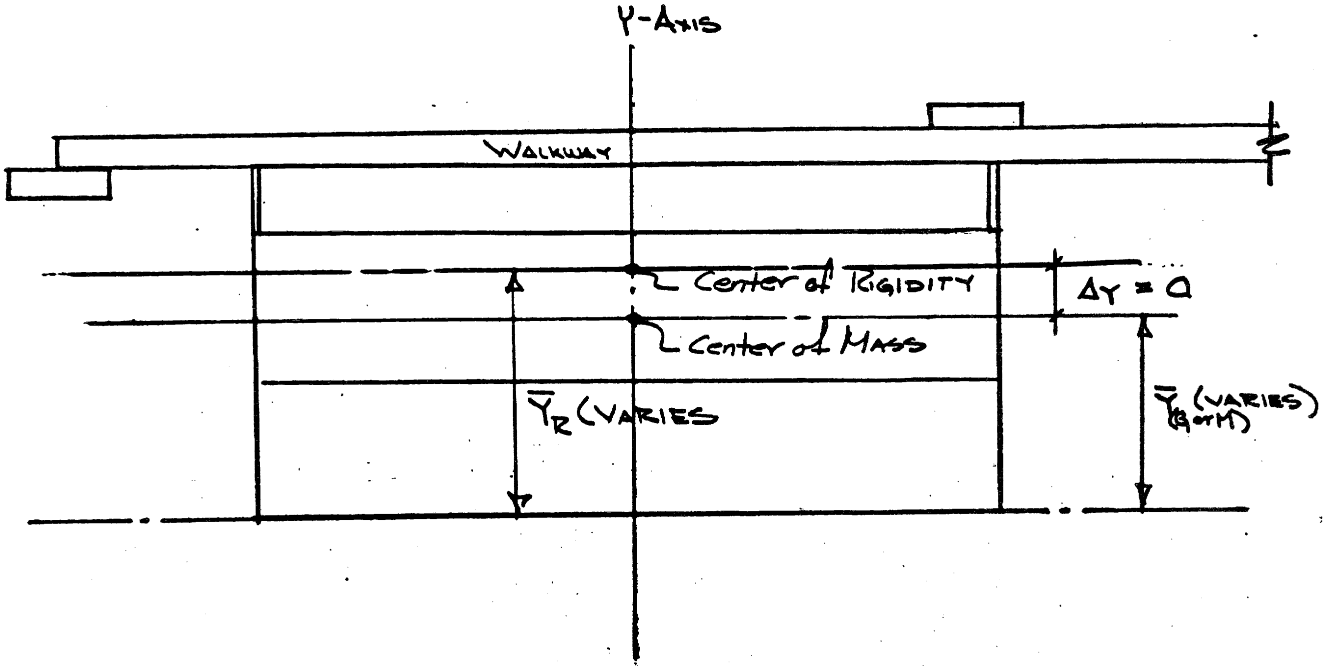
CHECK

W_x (SHORT) = 7732.7^k = W_x (LONG) = 2436 + 4006.2 + 1292 = 7734.2^k

PROJECT C.R.D ACC. NO. 1856.70
SUBJECT Compute Additional Torsional Shears SHEET NO. ___ OF E-12
DATE 6-15 1973
COMP. WHR CHECK KS CONT. NO. ______

DETERMINE - Center of Mass & Center of Rigidity @ All Levels of Structure. Due To The Physical Layout of the Building, The Center of Rigidity & Center of Mass Are Located On An Axis of Symmetry (Y-Axis)

The Centers of Mass & Rigidity Vary In Location Along This Common Axis



Procedure: Determine $\bar{Y}_R$ & $\bar{Y}_M$ @ All Levels. The difference Yields ΔY or a By Which The Torsional Moments Are Determined

(1.1) $T_x = F_x \cdot a_x$

F_T is Then Determined By

$$F_T = \frac{Tc}{J} \quad \text{UBC (2314-9)}$$

PROJECT C.R.D ACC. NO. 1456.70

SUBJECT Determine Center of Mass SHEET NO. __ OF E-13

DATE 6-15 1973

COMP. WHR CHECK KS CONT. NO.

Penthouse Roof

* Refer. Axis - A-Line

(Pg E-4)

Level	Area	P	$\bar{Y}$	$P \cdot \bar{Y}$		
Penthouse Roof (174.50)	6080 2280	1003.2 342. 1345.2	47.25' 72.00	47401.2 24624.0 72025.2	$\bar{Y} = \frac{72025.2}{1345.2}$ = 53.5' from A Line or 9.5' from B Line	

Main Roof & Penthouse Floor (Pg E-5) Inc Plat Form

Level	Element	$\bar{W}_{AVE}$	P	$\bar{Y}$	$P \cdot \bar{Y}$	
162±	Roof	12.55 k/1	2384.5	36' ±	85842	
	Louvers	.125	23.8	33 Ave	785	
	Skylight	.250	47.5	33 Ave	1568	
	Back Louv.	.07	13.3	64'	851	
	EndWall		75k	47.25	3544	$\bar{Y}$ = 41.9' - A-Line
	Columns	.65	123.5	63'	7781	21.1 - B-Line
	Equip	1.02	193.8	47'	9109	
			≈ 2861			
	Plat		342k	72'	24624	
			3203k		134,104	

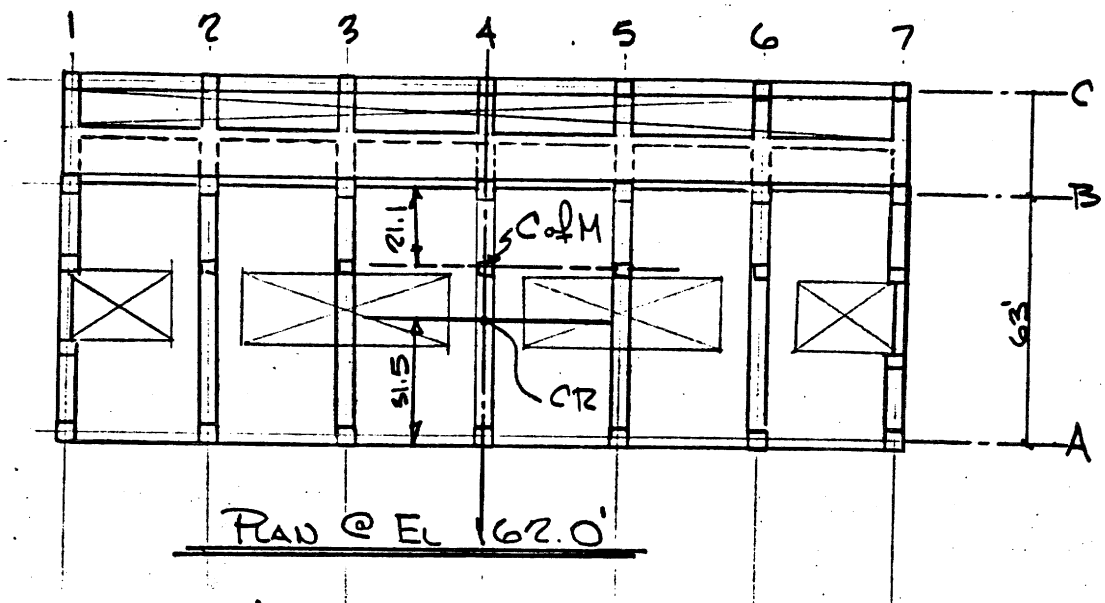
First Spandrel Level (See Pg E-6) (2nd Spandrel Similar)

Level	Element	W	P	$\bar{Y}$	$P \cdot \bar{Y}$	
150±	Crane & C.R.		41.5k	3'	124.5	
	Bm	.60	152	0	0	
	Brick	.84	212	0	0	
	Columns	.37	93	0	0	
C-Line			190k	-83	15770	$\bar{Y}$ = 47.2' A-Line
	Bm, Mas, Col	1.574	300	63	18900	or 15.8 B-Line
	Crane & Rl		41.5	60	2490	
	Slab	.756	144k	73	10512	
	Equip & Mech		51k	73	3723	
	other	.70	133	73	9709	
	Walkway		55	88	4840	
			50	60	3000	
			1463			

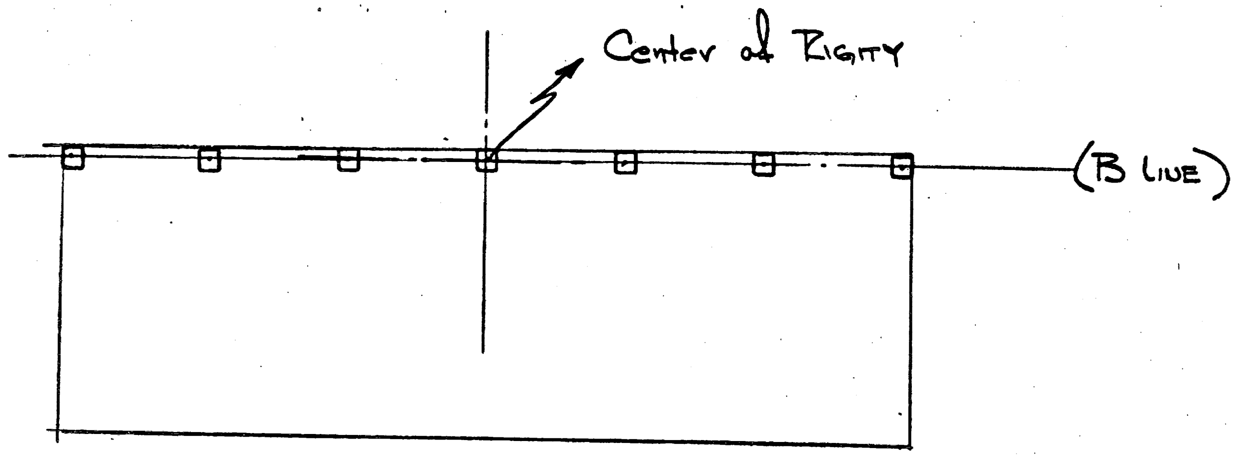
$\bar{Y} = \frac{69055}{1463}$ =

A-145

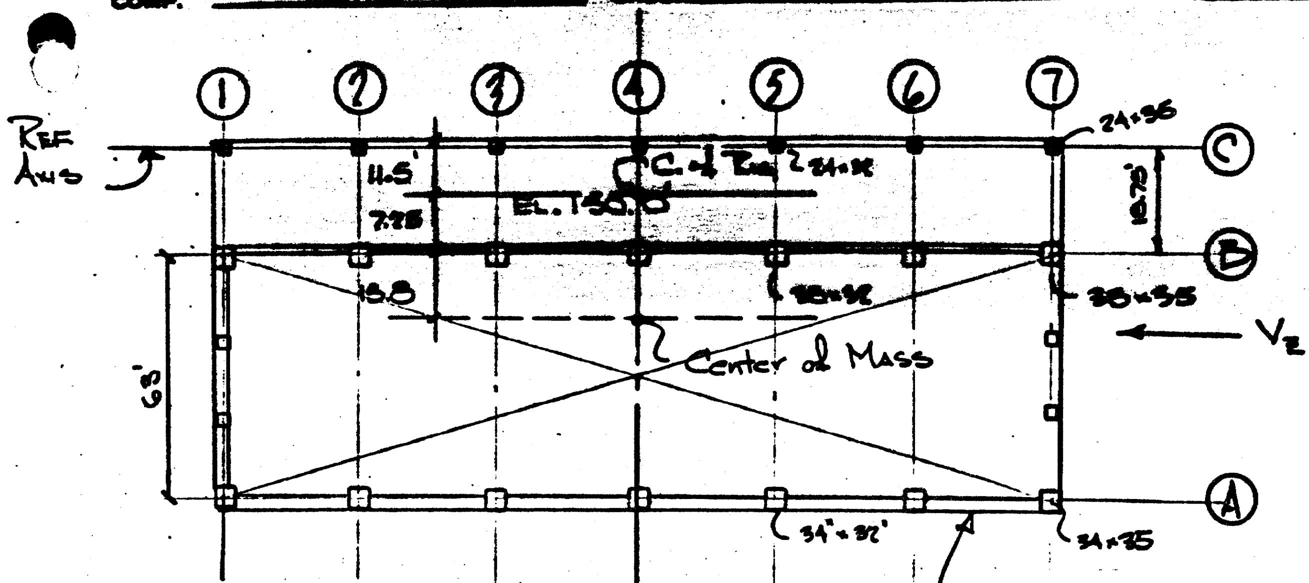
PROJECT	C.R.D	ACC. NO.	1856.70
SUBJECT		SHEET NO. ___ OF	E-14
		DATE	6-18 19 23
COMP. WHR	CHECK KS	CONT. NO.	



Center of Rigidity @ 31.5' From B or A By Symmetry



PROJECT: C.P.D. ACC. NO. 1856.70
SUBJECT: Center of Rigidity SHEET NO. ___ OF E-15
DATE 6-18 1975
COMP. WHR CHECKED: KS CONT. NO. ___



NO TORSION SHEAR TRANSF. TO A-LINE AT THIS LEVEL. SHEAR CAN BE TRANSFERED TO B-C LINES BY SLAB

ELEMENT	AREA	$\bar{Y}$	Ax
C-1	5.83	0	0
C-2	5.33	0	0
C-3	5.33	0	0
C-4	5.33	0	0
C-5	5.33	0	0
C-6	5.33	0	0
C-7	5.83	0	0
B-1	9.24	18.75	173.25
B-2	8.44	↑	158.25
B-3	8.44		158.25
B-4	8.44		158.25
B-5	8.44		158.25
B-6	8.44	↓	158.25
B-7	9.24	18.75	173.25
	99.0		1137.75

$$\bar{Y} = \frac{1137.75}{99} = 11.50$$

Center of RIGIDITY IS THE SAME AT EL 139

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11-72B

PROJECT: CARLES RIVER BASIN
SUBJECT: PUMPING STA. REDESIGN
SLAB @ EL. 122

COMP. JAB CHECK BLC

ACC. NO.
SHEET NO. 20 OF
DATE 9-19 1972
CONT. NO. 1856

SLAB ANALYSIS @ EL. 122.

LOADS

GEAR ASSEMBLY D.W. 20.0^k ~ (125^k THRUST)

ENGINE D.W. 35.7^k

L.L. 200#/□'

D.L. 600 48" SLAB

HS20 TRUCK LOAD

NOTES:

1. THE SLAB SHALL BE CAPABLE OF CARRING THE DEAD WT. OF THE MOTOR OR GEAR AT ANY LOCATION DUE TO THE POSSIBILITY OF MOVING DURING REPAIRS.

2. THE SLAB SHALL BE CAPABLE OF CARRING MOVING HS20 TRUCK LOADS IN THE DIRECTION OF THE SPAN AND STATIONARY HS20 LOADS IN BOTH DIRECTIONS, IN ANY OR ALL SPANS DUE TO THE POSSIBILTY OF TRUCK STORAGE.

3. THE SLAB STRIP LEFT OUT FOR THE PUMP INSTALLATION SHALL BE CAPABLE OF CARRY THE GEAR THRUS. CASE I

PROJECT: CHARLES RIVER BASIN
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4. ASSUME GEAR ASSY. AND ENGINE LOADS ACT AS A CONCENTRADED LOAD IN THE SPAN DIRECTION AND IS DISTRIBUTED OVER AN EFFECTIVE WIDTH IN THE DIRECTION PERPENDICULAR TO THE SPAN (USE ASSHO SPECS FOR EFFECTIVE WIDTH)

DESIGN CRITERIA

$f'_c = 3000$ PSI

$f_c = 1050$ PSI

$v_c = 60$ PSI

PROJECT: Charles River Basin
SUBJECT: Pumping Sta. Redesign
Slab @ El. 122.0
COMP.: [illegible] CHECK: GLC
ACC. NO.:
SHEET NO. 22 OF
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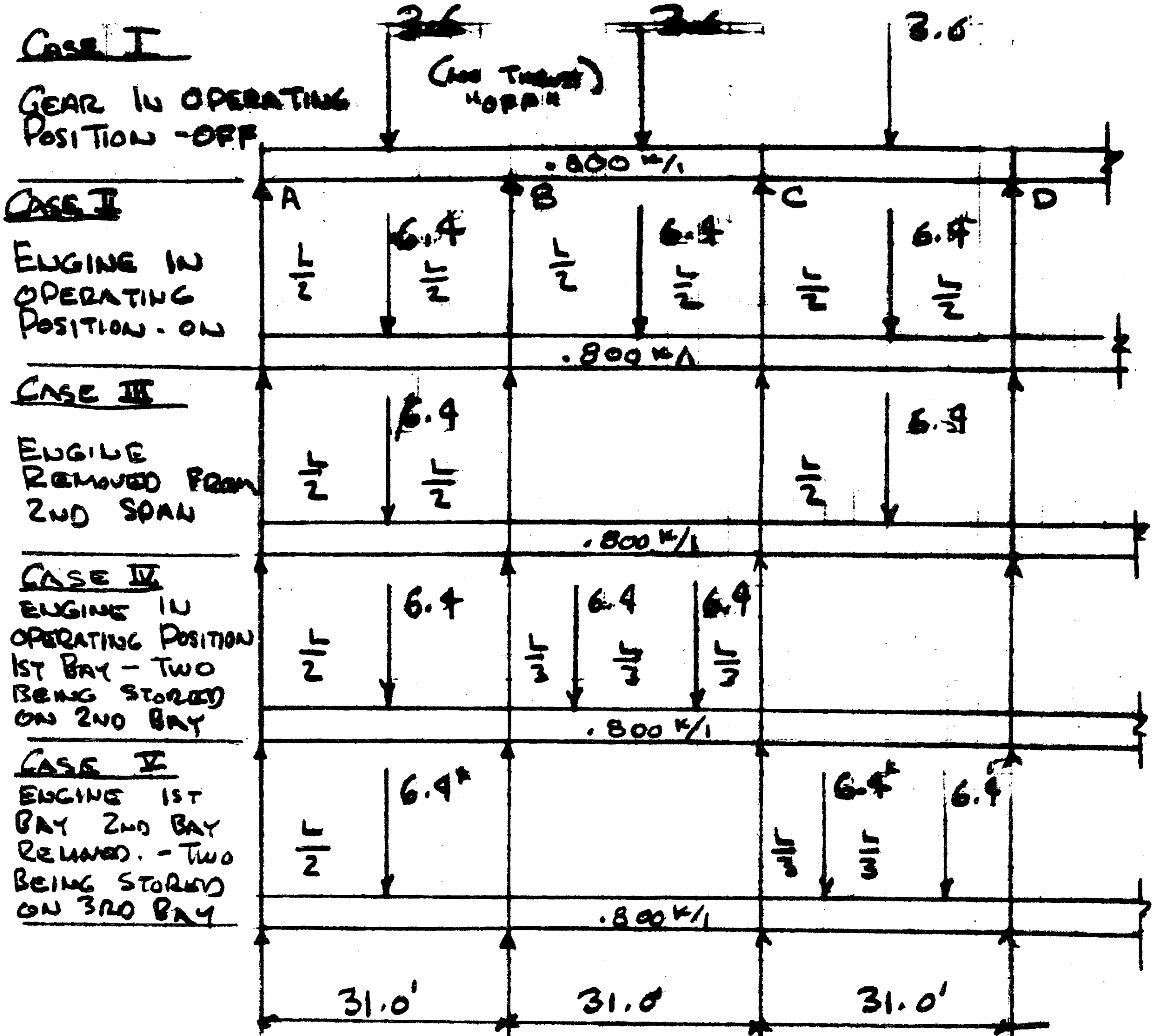
CONT. NO. 1856



HS20 WHEEL LOAD = 16.0^k

EFFECTIVE WIDTH = $4 + .06 \times 27 = 5.62$

GEAR WT = 20^k ~ LOAD/1.0 STRIP = $\frac{20}{5.62}$ - $\underline{3.6^k}$

ENGINE WT. = 35.7^k ~ LOAD/1.0 STRIP = $\frac{35.7}{5.62} = \underline{6.4^k}$

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PROJECT: Charles River Basin
SUBJECT: Pumping Sta. Redesign
Slab @ El. 122.0
COMP.: AB   CHECK: GLC

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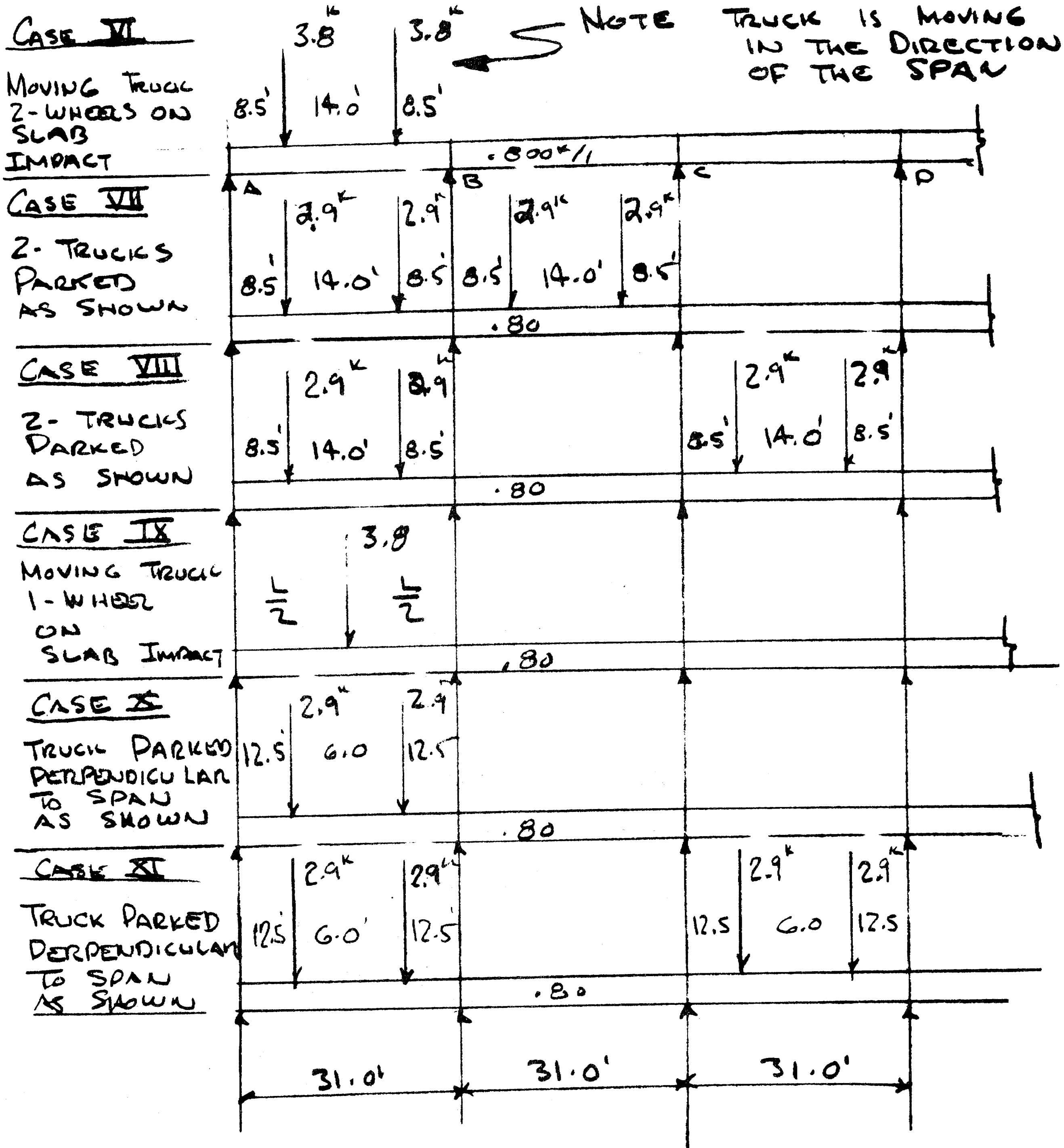
SHEET NO. 23 OF ____
DATE 9-19 1972
CONT. NO. 1856



$HS20 = 16^k \sim P = \frac{16}{5.62} = 2.9^k$

$\text{Impact} = 30\% \quad P_E = 2.9 \times 1.3 = 3.8^k$

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PROJECT: CHARLES RIVER BASIN — ACC. NO. ____
SUBJECT: PUMPING STA. REDESIGN — SHEET NO. 24 OF ____
SLAB @ EL. 122.0 — DATE 9-19 1972
COMP. JAB — CHECK. BLC — CONT. NO. ____

ASSUME ALL WALLS HINGED

CASE I

FEM $\overline{BA}$ = $\frac{3}{16} \times 3.6 \times 31.0 = 20.8^{1K}$

$.80 \times 31.0^2 \times \frac{1}{8} = 96.0^{1K}$

116.8

FEM $\overline{BC}$ ETC. = $\frac{1}{8} \times 3.6 \times 31.0 = 13.6^{1K}$

$.8 \times 31.0^2 \times \frac{1}{12} = 64.0^{1K}$

77.6

CASE II

FEM $\overline{BA}$ = $\frac{3}{16} \times 6.4 \times 31.0 = 37.0^{1K}$

$.8 \times 31.0^2 \times \frac{1}{8} = 96.0^{1K}$

133.0

FEM $\overline{BC}$ ETC. = $\frac{1}{8} \times 6.4 \times 31.0 = 24.8^{1K}$

$.8 \times 31.0^2 \times \frac{1}{12} = 64.0^{1K}$

108.8

CASE III

FEM $\overline{BA}$ = SAME AS CASE II = 133.0^{1K}

FEM $\overline{CD}$ = FEM BC " " = 108.8

CASE IV

FEM $\overline{BA}$ = SAME AS CASE II = 133.0

FEM $\overline{BC}$ ETC. = $\frac{2}{9} \times 6.4 \times 31 = 44.0$

$.8 \times 31.0^2 \times \frac{1}{12} = 64.0^{1K}$

108.0

CASE V

FEM $\overline{BA}$ = SAME AS CASE II = 133.0^{1K}

FEM $\overline{CD}$ = SAME AS $\overline{BC}$ CASE IV = 108.0

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PROJECT CHARLES RIVER BASIN
SUBJECT PUMPING STA. REDESIGN
SLAB @ EL. 122.0'
COMP. DTS CHECK GLC

ACC. NO.
SHEET NO 25 OF
DATE 9-19 1972
CONT. NO. 1856

CASE VI

$$FEM\ \overline{AB} = \frac{3 \times 3.8 \times 8.5}{2 \times 31.0} \times 22.5 = 35.4^{1K}$$

$$.8 \times 31.0^2 \times \frac{1}{8} = 96.0$$

$$131.4$$

$$FEM\ \overline{BC} = .8 \times 31.0^2 \times \frac{1}{12} = 64.0^{1K}$$

CASE VII

$$FEM\ \overline{AB} = \frac{3 \times 2.9 \times 8.5}{2 \times 31} \times 22.5 = 26.6$$

$$.8 \times 31.0^2 \times \frac{1}{8} = 96.0$$

$$122.6^{1K}$$

$$FEM\ \overline{BC} = \frac{2.9 \times 8.5}{31} \times 22.5 = 17.9^{1K}$$

$$.8 \times 31^2 \times \frac{1}{12} = 64.0$$

$$81.9^{1K}$$

CASE VIII

FEM $\overline{AB}$ = SAME $\overline{AB}$ CASE VII 122.6[1K]

FEM $\overline{CD}$ = SAME BC CASE VII 81.9[1K]

CASE IX

$$FEM\ \overline{AB} = \frac{3}{16} \times 3.8 \times 31 = 22.0^{1K}$$

$$.8 \times 31^2 \times \frac{1}{8} = 96.0$$

$$118.0$$

$$FEM\ \overline{BC}\ ETC = .8 \times 31^2 \times \frac{1}{12} = 64.0^{1K}$$

PROJECT Charles River Basin ACC. NO.
SUBJECT Pumping Sta. Redesign SHEET NO. 26 OF
Slab @ El. 122.0 DATE 9-19 1972
COMP. AB CHECK GLC CONT. NO. 1856

CASE X

FEM $\overline{AB} = \frac{3 \times 2.9 \times 12.5}{2 \times 31} \times 18.5 = 32.4^{1K}$

$.8 \times 31^2 \times \frac{1}{8} = 96.0$

128.4

CASE XI

FEM $\overline{AB}$ = SAME $\overline{AB}$ X = 128.4

FEM CD $= \frac{2.9 \times 12.5}{31} \times 18.5 = 21.5$

$.8 \times 31^2 \times \frac{1}{12} = 64.0$

85.5

$K\overline{BA} = \frac{3}{31} = .097$ ~ $S\overline{BA} = \frac{.097}{.225} = .43$

$K_{BC} = \frac{4}{31} = \frac{.128}{.225}$ ~ $S\overline{BC} = \frac{.128}{.225} = .57$

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PROJECT: CHARLES RIVER BASIN ACC. NO.

SUBJECT: PUMPING STA. REDESIGN SHEET NO. 27 OF

SLAB @ EL. 122.0 DATE 9-19 1972

COMP. JAD CHECK GLC CONT. NO. 1856

	.43	.57	.5	.5	.5	.5	MAX. V
	-117	+78	-78	+78	-78	+78	
	+12	+22	+11		-3		
		+3	-6	-6	+2	+2	
	-1	-2					
CASE I	-101	+101	-73	+72	-79	+80	14.8 K
	-133	+109	-109	+109	-109	+109	
	+10	+14	+7				
			-3	-3			
CASE II	-123	123	105	107	109	109	15.6 K
	-133	+64	-64	+109	-109	+109	
	+30	+39	-23	-23	-12		
		-12	-20		+6	+6	
	+5	+7	+10	+10			
CASE III	-98	+98	-97	+97	115	+115	15.6 K
	-133	+108	-108	+64	-64	+64	
	+11	+14	+22	+22	-11		
		-11	+7		+6	+6	
	+5	+6	-4	-4			
CASE IV	-117	+117	-83	+83	-69	+70	18.8 K
	-133	+64	-64	+108	-108	+64	
	+30	+40	-22	-22	+22	+22	
		+11	+20	-11			
	-5	-6	-5	+5			
CASE V	-108	+109	-71	+70	-86	+86	18.8 K

[redacted] INDICATES SIMPLE SPAN POS MOMENT.

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PROJECT: CHARLES RIVER BASIN — ACC. NO.

SUBJECT: PUMPING STA. REDESIGN — SHEET NO. 28 OF

SLAB AT EL. 122.0' — DATE 9-20 1972

COMP. [illegible] — CHECK ALC — CONT. NO. 1856

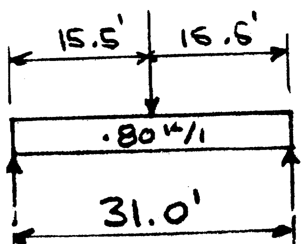
		.43	.57		.5	.5		.5	.5	MAX V
	127	-131 +29	+64 +38	96	-64 +19 -10	+64 -10	96			
CASE VI		-102	+102		-55	+54		+64		16.2k
	124	-123 +18 -105	+82 +23 +105	124	-82 +9 +12 -6	+64 +9 -6	96	-64	+64	
CASE VII		-105	+105		-67	+67		-64		15.3
	124	-126 +27	+64 +35	96	-64 -9 +18 -9	+82 -9 -9	124	-82 +9	+64 +9	
CASE VIII		-99	+99		-64	+64		-73	+73	15.3
	125	-118 +23	+64 +31	96	-64 +15 -8	+64 -8	96	-64		
CASE IX		-95	+95		-57	+56		-64		15.3
	135	-128 +27	+64 +37	96	-64 +19 -9	+64 -9	96	-64		
CASE X		-101	+101		-54	+55		+64		15.5
	135	-128 +27 +2	+64 +37 -5 +3	96	-64 -10 +19 -12	+85 -10 +5 -12	135	-85 +10 -5 +3	+64 +10 +3	
CASE XI		-99	+99		-67	+68		-77	+77	15.5

PROJECT: CHARLES RIVER BASIN
SUBJECT: PUMPING STA. REDESIGN
SLAB AT EL. 122.00
COMP.: BB CHECK: GLC

ACC. NO.
SHEET NO. 29 OF
DATE 9-21-72
CONT. NO. 1856

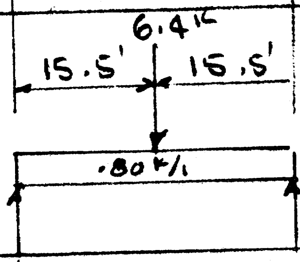
POS. MOMENTS BASED ON SIMPLE SPAN. DUE TO LOADING CONDITIONS FROM CASE I TO XI

1.8
12.4
14.2



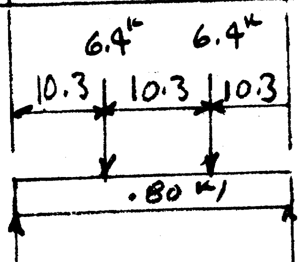
M = 3.6 × 31 × 1/4 = 28.0
.8 × 31² × 1/8 = 96.0
124.0

3.2
12.4
15.6



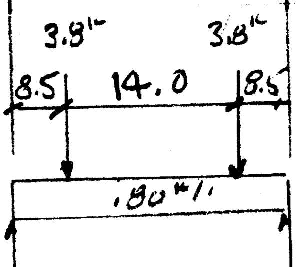
M = 6.4 × 31 × 1/4 = 50.0
.8 × 31² × 1/8 = 96.0
146.0

6.4
12.4
18.8



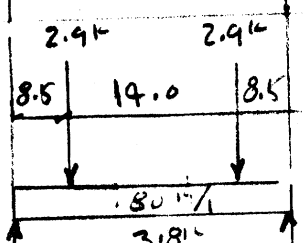
M = 18.8 × 15.5 = 290.0 ↻
6.4 × 5.1 = 33.0 ↺
.8 × 15.5²/2 = 96.0 ↺
161.0

3.8
12.4
16.2

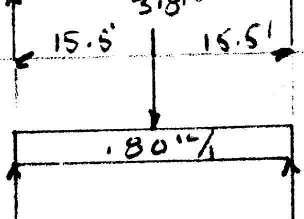


M = 16.2 × 15.5 = 250.0 ↻
3.8 × 7 = 27.0 ↺
.8 × 15.5²/2 = 96.0 ↺
127.0''

2.9
12.4
15.3



M = 15.3 × 15.5 = 240.0 ↻
2.9 × 7.0 = 20.0 ↺
.8 × 15.5²/2 = 96.0 ↺
124

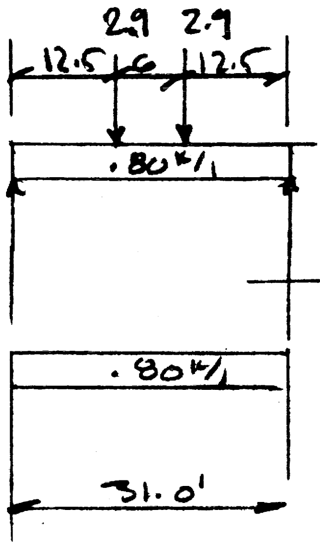


M = 3.8 × 31 × 1/4 = 29
.8 × 15.5²/8 = 96
125

A 157

PROJECT	Charles River Basin	ACC. NO.	
SUBJECT	Pumping Sta. Redesign	SHEET NO.	30 OF
	Slab @ El. 132.00	DATE	9-21 1972
COMP.	[illegible] CHECK GLC	CONT. NO.	185

$$\begin{array}{r} 2.9 \\ 12.4 \\ \hline 15.3 \end{array}$$



$M = 15.3 \times 16 = 240$ ↻
$2.9 \times 3 = 9$ ↺
$.8 \times \frac{31^2}{8} = 96$ ↺
$= 135$

$M = .8 \times \frac{31^2}{8} = 96^{\text{'k}}$

Max Pos. Mom = 96'k ~ Case III
Max Neg Mom = 133'k ~ Case II

Based on 31'-0" span

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A 158

PROJECT: CHARLES RIVER BASIN　　ACC. NO.
SUBJECT: PUMPING STA. REDESIGN　　SHEET NO. 31 OF
SLAB AT EL. 122.00　　DATE 9-22 1972
COMP. [illegible]　　CHECK GLC　　CONT. NO. 1856

SLAB OVER DEVIDER WALL (15'-6" SPAN)

FROM INSPECTION OF 31'-0" SPAN IT IS OBVIOUS THAT CASE II WILL CONTROL MAX. NEG. MOM. & CASE III WILL CONTROL MAX. POS. MOM.

CASE II FOR 15'-6" SPAN

$$FEM\ \overline{BA} = \frac{3}{16} \times 6.4 \times 15.5 = 18.6^{1k}$$

$$.8 \times 15.5^2 \times \frac{1}{8} = \underline{24.0}$$

$$42.6$$

$$FEM\ \overline{BC} = \frac{1}{8} \times 6.4 \times 15.5 = 12.4^{1k}$$

$$.8 \times 15.5^2 \times \frac{1}{12} = \underline{16.0}$$

$$28.4$$

CASE III FOR 15'-6" SPAN

FEM $\overline{BA}$ = 18.6 SAME CASE II

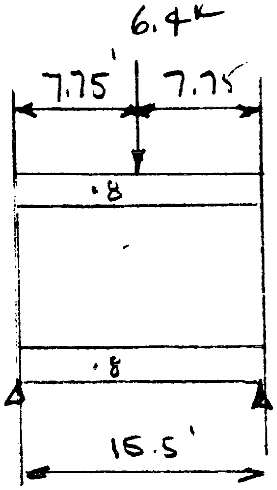
FEM BC = 28.4 " " "

		.43	.57		.5	.5		.5	.5
	49.0	-43 +6	+28 +9	49.0	-28	+28	49.0	-28	
CASE II		-37	+37		-28	+28		-28	
	49.0	-43 +12	+16 +15	49.0	-16 -14	+43 -14	49.0	-43 +14	+16 +14
CASE III		-31	+31		-30	-29		-29	+30

RED NO INDICATE MAX POS MOMENTS SIMPLE SPAN

(A159)

PROJECT	CHARLES RIVER BASIN	ACC. NO.	
SUBJECT	PUMPING STA. REDESIGN	SHEET NO	32 OF
	SLAB @ EL. 122.00'	DATE	9-22 1972
COMP.	[illegible] CHECK GLC	CONT. NO.	18



POS. MOMENTS BASED ON SIMPLE SPAN HAVING LOADING CONDITIONS DUE TO CASE II ~~III~~

$M = 6.4 \times 15.5 \times \frac{1}{4} = 25$

$.8 \times 15.5^2 \times \frac{1}{8} = \underline{24.0}$

44.0

$M = .8 \times 15.5^2 \times \frac{1}{8} = 24.0^{'k}$

MAX. + M = 33.0'k – CASE III

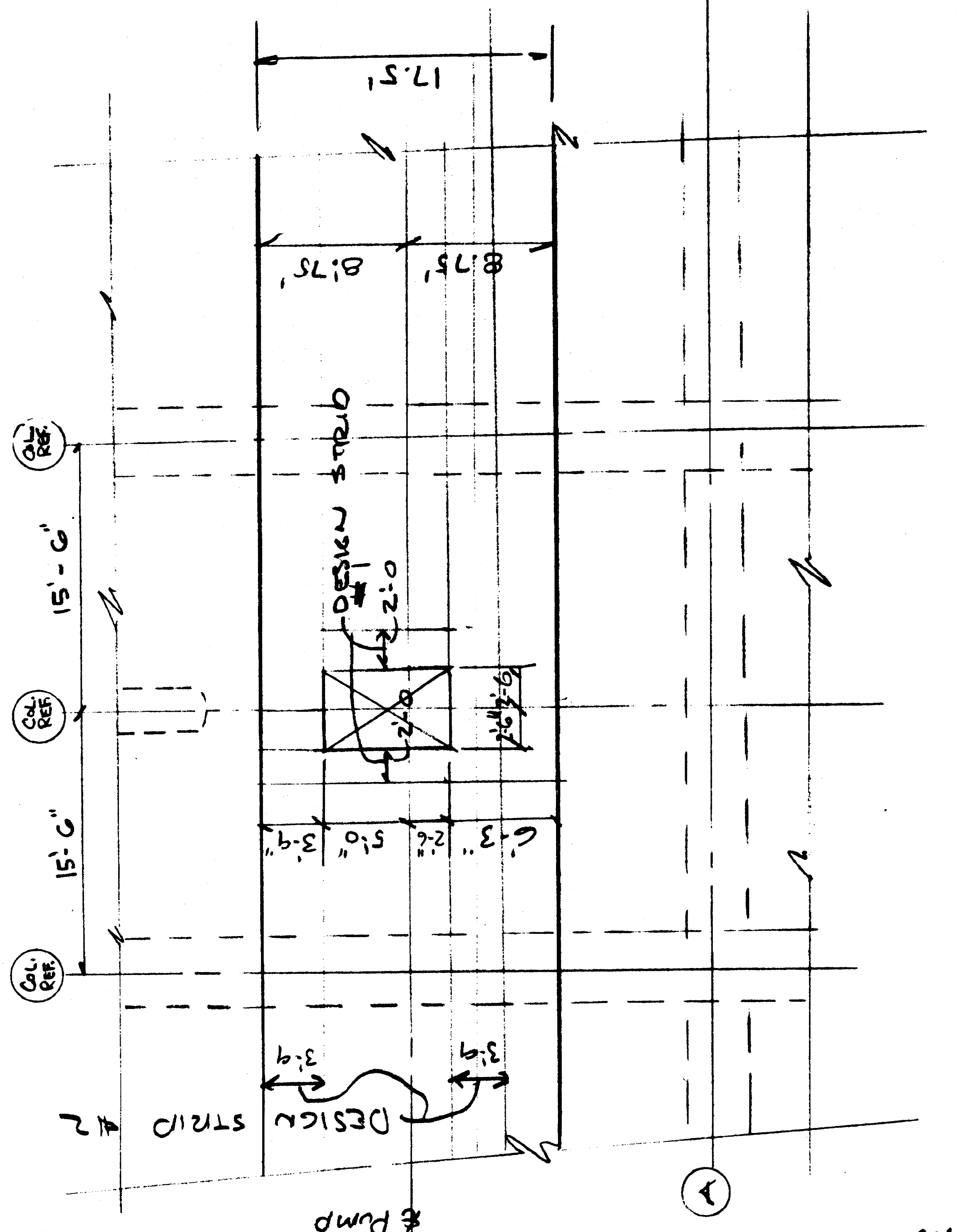
MAX – M = 37.0'k – CASE II

— BASE ON 15'-6" SPAN

A160

PROJECT: Charles River Basin
SUBJECT: Pumping Sta. Redesign
Slab @ El. 122.00' over Pump Rm.
COMP. [illegible] CHECK GLC

ACC. NO.
SHEET NO. 33 OF
DATE 9-22 1972
CONT. NO. 1856



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A161

PROJECT: CHARLES RIVER BASIN — ACC. NO.

SUBJECT: PUMPING STA. — SHEET NO. 34 OF

SLAB @ EL 122.0 OVER PUMP RM — DATE 4-4 1973

COMP. DB — CHECK — CONT. NO. 1856

SLAB STRIP LEFT OUT IN INITIAL POUR TO ALLOW FOR PUMP INSTALLATION

LOADS

20.0^k GEAR AND ASSY
125.0 THRUST
145.0^k

.200 k/# = L.L.
.625 = D.L. = 4.167 × .15

PERIMETER OF OPENING

5.0' + 5.0' + 7.5' + 7.5 = 25.0'

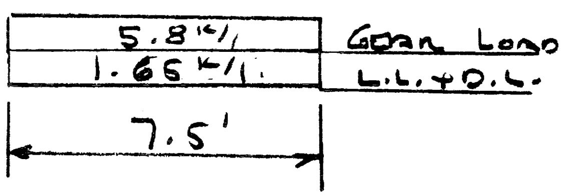
DISTRIBUTE GEAR LOADS AROUND PERIMETER

$\frac{145^k}{25} = 5.8^{k/l}$

DESIGN STRIP #1

ASSUME: 2.0' WIDE
SPAN 7.5'

21.8^k
6.2^k
28.0^k



$M = \frac{7.45 \times 7.5^2}{8} = 52.2^{\prime k}$

$d_M = \sqrt{\frac{52.2 \times 12}{.152 \times 24}} = 13.2''$ ok

$d_V = \frac{28.0}{.06 \times 24} = 19.5''$ ok

$d_A = 46 - 6.7 = 39.3''$

SIMPLE SPAN $\frac{wl^2}{8}$

L.L. + DL. = .825 × 2 = 1.65 k/l
GEAR = 5.8
7.45

REINF

$\pm A_s = \frac{52.2}{1.44 \times 39.3} = 0.92 \square''$

USE 6-#9
3-T & 3-B
OVER 2'-0" STRIP EA. SIDE OF OPEN'G.

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PROJECT: CHARLES RIVER BASIN — ACC. NO.
SUBJECT: PUMPING STA. REDESIGN — SHEET NO. 35 OF
SLAB @ EL. 122.0' OVER PUMP RM. — DATE 4-4 1973
COMP. DB — CHECK. — CONT. NO. 1856

DESIGN STRIP #2

ASSUME: 3.75' STRIP

DESIGN CONDITIONS

LOADS w2 & w3 SHALL BE DESIGNED TO GIVE POSITIVE & NEG. MOMENTS BASED ON A CONTINUOUS SLAB ANALYSES.

LOAD w1 & F SHALL BE DESIGNED TO GIVE MAX. POS. MOMENT BASED ON A SIMPLE SPAN ANALYSES.

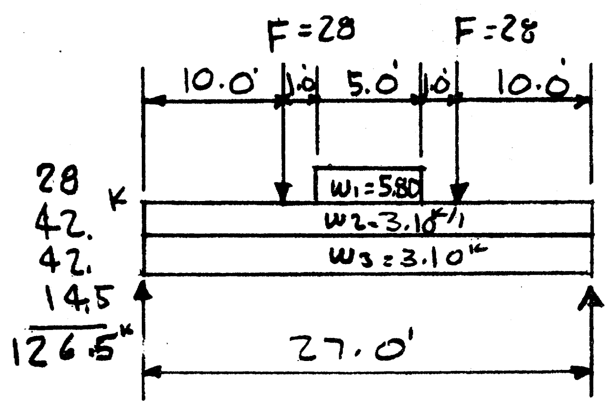
LOADS SAME AS SHOWN ON SH. # 34

$w_2 = .825 \times 3.75 = 3.1^{k}/'$ = (L.L. + D.L. SLAB BETWEEN STRIPS)

$w_3 = .825 \times 3.75 = 3.1^{k}/'$ = (L.L. + D.L. 3.75' STRIP)

$w_1 = 5.80^{k}/'$ = (GEAR LOADS)

$F = 28^{k}$ = STRIP #1



$M_{w2 \& w3} = 6.2 \times 27^2 \times \frac{1}{10} = \underline{450}^{'k}$

$M_{w1 \& F} = 42.5 \times 13.5 - \frac{5.80 \times 2.5^2}{2} - 28.0 \times 3.5 = \underline{459}$

$M_{TOTAL} = 450 + 459 = \underline{909}^{'k}$

PROJECT CHARLES RIVER BASIN ACC. NO.
SUBJECT PUMPING STA. SHEET NO. 36 OF
SLAB @ EL. 122.0' OVER PUMP RM DATE 4-4 1973
COMP. JB CHECK CONT. NO. 1866

DESIGN STRIP #2 CONT.

$$d_M = \sqrt{\frac{909 \times 12}{.152 \times 45}} = 39.9'' > 39.3$$ 1.6% OVER AVAILABLE d

R @ FACE OF SUPPORT = 126.5^k

MAX SHEAR @ d FROM FACE OF SUPPORT = $126.5 - 3.25 \times 6.2 = \underline{106.3^k}$

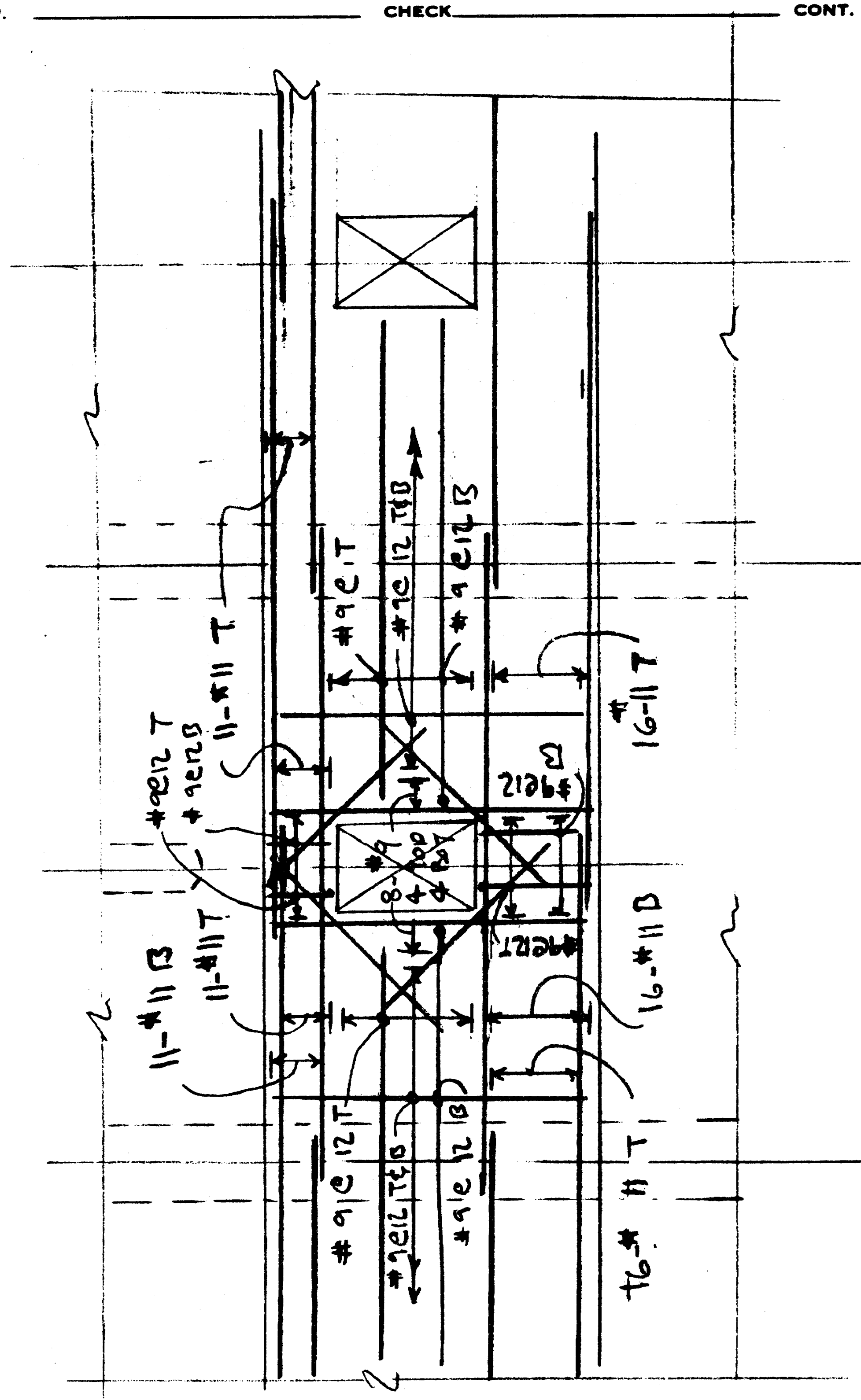
$$d_V = \frac{106.3}{45 \times .06} = 39.2 \text{ ok}$$

$$A_s = \frac{909}{1.44 \times 39.3} = 16.2^{\#}$$

39.3" SLAB ok
WITH 11-#11 T&B

TEMP: $39.3 \times 45 \times .004$
$= 7.0$

PROJECT Charles River Basin — ACC. NO.
SUBJECT Pumping Sta — SHEET NO. 32 OF
Slab @ El. 122.00 — DATE 19
COMP. — CHECK — CONT. NO.



PROJECT: CHARLES RIVER BASIN — ACC. NO. ____
SUBJECT: PUMPING STA REDESIGN — SHEET NO. __ OF ____
SLAB AT EL. 122.00 — DATE 9-22 1972
COMP. XB — CHECK GLC — CONT. NO. 1856

DESIGN SLAB @ EL. 122.00' (31.0' SPAN)

$+M_{MAX} = 96.0^{1K}$ ∴ $+A_s = \frac{96}{1.44 \times 41.3} = 1.62\square''$ ∴ #10@9"

$-M_{MAX} = 133.0^{1K}$ ∴ $-A_s = \frac{133}{1.44 \times 41.3} = 2.24\square''$ #10@6"

$d_R = \sqrt{\frac{133}{.152}} = 29.5''$

$d_A = 48.0'' - 6.7'' = 41.3''$

TEMP. STEEL - "RESTRAINED BOTH ENDS"

PARRALLEL & PERPENDICULAR TO SPAN

$= .004 \times 48 \times 12 = 2.3$ ∴ MAX $A_s = 1.0$

USE 48" SLAB
MAIN REINF
#10@9" BOTT.
#10@6" TOP
#9@12 TEMP

DESIGN SLAB AT EL. 122.00 (15.5' SPAN)

MAX $+M = 33.0^{1K}$ ∴ $A_s = \frac{33.0}{1.44 \times 41.3} = .56\square''$ — #6@9

" $-M = 37.0$ ∴ $A_s = \frac{37}{1.44 \times 41.3} = .62\square''$ — #5@6

TEMP REINF GOVERNS DESIGN.

USE 48" SLAB
MAIN REINF #9@12
T&B
#9@12 TEMP

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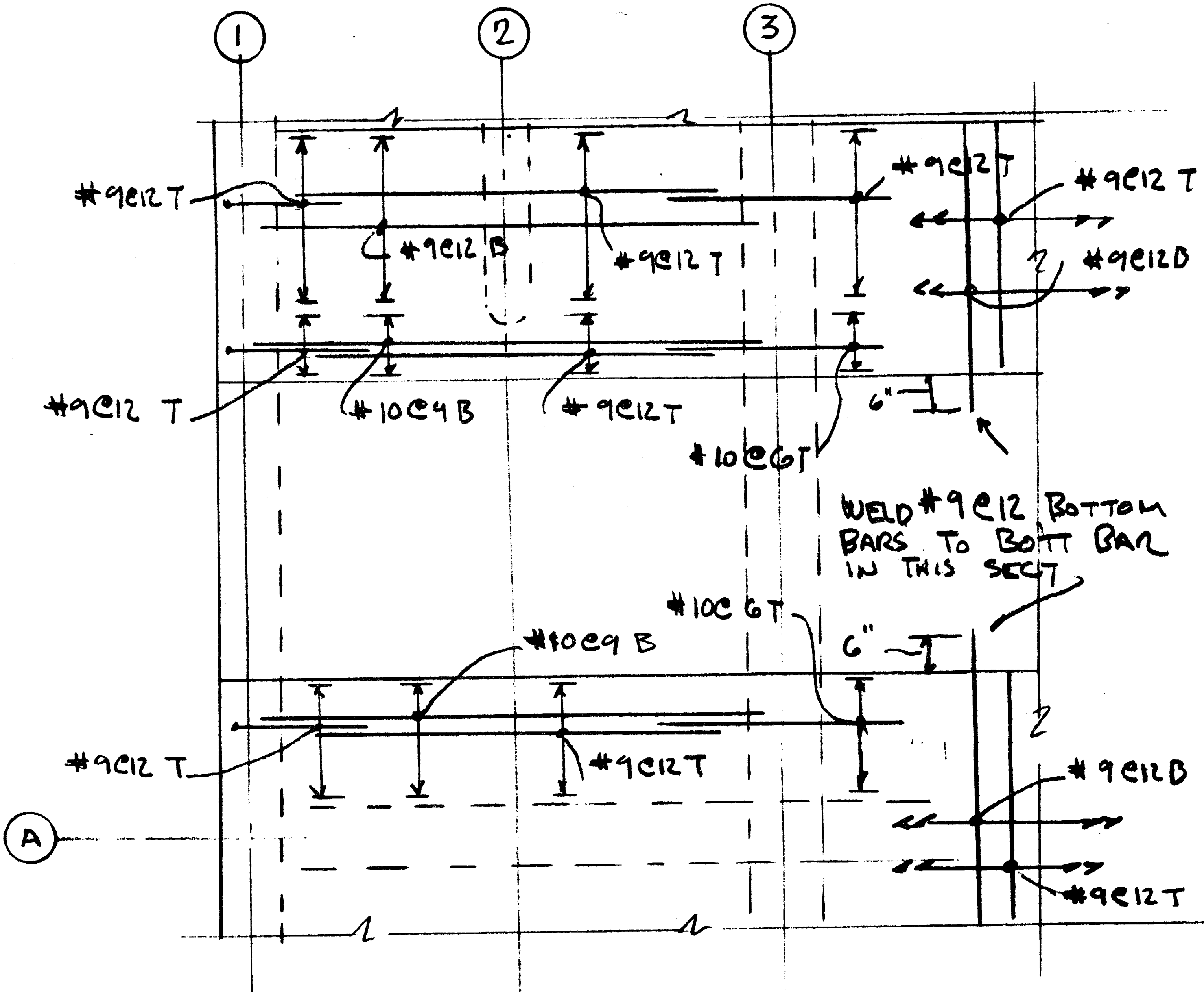
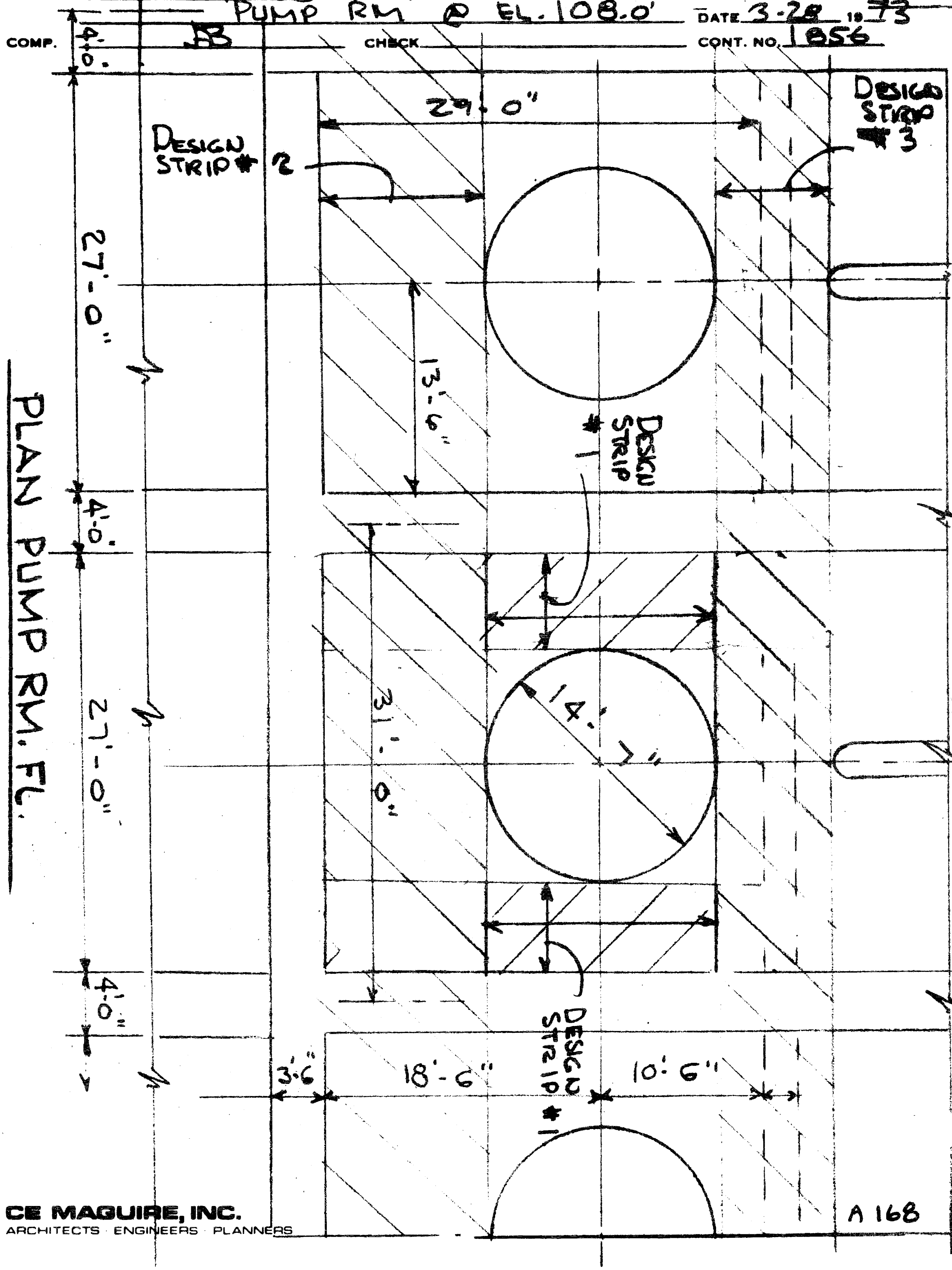
PROJECT Charles River Basin
SUBJECT Pumping Sta. Redesign
Slab @ El. 122.00
COMP.
CHECK GLC
ACC. NO.
SHEET NO. 39 OF
DATE 9-22 1972
CONT. NO. 1856



TYPICAL SLAB REINFORCING IN SLAB AT. EL. 122.0' EXCEPT AS NOTED

A167

PROJECT: Charles River Basin
SUBJECT: Pumping Sta. — Pump Rm @ El. 108.0'
COMP.: JB CHECK:
ACC. NO.:
SHEET NO. 1 OF
DATE 3-28 1973
CONT. NO. 1856



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A 168

PROJECT: CHARLES RIVER BASIN — ACC. NO. ____
SUBJECT: PUMPING STA. PUMP RM — SHEET NO. 9 OF ____
DATE 3-28 1973
COMP. DB — CHECK ____ — CONT. NO. 1856

LOADS

1. PUMP & ASSY. 122^k

DISTRIBUTE BY PROPORTION TO EA. STRIP

2. ASSUME RM. FILLED WITH WATER

$29.0' \times 31.0' = 900^{\#}$ FL. × 10 HIGHT × 0.064 WATER

$= 576^k - 125^k$ (THRUST) $= \underline{451^k}$

3. WT. CONCRETE

FL. AREA = $900^{\square'}$
-ONENG. = $166 = \pi\, 7.292^2$
$734^{\square}$

$5.0'$ THICK SLAB × 734 × .150 = $\underline{550^k}$

DESIGN ASSUMPTIONS

1. DESIGN STRIP #1 — SIMPLE SPAN $wl^2 \times \frac{1}{8}$

SPAN FROM DESIGN STRIP # 2 TO # 3

SPAN = $15.0'$ & WIDTH = $6.21'$

LOAD AREA = $15 \times 6.21 = \underline{93}^{\square'}$

2. DESIGN STRIP # 2 — SIMPLE SPAN $wl^2 \times \frac{1}{8}$

SPAN = $27.0'$ & WIDTH = $11.2'$

LOAD AREA = $27.0 \times 11.2 = 302^{\square'}$

3. DESIGN STRIP #3 — SIMPLE SPAN $wl^2 \times \frac{1}{8}$

SPAN = $27.0'$ & WIDTH = $7.0'$

LOAD AREA = $27 \times 7 = 189^{\#}$

PROJECT Charles River Basin ACC. NO.
SUBJECT Pumping Sta. SHEET NO. 3 OF
Pump Rm DATE 3-29 1973
COMP. AB CHECK CONT. NO. 1856

DISTRIBUTION OF LOADS

PUMP LOAD = 122.0^k

LOAD AREA

STRIP #1 = 93 #
#2 = 302
#3 = 189
584 #

UNIFORM LOAD = $w_p = \frac{122.0}{584} = .209^{k}/\#$

CONCRETE D.L. " " $w_c = \frac{550}{584} = .940$

WATER wt " " $w_w = \frac{451}{584} = .77?$

TOTAL " " = $w_T = 1.92^{k}/\#$

PROJECT CHARLES RIVER BASIN
SUBJECT PUMPING STA.
PUMP RM

COMP. [initials] CHECK

ACC. NO.
SHEET NO. 4 OF
DATE 3-27 19
CONT. NO. 1856

DESIGN SLAB

DESIGN STRIP 1 — 1'-0" STRIP

$\pm M = 1.92 \times 15.0^2 \times \frac{1}{8} = \underline{54}^{'K}$ — USING 5' Slab

$d_A = \sqrt{\frac{54}{.152}} = 18.9''$ — $d_a = 60'' - 6.8 = 53.2''$

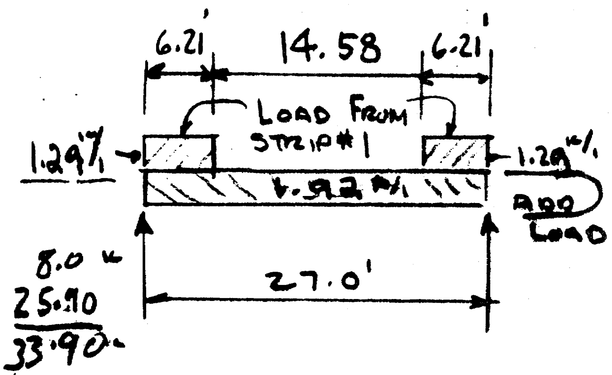
$V = 1.92 \times 15.0 \times \frac{1}{2} = 14.4^{K}/1'\text{-}0''$ STRIP

$A_s = \frac{53}{53.2 \times 1.44} = \underline{0.69\square''}$

USE 5'-0" SLAB
REINF. #9 @ 6"
TOP & B

USE MIN REINF 1.25 T & B
SEE SHEET #5

DESIGN STRIP #2 — 1'-0" STRIP



ADDITIONAL LOAD FROM STRIP 1

$V = 14.4/1.0'$

STRIP #2 = 11.2' WIDE

DISTRIBUTE LOAD OVER FULL WIDTH

$\frac{14.4}{11.2} = 1.29^{K/1}$

$\mp M = 33.90 \times 13.5 = 457^{'K}$
$1.92 \times \frac{13.5^2}{2} = 175$
$1.29 \times 6.21 \times 10.4 = 83$
$199^{'K}$

$A_s = \frac{199}{1.44 \times 53.2} = 2.60\square''$

USE 5'-0" SLAB
#11 @ 6 T & B

$d_M = \sqrt{\frac{199}{.152}} = 36.3$ ok — $d_V = \frac{33.9}{.06 \times 12} = 47''$ ok

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A171

PROJECT	CHARLES RIVER BASIN	ACC. NO.	
SUBJECT	PUMPING STA. / PUMP RM.	SHEET NO.	5 OF
		DATE	3-29 19
COMP.	[illegible]	CHECK	CONT. NO. 1856

DESIGN STRIP # 3 1-0" STRIP

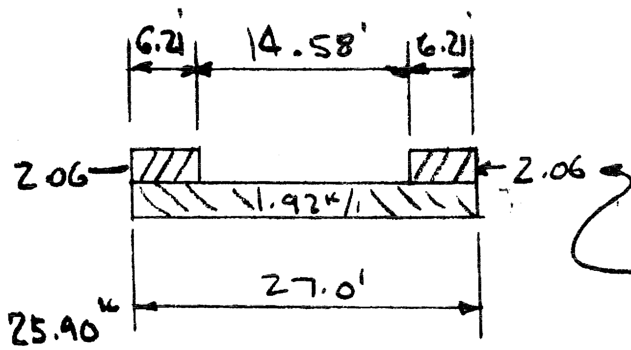
ADDITIONAL LOAD FROM STRIP 1

$V = 14.4^k/1.0"$

STRIP # 3 = 7.0 WIDE

DISTRIBUTE LOAD OVER FULL WIDTH

$\frac{14.4}{7} = 2.06$



25.90k
12.8
38.70

$FM = 38.7 \times 13.5 = 523\ 2$ (1K)

$1.92 \times \frac{13.5^2}{2} = 175\ 6$

$2.06 \times 6.21 \times 10.4 = 134\ 6$

$214^{1K}\ 2$

$d_M = \sqrt{\frac{214}{.152}} = 37.6$ ok

$d_V = \frac{38.70}{.06 \times 12} = 53.6$ ok

$As = \frac{2\bcancel{14}}{1.44 \times 53.2} = 2.8^{\square}$

USE

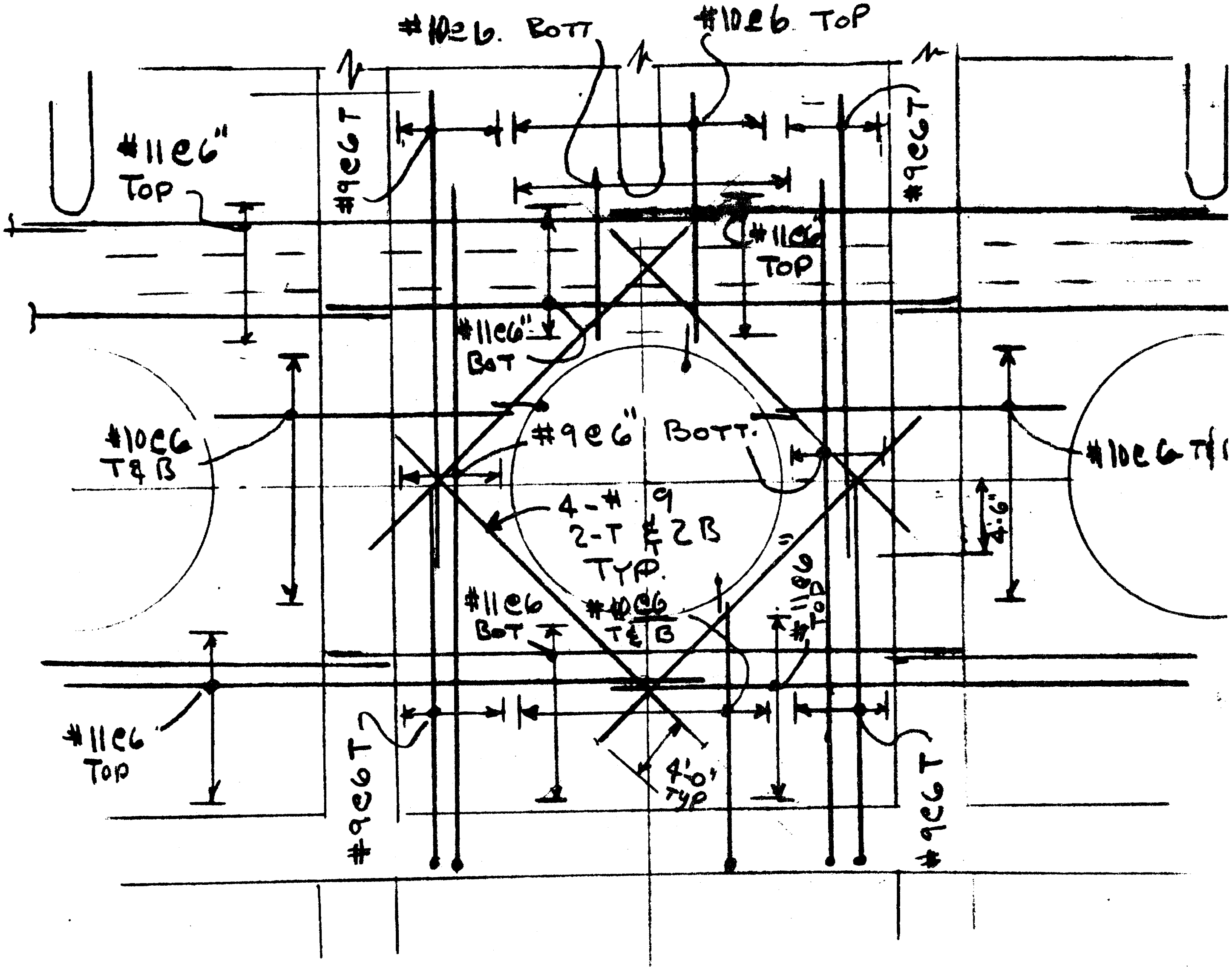
5'-0" SLAB

REINF. #11 @ 6" T&B

MIN REINF = 12 × 53.2 × .004 = 2.52 □"

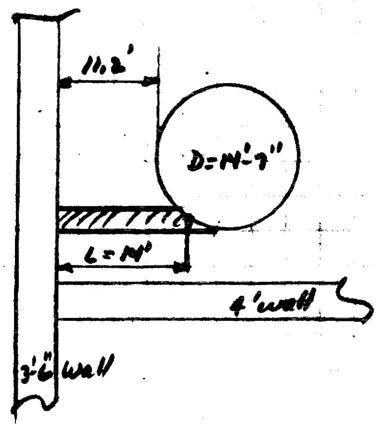
1.25 E.F #9 @ 12 T&B

PROJECT CHARLES RIVER BASIN ACC. NO.
SUBJECT PUMPING STA. SHEET NO. 6 OF
PUMP ROOM @ EL. 108 DATE 3-29 1973
COMP. AB CHECK CONT. NO. 1856



PLAN OF PUMP ROOM
FLOOR REINF
AT EL. 108

PROJECT Charles River Dam ACC. NO. ____
SUBJECT Pumping Sta - Pump slab @ El. 108 SHEET NO. 2 OF ____
Investigation DATE 3/30 1973
COMP. L.E. CHECK [initials] CONT. NO. 1856.30



Analyze the longest unit strip in area of pump opening as a cantilever.

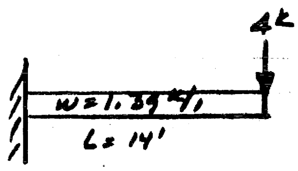
Pump weight = 122^k

Pump load/ft = $\frac{122^k}{44} = 2.8^k/'$

Use $4^k/'$ @ end of unit strip to account for unbalanced load distribution.

Dead load 5' slab = 750#/SF
Live load 10' water = 640#/SF

Total load = 1390#/SF



$M = \frac{1.39 \times 14^2}{2} + 4 \times 14 = 193^{'k}/ft$

$V = 1.39 \times 14 + 4 = 23.5^k/ft$

Top Reinf $= \frac{193}{1.44 \times 53} = 2.5\ in^2/ft$

Requires #10@6" Top (2.54) @ longest strip, on West side

use #10@6 T & B

END WALL DESIGN

LOADS

✓ $5.02^{k}/_{1}$ DUE FROM SUPERSTRUCTURE

1.13 $= \frac{122}{4} \times \frac{1}{27}$ ASSUME THAT THE PUMP LOAD WILL DISTRIBUTE 25% OF THE TOT. LOAD UNIFORMLY ALONG THE END WALL

9.7 $= \frac{(3.87 \times 10 \times 27)}{27} \times \frac{1}{4}$ ASSUMED LOAD DISTRIBUTION. DUE TO D.L. + WATER

5.2 $= 10 \times 3.5 \times .15$ D.L. BEAM SIZE ASSUMED 3'-6 x 10'-0"

$26.05^{k}/_{1}$

$M = 26.05 \times 27.0^2 \times \frac{1}{10} = 1900^{1k}$

$d = \sqrt{\frac{1900 \times 12}{.152 \times 42}} = 60"$

$d = 120.0 - 6.7 = 113.3$

$A_s = \frac{1900}{1.44 \times 113.3} = 11.7$

USE 8-#11 -2-LAYERS

OTHER #9@12

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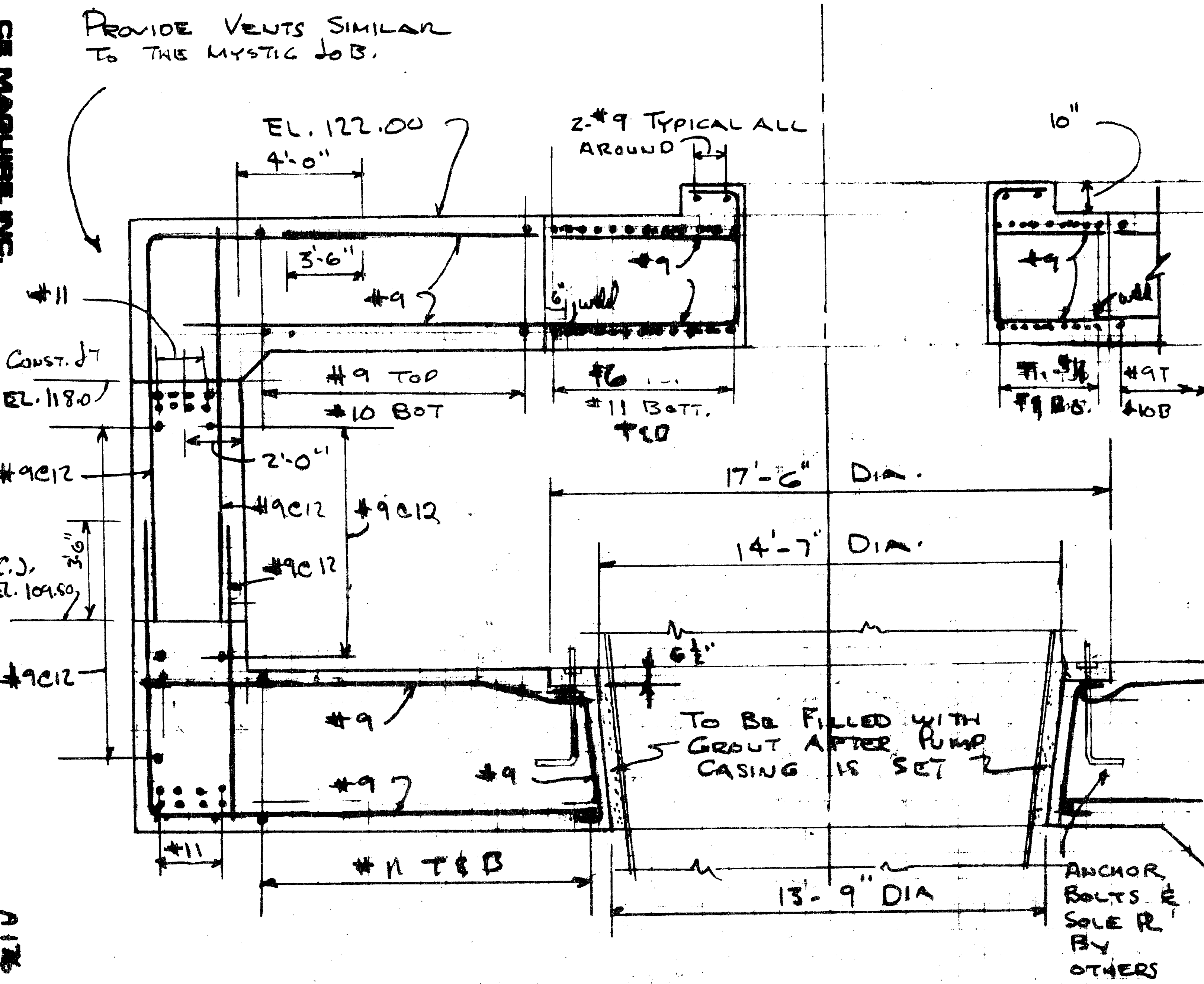
PROJECT CHARLES RIVER BASIN
SUBJECT PUMPING STA. REDESIGN
SECTION THRU PUMP RM
COMP. JB
CHECK GJC
ACC. NO.
SHEET NO. OF
DATE 19
CONT. NO.



A176

PROJECT: CHARLES RIVER BASIN | ACC. NO.
SUBJECT: PUMPING STA. REDESIGN | SHEET NO. 1 OF
PUMP ROOM @ EL. 108.0' | DATE 9-12 1972
COMP. AB | CHECK BLC | CONT. NO. 1856

LOAD ON "A" LINE 5.02 K/' (FROM SUPERSTRUCTURE DESIGN)

PUMP AND ASSY WT
= 122 K AROUND PERIPHERY OF PUMP OPN'G.

WT. DUE TO WATER IN PUMP RM MAX 550 K

EFFECTIVE FLOOR AREA = 27 × 30 = 810 ⧆
−(OPENING @ PUMP) = $\pi \times 14.25^2$ = 635
175 ⧆

UNIFORM LOAD DUE TO WATER = $\frac{550^K}{175 ⧆}$ = 3.12 K/⧆
5'-0" t SLAB = 5 × .15 = 0.75
3.87 K/⧆

DESIGN DATA (W.S.D.)

f'c = 3000 psi
fc = 1050 "
vc = 60 "
fs = 20,000 "

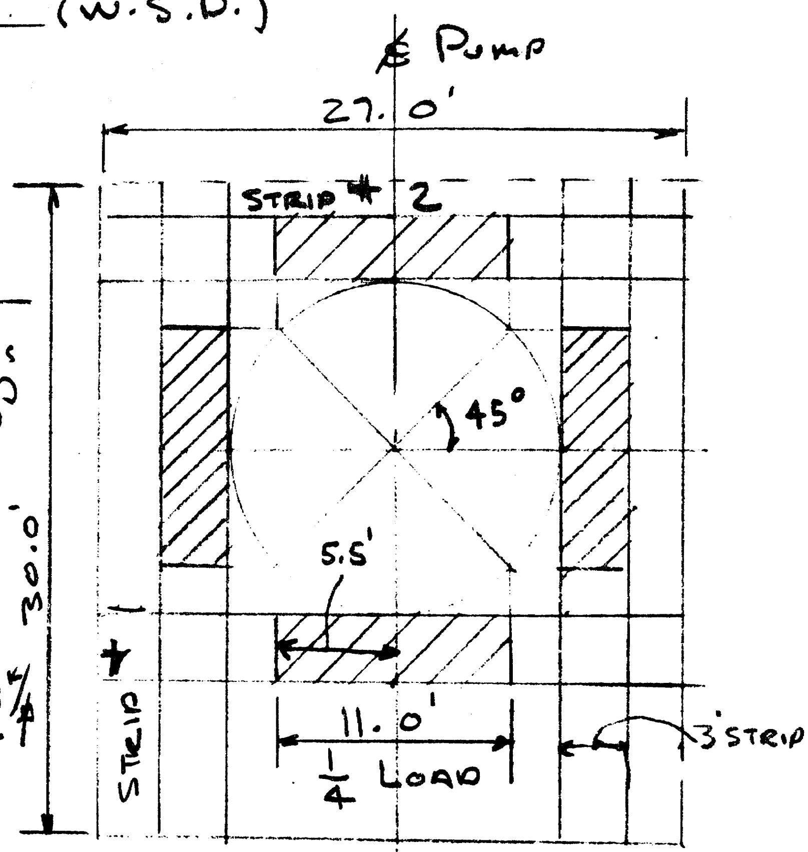
DESIGN SLAB

ASSUME: PUMP LOAD IS CARRIED BY 3.0 STRIP ON 4 SIDES AS SHOWN ON LOADING DIA.

ASSUME STRIP

LENGTH 11'-0"

UNIFORM LOAD = $\frac{122}{4} \times \frac{1}{11}$ = 2.8 K/⧆



LOADING DIA

A177

VOID

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PROJECT: Charles River Basin — ACC. NO. ______

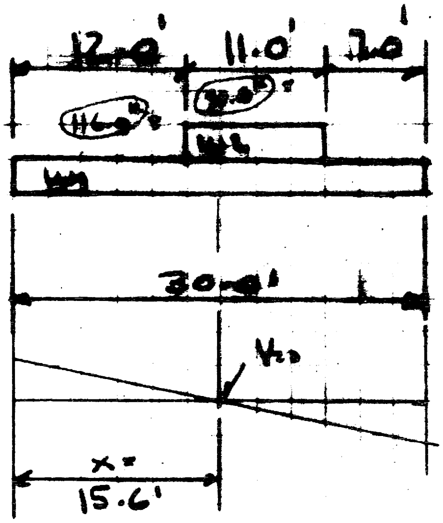
SUBJECT: Pumping Sta. Re[illegible] — SHEET NO. 2 OF ______

Blind Room @ El. 108.0' — DATE 9-14 1972

COMP. [illegible] — CHECK ALL — CONT. NO. 1856

Slab Design

Strip #1



$w_1 = 3.87^k/\phi$ (See Sh. #1)

$w_2 = \frac{2.8}{6.67}$ " (" " ")

12.8
58.0
70.8

18.0
58.0
76.0

$$\begin{array}{r} 70.8 \\ -46.5 \\ \hline 24.3 \end{array} \quad x = \frac{24.3}{6.67} + 12$$

$$= 3.6 + 12 = \underline{15.6'}$$

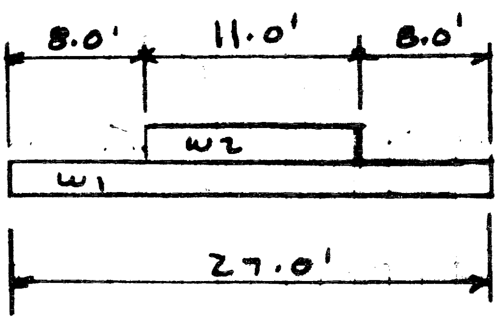
$$M = 70.8 \times 15.6 = 1100$$

$$-3.87 \times 15.6^2 \times \tfrac{1}{2} = 470$$

$$-2.8 \times 3.6^2 \times \tfrac{1}{2} = \underline{18}$$

$$612^{1k}$$

Strip #2



15.4^k
52.0
67.4^k

$$M = 67.4 \times 13.5 = [illegible]$$

$$-3.87 \times 13.5^2 \times \tfrac{1}{2} = 342$$

$$-2.80 \times 5.5^2 \times \tfrac{1}{2} = \underline{-42}$$

$$526$$

PROJECT Charles River Basin | ACC. NO. ________
SUBJECT Pumping Sta. Redesign | SHEET NO. 3 OF ____
Pump Room @ El. 108.0' | DATE 9-14 1972
COMP. JDB | CHECK GLG | CONT. NO. 1850

Slab Design

Strip #1

$M = 612^{1K}$ using 1.0' strip $M = 204^{1K}$

$d_n = \sqrt{\frac{204}{.152}} = 37$

$d_a = 60.0'' - 6.7 \text{ (cover)} = 53.3$

USE 5'-0" SLAB
#11@5 T&B

$A_s = \frac{204}{1.44 \times 53.3} = 2.72^{\square''} \sim \#11@5$

$V = 76 \times \frac{1}{3} = 25.4^{K} / \text{1 ft strip}$

$v = \frac{25.4}{12 \times 53.3} = .04 \text{ ksi}$

Strip #2

$M = 526$ using 1'-0" strip $M = 175^{1K}$

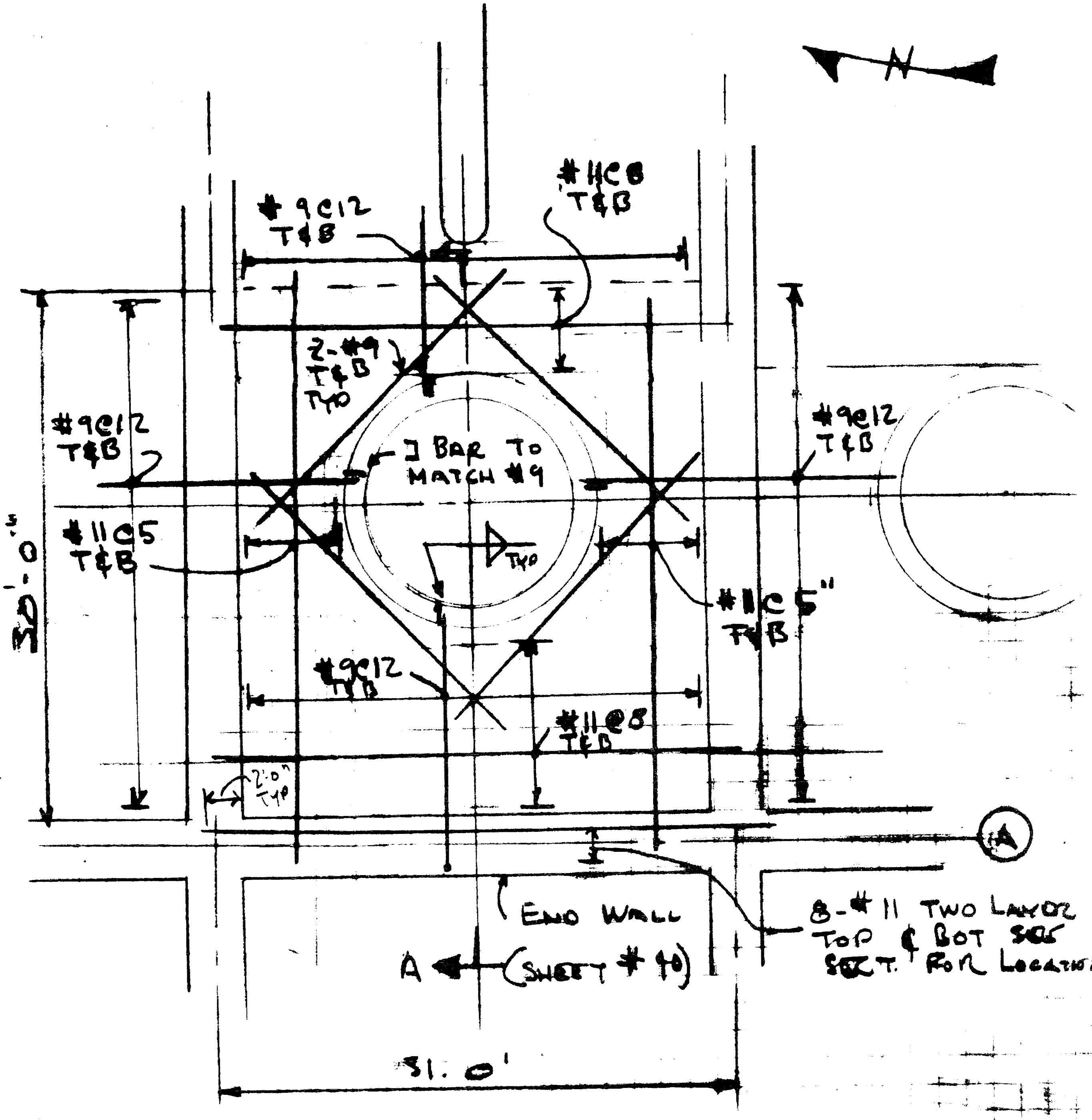
$V = 67.4$ " " " $V = 23^{K}$

$A_s = \frac{175}{1.44 \times 53.3} = 2.3^{\#}$ USE #11@8

Additional Reinf #9@12

VOID

PROJECT	CHARLES RIVER BASIN	ACC. NO.	
SUBJECT	PUMPING STA. REDESIGN	SHEET NO.	5 OF
	PUMP ROOM SLAB @ EL. 108.0'	DATE	9-13 1972
COMP.	XAB	CHECK GLC	CONT. NO. 1856



PUMP RM SLAB
@ EL. 108.0'

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A180

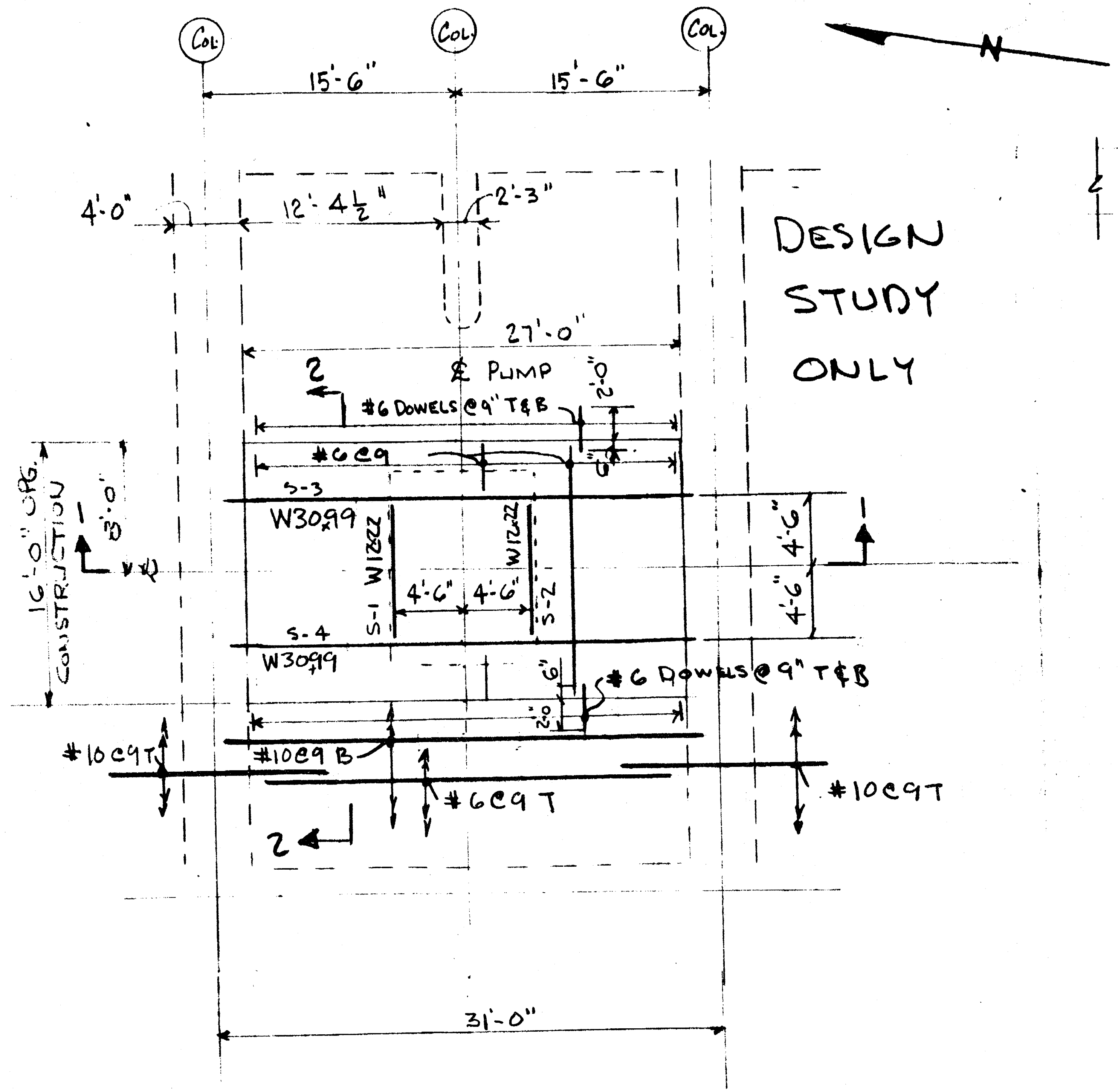
V010

CHARLES A. MAGUIRE & ASSOCIATES, INC.

Subject: CHARLES RIVER BASIN PUMPING STA. REDESIGN
Job No. 1856

Sheet No. 1? of
Date AUG. 22, 72
Computed JAD
Checked

CASE I 16'-0" WIDE CONSTRUCTION OPENING FULL LENGTH. TO BE FILLED IN AFTER INSTALLATION IS COMPLETED



PLAN @ EL. 122.00

A-181

OPENING FOR PUMP ASSEMBLY INSTALLATION

CHARLES A. MAGUIRE & ASSOCIATES, INC.

Subject CHARLES RIVER BASIN PUMPING STA. REDESIGN

JOB No. 1856

Sheet No. 15 of

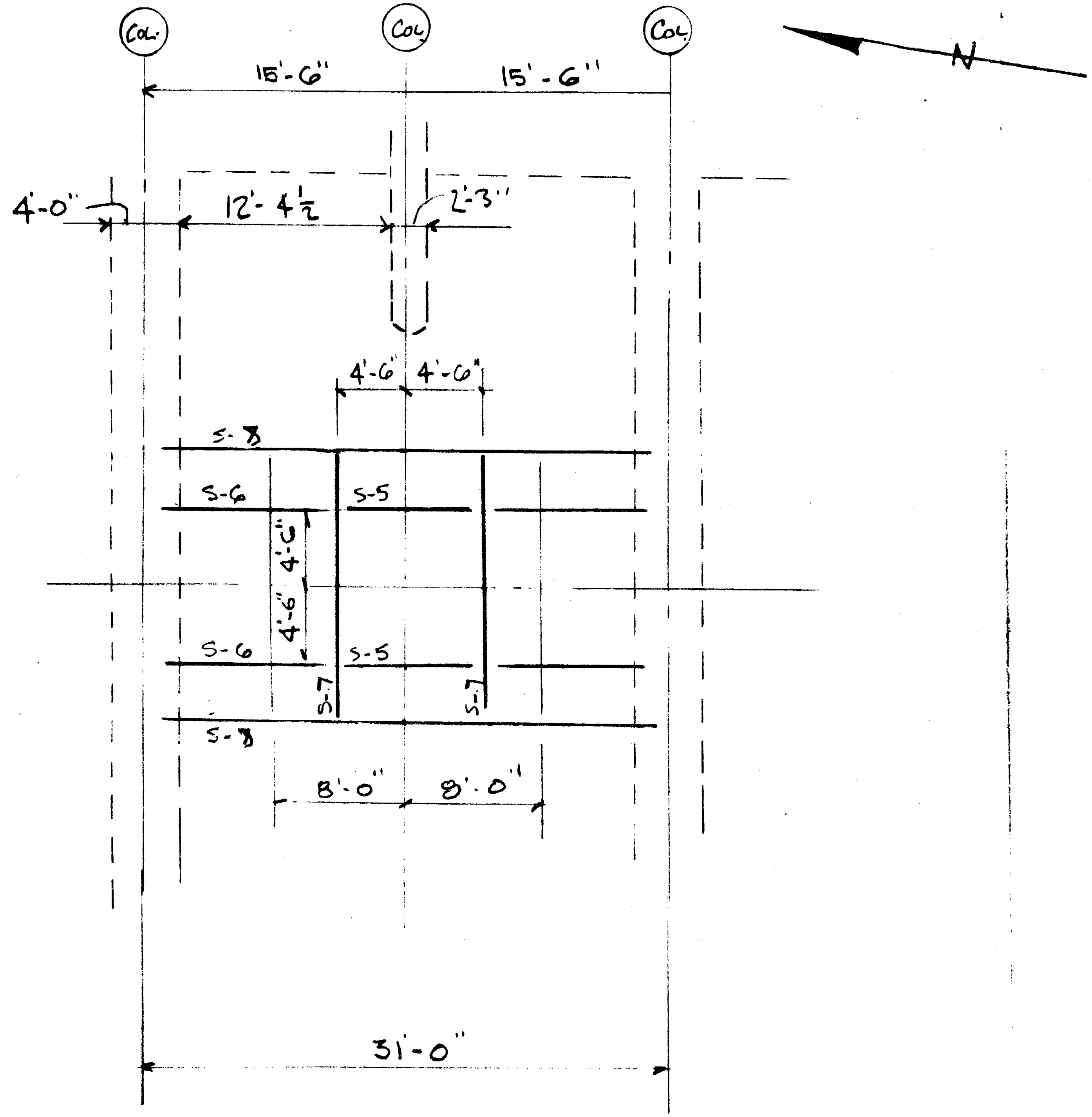
Date 8-22-72

Computed

Checked

CASE II

16'-0 SQ. PERMANENT OPENING WITH REMOVABLE STEEL SUPPORT FOR GEAR, WITH REMOVABLE COVER PL'S



PLAN @ EL 122.00

A 182

OPENING FOR PUMP ASSEMBLY INSTALATION

GO-406

PROJECT: CHARLES RIVER BASIN — ACC. NO. ______

SUBJECT: PUMPING STA. REDESIGN — SHEET NO. 12 OF ____

SLAB @ EL. 122.00 PUMP OPNG. — DATE 8-22 1972

COMP. JAB — CHECK GLC — CONT. NO. 1856

DESIGN SLAB @ EL. 122.00' W.S.D. (USE 200#/□ L.L. + EQUIP. WT)

f'_c = 3000 psi
f_c = 1050 psi
v_c = 60 psi

GEAR ASSY WT = 20ᴷ
ENGINE " " = 35.7ᴷ

"REF." FAIRBANKS MORSE POWER SYSTEMS JULY 26, 1971

DESIGN LOADS

L.L. = 200#/□'
D.L. = 600 (ASSUME 48" t)

ADDITIONAL LOAD ON SLAB

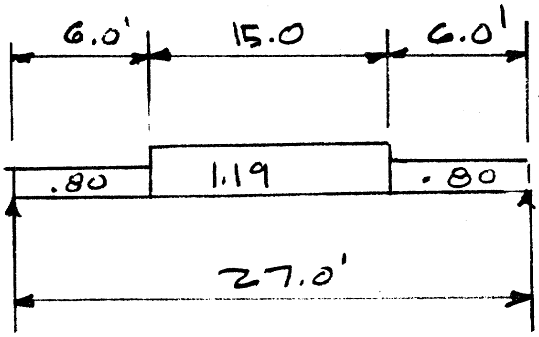
SLAB SHALL BE DESIGNED SO IT WILL BE CAPABLE TO SUPPORT THE DEAD WT OF THE MOTOR OR GEAR AT ANY LOCATION DUE TO THE POSSIBILITY OF MOVING DURING REPAIRS.

ASSUMED GEAR AREA = 9.0 × 9.0 = 81 Sq. Ft.
$w_g = \frac{20}{81} = .246^{k}/□$

ASSUMED ENGINE AREA = 15 × 2 = 30 Sq Ft.
$w_e = \frac{35.7}{30} = 1.19^{k}/□$

USING A SLAB "48" thick" IT IS OBVIOUS THAT MIN. REINF. WILL CONTROL THE DESIGN SECTION. THEREFORE DESIGN FOR SIMPLE SPAN

8.9
4.8
13.7



DESIGN FOR 800#/□' AND ASSUME ENGINE PLACED @ THE CENTER OF SPAN

+M = 13.7 × 13.5 = 185 ↻
1.19 × 7.5² × ½ = 33 ↺
.8 × 6 × 10.5 = 50 ↺
= 102ᴵᴷ

A183

PROJECT: CHARLES RIVER BASIN
SUBJECT: PUMPING STA. REDESIGN
SLAB @ EL. 122.00 PUMP OPNG.
COMP.: JAB CHECK: GLC
ACC. NO.
SHEET NO. 12 OF
DATE 8-22- 1972
CONT. NO. 1856

DESIGN STEEL BEAMS TO CARRY GEAR ASSEMBLY

USE: ASTM-A36 (AISC H.B., 1970)

DESIGN LOADS

GEAR ASSEMBLY TOTAL WT = 20^k

MAX. THRUST FROM GEAR = 125^k

200 #/S.F. L.L.

ASSUME TOTAL WT PLUS MAX. THRUST OF GEAR ACT ON BEAMS AS A UNIFORM LOAD DISTRIBUTING EQUALLY AROUND IT'S SUPPORT.

TOTAL LOAD = $125^k + 20^k = 145^k$

UNIFORM LOAD = $9.0' \times (4\text{-SIDES}) = \frac{145^k}{36'} = 4.03^{k}/'$ SAY $4.1^{k}/'$

S-1 / S-2 $M = 4.1 \times 9^2 \times \frac{1}{8} = 41.5^{1k}$

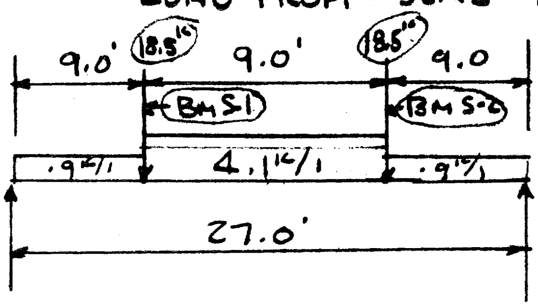
$S_R = 41.5 \times .545 = 22.7$

TRY W12x22 — USE W12x22 S-1 & S-2

$R = 4.1 \times 9 \times \frac{1}{2} = 18.5^k$

S-3 / S-4 ADDITIONAL LOAD ON BEAMS S-3 & S-4

LOAD FROM SLAB $.20^{k}/\square'$ $w_s = .2 \times 4.5 = 0.9^{k}/'$



18.5
18.5
8.1
45.1k

$M = +45.1 \times 27 \times \frac{1}{2} = +610^{1k}$

$-4.1 \times 4.5^2 \times \frac{1}{2} = -42$

$-.9 \times 9 \times 13.5 = -110$

458^{1k}

$S_R = 458 \times .545 = 250$

LOAD. DIAG.

USE W30 99 S-3 / S-4

PROJECT: CHARLES RIVER BASIN
SUBJECT: PUMPING STA. REDESIGN
SLAB @ EL. 122.00 PUMP OPNG.
COMP. JAB CHECK GLC

ACC. NO.
SHEET NO. 8 OF
DATE 8-22 1972
CONT. NO. 1856

$d_R = \sqrt{\frac{102}{.152}} = 26''$ ~ $d_A = 48''$ (4" COVER)

$$\begin{array}{r} 48 \\ -\ 4.7 \\ \hline 43.3 \end{array}$$

$A_s = \frac{102}{1.44 \times 43.3} = 1.64^{\square''}$

TEMP. REINF. (UNRESTRAINED MEMBER)

$A_{s_t} = .002 \times 48 \times 12 = 1.15 \times (.5) = .575$ #6@9

#10@9 $\Sigma = \frac{13.7}{.147 \times .872 \times 433} = 2.5\ \checkmark$

USE 48" SLAB
MAIN REINF #10@9"
TEMP. #6@9"

SLAB AROUND GEAR OPENING.

$M = .8 \times 16^2 \times \frac{1}{8} = 26^{1k}$ ~ USE MIN REINF #6@9"

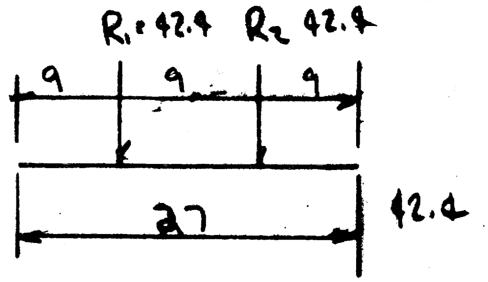
– PROVIDE DOWELS IN MAIN SLAB THEN REINF CAN BE WELDED BEFORE CONST.

PROJECT: CHARLES RIVER BASIN
SUBJECT: PUMPING STA. REDESIGN
SLAB @ EL. 122.0
COMP. JAB　CHECK GLC

ACC. NO.
SHEET NO. 6 OF
DATE 8-22 1972
CONT. NO. 1856

S-7

$R_1 = R_2 =$ S5 = 18.6
S6 = 23.9
42.4



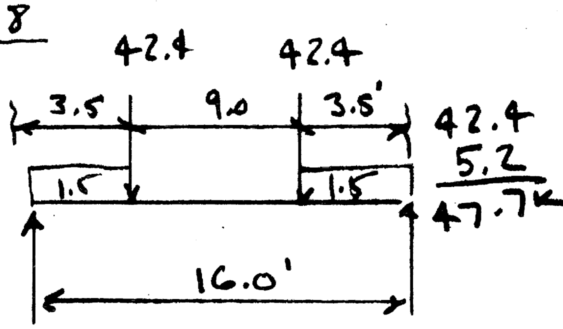
M = 42.4 x 9 = 38

S = .545 x 38 = 20.8

USE 14W22

S-8

w = .846 x 1.75 = 1.5 k/'



USE W21 x 49

M = 47.7 x 3.5 = + 168
1.5 x 3.5² x ½ = − 9
157

S = .545 x 157 = 86

USE SLAB CASE I DESIGN.

CHARLES A. MAGUIRE & ASSOCIATES INC. ENGINEERS

PROJECT CHARLES RIVER BASIN — ACC. NO.
SUBJECT PUMPING STA. REDESIGN — SHEET NO. 17 OF
SLAB @ EL. 122.0 PUMP OPNG. — DATE 8-22- 1972
COMP. JB — CHECK GLC — CONT. NO. 1856

DESIGN STEEL BEAMS TO CARRY GEAR ASSY.

USE: ASTM - A36 (AISC H.B., 1970)

DESIGN LOADS

GEAR ASSEMBLY TOTAL WT = 20^k

MAX. THRUST = 125

145^k

ASSUME TOT. WT OF GEAR PLUS MAX THRUST OF GEAR ACT ON BEAMS AS A UNIFORM LOAD DISTRIBUTING EQUALLY AROUND ITS SUPPORTS

UNIFORM LOAD = $9.0 \times (4\text{-SIDES}) = \frac{145}{36} = 4.04$

S-5

$M = 4.1 \times 9^2 \times \frac{1}{8} = 41.5$

$S = 41.5 \times .545 = 22.7$ — USE W12x22

$R = 4.1 \times 9 \times \frac{1}{2} = 18.5$

S-6

D.L. = .246 GEAR
D.L. = .600 SLAB
.846

$w = .846 \times 6.25 = 5.3^{k/'}$

$M = 5.3 \times 9^2 \times \frac{1}{8} = 54.7$

$S = .546 \times 54.7 = 29.3$

$R = 5.3 \times 4.5 = 23.9^k$

NOTE: THE REMOVABLE COVER PLS SHALL BE DESIGNED USING 200#/□' L.L. ONLY.
DESIGN BEAM S-6 TO CARRY GEAR ASSY. DUE TO THE POSSIBILITY OF MOVING GEAR OVER DURING REPAIRS

USE W14x22

DESIGN CRITERIA.— (L.E)

DESIGN WATER ELEVATIONS

	BASIN SIDE	TIDE SIDE
HIGH WATER	El 112	El. 116
LOW WATER	El 106	El. 98

DEWATERED MAINTENANT CONDITION USE MAX El. 114

REINFORCED CONCRETE DESIGN

- WORKING STRESS method :
- $f'_c = 3000$ psi , $f_c = .35 f'_c = 1050$ * $f_s = 20{,}000$ (ASTM A615 GRADE 40)
  $v_c = 1.1\sqrt{f'_c} = 60$ psi
- DESIGN COEFICIENTS : $K = \frac{1}{2} f_c j k = 152$
  $n = E_s/E_c = 9.2$
  $a = 1.44$

DESIGN DEAD LOADS.

CONCRETE	: 150 #/FT³
WATER	: 64 "
EARTH (DRY)	: 125 "
" (WET)	: 140 "

SEISMIC LOADING .— ZONE 3

- Apply horizontal force of 0.1 g at center of gravity -
- allowable stress to be increased by 1/3

A 188

PROJECT CHARLES RIVER BASIN — ACC. NO. 1353-3a
SUBJECT PUMP STATION – FOREBAY WALLS — SHEET NO. 3 OF
FLOOR @ El 118.61 — DATE 8-26 197_
COMP. — CHECK CLC — CONT. NO.

CONC SLAB El 118.61 : Assumed 18" slab, HS 20-44 TRUCK LOAD

Clear span

$S = 27 + 1.5 = 28.5$ ft

Moment for the case main re-inforcement parallel to traffic

$LLM = .900 S = .900 \times 28.5 = 25.62$ k-ft

$DLM = \frac{.225(28.5)^2}{8} = 22.90$ k-ft

$M_t = 48.52$ k-ft

depth required by moment :

$d = \sqrt{\frac{48.52}{.152}} = 17.9" + 1" = 18.9" > 18"$

TRY 20" SLAB – $d = 19"$

$A_s = \frac{48.52}{1.44 \times 19} = 1.78 \text{ in}^2$ (HIGH)

TRY BEAM & SLAB SYSTEM—

Assumed using 18" wide beam – clear span: 11'-6"
12" slab 1'-6"
S = 13'-0"

Main re-inforcement perpendicular to traffic

$M_L = \frac{S+2}{32} \times P_{20} = \frac{13.00+2}{32} \times 16 = 7.5$ k-ft

$M_D = \frac{.15(13.00)^2}{8} = 3.16 = 3.16$ k-ft

M_t 10.66 k-ft

$d = \sqrt{\frac{10.66}{.152}} = \sqrt{70.1} = 8.4 + 1 = 9.4"$

Try 12" slab –

PROJECT: CHARLES RIVER BASIN	ACC. NO. 1856.30
SUBJECT: PUMPING STATION – FOREBAY WALLS	SHEET NO. 3 OF
	DATE 8.28 1972
COMP. T.L. CHECK GLS	CONT. NO.

$A_s = \frac{10.66}{1.44 \times 11} = .672 \text{ in}^2$ #7@ 12" (B)

Try ~~14"~~ 14 slab. ~~d = 13"~~ d = 13.5"

$A_s = \frac{10.66}{1.44 \times 13} = .570 \text{ in}^2$

SEE SHEET # 3a REV. USING 2" COVER

USE :	14" SLAB	# 7 @12" (B)
		# 5@ 12" (T)

Temp steel $A_s = .002 \times 12 \times 14 = .336$

Each Face $= .168 \text{ in}^2$

Slab expose to weather, increase 25% as specify by EM 1110-2-2103

$A_s = 1.25 \times .168 = .21 \text{ in}^2$

5 @ 12" T & B

A 190

PROJECT Charles River Basin ACC. NO.
SUBJECT Pumping Station SHEET NO. 3A OF
DATE 19
COMP. DB CHECK CONT. NO.

CHECK SLAB 14"

2" COVER

$\frac{5"}{7"}$ POCKET FOR TRASH RACK CLEANER

TRY 16" t SLAB $w = 1.33 \times .15 = .2$

$M_D = .2 \times 13.0^2 \times \frac{1}{8} = 4.2^{k}$

ΣM REVISED $= M_L + M_D$

$= 7.5 + 4.2 = 11.7^{k}$

$d = \sqrt{\frac{11.7}{.152}} = 8.8$ $d_A = 16 - 7.5 = 8.5$ ok

$A_{s_+} = \frac{11.7}{1.44 \times 8.5} = .96$

Use 16" slab
#7 @ 6 B
#5 @ 12 T

PROJECT CHARLES RIVER BASIN — ACC. NO.
SUBJECT PUMP STATION - FOREBAY WALLS — SHEET NO. 4 OF
DATE 8.29 1972
COMP. TA — CHECK GLC — CONT. NO.

BEAM B1, B2, B3 —

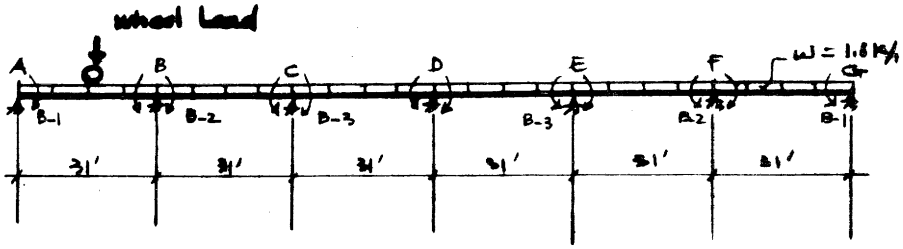
Assumed sized 18" x 36"

own weight $= \frac{18 \times 36}{144} \times .15 = .675$ K/1

Floor's Dead load $= \frac{13'}{2} \times \frac{14}{12} \times 1 \times .15 = 1.040$ K/1

$w_{DL} = 1.715$ K/1 say 1.8 K/1

wheel Load



BEAM CONSTANCES —

$I = \frac{bh^3}{12} = \frac{18 \times (36)^3}{12} = 70,200 \text{ in}^4$

$K_{BA} = K_{FG} = \frac{3(70,200)}{31 \times 12} = 567$

$K_{BC} = K_{FE} = \frac{4(70,200)}{31 \times 12} = 756$

1323

$DF_{BA} = \frac{567}{1323} = .43$

$DF_{BC} = \frac{767}{1323} = .57$

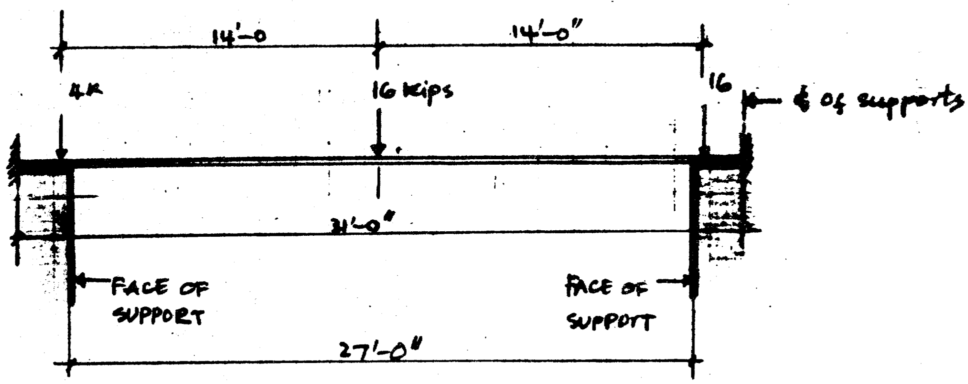
$DF_{CB} = \frac{767}{1534} = .5$

$DF_{CD} = \frac{767}{1534} = .5$

wheel load will be placed at location that yield the max moments —

$FEM_{DL} = \frac{1.8(31)^2}{12} = \pm 144$ k-ft

FOR wheel load, let assumed the truck runs parallel with the beam - by inspection, let position the truck so that it will yield max Fixed end moments:



$$FEM_{LL} = \frac{PL}{8} = \frac{16 \times 31}{8} = 62\ K\text{-}Ft$$

Note: the Front wheels and the rear wheel load were neglected here, because the designer believe that these load will not create any bending since the center of loads are 6" beyond the face of supports - therefore they can be transfered to the walls below -

$$FEM_{D+L} = \pm(144 + 62) = \pm 206$$

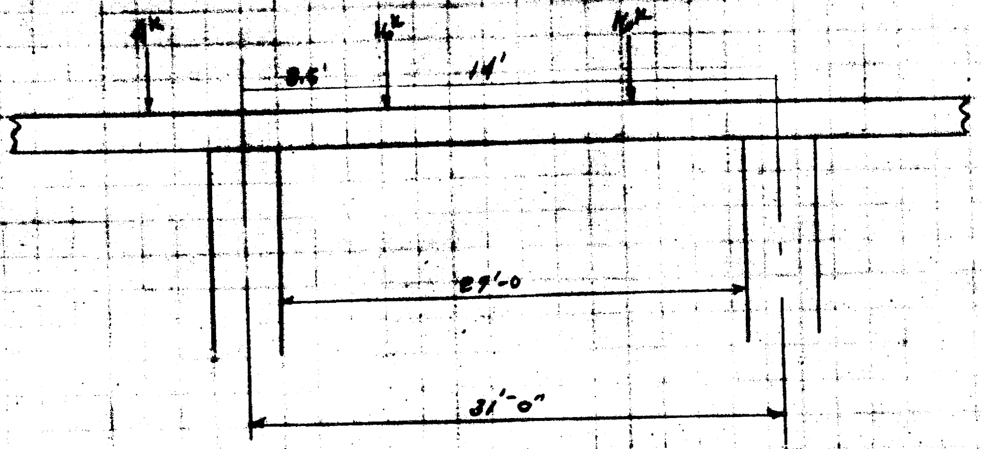
PROJECT CHARLES RIVER DAM ACC. NO.

SUBJECT PUMP STATION - FLOOR BAY MARKS SHEET NO. 21 OF

FLOOR 36.6 @ HP.61 DATE 14 Sept 1972

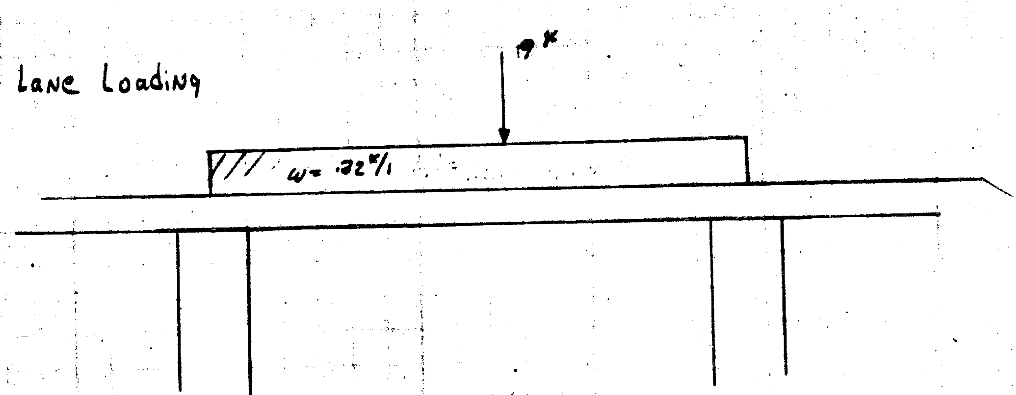
COMP. [illegible] CHECKED BY [illegible] CONT. NO.

Truck Loading



$$F.E.M. = 16(8.5)\left(\frac{8.5+14.0}{31}\right) = 98.9^{1\text{-}k} > 62^{1\text{-}k}$$

Lane Loading



$$F.E.M. = \frac{wl^2}{12} + \frac{PL}{8}$$

$$= \frac{.32(31)^2}{12} + \frac{9(31)}{8} = 25.6 + 34.9 = 60.5^{1\text{-}k}$$

PROJECT: CHARLES RIVER BASIN — ACC. NO. 1856-30

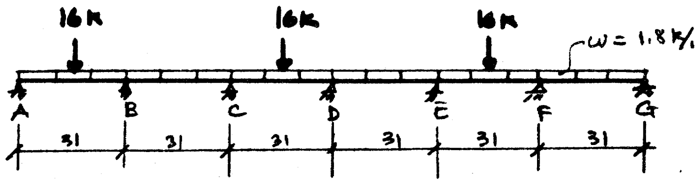
SUBJECT: PUMPING STATION – FOREBAY WALLS — SHEET NO. 6 OF

DATE: 8-28 1972

COMP. TC — CHECK: GLC — CONT. NO.

CASE ONE :–

POSITION OF TRUCK THAT WILL CREATE MAX POSITIVE MOMENT in span

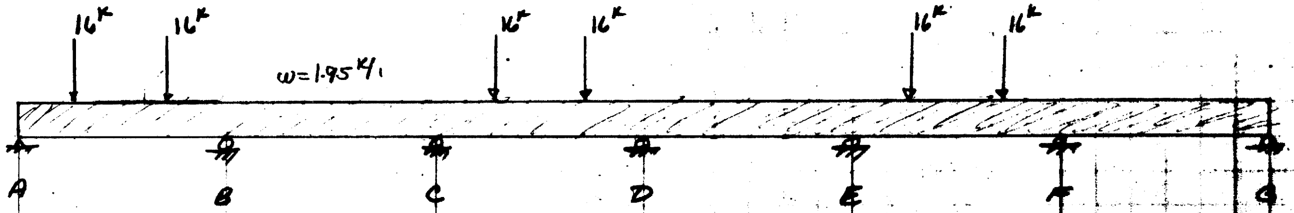


	A	B	B	C	C	D	D	E	E	F	F	G
I	70,200		70,200		70,200		70,200		70,200		70,200	
K	567		756		756		756		756		567	
DF		.43	.57	.5	.5	.5	.5	.5	.5	.57	.43	
FEM	-206	+206	-144	+144	-206	+206	-144	+144	-206	+206	-144	+144
	+206	+103									-72	-144
	0	+309	-144	+144	-206	+206	-144	+144	-206	+206	-216	0
		-71	-94	+31	+31	-31	-31	+31	+31	+5.7	+4.3	
		0	+15.5	-47	-15.5	+15.5	+15.5	-15.5	+2.9	+15.5	0	
		-6.7	-8.8	+31.3	+31.3	-15.5	-15.5	+6.3	+6.3	-8.8	-6.7	
		0	+15.7	-4.4	-7.8	+15.7	+3.2	-7.8	-4.4	+3.2	0	
		-6.8	-8.9	+6.1	+6.1	-9.5	-9.5	+6.1	+6.1	-1.8	-1.4	
		0	+3.	-4.5	-4.8	+3	+3	-4.8	-.9	+3	0	
		-1.2	-1.7	+4.6	+4.6	-3	-3	+2.9	+2.9	-1.7	-1.2	
		0	+2.3	-.9	-1.5	+2.3	+1.5	-1.5	-.9	+1.5	0	
		-1.0	-1.3	+1.2	+1.2	-1.9	-1.9	+1.2	+1.2	-.8	-.7	
M	0	+222.3	-222.3	+161.3	-161.3	+181.7	-181.7	+161.8	-161.8	+222.7	-222.7	0
V_w	27.9↑	+27.9	27.9	27.9	27.9	27.9	27.9	27.9	27.9	27.9	27.9	+29.7
V_p	8↑	+8	0	0	8	8			8	8		
V Elastic	-7.2↓	+↑7.2	+1.97	-1.97	-.66	+.66	+.66	-.66	-1.96	+1.96	+7.2	-7.2
SHEAR	+28.9	+43.1	+29.87	+25.93	26.24	+36.56	+28.56	+26.24	+26.94	+29.86	35.1	+22.5

$$\text{Max } M^{+} (\text{in } AB) = (1.5 \times 144 + 2 \times 62) - \frac{222.3}{2}$$

$$= 216 + 124 - 111.1 = 228.9 \text{ k-ft}$$

Case I



A	B	B	C	C	D	D	E	E	F	F	G
0	.43	.57	.50	.50	.50	.50	.50	.50	.57	.43	0
	+382.5	-156.0	+156.0	-255.0	+255.0	-156.0	+156.0	-255	+255.0	-234.0	0
	-97.0	-129.5	+49.5	+49.5	-49.5	-49.5	+49.5	+49.5	-12.0	-9.0	
		+24.7	-64.7	-24.7	+24.7	+24.7	+24.7	-6.0	+24.9		
	-10.6	-14.1	+49.7	+49.7	-24.7	-24.7	+15.3	+15.3	-14.1	-10.6	
		+22.3	-7.0	-12.3	+22.3	+7.6	-12.3	-7.0	+7.6		
	-9.6	-12.7	+9.6	+9.6	-14.9	-14.9	+9.6	+9.6	-4.3	-3.3	
		+4.8	-6.3	-7.4	+4.8	+4.8	-7.3	-2.1	+4.8		
	-2.1	-2.7	+6.6	+6.6	-4.8	-4.8	+4.7	+4.7	-2.7	-2.1	
		+3.3	-1.3	-2.4	+3.3	+3.3	-2.9	-1.3	+3.3		
	-1.4	-1.9	+1.8	+1.8	-2.8	-2.8	+1.8	+1.8	-1.3	-1.0	
		+0.9	-0.9	-1.9	+0.9	+0.9	-1.4	-0.6	+0.9		
	-0.4	-0.5	+1.4	+1.4	-0.9	-0.9	+1.0	+1.0	-0.5	-0.4	
		+0.7	-0.2	-0.4	+0.7	+0.5	-0.4	-0.2	+0.5		
	-0.3	-0.4	+0.3	+0.3	-0.6	-0.6	+0.2	+0.2	-0.2	-0.2	
	+261.1	-261.1	+189.5	-190.2	+213.5	-213.4	+189.7		+260.6	-260.6	

$M = \frac{1.95(31)^2}{8} + 16(8.5) - 130.5$

$= 234 + 136 - 130.5 = 239.5$ '-k

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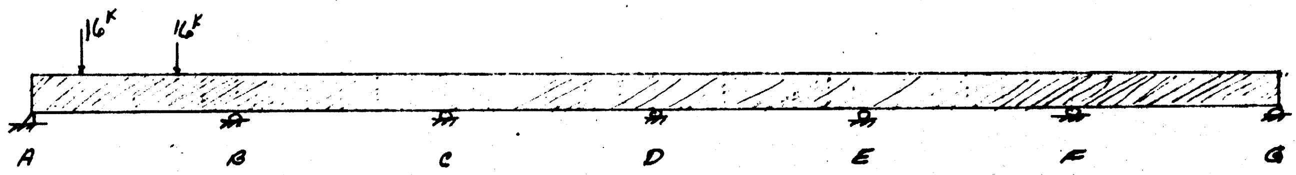
PROJECT CHARLES RIVER DAM
SUBJECT [illegible]
COMP. [illegible]
CHECKED [illegible]
JOB NO.
SHEET NO. [illegible]
DATE 15 Sept [illegible]
CONT. NO.



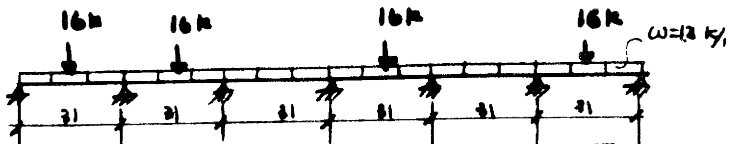
A		B		C		D		E		F	G
0	.43	.57	.50	.50	.50	.50	.50	.50	.57	.43	0
	+382.5	-156.0	+156.0	-156.0	+156.0	-156.0	+156.0	-156.0	+156.0	-234.0	
	-97.0	-129.5	-69.7		+32.3		-16.1		+9.0		
		+16.1	+32.3	+32.3	-16.1	-16.1	+8.0	+8.0	+42.3	+31.7	
		+1.9		-8.0		+4.0		+21.1			
	-7.7	+10.3		+0.3		-5.2	-10.5	-10.5	-15.2		
		-1.2	+3.8	+3.8	+0.6	+0.6	+0.3		+8.0	+2.2	
	+0.5	+0.7	+5.2		+1.9		-0.1				
			-2.5	-2.5	-1.2		-0.1	-0.1			
	+278.3	-278.3		-0.1	-0.3	-0.3		+1.5			
				+0.1		0.2	-0.2	-0.7			
					+0.1	0.1					

$M = 234 + 136 - \frac{278}{2} = 370 - 139 = 231^{1-k}$

A197

PROJECT: CHARLES RIVER BASIN | ACC. NO. 1856.30
SUBJECT: PUMP STATION – FOREBAY WALLS | SHEET NO. 8 OF
DATE 8.29 1972
COMP. T.L. | CHECK GLC | CONT. NO.

CASE TWO .– position of wheels that will give max Negative moment

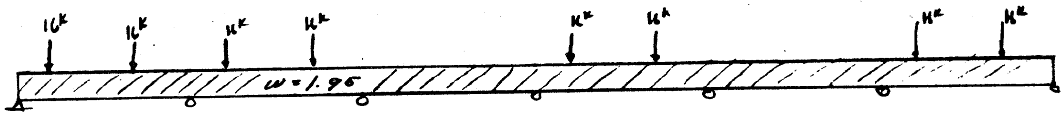


	A	B	B	C	C	D	D	E	E	F	F	G
K	567		756		756		756		756		567	
DF		.43	.57	.5	.5	.5	.5	.5	.5	.57	.43	
FEM	-206	+206	-206	+206	-144	+144	-206	+206	-144	+144	-206	+206
	+206	+103									-103	-206
	0	+309	-206	+206	-144	+144	-206	+206	-144	+144	-309	0
		-44	-59	-31	-31	+31	+31	-31	-31	+94	+71	
		0	-15.5	-29.5	+15.5	-15.5	-15.5	+15.5	+47	-15.5	0	
		+6.7	+8.8	+7.0	+7.0	+15.5	+15.5	-31.3	-31.3	+8.8	+6.7	
		0	+3.5	+4.4	+7.8	+3.5	-15.6	+7.8	+4.4	-15.6	0	
		-1.5	-2.0	-6.1	-6.1	+6	+6	-6.1	-6.1	+8.9	+6.7	
		0	-3.0	-1	+3	-3	-3	+3	+4.9	-3	0	
		+1.3	+1.7	-1	-1	+3	+3	-3.9	-3.9	+1.7	+1.2	
		0	-.5	+.9	+1.5	-.5	-1.9	+1.5	+.9	-1.9	0	
		+.2	+.3	-1.2	-1.2	+1.2	+1.2	-1.2	-1.2	+1.1	+.8	
	0	+267.7	-267.7	+148.5	-148.5	+186.2	-185.3	+160.3	-160.3	+222.5	-222.6	
V_w	27.9	27.9	27.9	27.9	27.9	27.9	27.9	27.9	27.9	27.9	27.9	27.9
V_p	.80	8.0	8.0	8.0			8.0	8.0			8.0	8.0
$\Sigma M / \ell$	-8.6	+8.6	+3.85	-3.85	-1.18	+1.18	+.81	-.81	-.72	+.72	+7.3	-7.3
SHEAR	+27.3	+44.5	39.75	+32.05	+26.72	29.08	36.71	35.09	+27.18	28.62	42.2	+28.6

A198

PROJECT	CHARLES RIVER DAM
SUBJECT	P
COMP.	GLC
CHECKING	TL
ACC. NO.	
SHEET NO.	8a OF
DATE	14 Sept 1982
CONT. NO.	

Case II

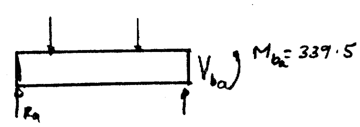


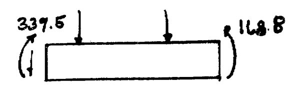
A B C D E F G

0	.43	.57	.50	.50	.50	.50	.50	.50	.57	.43	0
-156.	+156	-156	+156	-156	+156	-156	+156	-156	+156	-156	+156
-99	+99	-99	+99			-99	+99			-99	+99
	+127.5									-127.5	
0	+382.5	-255.0	+255.0	-156.0	+156.0	-255.0	+255.0	-156.0	+156.0	-382.5	0
	-54.8	-72.7	-49.5	-49.5	+49.5	+49.5	-49.5	-49.5	+129.5	+99.0	
		-24.7	-36.3	+24.7	-24.7	-24.7	+24.7	+64.7	-24.7		
	+10.6	+14.1	+5.8	+5.8	+24.7	+24.7	-44.7	-44.7	+14.1	+10.6	
		+2.9	+7.0	+12.3	+2.9	-22.3	+12.3	+7.0	-22.3		
	-1.3	-1.6	-9.6	-9.6	+9.7	+9.7	-9.6	-9.6	+12.7	+9.6	
		-4.8	-0.8	+4.8	-4.8	-4.8	+4.8	+6.3	-4.8		
	+2.1	+2.7	-2.0	-2.0	+4.8	+4.8	-5.5	-5.5	+2.7	+2.1	
		-1.0	+1.3	+2.4	-1.0	-2.7	+2.4	+1.3	-2.7		
	+0.4	+0.6	-1.8	-1.8	+1.8	+1.8	-1.8	-1.8	+1.5	+1.2	
	+339.4	-339.5	+169.1	-168.8	+218.9	-219.0	+188.1	-189.8	261.9	262.0	

0 339.5 168.8 219.0 188.0 262.0

Shear =





$$V_{ba} = \frac{16(8.5+22.5)+1.95(31)(15.75)-339.5}{31} = 16+30.7-10.95 = 35.75$$

$$V_{bc} = \frac{16(31)+30.7(31)+168.8-339.5}{31} = 16+30.7-5.5 = 41.2$$

$V = 35.75$

$$M_{Des} = M_1 - \frac{Va}{2} = 339.5 - \frac{35.75(4)}{2} = 268.0$$ Moment @ face of support

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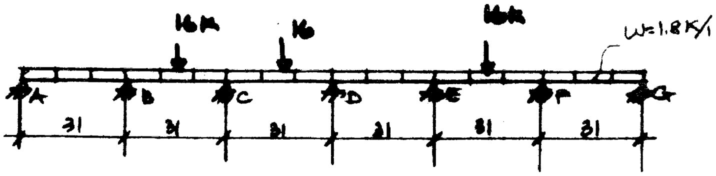
PROJECT CHARLES RIVER BASIN — ACC. NO. 1952-3[illegible]

SUBJECT PUMP STATION - FOREBAY WALLS — SHEET NO. 9 OF

DATE 9.29 1972

COMP. T.L. — CHECK — CONT. NO.

CASE THREE.- position of wheels to give max M^-.



	A	B	B	C	C	D	D	E	E	F	F	G
K	567		756		756		756		756		567	
DF		.43	.57	.5	.5	.5	.5	.5	.5	.57	.43	
	-144	+144	-206	+206	-206	+206	-144	+144	-206	+206	~~-144~~	~~+144~~
	+144	+72									-72	-144
	0	+216	-206	+206	-206	+206	-144	+144	-206	+206	-216	0
		-4.3	-5.7	0	0	-31	-31	+31	+31	+5.7	+4.3	
		0	0	-2.9	-15.5	0	+15.5	-15.5	+2.9	+15.5	0	
		0	0	+9.2	+9.2	-7.8	-7.8	+6.3	+6.3	-8.9	-6.6	
		0	+4.6	0	-3.9	+4.6	+3.2	-3.9	-4.5	+3.1	0	
		-2.0	-2.6	+1.9	+1.9	-3.9	-3.9	+4.2	+4.2	-1.8	-1.3	
		0	.9	-1.3	-1.9	+.9	+2.1	-1.9	-.9	+2.1	0	
		-.4	-.5	+1.6	+1.6	-1.5	-1.5	+1.4	+1.4	-1.2	-.9	
		+209.3	-209.3	+214.5	-214.5	+167.3	-167.3	+165.6	-165.6	+220.5	-220.5	

CASE II governed for M^- max = 267.7 K-ft

max Shear = 44.5 K. (left side of B)

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SUBJECT	PUMPING STATION - FOREBAY WALLS -	SHEET NO.	10 OF
		DATE	8.29 1972
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Max $M^- = 267.7$ k-ft

Max $M^+ = 228.9$ k-ft

max $V = 44.5$ k-ft

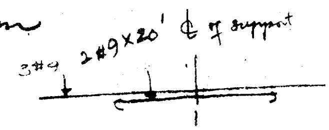
Design Beams.-

depth required by moment

$d = \sqrt{\frac{267 \times 12}{.152 \times 18}} = \sqrt{1170} = 34.2''$ (at supports)

$d = \sqrt{\frac{228.9 \times 12}{.160 \times 18}} = \sqrt{1005} = 31.8 + 2.5 = 34.3''$

try : 18" x 42" beam



$A_s(top) = \frac{267.7}{1.44 \times 40} = 4.64\ in^2$ 5 # 9 top

$A_s(bott) = \frac{228.9}{1.44 \times 40} = 4.00\ in^2$ 4 - #9 Bottom

unit shear at the supports

$v_s = \frac{44,500}{18 \times 42} = 58.8$ psi

allowable shear : $v = 1.1\sqrt{f'_c} = 60$ psi > 58.8 psi

∴ use min stirrup re-inforcement

say #4 ⊔ @ 12"

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Try 24" wide beam

$$d = \sqrt{\frac{267 \times 12}{.152 \times 24}} = \sqrt{880} = 29.6" + 2 = 31$$

Try 30"x30"

$$d = \sqrt{\frac{267 \times 12}{.152 \times 30}} = \sqrt{704} = 26.7" + 2 = 28.7$$ *

use 30" deep beam

d = ~~28"~~ 27.4"

USING 2" COVER CHECK Bm
2-9-73 AB

$$A_s\ top = \frac{267}{1.44 \times \sout{28}\ 27.4} = \sout{6.6}\ 6.7\ in^2 \quad 5\#10$$

$$A_s\ bott = \frac{229}{1.44 \times \sout{28}\ 27.4} = \sout{5.68}\ 5.84\ in^2 \quad 5\#10$$

unit shear at support

$$v_s = \frac{44{,}500}{30 \times 30} = 49.5\ psi < 60\ psi \quad O.K$$

∴ use min stirrup re-inforcement

~~say~~ #4 U @ 12" for 5' from face of supports

Rest #4 @ 18"

PROJECT C.R.D. ACC. NO.
SUBJECT Pump Sta Forebay ~~Beam~~ SHEET NO. 106 OF
Check Deflections DATE 1/22 19 73
COMP. LE CHECK CONT. NO.

Check Approx. Defl. of 30x30 Beam, $L = 28' \pm$

D.L. Beam and slab ~ 2 k/'
16k Concentrated load @ ℄

Assume simply supported (conservative)

$$I = \frac{1}{12} \times 30^4 = 67{,}500 \text{ in}^4$$

$$E_c = 3.5 \times 10^6 \text{ psi}$$

$$\Delta = \frac{5wl^4}{384EI} + \frac{Pl^3}{48EI} = \left(\frac{5wl}{384} + \frac{P}{48}\right)\frac{l^3}{EI}$$

$$= \left(\frac{5 \times 2 \times 28}{384} + \frac{16}{48}\right) \times \frac{28^3 \times 1728 \times 1000}{3.5 \times 10^6 \times 67{,}500}$$

$$= (0.73 + 0.33) \times 0.16 = 0.17 \text{ inches}$$

Deflection no problem

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PROJECT: CHARLES RIVER BASIN
SUBJECT: PUMPING STATION - FOREBAY WALLS
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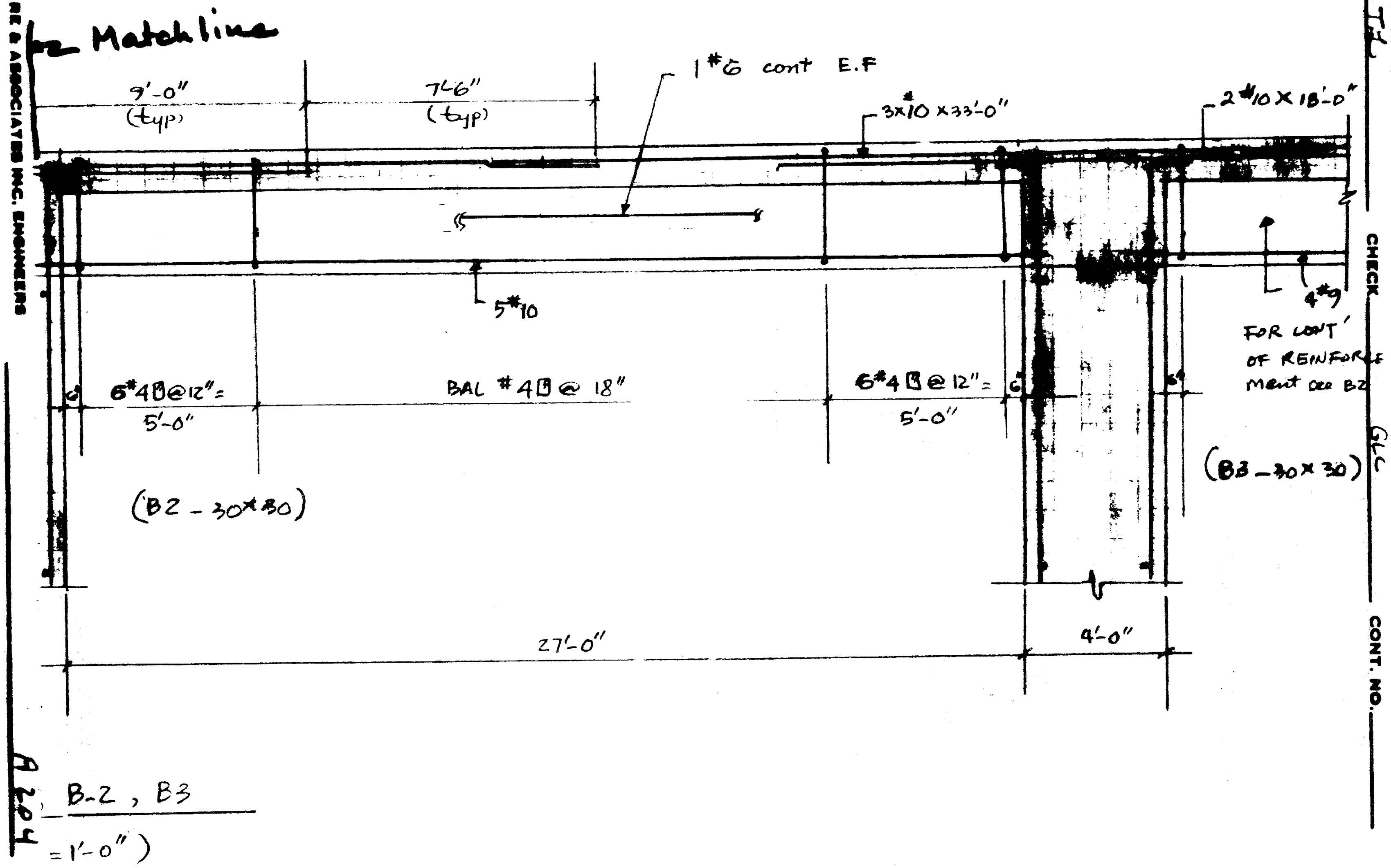
CHECK: GLC
ACC. NO. 185630
SHEET NO. 4 OF
DATE: 8.29 1972
CONT. NO.



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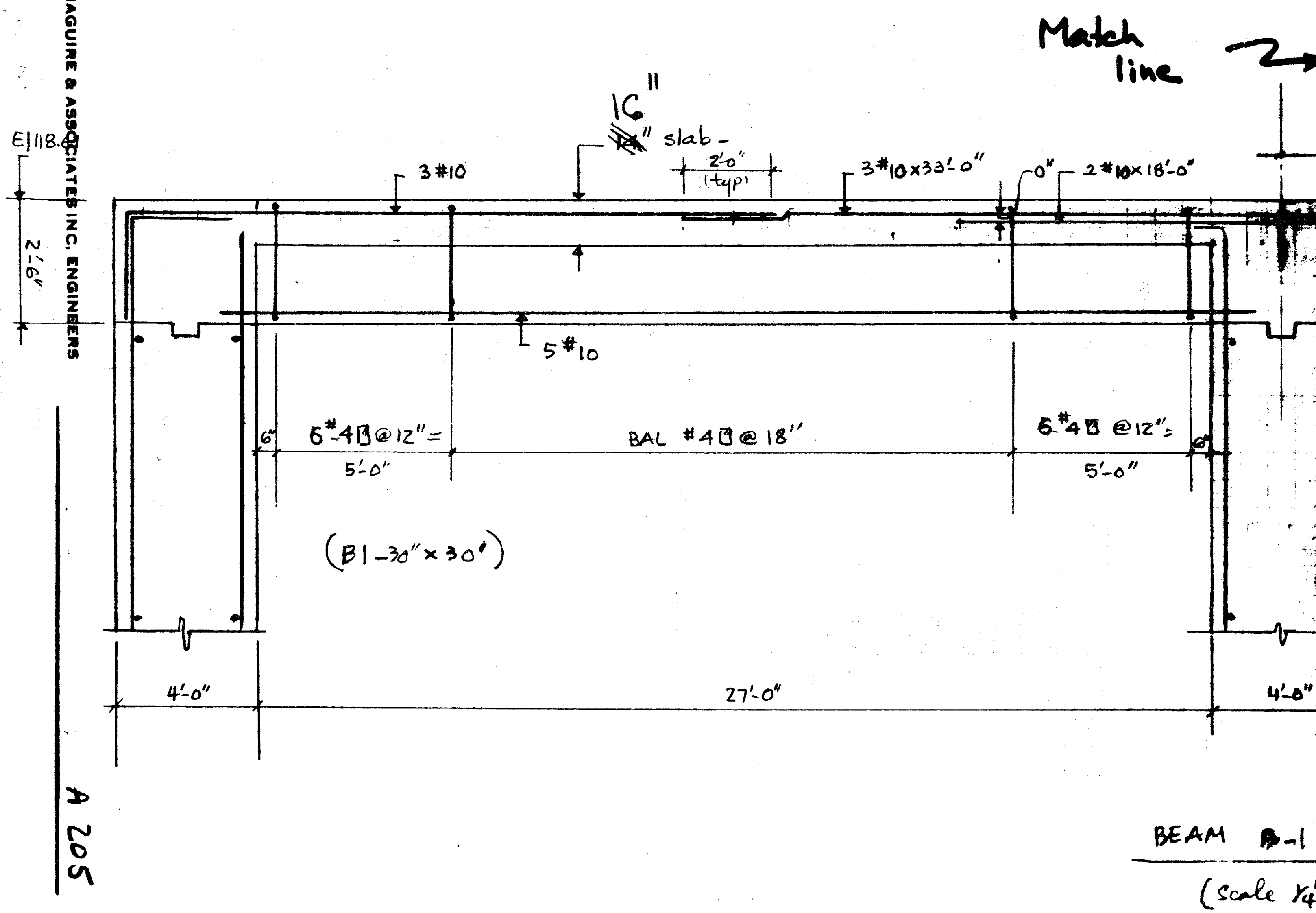
PROJECT ____ ACC. NO. ____

SUBJECT ____ SHEET NO. ____ OF ____

DATE ____ 19 ____

COMP. ____ CHECK ____ CONT. NO. ____

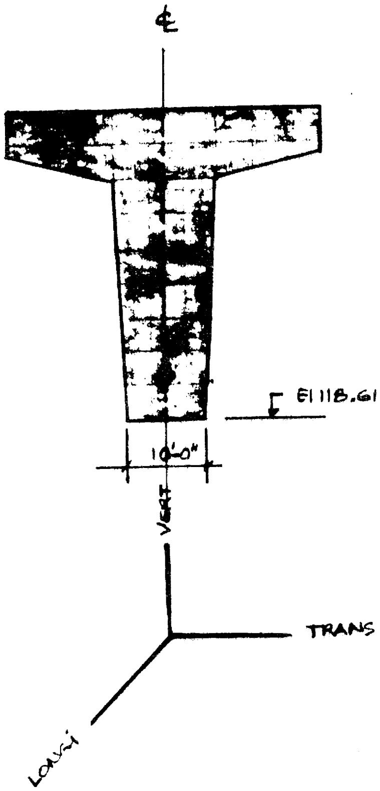
Part 2 of Sheet 11



A 205

PROJECT	CHARLES RIVER BASIN	ACC. NO.	1856.30
SUBJECT	P.S. FOREBAY WALLS	SHEET NO.	29 OF
		DATE	8.24 1972
COMP.	T.	CHECK GLC	CONT. NO.

WALLS SUPPORTING BRIDGE PIER



Assumed the width of pier at El 118.61 is 10'-0"

Data from GLC:

$P = 995$ Kips D+L+I

✓ $M_{LONGI} = 2447$ K-FT EQ

$M_{TRANS} = 2447$ K-FT EQ

✓ $V_{LONGI} = 40.25$ Kips W

$V_{TRANS} = 36.85$ Kips W

$V_{EQUAKE} = 81.6$ Kips

* We interest only in longitudinal forces since the wall run in other direction and strong enough to take care of the load

contributed moment per foot width of pier:

$$M = \frac{2447}{10} = 244.7 \text{ K-Ft}$$

contributed Axial load per foot width of pier

$$P = \frac{995}{10} = 99.5 \text{ Kips}$$

contributed shear per foot of pier

$$V = \frac{40.25}{10} = 4.25$$

Shear due to earth quake from bridge

$$V_{EQ} = \frac{81.6}{10} = 8.16 \text{ Kip}$$

$$V_{total} = 4.25 + 8.16 = 12.41 \text{ Kips.}$$

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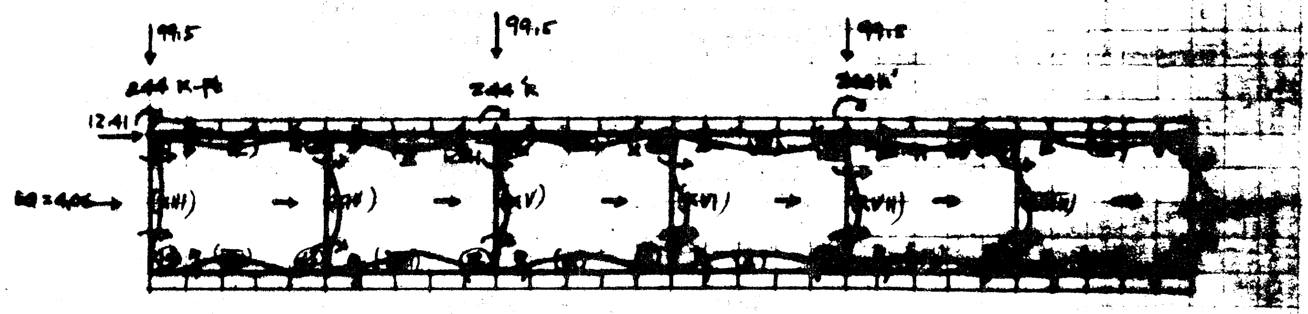
PROJECT CHARLES RIVER BASIN ACC. NO. [illegible]
SUBJECT P.S. – FOREBAY WALLS SHEET NO. [illegible]
DATE [illegible] 197[illegible]
COMP. [illegible] CHECK [illegible] CONT. NO.

CASE ONE : STUDY THE EARTH QUAKE CONDITION –



* for Fixed End Mo

Base slab 4'-6" = 54"

Walls thickness 4' = 48"

Assumed 1'-6" roof slab –

$I_R = \frac{bh^3}{12} = \frac{12 \times 18^3}{12} = 5830 \ in^4$

$I_W = \frac{12 \times (48)^3}{12} = 110,592 \ in^4$

$I_{base} = \frac{12 \times 54^3}{12} = 158,364 \ in^4$

Wall's height = 33.5'

Roof & Base L = 27+4 = 31'-0"

$K_R = \frac{4EI}{L} = \frac{5830}{31} = 188$

$K_W = \frac{4EI}{L} = \frac{110,592}{33.5} = 3300$

$K_b = \frac{4EI}{L} = \frac{158,364}{31} = 5120$

DISTRIBUTION FACTORS.–

JOINT 1 .– $DF_{12} = \frac{188}{3300+188} = \frac{188}{3488} = .06$

$DF_{1-14} = \frac{3300}{3300+188} = \frac{3300}{3488} = .94$

JOINT 2 —

$DF_{2-1} = \frac{188}{188+188+3300} = \frac{188}{3676} = .05$

$DF_{2-3} = \quad " \quad = .05$

$DF_{2-13} = \frac{3300}{188+188+3300} = \frac{3300}{3676} = .90$

JOINT 14 —

$DF_{14-1} = \frac{3300}{3300+5120} = \frac{3300}{8420} = .39$ say .40

$DF_{14-13} = \frac{5120}{3300+5120} = .61$ say .60

JOINT 13 —

$DF_{13-14} = \frac{5120}{5120+5120+3300} = \frac{5120}{13,540} = .38$ say .4

$DF_{13-12} = \frac{5120}{13,540} = .38$ say .4

$DF_{13-2} = \frac{3300}{13,540} = .24$ say .2

LOADING & FIXED END MOMENTS —

Roof LL = 150 #/1
DL = 225
375 #/1 say .4 K/1

BASE — Neglect own dead weight — consider only earth pressure upward, due to weight of structure

wall's height = 33.5 ft

Dead weight per wall = 33.5 × 4 × .15 = 20.1 Kips/1

wall pressure on base $\frac{20.1}{31} = .65$ K/1

Pier's pressure on base $\frac{99.5}{62} = 1.61$ K/1

2.26 K/1 + .4 (From Roof) = 2.66 K/1

Water is full in channels, there is no differentiate in water level between channels –

Assumed that all walls will be subjected to earthquake load –

FIXED – END MOMENTS. –

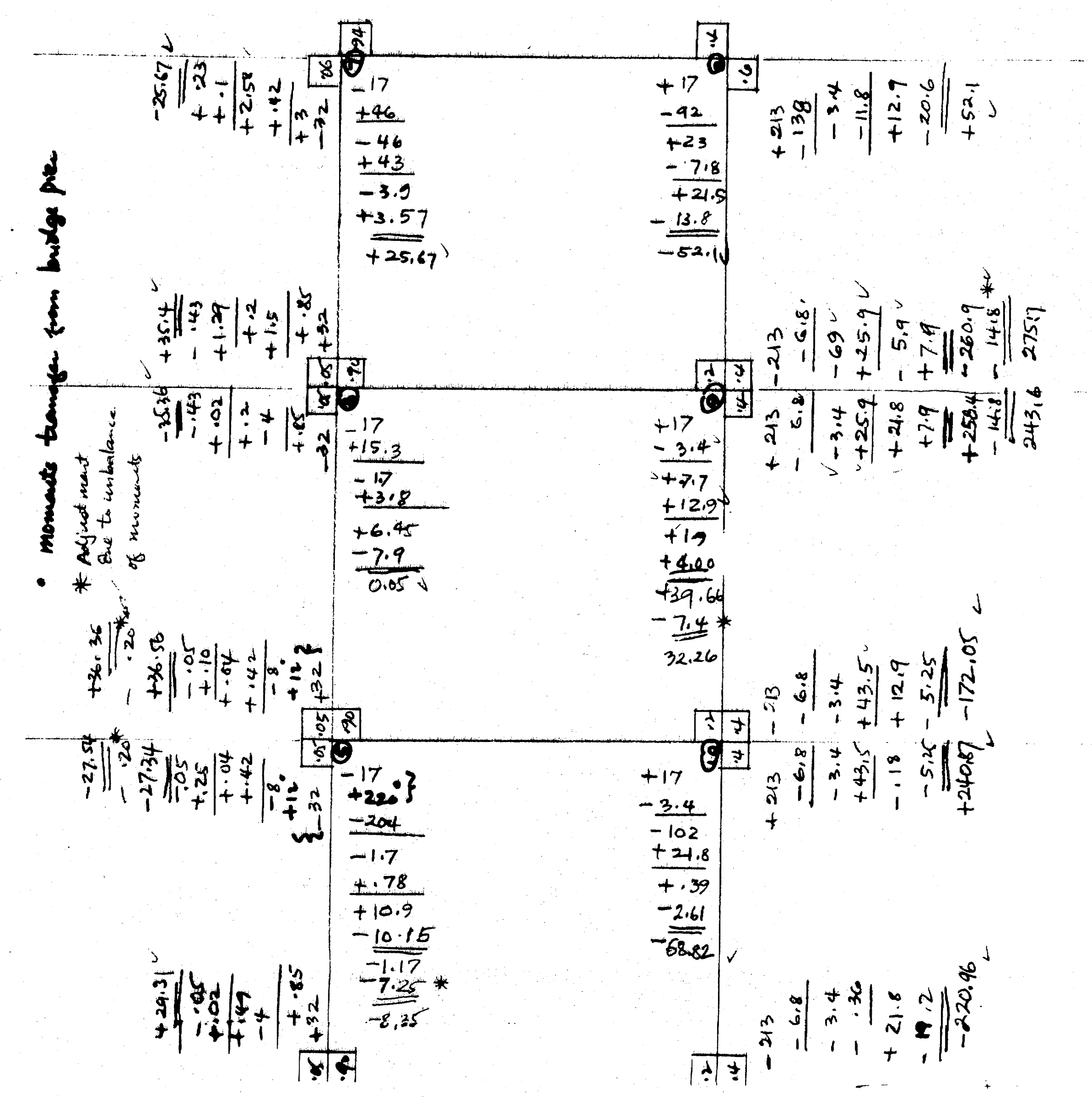
$$FEM_R = \frac{.4(31)^2}{12} = \pm 32 \text{ k-ft}$$

$$FEM_B = \frac{2.66(31)^2}{12} = \pm 213 \text{ k-ft}$$

$$FEM_W = \frac{4.06(33.5)}{8} = \pm 17 \text{ k-ft}$$

Assumed that the shear force of 12.41 k transfer from the viaduct's pier to the wall will be carried by slab El 118.61 to the end bay where it will resist by adjacent structures.

PROJECT CHARLES RIVER BASIN — ACC. NO. 1856.30
SUBJECT PUMP STATION - FOREBAY WALL — SHEET NO. 33 OF
DATE 8.25 1972
COMP. TL — CHECK GLC — CONT. NO.



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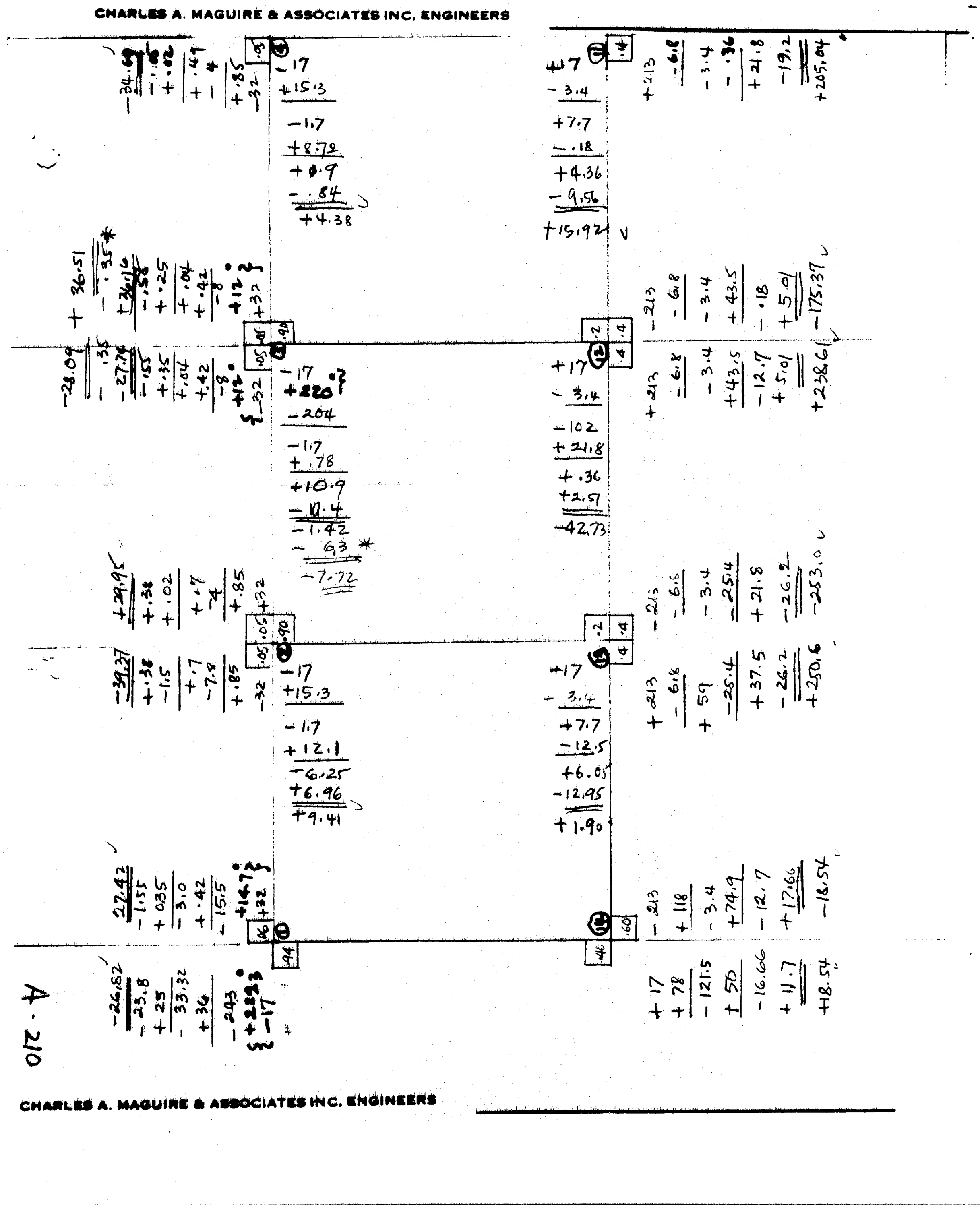

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PROJECT CHARLES RIVER BASIN | ACC. NO. 1858.30
SUBJECT PUMP STATION - FOREBAY WALLS | SHEET NO. 34 OF
DATE 8.28 1972
COMP. TL | CHECK GLC | CONT. NO.

Max M^- on roof slab:

$M = -39.37$ k-ft @ joint ②

Max M^- @ Base :

$M^- = -275.7$ k-ft @ joint ⑨

Since this is for seismic condition so we can reduce moment by 1/4 (i.e increase stress by 1/3)

M^- (Roof slab) = $\frac{3}{4}(39.37)$ = 29.6 k-ft

M^- (Base) = $\frac{3}{4}(275)$ = 206 k-ft

M^- (wall) = $\frac{3}{4}(68.82)$ = 51.6 k-ft

Max positive moments:

$M^+(\text{Roof}) = \left(1.5 \times 32 - \frac{29.3 + 27.54}{2}\right)\frac{3}{4} = (48 - 28.42)\frac{3}{4} = 19.58\left(\frac{3}{4}\right)$ 19.58

$M^+ \text{BASE} = \left(1.5 \times 213 - \frac{18.54 + 250.6}{2}\right)\frac{3}{4} = \frac{3}{4}(185.38) = 139$

use 185

d required by moment:

$d_{roof} = \sqrt{\frac{29.6}{.152}}$ $\sqrt{195}$ = 14" + 2 = 16"

use 18" slab

$d_{base} = \sqrt{\frac{206}{.152}} = \sqrt{1354} = 36.8 + 6 = 42.8''$

use 54" slab

A 211

PROJECT
SUBJECT Dimensioning – Subway works   SHEET NO. __ OF __
DATE __ 1972
COMP. __   CHECK GLC   CONT. NO. __

Re-inforcements –

4'-0" Base : $A_s = \frac{183.2}{1.44 \times 48} = 2.66 \text{ in}^2$ — #11 @ 6" Bottom

$A_s = \frac{185.38}{1.44 \times 49} = 2.63 \text{ in}^2$ — #11 @ 6" top

temp $A_s = 12 \times 54 \times .002 = \frac{1.3}{2} \text{ in}^2 = .65$ — #8 @ 12" T & B

1'-6" Roof

$A_s = \frac{29.6}{1.44 \times 19} = 1.10 \text{ in}^2$ — #7 @ 6" top

$A_s = \frac{19.52}{1.44 \times 16} = .85 \text{ in}^2$ — #7 @ 6" Bottom

exposed concrete ↑ 25%

temp steel $A_s = 18 \times 12 \times .002 = \frac{.432}{2} = .216 \times 1.25 = .27 \text{ in}^2$

#5 @ 12"

4'-0" wall –

$A_s = \frac{45.9}{1.44 \times 43} = .74 \text{ in}^2$

Min steel required by ACI for wall re-inforcement

vertical $A_s = .0015 \times 12 \times 48 = .87 \text{ in}^2$ — #9 @ 12" V

Horizontal $A_s = .0040 \times 12 \times 48 = 2.3 \text{ in}^2$

each face $\frac{2.3}{2} = 1.15$ — #9 @ 12" (H)

(also see case Two for re-inforcements)

PROJECT CHARLES RIVER BASIN
SUBJECT PUMP STATION - FOREBAY WALLS
COMP. IL CHECK GLC
SHEET NO. 36 OF
DATE 9.81 1973

CASE TWO -

Since the slab at El 118.61 is relatively thin as comparing to the walls, so let us assumed that walls are hinged at top (i.e no moment from wall will be transfered to slab).

Base slab 4'-6" = 54" L = 31'-0

Walls thickness 4'-0" = 48" H = 33.5'

$I_{wall} = 110,592 \ IN^4$

$I_{base} = 158,364 \ IN^4$

$$K_W = \frac{3EI}{L} \approx \frac{3I}{L} = \frac{3 \times 110,592}{33.5} = 9920$$

$$K_B = \frac{4EI}{L} \approx \frac{4I}{L} = \frac{4 \times 158,364}{31} = 20,420$$

Exterior joints -

$$DF_W = \frac{9920}{9920 + 20,420} = \frac{9920}{30,340} = .33$$

$$DF_B = \frac{20,420}{30,340} = .67$$

Interior joints

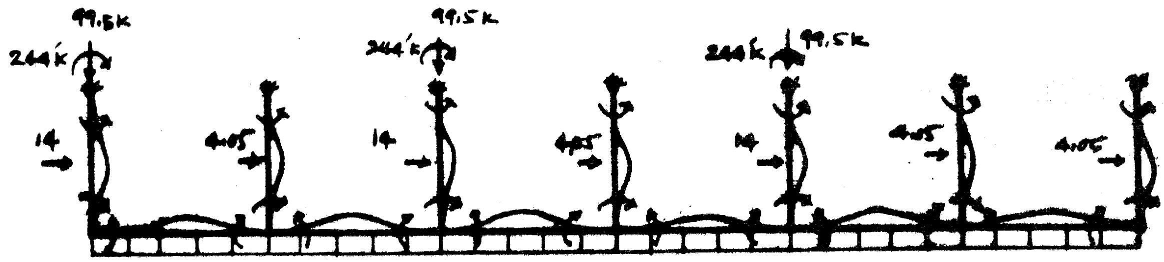
$$DF_W = \frac{9920}{9920 + (2 \times 20,420)} = \frac{9920}{50,760} = .196 \text{ say } .2$$

$$DF_{B_L} = \frac{20,420}{50,760} = .4$$

$$DF_{BR} = \frac{20,420}{50,760} = .4$$

A 213

PROJECT CHARLES RIVER BASIN ACC. NO. 1856.30
SUBJECT PUMP STATION - FOREBAY WALLS SHEET NO. 37 OF
DATE 9.21 1972
COMP. TA CHECK GLC CONT. NO.



sign convention 

LOADING -

From slab @ EL 118.61 LL = 150 #/1
DL = 225 #/1
375 #/1 say .4 K/1

WALL W = 38.5 X 4 X 1 X .15 = 20.1K/1 strip

Wall pressure on base = $\frac{20.1}{31}$ = .65 K/1

LOAD From pier on base = $\frac{99.5}{62}$ = 1.61 K/1

2.26 K/1 + .4 = 2.66 K/1

- There is no differentiate in water Elev. between channels -
- No sidesway permitted. transverse load will be transmitted to the lock or fish ladder.
- Assumed an earth quake force of 4.06 K acting at mid height of walls without bridge's pier
- An earth quake force of 4.06K + 9.95 = 14.01K acting at mid height of walls with bridge pier (very conservative Assumption - since 244'K is combined moments of L+D+E already

A 214

transfer to wall to be distributed to all members

FIXED END MOMENTS

BASE: $FEM = \frac{2.66(31)^2}{12} = \pm 213$ k-ft

WALL: $FEM = \frac{4.05 \times 33.5}{8} = \pm 17$ k-ft

$= \frac{14}{4.06} \times 17 = \pm 59$ k-ft

Max $M^+_{wall} = 2 \times 17 - \frac{43.76}{2}$

$= 34 - 21.88 = 12.12$ k-ft

Max $M^+_{base} = 1.5 \times 213 - \frac{249.17 + 81.08}{2} = 154.87$ k-ft (2 bay from left)

walls: Max M^- = 287.89 k-ft

Base Max M^- = 339.69 Since code permit an increase of working stress by 1/3 or reduce M by 1/4

So

M^-_{max} (wall) = 3/4 (287.89) = 216 k-ft

M^-_{max} (base) = 3/4 (339.69) = 254.5 k-ft

M^+_{max} (base) = 3/4 (154.87) = 116.2 k-ft

Re-inforcement

wall $A_s = \frac{216}{1.44 \times 43} = 3.48$ in² #11 @ 5"

Base $A_s = \frac{254.5}{1.44 \times 48} = 3.69$ in² #11 @ 5"

PROJECT CHARLES RIVER BASIN — ACC. NO. 1858:30
SUBJECT PUMPING STATION - FOREBAY WALLS — SHEET NO. 50 OF
DATE 8.01 1972
COMP. — CHECK GLC — CONT. NO.

For design of Re-inforcement, let take the worst case of the two

Wall	# 11 [illegible]" EF - V.	
Base	# 11@ 5" B at support	
	# 11 @ 10" other places -	

temperature reinforcement # 9 @ 12" all -

PROJECT CHARLES RIVER BASIN | ACC. NO. 1856.38
SUBJECT PUMP-STATION - FOREBAY WALLS | SHEET NO. 39 OF
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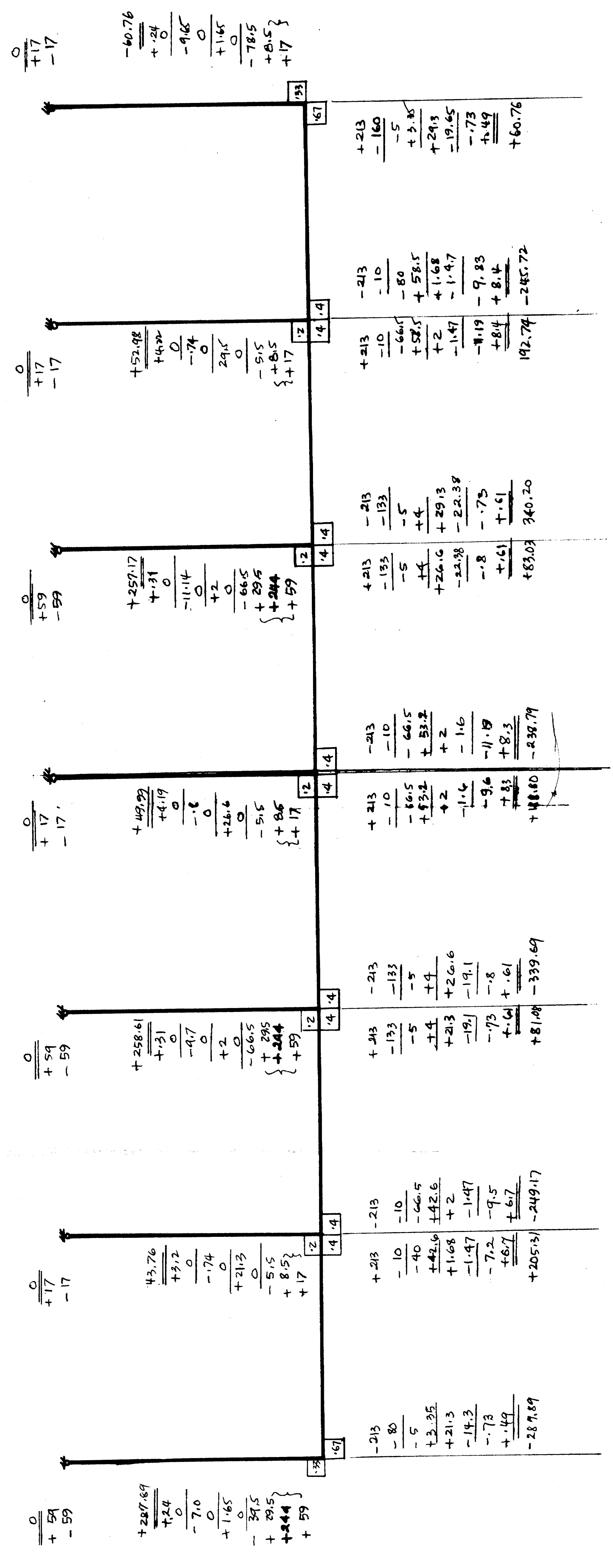
COMP. | CHECK GLC | CONT. NO.



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PROJECT CHARLES RIVER BASIN	ACC. NO. 1058.30
SUBJECT P.S - FOREBAY WALLS	SHEET NO. 42 OF
	DATE 8.17 1972
COMP. [illegible] CHECK GLC	CONT. NO.

FOREBAY WALL (PUMPING STATION SIDE)

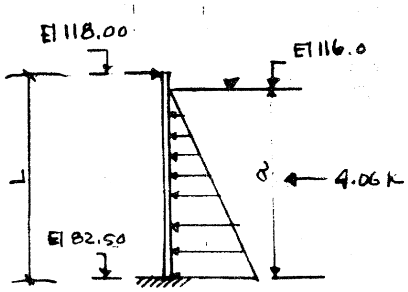
Max. water Elevation (tide side) El. 116.00

bottom wall El 82.50 (top wall El. 11[illegible]

$H_w = 33.50$ ft

For interior walls max water differentiate is 33.50 ft with seismic loading of 4.06 K applied at mid height

Design Assumptions: wall is hinged at top and fixed at bottom



$$M = \frac{wa^2}{120L^2}(40L^2 - 45aL + 12a^2)$$

$w = 33.5 \times .064 = 2.14$ K/1

$$M_w = \frac{2.14 \times \overline{33.5}^2}{120 \times \overline{35.5}^2}(40 \times \overline{35.5}^2 - 45 \times 33.5 \times 35.5 + 12 \times \overline{33.5}^2)$$

$= .0159(50400 - 53600 + 13,500) = .0159 \times 10,300 = 164$ K.ft

$M_{seismic} = 4.06 \times 35.50 \times \frac{3}{16} = 26.7$ say 27

$M_{total} = 191$ K-ft

$$d = \sqrt{\frac{191}{.152}} = 35.4''$$

$t = 35.4 + 4 = 39.4'' < 48''$ OK ✓

USE 4'-0" wall - ($d = 44''$)

A217

Since seismic load was included in the design, the code suggested an increase in working stress by 33%.

OR using: $M = \frac{3}{4}(191) = 143.5$ K-ft < 164 K-ft

∴ use $M = 164$ K-ft

$$A_s = \frac{164}{1.44 \times 43} = 2.65 \text{ in}^2$$

Min Horizontal re-inforcement required by A.C.I

$$A_s = .0025 \times 48 \times 12 = 1.44 \text{ in}^2$$

$$\text{Each Face} = \frac{1.44}{2} = .72 \text{ in}^2$$

Note - Since the system will behave as a unit, so let us further analyze the wall and base slab as a frame - rigid at base and hinged at top of wall -

PROJECT: CHARLES RIVER BASIN — JOB NO. [illegible]
SUBJECT: P.S. FOREBAY WALLS — SHEET NO. [illegible]
DATE: 9/21 1977
COMP.: [illegible] CHECK: [illegible] — CONT. NO.

General Assumptions:

- 4'-0" walls — Top of wall El 116.0
- 4'-6" Base — TAKE 12" STRIP

$$I_{WALL} = \frac{bh^3}{12} = \frac{12 \times 48^3}{12} = 119592 \text{ in}^4$$

$$I_{base} = \frac{12 \times 54^3}{12} = 158,364 \text{ in}^4$$

- L (wall) = 33.5 ft

L (base) = 27 + 4 = 31'-0" to ℄ of supports

$$K_{(wall)} = \frac{3I}{L} = \frac{3 \times 110,592}{12 \times 33.5} = 865$$

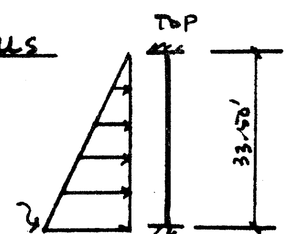
$$I_{base} = \frac{4I}{L} = \frac{4 \times 158,364}{12 \times 31} = \frac{1700}{2565}$$

$$DF_{wall} = \frac{865}{2565} = .34$$
$$DF_{base} = \frac{1700}{2565} = .67$$
} Exterior walls

$$DF_{wall} = \frac{865}{2565 + 1700} = \frac{865}{4265} = .20$$
$$DF_{base} = \frac{1700}{4265} = .40$$
} interior walls

LOADING ON STRUCTURE — Top H_2O El 116.00

WALLS — bottom wall El 82.50 ⟶ H_w = 33.50 ft



w = 64 × 33.5
= 2.14 K/ft

$$FEM_{TOP} = \frac{WL^2}{30} = \frac{2.14(33.5)^2}{30} = 80 \text{ k-ft}$$

$$FEM_{BOTT} = \frac{WL^2}{20} = \frac{2.14(33.5)^2}{20} = 120 \text{ k-ft}$$

SEISMIC LOADING .- ZONE 3.

- apply a horizontal force of 0.1 g at center of gravity -
- allowable working stress to be increased by 1/3.
- Assumed using 18" slab at El 119.61 (w = .225 K/□)

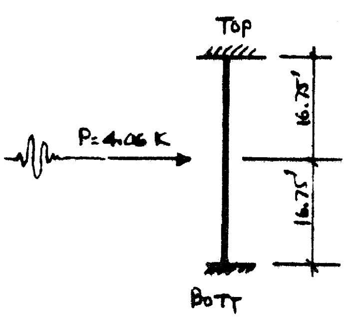
Weight of slab on wall $\frac{27}{2} \times .225 = 3.04$ K/1

max truck load = 16.00 K/1

own weight = .15 x 4 x 36 = 21.6 K/1

$P_{(t)} = 40.64$ K/1

Seismic load = 10% P = .1 x 40.64 = 4.06 K/1



$FEM_{TOP} = \frac{PL}{8} = \frac{4.06 \times 33.5}{8} = 17$ Kip-FT

$FEM_{BOTT} = \frac{PL}{8} = \frac{4.06 \times 33.5}{8} = 17$ K-FT

FOR BASE SLAB -

Max loading on slab occurs when the channel is empty.
Full hydro static load, neglecting the base's weight.

h = 33.5 + 4.5 = 38' p = 64 x 38 = 2.44 K/□ ↑

Min loading when channel is full - dead weight of structure will be considered only.

$p = \frac{40.64 \text{ Kip}}{31} = 1.3$ K/1

Consider the moment to center of support - max span 27'+4 = 31'-0"

Max loading $FEM = \frac{2.44(31)^2}{12} = 196$ k-ft

Min loading $FEM = \frac{1.13(31)^2}{12} = 104$ k-ft

Walls are Hinged at top, so that no moment will be transfered to the floor slab.

Note The seimic loads contributed to the structure are small, compare to the actual loading due to water pressure. Since the code permit an increase 33% of working stress for concrete (i.e reducing the final moment 25%), the reduced final moment will be less than of actual one. Therefore it is safe to neglect seimic load on structure.

Refer to the analysis of case V (Pg) for the study of one case where earth quake forces were included in the analysis.

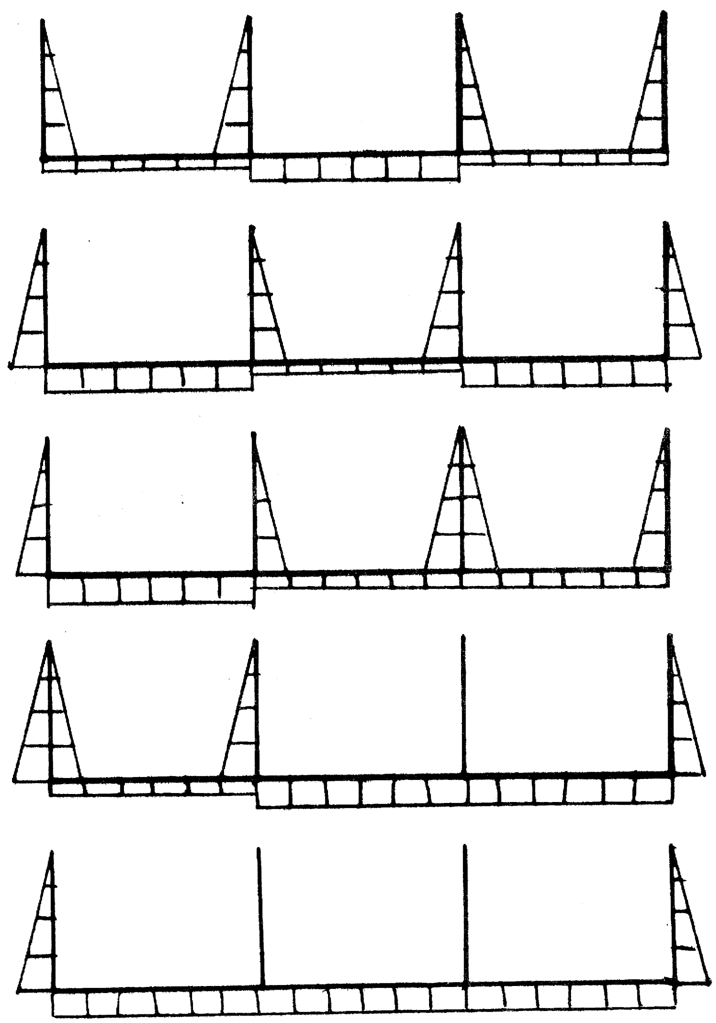
$f = \frac{M}{S}$ where S : constance

increase f by 1/3 or multiply f by $\frac{4}{3}$

$f \times \frac{4}{3} = \frac{3}{4} M \times \frac{1}{S}$

* Since all channels are of the same dimensions and loading condition on all are simmilar, therefore it is safe to study any group of three channels —

The following loading conditions will be studied.



CASE I

CASE II

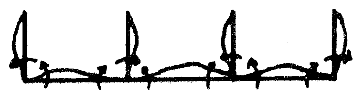
CASE III

CASE IV

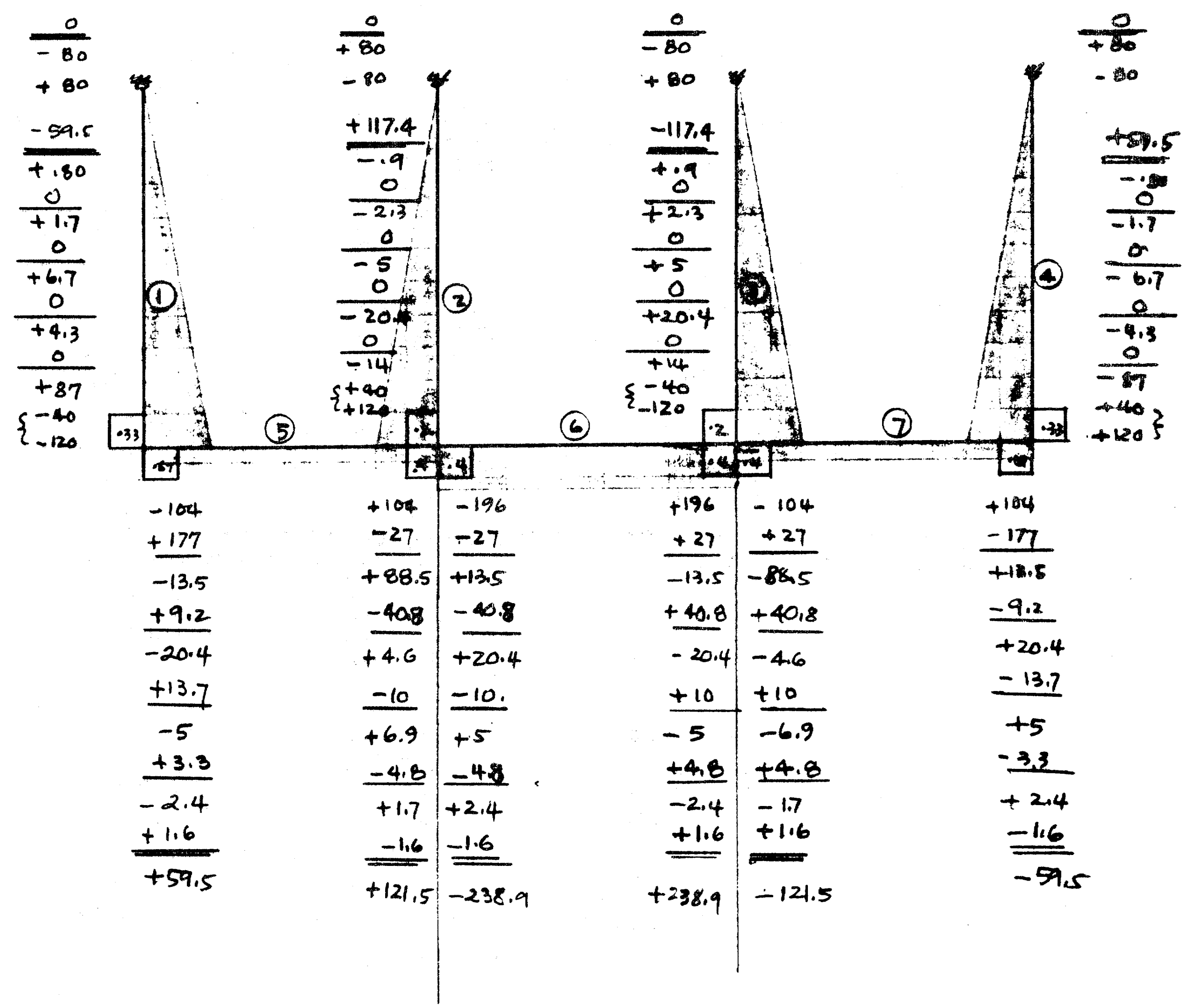
CASE VI

PROJECT	CHARLES RIVER BASIN	ACC. NO.	1856-3?
SUBJECT	P.S. - FORE BAY WALLS	SHEET NO.	48 OF
		DATE	8-24 1972
COMP.	T.L. CHECK G.L.C.	CONT. NO.	

CASE ONE : one channel is dewatering



* Sign convention : Clockwise is positive (+)

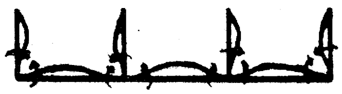


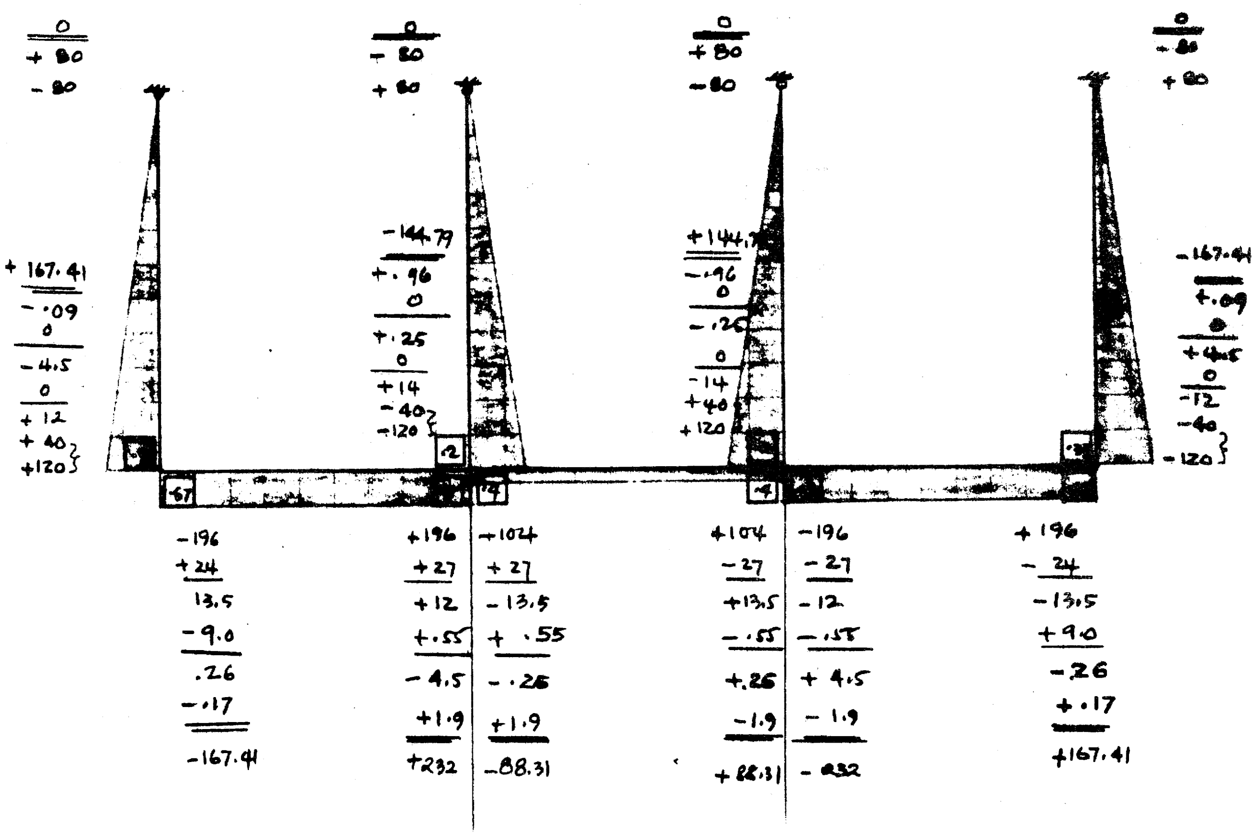
PROJECT CHARLES RIVER BASIN ACC. NO. 1953.30
SUBJECT P.S.- FOREBAY WALLS SHEET NO. 44 OF
DATE 8.22 1972
COMP. T.L. CHECK GLC CONT. NO.

CASE TWO

Sign convention :

clockwise → positive ⟳





$$d = \sqrt{\frac{232{,}000}{152}} = \sqrt{1530} = 39.5 + 6 = 45.5'' < 54''$$

OK ✓

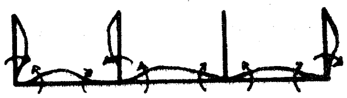
PROJECT CHARLES RIVER DAM BASIN — ACC. NO. 1858.30

SUBJECT P.S - FOREBAY WALLS — SHEET NO. 52 OF

DATE 8.22 1972

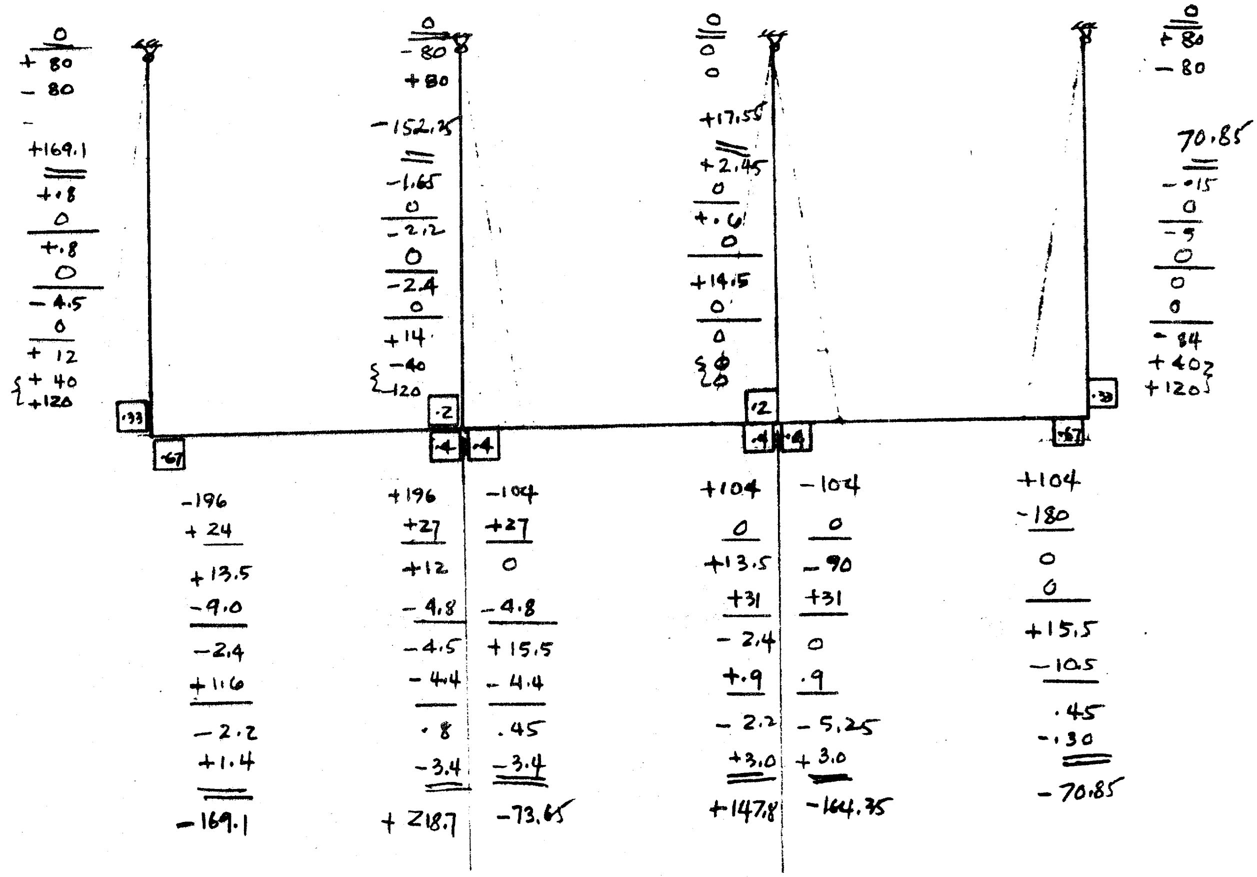
COMP. TL — CHECK GLC — CONT. NO.

CASE THREE



Sign convention

positive clockwise (+)



PROJECT: CHARLES RIVER BASIN — ACC. NO. 1856.30

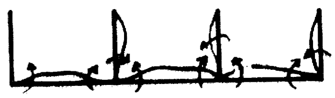
SUBJECT: P.S. FOREBAY WALLS — SHEET NO. ___ OF ___

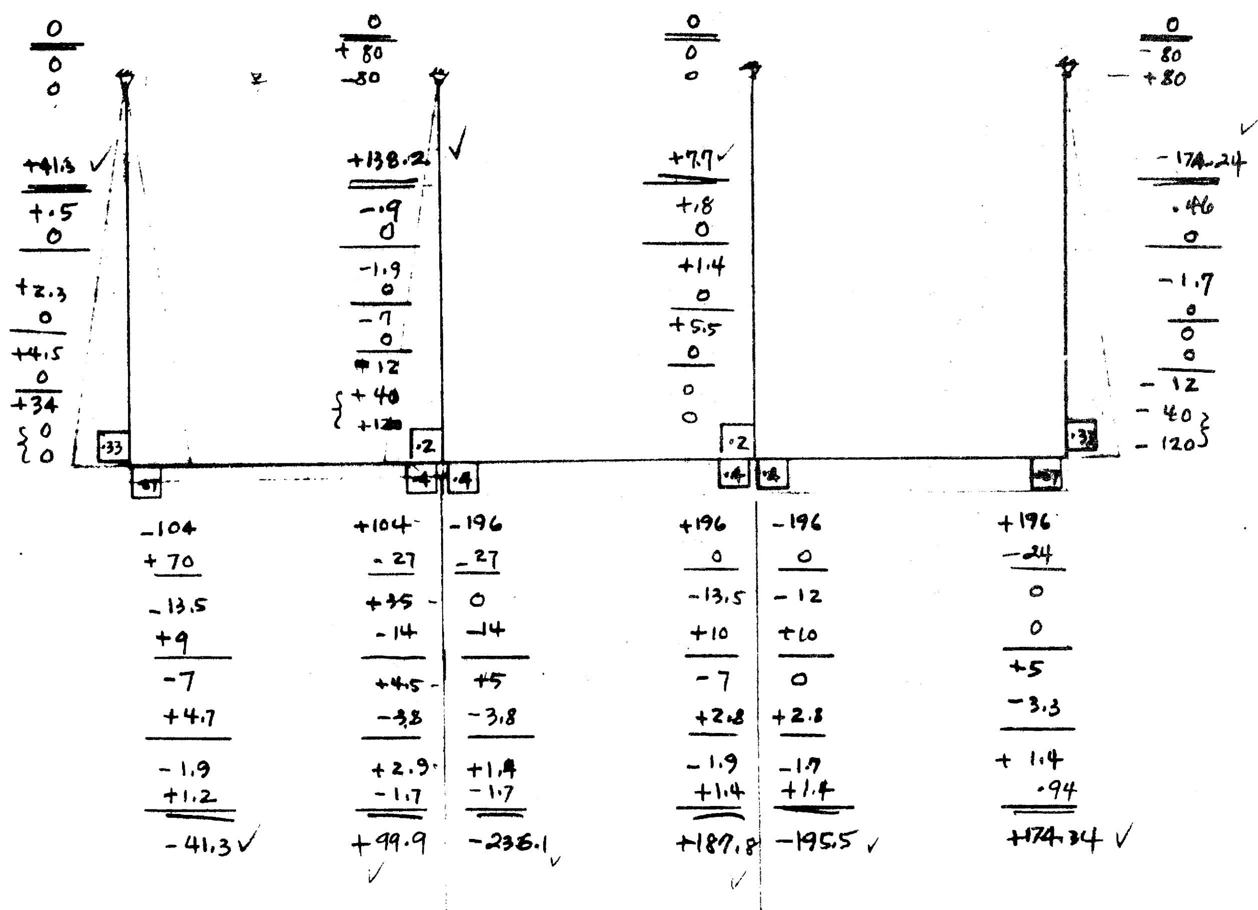
DATE: 8-22-1972

COMP.: TL — CHECK: GLC — CONT. NO. ___

CASE FOUR –

positive : clockwise direction





PROJECT CHARLES RIVER BASIN
SUBJECT P.S.- FOREBAY WALLS
COMP. T.Z. CHECK GLC
ACC. NO. 1956-30
SHEET NO. 52 OF
DATE 5-23 1972
CONT. NO.

STUDY SEIMIC CONDITION :—

By comparison the four cases analyzed above, case IV give us the max possible bending in wall and in base slab. Let take this condition and apply seimic load -

contributing moment into walls' moment by seimic force

$$FEM_{Top} = FEM_{BOTT} = 17 \text{ k-ft.} \qquad (Pg \quad)$$

The total moment in wall

$$FEM_{Top} = 80 + 17 = 97 \text{ k-ft}$$

$$FEM_{BOTT} = 120 + 17 = 137 \text{ k-ft}$$

THe code permit an increase in working stress by 33% — or reduce the final moment by 1/3 (1/4) —

Reducing factor

The max Moment in wall = 192.15 k-ft × 3/4 = 144.2 k-ft < 174

Max Moment in base = 229.4 k-ft × 3/4 = 172.1 k-ft < 236

∴ The max moment in case IV govern the design

Note : Since the earth quake force are very small compare to the water pressure acting on structure, and due to increasing of working stress of concrete (i.e decrease moment) so in all case the actual loading govern the design

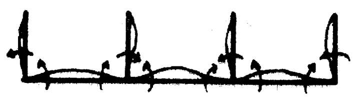
PROJECT CHARLES RIVER BASIN ACC. NO. 1856.30

SUBJECT P.S. - FOREBAY WALLS SHEET NO. 50 OF

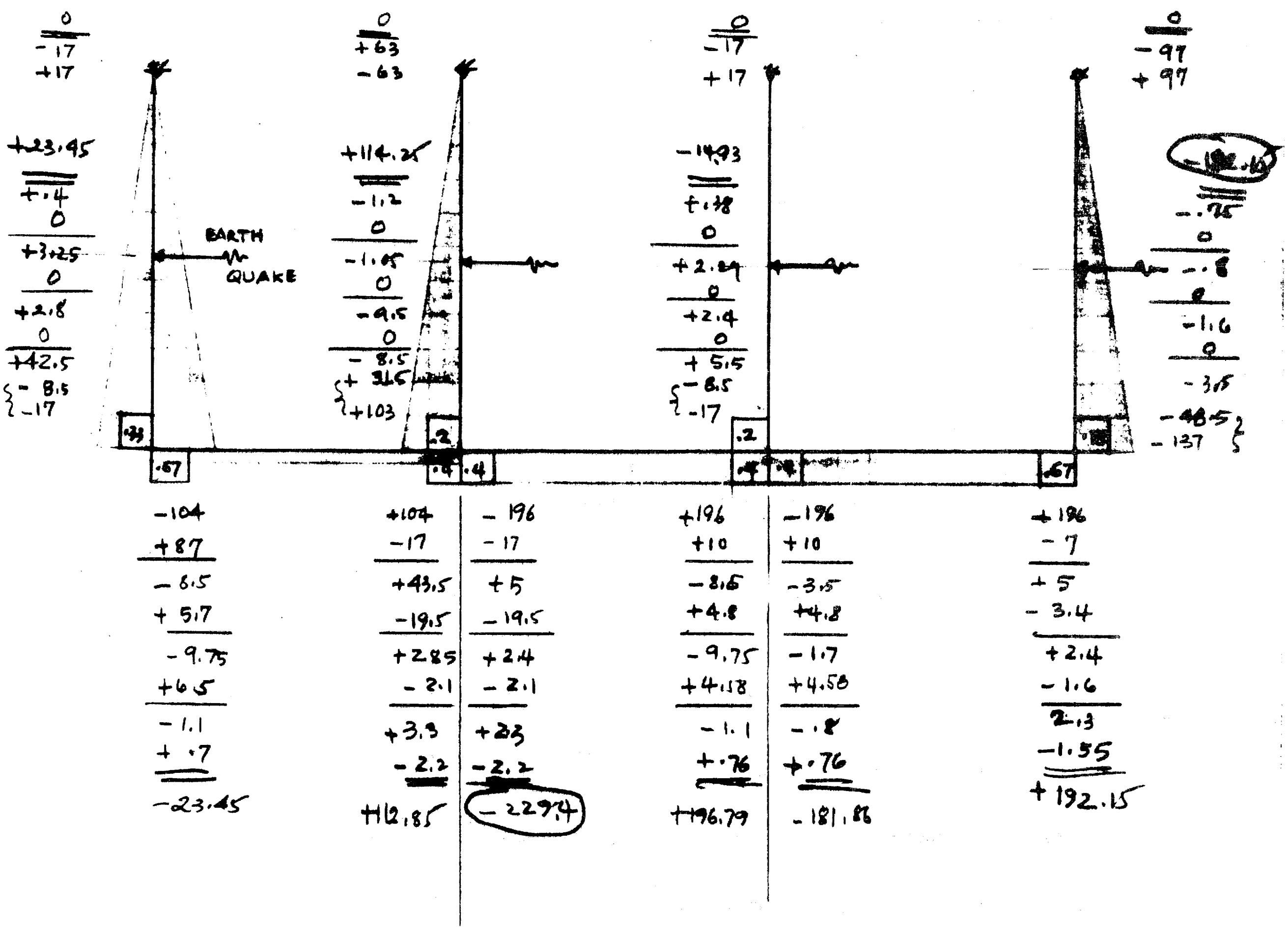
DATE 8.23 1972

COMP. T.L. CHECK GLC CONT. NO.

CASE FIVE - case IV + Earth Quake load -



* Sign convention - clockwise ↻ is positive

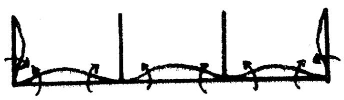


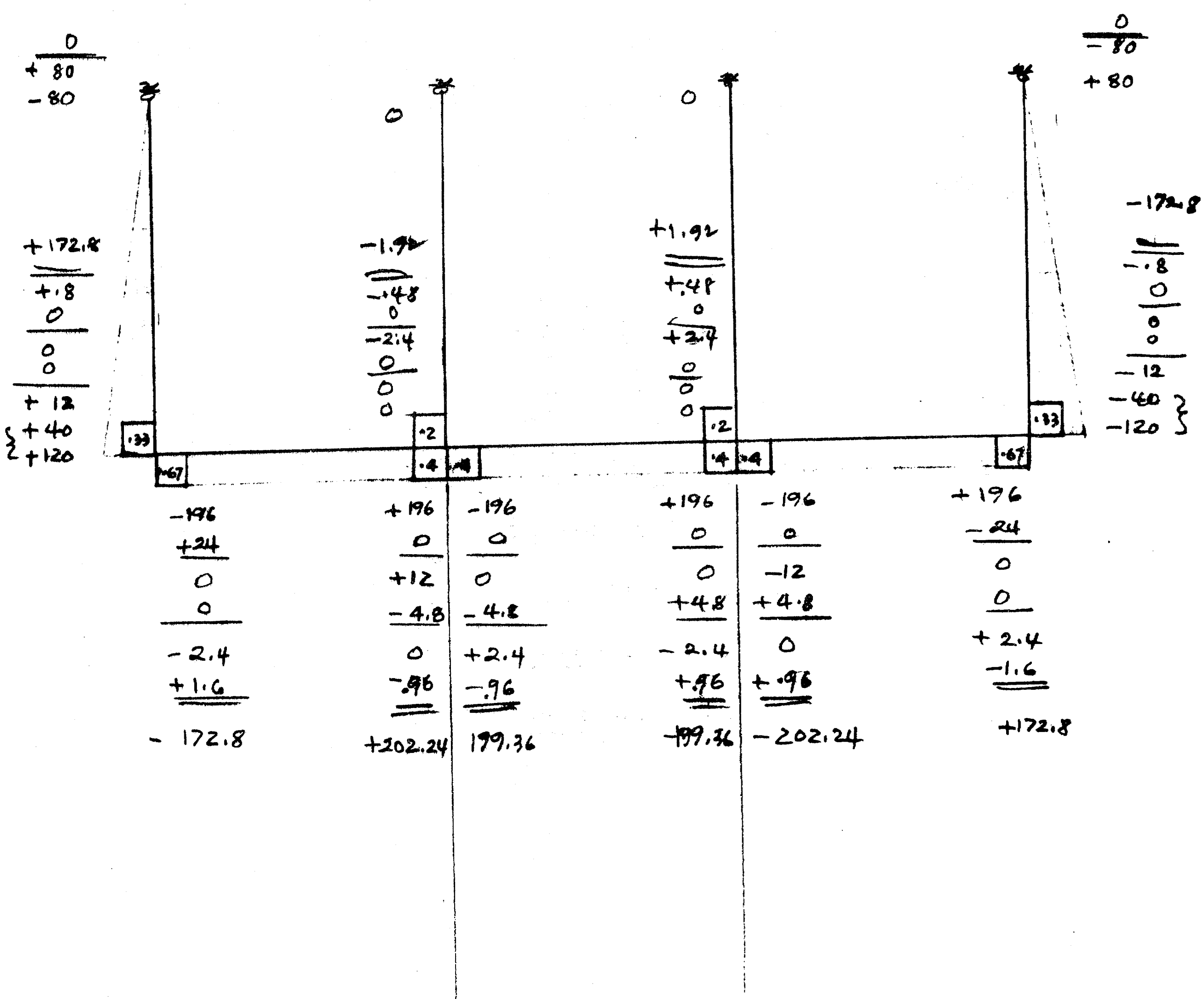
PROJECT: CHARLES RIVER BASIN | ACC. NO. 1853.30
SUBJECT: P.S. — FOREBAY WALLS | SHEET NO. 54 OF
DATE: 8.23 1972
COMP. T.L. | CHECK GLC | CONT. NO.

CASE SIX — All three channels are dewatered.

sign convention — clockwise is positive

NO EARTH QUAKE LOAD CONSIDERED !





PROJECT CHARLES RIVER BASIN — ACC. NO. 1856.30
SUBJECT P.S. – FOREBAY WALLS — SHEET NO. 55 OF
DATE 8.24 1972
COMP. JZ — CHECK GLG — CONT. NO.

POSITIVE MOMENTS :–

SIMPLE MOMENT FOR TRIANGULAR LOADING ON WALLS –

@ center of wall $M^+ = 0.25\,WL$

where $L = 33.5'$

$W = .064 \times 33.5 \times \frac{33.5}{2} = 36$ K

$M^+ = 0.25 \times 36 \times 33.5 = 301$ k-ft

SIMPLE MOMENT FOR BASE SLAB

$M_{min} = 1.5 \times 104 = 156$ k-ft

$M_{max} = 1.5 \times 196 = 294$ k-ft

CASE ONE :–

Members

① M = Not applicable

② $M = 301 - \frac{117.4}{2} = 242.3$

③ $M = 301 - \frac{117.4}{2} = 242.3$

④ M = Not applicable

⑤ $M = 156 - \frac{121.5 + 59.5}{2} = 65.5$

⑥ $M = 294 - \frac{238.9 + 238.9}{2} = 174.1$

⑦ $M = 156 - \frac{121.5 + 59.5}{2} = 65.5$

PROJECT CHARLES RIVER BASIN — ACC. NO. 1856-30
SUBJECT P.S. – FOREBAY WALLS — SHEET NO. 56 OF
DATE 8.24 19__
COMP. T.L. — CHECK GLC — CONT. NO.

CASE TWO :-

Members

① $M = 301 - \frac{167.41}{2} = 217.3$

② $M = 301 - \frac{144.79}{2} = 228.6$

③ $M = 301 - \frac{144.79}{2} = 228.6$

④ $M = 301 - \frac{167.41}{2} = 217.3$

⑤ $M = 294 - \frac{167.4 + 232}{2} = 94.3$

⑥ $M = 156 - \frac{88.3 + 88.3}{2} = 67.7$

⑦ $M = 294 - \frac{232 + 167.4}{2} = 94.3$

CASE THREE :-

① $M = 301 - \frac{169.1}{2} = 216.4$

② $M = 301 - \frac{152}{2} = 225$

③ M = Not applicable

④ M = Not applicable

⑤ $M = 294 - \frac{169.1 + 218.7}{2} = 100.1$

⑥ $M = 156 - \frac{73.45 + 147.8}{2} = 45.27$

⑦ $M = 156 - \frac{164.35 + 73.45}{2} = 38.4$

PROJECT CHARLES RIVER BASIN | ACC. NO. 1856.3B
SUBJECT P.S. FOREBAY WALLS | SHEET NO. 57 OF ___
DATE 8-24 1972
COMP. T.L. | CHECK GLC | CONT. NO. ___

CASE FOUR :–

Members

① M = Not applicable

② $M = 301 - \frac{138.2}{2} = 231.9$

③ M = Not applicable

④ $M = 301 - \frac{174.24}{2} = 213.88$

⑤ $M = 156 - \frac{41.3 + 99.9}{2} = 85.4$

⑥ $M = 294 - \frac{236 + 187.8}{2} = 82.1$

⑦ $M = 294 - \frac{195.5 + 174.34}{2} = 109.1$

CASE SIX :–

① $M = 301 - \frac{172.8}{2} = 214.6$

② M = Not applicable

③ M = Not applicable

④ $M = 301 - \frac{172.8}{2} = 214.6$

⑤ $M = 294 - \frac{172.8 + 202.2}{2} = 106.5$

⑥ $M = 294 - \frac{199.36 + 199.36}{2} = 94.64$

⑦ $M = 294 - \frac{172.8 + 202.2}{2} = 106.5$

MAX NEGATIVE MOMENTS .–

CASE	M_{max} IN WALL	M_{max} IN BASE
ONE	117.4 'k	238.9 'k
TWO	167.41 'k	232.0 'k
THREE	169.1 'k	218.7 'k
FOUR	138.2 'k	236 'k
FIVE	SEE FOUR	SEE FOUR
SIX	172.8 'k	202.24 'k

MAX POSITIVE MOMENTS .–

ONE	242.3 'k	174.1 'k
TWO	228.6 'k	94.3 'k
THREE	225 'k	100.1 'k
FOUR	231.9 'k	109.1 'k
FIVE	SEE FOUR	SEE FOUR
SIX	214.6 'k	106.5

compare all six cases analyzed above we get

WALL : M^{+}_{max} = 242 'k

M^{-}_{max} = 172.8 'k

BASE : M^{+}_{max} = 174.1 'k

M^{-}_{max} = 238.9 'k

PROJECT: CHARLES RIVER BASIN	ACC. NO. 1856.30
SUBJECT: P.S. FOREBAY WALLS	SHEET NO. 59 OF
	DATE 8.24 1972
COMP. T.L. CHECK GLC	CONT. NO.

The analysis of the forebay walls (in pump station side) was done with the assumption that max water El 116.00

The max water in basin side El 112.00 — so there is an over load of 4'-0" of water pressure on walls and base.

Try to find a reducing factor for the above results.

Fixed end moment $M = \frac{wL^2}{12}$: for uniform load

$$M = \frac{WL^2}{30}, \quad = \frac{WL^2}{20} \quad \text{For triangular loading}$$

we see that w or W varried in same degree, so we can take proportion of loads and moments.

$$\frac{\text{New depth}}{\text{old depth}} = \frac{34}{38} = .895 = \text{adjustment factor}$$

Assumed that this factor can also be used to reduce positive moment as well since $w \propto h$ —

∴ Wall $M^+ = .895 \times 242 = 216$ k-ft

$M^- = .895 \times 172.8 = 155$ k-ft

Base $M^+ = 174 \times .895 = 156$ k-ft

$M^- = 238.9 \times .895 = 213$ k-ft

depth required by moment

wall $d = \sqrt{\frac{216}{.152}} = 37.7 + 4 = 41.7'' < 48''$

Base $d = \sqrt{\frac{213}{.152}} = 37.4 + 4 = 41.8'' < 54''$

PROJECT: CHARLES RIVER BASIN   ACC. NO. [illegible]
SUBJECT: P/S – FOREBAY WALLS   SHEET NO. [illegible] OF
DATE: [illegible] 24 1973
COMP. TZ   CHECK GLC   CONT. NO.

WALLS

4'-0" wall $d = 48" - 4.0 = 44.0"$

$A_s^- = \frac{216}{1.44 \times 43} = 3.50 \text{ in}^2$ #11 @ 5"

max @ $\frac{L}{\sqrt{5}} = \frac{33}{\sqrt{5}} = 14.8'$ from base $A_s^+ = \frac{155}{1.44 \times 44} = 2.45 \text{ in}^2$ #10 @ 5"

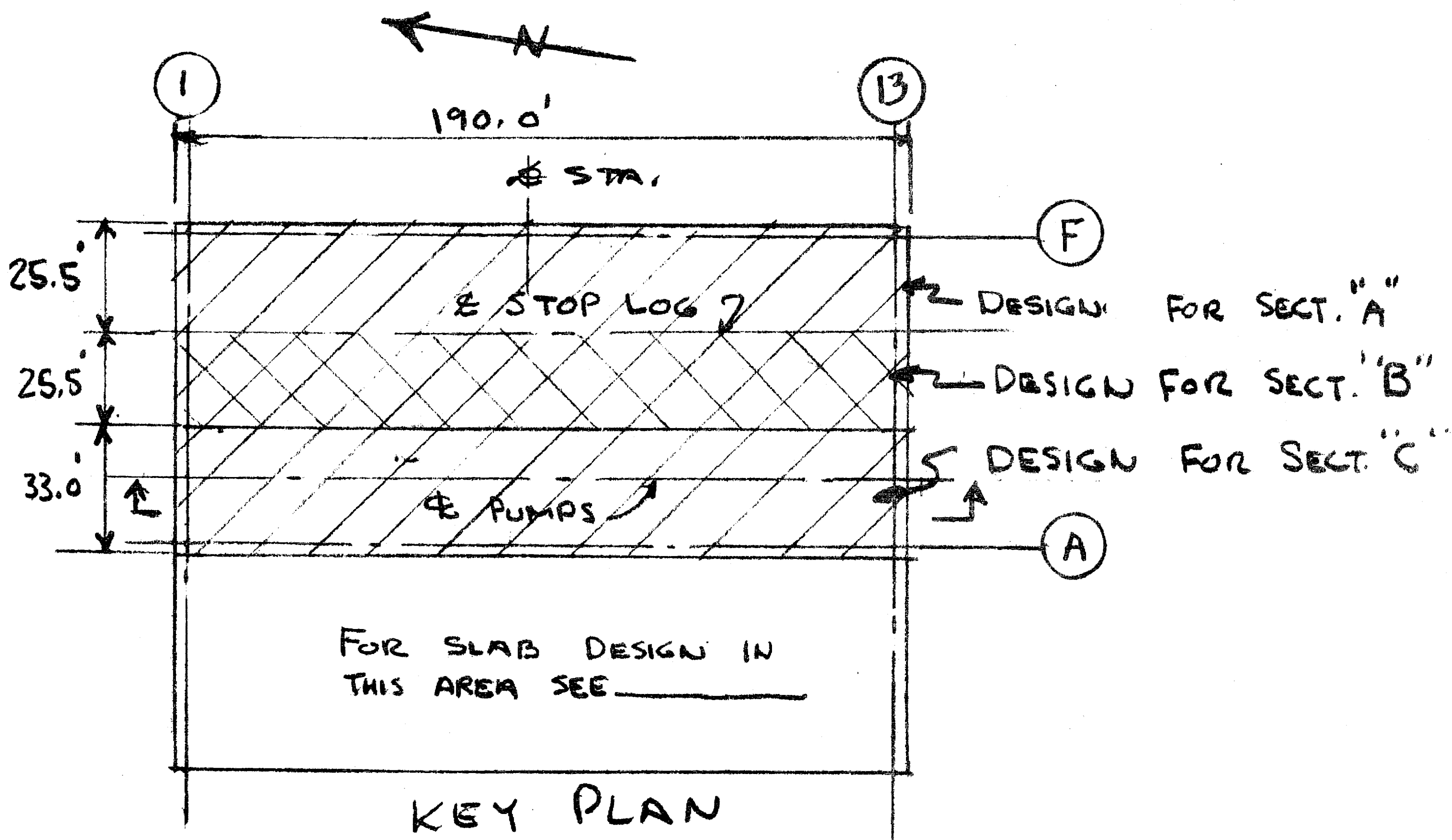
BASE

bottom steel: $A_s = \frac{213}{1.44 \times 48"} = 3.1 \text{ in}^2$ #11 @ 5"

top steel $A_s = \frac{156}{1.44 \times 50} = 2.16 \text{ in}^2$ #10 @ 5"

PROJECT CHARLES RIVER BASIN ACC. NO.
SUBJECT PUMPING STA. REDESIGN SHEET NO. 31 OF
FOUND. SLAB @ EL. 85.0 DATE 8-16 1972
COMP. JAB CHECK GLC CONT. NO. 1856

FOUND. SLAB DESIGN



KEY PLAN

PUMPING STA. FOUND PLAN

DESIGN LIMITS

DESIGN DATA (W.S.D.)

f'_c 3000 psi
f_c = 1050 psi
v_c = 60 psi
f_s = 20,000 psi

NOTE: NORTH WALL WILL NEVER GOVERN SOUTH WALL DESIGN, HOWEVER THE SOUTH WALL CAN GOVERN THE NORTH WALL DUE TO THE POSSIBILITY OF THE ABSENCE OF LAT. PRESSURE ON THE NORTH WALL.

SUMMARY

SLAB MOMENTS AND SHEARS

LOCATION	SECTION A			SECTION B			SECTION C			REMARKS
	MOM.	V	CASE	MOM	V	CASE	MOM	V	CASE	
END WALL	-66 +17	30	I	-36 +68	30	IIA	-99 +184	48	IIA	
1ST. INTERIOR WALL	-110 +23	30	↓	-87 +62	30	I, IIA	-244 +109	48	I	
INTERIOR WALL	-24		↓	-62		↓	-185			

WALL MOMENTS AND SHEARS

LOCATION	SECTION A			SECTION B			SECTION C			REMARKS
	MOM	V	CASE	MOM	V	CASE	MOM	V	CASE	
END WALL	-66 ~~+[illegible]~~ 48	24	I	~~-35~~ 46	26	IIA	-99 +150	21	I, IA	
1ST. INTERIOR WALL	-11 —	—	↓	63+	26	↓	+176	17	IIIA	
INTERIOR WALL	-2 —	—	↓	+63	26	↓	+176			

"SECTIONS FROM SOUTH WALL TO ℄ STA."

— NEG. MOMENT
— POS. MOMENT

SUMMARY

SLAB MOMENTS AND SHEARS

LOCATION	SECTION A			SECTION B			SECTION C			REMARKS
	MOM.^k	V^k	CASE	MOM	V	CASE	MOM	V	CASE	
END WALL	-60			-111		IA/IIB	-168		IIB	
	+35	27		+41	30	↓	+132	42	↓	
1ST. INTERIOR WALL	-72			-74			-215			
	+35	27		+105	28		+114	42		
INTERIOR WALL	-72			-73			-204			

WALL MOMENTS AND SHEARS

LOCATION	SECTION A			SECTION B			SECTION C			REMARKS
	MOM	V	CASE	MOM	V	CASE	MOM	V	CASE	
END WALL	-66	15		-111	27	IA	+168	25	IIB	
	+148			+100		↓	+92		↓	
1ST. INTERIOR WALL	79			+63	8		±176	17		
INTERIOR WALL	72			±63	8		±176	17		

"SECTIONS FROM NORTH WALL TO ℄ STA."

▬ NEG. MOMENT
— POS MOMENT

PROJECT: CHARLES RIVER BASIN
SUBJECT: PUMPING STA. REDESIGN
FOUND SLAB @ EL. 85
COMP.: [illegible] CHECK: [illegible]

ACC. NO.
SHEET NO. 32 OF
DATE 9-1 1972
CONT. NO. 1856

DESIGN SLAB SECT "A"

SECTION FROM SOUTH WALL TO ℄ STA.

DEPTH OF SLAB 48"

$d_a = 48.0" - 6.7" = 41.3"$

MAX. MOM = 110.0^k ~ $d_u = \sqrt{\frac{110}{.158}}$ = 26.4" ok

MAX V = 30.0^k ~ $v = \frac{30}{12 \times 41.3}$ = 60.4 psi ok

AT END WALL ($ad = 1.44 \times 41.3 = 59.4$)

$-M = 66^{k}$ ~ $A_s = \frac{66}{1.44 \times 41.3}$ = 1.11 □" < 1.16

$+M = 17^{k}$ ~ $A_s = \frac{17}{59.4}$ = .29 □"

A_{s_T} = 1.16 TEMP. REINF. CONTROLS DESIGN. EXCEPT #9@12 = 1.0 □"

TEMP. REINF (REF. EM 1110-2-2103) "RESTRAINED"

MIN. REINF PERPENDICULAR TO RESTRAINED EDGE = .004×12×48 = 2.3□"

HALF EACH FACE = 2.3 × .5 = 1.15 □" ~ #10@12" *

MIN REINF PARALLEL TO RESTRAINED EDGE = .004×12×48 = 2.3□"

FOR A DISTANCE OF ONE FOURTH THE LENGTH OF RESTRAINT .25 × 31.0' = 7.75'. HALF EACH FACE $\frac{2.3}{2}$ = 1.15 #10@12

REMAINDER OF REINF. = .002 × 12 × 48 = 1.15 □"

HALF EACH FACE = 1.15 × .5 = 0.57 □" ~ #7@12

CHECK BOND

(#10@12) TOP $\Sigma_0 = \frac{30}{.147 \times .872 \times 41.3}$ = 5.7 N.G.

BOND CONTROLS TOP BARS USE #8@8"

(#10@12) BOTT $\Sigma_0 = \frac{30}{.207 \times .872 \times 41.3}$ = 4.2 ok

USE

48" ± SLAB

MAIN. REINF
#10@12 BOT
#8@8 T

TEMP #9@12 T&B 7'-9" FROM SUPPORT
REM. #7@12 T&B

NOTE #9@12 MAX TEMP REINF. HOWEVER BOND CONTROLS

PROJECT: CHARLES RIVER BASIN — ACC. NO. ____
SUBJECT: PUMPING STA. REDESIGN — SHEET NO. 33 OF ____
FOUND SLAB @ EL. 86.0 — DATE 9-1 1972
COMP. JJB — CHECK GLC — CONT. NO. 1856

SLAB SECT. "A" CONT.

SECTION FROM NORTH WALL TO ℄ STA.

AT END WALL

+M = 66[1K] (USE SAME DESIGN AS FOR SOUTH WALL SH. #32

+M = 138[1K]
V = 27[K]

THESE DESIGN MOMENTS & SHEARS ARE EQUAL OR LESS THEN IN A PREVIOUS CASE. REF. SH. #32 "TEMP. REINF. CONTROLS"

USE 48" t SLAB

MAIN REINF. #10 @ 12 BOT. / #8 @ 8 TOP

TEMP. REINF #8 @ 12 7'-9" FROM SUPPORT. REM. #7 @ 12 T&B

SECT "A" CONT.

SECTION FROM SOUTH WALL TO ℄ STA.

AT 1ST INTERIOR SUPPORT

TEMP REINF SEE SH. 32

$-M = 110^{K} \rightarrow A_s = \frac{110}{59.4} = 1.86^{\square''} > 1.15^{\square''}$ THE MOMENT GOVERNS

$+M = 23^{1K} \rightarrow$ TEMP. REINF CONTROL (REF. SH #32)

$V = 30^{K}$

$-A_s = 1.86 \rightarrow \#11 @ 10$

CHECK BOND (NOTE V IS TAKEN @ ℄ OF SUPPORT AND CAN BE REDUCED BY V AT THE FACE)

$\Sigma_0 = \frac{30}{.132 \times .872 \times 41.3} = 6.3$ BOND CONTROLS

TRY ~~#10 @ 8~~ US #8 @ 8

$\Sigma_0 = \frac{30}{.147 \times .872 \times 41.3} = 5.6$ ok

USE 48" t SLAB

MAIN REINF #11 @ 10 BOT. / #8 @ 8 TOP.

TEMP. #8 @ 12 T & B - 7'-4" FROM SUPPORT. REMAINDER REINF. #7 @ 12 T&B

SEE SH. #32 FOR TEMP REINF PERPENDICULAR TO MAIN REINF.

PROJECT: CHARLES RIVER BASIN — ACC. NO. ______
SUBJECT: PUMPING STA. REDESIGN — SHEET NO. 34 OF ____
FOUND. SLAB @ EL. 86.0 — DATE 9-1-1972
COMP. SRS — CHECK GLC — CONT. NO. 1856

SECT. "A" CONT.

SECTION FROM NORTH WALL TO ℄ STA.

AT 1ST INTERIOR SUPPORT

$-M = 72$ ∴ $-A_s = \frac{72}{59.4} = 1.21^{\square''} > 1.15$* THE MOM. CONTROLS (↙ TEMP REINF)

$+M = 35$ ←
$V = 27$
TEMP * CONTROLS

TRY ~~#9 @ 10~~ #8 @ 8

$\Sigma_o = \frac{27}{.165 \times .872 \times 41.3} = 4.5$ ok

USE 48" ⊥ SLAB

MAIN REINF. ~~#10 @ 12~~ BOT
~~# 9 @ 8~~ TOP

TEMP #9 @ 12 T&B — 7'-9" FROM SUPPORT. REMAINDER REINF. #7 @ 12 T&B

SECT "A"

SECTION FROM SOUTH WALL TO ℄ STA

@ INTERIOR SUPPORT

$-M = 24^{1k}$
$+M = 23^{1k}$

} ← THESE DESIGN MOM'S & SHEARS ARE LESS THE PREVIOUS CASE. REF SH. #32
TEMP REINF GOVERNS

USE SAME DATA AS FOR "AT END WALL" SH. #32

A 241

CHARLES A. MAGUIRE & ASSOCIATES INC. ENGINEERS

* REF. SH. #32

PROJECT: CHARLES RIVER BASIN ACC. NO.
SUBJECT: PUMPING STA. REDESIGN SHEET NO. 35 OF
FOUND. SLAB @ EL. 85.0' DATE 9-1 1978
COMP. JAR CHECK GLC CONT. NO. 1866

SECT "A"

SECTION FROM NORTH WALL TO ℄ STA.

@ INTERIOR SUPPORT

$-M = 70 \rightarrow A_s = \frac{70}{59.4} = 1.18 > 1.15$ (TEMP. REINF)

$V = 27^k$ — #9 @ 10 ok

+M 35''k (USE TEMP. STEEL ONLY)

TEMP REINF #9 @ 12 MAX

$\Sigma_o = \frac{27}{.233 \times .872 \times 41.3} = \underline{3.2} < 3.5$ ok

USE 48" t SLAB.

MAIN REINF #9 @ 12 B
#9 @ 10 T

TEMP. REINF
#9 @ 12 T & B
7'-9" FROM SUPPORT
R = #7 @ 12 T & B

PROJECT: CHARLES RIVER BASIN
SUBJECT: PUMPING STA. REDESIGN
FOUND SLAB @ EL. 85.0
COMP.: JAT CHECK: GLC

ACC. NO.
SHEET NO. 36 OF
DATE 9-1 1972
CONT. NO. 1856

DESIGN SLAB SECTION B

SECTION FROM SOUTH WALL TO ℄ STA.

AT END WALL

$-M = 36$ TEMP REINF #9@12

$+M = 68$ — $A_s = \frac{68}{59.4} = 1.15$ — # #8@8

$V = 30^K$ Bond ok see Sh. #32

USE: 48" SLAB

MAIN REINF
#9@12 BOT
#8@8 TOP
TEMP #9@12" T. & B
7'-9" FROM SUPP.
REM. #7@12 T & B

SECT. B

SECTION FROM SOUTH WALL TO ℄ STA.

AT 1ST. INTERIOR SUPPORT

$-M = 87^{1K}$ ~ $A_s = \frac{87}{59.4} = 1.46^{\square''}$ ~ #9@8"

$+M = 62^{1K}$ ~ TEMP CONTROLS
BOND " "

V 30

USE 48" T SLAB

MAIN REINF
#9@8" B
#8@8 T
TEMP. #9@12 T&B
7'-9" FROM SUPP.
R = #7@12 T&B

PROJECT CHARLES RIVER BASIN — ACC. NO.
SUBJECT PUMPING STA. REDESIGN — SHEET NO. 37 OF
FOUND. SLAB @ EL. 85.00 — DATE 9-1 1972
COMP. JAB CHECK GLG — CONT. NO. 1856

SECT. "B"

SECTION FROM SOUTH WALL TO ℄ STA.

AT INTERIOR SUPPORT

−M = 62 As ≠ = 62 / 59.4 = 1.05 TEMP. REINF. CONTROLS
+M = 62

USE 1ST INT. SUPPORT DESIGN

SECT. "B"

SECTION FROM NORTH WALL TO ℄ STA

AT END WALL

−M = 111'k — SIMILAR TO 1ST INTERIOR WALL SUPPORT LOCATION SECTION A SOUTH WALL END. SEE SH. #33
+M = 41'k
V = 30

USE 48" SLAB

MAIN REINF. #11 @ 10" B, #8 @ 8" T

TEMP. #9 @ 12 T&B
7'-9" FROM SUPPORT
R = #7 @ 12 T&B

SECT. "B"

AT 1ST & TYPICA INTER. WALL

−M = 74'k ~ −As = 74 / 59.4 = 1.25"' ~ #10 @ 12
+M = 105 ~ +As = 105 / 59.4 = 1.95"' ~ #11 @ 9
V = 28

#10 @ 12 Σ_o = 28 / (.207 × .872 × 41.3) = 3.8 ok
#11 @ 9 Σ_o = 28 / (.132 × .872 × 41.3) = 5.9 ok

USE 48" SLAB

MAIN REINF #10 @ 12 B, #11 @ 9 T

TEMP SAME AS ABOVE

CHARLES A. MAGUIRE & ASSOCIATES INC. ENGINEERS
A244

PROJECT: CHARLES RIVER BASIN
SUBJECT: PUMPING STA. REDESIGN
BOUND SLAB @ EL. 85.00'
COMP. [illegible] CHECK GLC

ACC. NO.
SHEET NO. 38 OF
DATE 9-1 1972
CONT. NO. 1856

DESIGN SLAB SECT. "C"

SECTION FROM SOUTH WALL TO STA.

Max. Mom = 244^{1k} $d = \sqrt{\frac{244}{[illegible]}} = $ [illegible]"

Max V = 48^k

Max As = $\frac{244}{57.4}$ = 4.1$^{\square''}$ — MAX. BAR SIZE & SPACING = #11 @ 6

TRY 54" SLAB ∴ d = 54 − 6.7 = 47.3"

MAX AS = $\frac{244}{1.44 \times 47.3}$ = 3.6 S.I. ∴ #11 @ 5 As = 3.74 ok

CHECK FOR MAX BOND REQUIREMENTS:

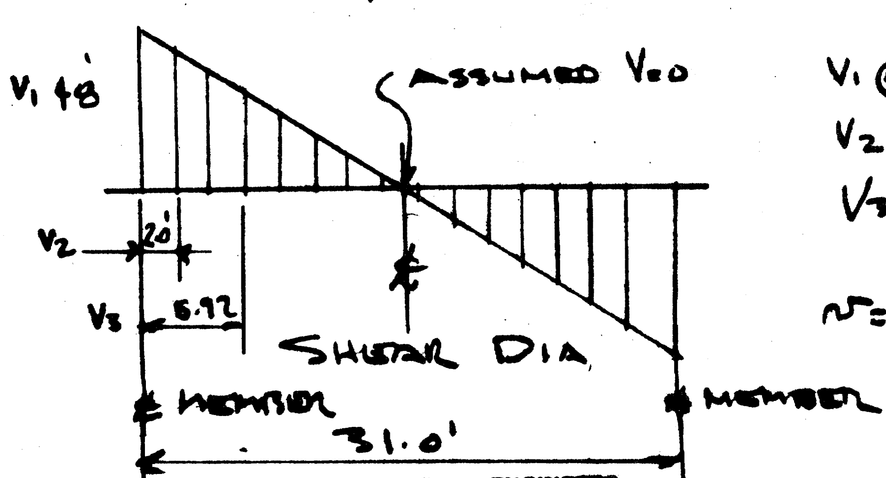
#11 BARS (BOTTOM ONLY)

$\Sigma_o = \frac{48}{.187 \times .872 \times 47.3}$ = 6.2 ok

CHECK MAX SHEAR $v = \frac{48}{12 \times 47.3}$ = 85 psi

NOTE: AS INDICATED ON SH.# 33. THE MAX. V IS AT THE ℄ OF MEMBER

COMPUTE ACTUAL SHEAR @ FACE OF SUPPORT



V1 @ ℄ = 48^k

V2 @ ℄ = 42^k

V3 @ ℄ = 29.8

$v = \frac{29.8}{12 \times 47.3}$ = 52 psi ok

PROJECT CHARLES RIVER BASIN ACC. NO.
SUBJECT PUMPING STA. REDESIGN SHEET NO. OF
FOUND. SLAB @ B. 86 DATE 9-1 1972
COMP. [illegible] CHECK GLC CONT. NO. 1856

SLAB "C"

SECTION FROM SOUTH WALL TO STA.

~~AT END WALL~~ (d = 47.3) (ad = 1.44 x 47.3 = 68.1)

$-M = 99^{1k}$ ~ As = 99 / 68.1 = 1.46 — #9 @ 8

+M = 184 ~ As = 184 / 68.1 = 2.6 — #11 @ 7"

V = 48 @ ℄

V = 42 @ FACE OF WALL SEE SK. # 38

MIN TEMP. REINF. PERPENDICULAR TO RESTRAINED EDGE = .004 x 12 x 54 = 2.59 □"

HALF EA. FACE OF WALL = 2.59 x .5 = 1.29 □"

OR MAX. # 9 @ 12"

MIN TEMP REINF PARALLEL TO RESTRAINED EDGE = 1.29 □" EA. FACE OF WALL 7'-9" FROM FACE OF SUPPORT. REMAINDER = .002 x 12 x 54 = 1.3 □" .65 EA. FACE

CHECK BOND (THE MOM'S. CONTROL)

Σ_0 #9 (BOTT) = 42 / (.233 x .872 x 47.3) = 4.3 ok

Σ_0 #11 (TOP) = 42 / (.132 x .872 x 47.3) = 7.7 ok

USE 54" SLAB
MAIN REINF. #9 @ 8 B
#11 @ 7 T
TEMP #9 @ 12 T & B
7'-9" FROM SUPPORT
REMAINDER #7 @ 11 T & B

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PROJECT: CHARLES RIVER BASIN — ACC. NO.
SUBJECT: PUMPING STA. REDESIGN — SHEET NO. 40 OF
BOND - SLAB AT EL. 85.00 — DATE 9-1 1972
COMP. JAB — CHECK GLC — CONT. NO. 1856

SLAB "C"   SO. WALL CONT.

AT 1ST. & TYP. INT.

$-M = 244^{IK} \sim As = \frac{244}{68.1} = 3.61 \sim \#11 @ 5"$

$+M = 109^{IK} \sim As = \frac{109}{68.1} = 1.6 \sim \#10 @ 9$

BOND (CONTROLS)

$V = 42^{K}$ @ FACE OF SUPPORT.

CHECK BOND

$\Sigma_0^1 (\#11 \text{ BOTT}) = \frac{42}{.187 \times .872 \times 47.3} = 5.4$ ok

$\Sigma_0^1 (\#10 \text{ TOP}) = \frac{42}{.147 \times .872 \times 47.3} = 6.9$ N.G.

TRY #9 @ 7 $\Sigma_0^1 = \frac{42}{.165 \times .872 \times 47.3} = 6.1$ ok

USE 54" SLAB

MAIN REINF. #11 @ 5 BOTT.

#10 @ 9 T

TEMP #9 @ 12 T & B

7'-9" FROM SUPPORT,

REMAINDER = #7 @ 11 T & B

PROJECT: Charles River Basin
SUBJECT: Pumping Sta. Redesign
Bound. Slab [illegible]
COMP. JAB CHECK GLC

ACC. NO.
SHEET NO. 41 OF
DATE 9-1-72
CONT. NO. 1756

Design Slab Sect. C

Section from North Wall to Sta.

At End Wall

$-M = 168 \sim A_s = \frac{168}{68.1} = 2.47^{\square} \sim$ #11 @ 7"

$+M = 132 \sim A_s = \frac{132}{68.1} = 1.95^{\square} \sim$ #11 @ 9" N.G.

Check #11@7" top for Bond $\Sigma_o = \frac{42}{.132 \times .872 \times 473} = 7.6$ N.G.

Bond Governs ~ use #9@6

USE 54" SLAB

MAIN REINF #11@7 B

#9@6 T

TEMP #9@12 T&B

7'-9" AWAY FROM SUPP.

THEN USE #7@11 T&B

SECT. C

At 1st Inter. Wall & Typ Int. Wall

$-M = 215 \sim A_s = \frac{215}{68.1} = 3.15 \sim$ #11 @ 6"

$+M = 114 \sim A_s = \frac{114}{68.1} = 1.68 \sim$ #9 @ 7

$V = 42$

USE 54" SLAB

MAIN REINF #11@6 B

#9@7 T

Temp. #9@12 T&B

7'-9" FROM SUPPORT

Rc #7@11"

PROJECT: CHARLES RIVER BASIN
SUBJECT: PUMPING STA. REDESIGN
FOUND. WALLS
COMP. AS   CHECK GLL
ACC. NO.
SHEET NO. 42 OF
DATE 9-3 1972
CONT. NO. 1056

DESIGN WALL SECT "A" (WALL t = 48")

SECTION FROM SOUTH WALL TO ℄ STA.

MAX WALL MOM. (ALL SECTIONS) = 204 ^k

" " V (" ") = 27 ^k

$d_M = \sqrt{\frac{204}{.226}} = 30.5''$ ok ∴ $A_{s\,max} = \frac{204}{1.44 \times 41.3} = 3.4^{\square''}$ ok

$\max v = \frac{27}{41.3 \times 12} = 54$ psi ok

$d \times j = 41.3 \times 1.44 = 59.4$

END WALL

$-M = 66^{1k}$ ∴ $-A_s = \frac{66}{59.4} = 1.11^{\square''}$ ~ #10 @ 12 I.F (TEMP OUTSIDE)

$+M = 204$ ∴ $+A_s = \frac{204}{59.4} = 3.4^{\square''}$ ~ #11 @ 5" O.F

TEMP. REINF

PERPENDICULAR TO RESTRAINED EDGE

$= .0012 \times 48 \times 2.3^{\square''}$ TOT. $\frac{1}{2}$(EA. FACE) = $1.15^{\square''}$

PARALLEL TO RESTRAINED EDGE SAME $1.15^{\square''}$ E.F

$\frac{1}{4}$ THE LENGTH OF RESTRAINT = $\frac{1}{4} \times 16.5' = 3.9$ SAY 4'

REMAINDER = $12 \times 48 \times .002 = 1.15^{\square''}$ TOT

HALF EA. FACE = 0.57

USE 48" WALL

MAIN. REINF. #10 @ 12 I.F
VERT. #11 @ 5 OF

TEMP. #9 @ 12" E.F 4'0"
HORZ. FROM SUPPORT
R = #7 @ 12" EF

A 249

PROJECT: CHARLES RIVER BASIN ACC. NO. ____
SUBJECT: PUMPING STA. REDESIGN SHEET NO. 43 OF ____
FOUND. WALLS DATE 9-5 1972
COMP. JAB CHECK GLC CONT. NO. 1050

WALL SECT. "A" CONT.

+M = 11'k
+M = 0
V = 24k
} USE MIN. REINF TEMP STEEL.

USE 48" WALL
MAIN. REINF
#9 @12 E.F. VERT
#9 @ 12 HORZ E.F.
4'-0" FROM SUP.
R = #7@12 HORZ EF

SECT. FROM SOUTH WALL TO ℄ STA.
TYPICAL INTERIOR WALL
(48" WALL)

SECT. "A"

SECTION FROM NORTH WALL TO ℄ STA.

END WALL

−M = 66'k ~ As = 66/59.4 = 1.11 □" TEMP CONTROLS
+M = 148'k ~ As = 148/59.4 = 2.53□" ~ #10@6
V = 15k

USE SAM TEMP REINF AS FOR END WALL SH. # 42

USE 48" WALL
(MAIN VERT) REINF #10@12 I.F. / #10@6 O.F.
(TEMP HORZ) #9@12 E.F. 4'-0" FROM SUPPORT
R = #7@12 E.F.

SECT. "A"

SECT. FROM NORTH WALL TO ℄ STA.

TYP INT. WALL
48" WALL

USE SAME DESIGN AS FOR TYP. INTERIOR WALL SOUTH END THIS SH.

−M = 72'k ~ As = 72/59.4 = 1.2
+M = 0

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A 250

PROJECT: CHARLES RIVER BASIN
SUBJECT: PUMPING STA. REDESIGN
FOUND. WALLS
COMP. JAB CHECK GLC
ACC. NO.
SHEET NO.
DATE
CONT. NO.

TYPICAL INTERIOR DEVIDER WALLS. t=2'8"

DESIGN NOTES:

1. DESIGN EA. WALL THE SAME FOR THE FULL LENGTH
2. THERE CAN BE NO UNEQUAL PRESSURE ON ANY OF THE DEVIDER WALLS
3. PROVIDE A WALL DRAIN IN EA. WALL SO THAT IF ONE SECTION IS DEWATERED THE DRAIN WILL SIMULTANEOUSLY EMPTY EACH.

MIN. REINF

PERPENDICULAR TO RESTRAINED FACE

= .004 × 27 × 12 = 1.29□" TOT. OR .65□" E.F.

PARALLEL TO RESTRAINED FACE

= SAME .65□" E.F. 4'-0" FROM SUPPORT

REM = .002 × 12 × 27 × 1/2 = .33

USE 2'-3" t

REINF: VERT. #7@12 E.F.
HORZ #7@12 E.F. 4'-0" FROM SUPPORT
R = #5@12 EA F

PROJECT CHARLES RIVER BASIN ACC. NO.
SUBJECT PUMPING STA. REDESIGN SHEET NO. 45 OF
FOUND WALLS DATE 9-5 1972
COMP. JAR CHECK GLC CONT. NO. 1856

DESIGN SECT. "B"

SECTION FROM SOUTH WALL TO ℄ STA.

END WALL

-M = 35'k TEMP CONTROLS

+M = 63 → $\frac{63}{59.4}$ = 1.06

V = 26

USE SAME AS NORTH WALL

USE 48" ℄

#9@12 E.F V

#9@12 E.F H.

4'-0" FROM EDGE

R = #7@12 EF

SECT. "B"

SECT. FROM SOUTH WALL TO ℄ STA.

INTERIOR WALLS

±M = 63'k } TEMP. REINF

V = 26k } CONTROLS

USE SAME AS END WALL ABOVE

SECT. "B"

SECT. FROM NORTH WALL TO ℄ STA.

END WALL

-M = 94 → As = $\frac{94}{59.4}$ = 1.55"□ #11@12

+M = [illegible]

USE 48" ℄ WALL

MAIN REINF; #11@12 EF
VER #11@12 IF

TEMP
HORZ. #9@12 E.F
7'-0" FROM SUPP.
R = #7@12 E.F

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PROJECT: CHARLES RIVER BASIN ACC. NO. ____
SUBJECT: PUMPING STA. REDESIGN SHEET NO. 46 OF ____
FOUND. WALLS DATE 9-8 1972
COMP. ____ CHECK GLC CONT. NO. 1586

SECT. "B"

SECTION FROM NORTH WALL TO ℄ STA

INTERIOR WALLS

±M = 63'k ~ As = 63/59.4 = 1.06

"TEM REINF CONTROLS

USE 48" t
#9 @ 12 E.F. VERT
#9 @ 12 E.F. HORZ

DESIGN WALL SECTION "C"

SECTION FROM SOUTH WALL TO ℄ OF STA.

END WALL

−M = 99'k ~ As = 99/59.4 = 1.67□" ~ #10 @ 9

+M = 150 ~ As = 150/59.4 = 2.53□" ~ #11 @ 7

V = 17

TEMP. .004 × 12 × 48 × 1/2 = 1.15□" E.F.

USE 48" t
#11 @ 7 O.F.
#10 @ 9 I.F.
VERT
#9 @ 12 E.W. HOR

DESIGN SECTION "C"

SECT FROM SOUTH WALL TO ℄ STA

INTERIOR WALL

±M = 176 ~ As = 176/59.4 = 2.95□" #11 @ 6

USE 48" t

VERT #11 @ 6 E.F.
HORZ #9 @ 12

A253

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PROJECT: Charles River Basin — ACC. NO.
SUBJECT: Pumping Sta. Redesign — SHEET NO. 47 OF
Found. Walls — DATE 9-6 1972
COMP. XF3 — CHECK GLC — CONT. NO. 1856

Design Set "C"

Section from North Wall to ℄ Sta.

End Wall

$-M = 168 \rightarrow As = \frac{168}{59.4} = 2.83^{\square''} \rightarrow$ # 11 @ 6"

$+M = 92 \rightarrow As = \frac{92}{59.4} = 1.55^{\square''} \rightarrow$ # 11 @ 12

$V = 25$

USE 48" t

Vert #11 @ 6 I.F.

#11 @ 12 O.F.

Horz. #9 @ 12 E.F.

Section "C"

Section from North Wall to ℄ Sta

Int. Wall

$\pm M = 176^{'k} \rightarrow As = \frac{176}{59.4} = 2.95$

USE 48" t

Vert #11 @ 6 E.F.

Hor #9 @ 12 E.F.

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APPENDIX B

ELECTRICAL DESIGN COMPUTATIONS

PROJECT CHARLES RIVER DAM | ACC. NO. 1856
SUBJECT REVISED LOADING DATA | SHEET NO. 1 OF 4
DATE OCT 3 1973
COMP. G.A.R. | CHECK G.M. | CONT. NO.

PUMPING STATION

OUTSIDE LIGHTING (PARK & PLAZA AREA)	20.0	KVA
INSIDE LIGHTING	38.3	KVA
TOTAL LIGHTING	58.3	KVA
PUMP. STA POWER	249.3	KVA
MISCELLANEOUS POWER	97.7	KVA
TOTAL POWER	347.0	KVA
HEATING & VENTILATION	189.1	KVA
AIR CONDITIONING	53.5	KVA
FISHWAY (150 HP MOTOR)	150.0	KVA
FUTURE LOADING	155.0	KVA

RECAP

TOTAL LTG	58.3	KVA
TOTAL PWR	347.0	KVA
HEAT & VENT	189.1	KVA
FISHWAY	150.0	KVA
FUTURE	155.0	KVA
	899.4	KVA

SAY 900 KVA @ 65% DEMAND = 585 KVA

B-1

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PROJECT: CHARLES RIVER DAM
SUBJECT: REVISED LOADING DATA
COMP.: G.A.R. CHECK: G.M.

ACC. NO. 1856
SHEET NO. 2 OF 4
DATE OCT. 3 1977
CONT. NO.

LOCKS

OUTSIDE LIGHTING		27.4 KVA
GALLERY LIGHTING		16.3 KVA
	TOTAL LIGHTING	43.7 KVA
GALLERY HEATING & PWR.		112.4 KVA
GATE & SEAL HEATING		36.1 KVA
	TOTAL HEATING	148.5 KVA
GALLERY POWER (LARGEST MOTOR 50)		324.0 KVA
FUTURE LOADING		103.0 KVA

RECAP

TOTAL LTG	43.7 KVA
TOTAL HTG	148.5 KVA
TOTAL PWR	324.0 KVA
FUTURE	103.0 KVA
	619.2 KVA

SAY 620 KVA @ 65% DEMAND = 404 KVA

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PROJECT CHARLES RIVER [illegible] M — ACC. NO. 1856
SUBJECT REVISED LOADING DATA — SHEET NO. 3 OF 4
DATE OCT. 3 1973
COMP. C.G.R. — CHECK G.M. — CONT. NO.

M.D.C. BOAT FACILITY

OUTSIDE LIGHTING (PARKING AREA)	20.0 KVA
INSIDE LIGHTING	12.3 KVA
TOTAL LIGHTING	32.3 KVA
HEATING & VENTILATION	15.8 KVA
AIR CONDITIONING	42.0 KVA
MISCELLANEOUS POWER	90.5 KVA
FUTURE LOADING	40.0 KVA

RECAP

TOTAL LTG	32.3 KVA
TOTAL HTG	15.8 KVA
TOTAL PWR	90.5 KVA
FUTURE	40.0 KVA
	178.6 KVA

SAY 180 KVA @ 65% DEMAND = 93 KVA

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PROJECT: CHARLES RIVER DAM | ACC. NO. 1856
SUBJECT: REVISED LOADING DATA | SHEET NO. 4 OF 4
DATE: OCT. 2 19 77
COMP. G.A.R. CHECK G.M. CONT. NO.

CONTROL TOWER

LIGHTING	8.4 KVA
HEATING	77.4 KVA
FUTURE LOADING	8.5 KVA
TOTAL	94.3 KVA

SAY 95 KVA @ 85% DEMAND = 80.7 KVA

PROJECT RECAP

TOTAL PUMP STA	585.0 KVA
TOTAL LOCKS	404.0 KVA
TOTAL BOAT FACILITY	93.0 KVA
TOTAL CONTROL TOWER	80.7 KVA
	1162.7 KVA

CHARLES RIVER

1856

10-15-73

ELECTRICAL LOAD DATA

EMERGENCY GENERATOR

LOCKS & NAVIGATION

Connected Load - Lighting		90.0 KVA
- Power - Air Compressor 20HP Oil Purifier 3/4HP Hydr. Oil Pump 10HP		36.0 "
- Misc Power		140.0 "
Total Connected Load		266.0 KVA
Demand Load - Lighting 90 KVA @ 90%	= 81	
Power 36 KVA @ 80%	28.8	
Misc. Power 140 KVA @ 55%	77	
Total Demand Load		186.8 KVA

PUMPING STATION

Connected Load - Lighting		60.0 KVA
- Power - 14 - 1/2 HP 3 - 3/4 HP 1 - 1 HP 10 - 1½ HP 7 - 3 HP 6 - 5 HP 9 - 7½ HP 3 - 10 HP		215.0 KVA
- Misc Power		95.0 KVA
Total Connected Load		370.0 KVA
Demand Load - Lighting 60 KVA @ 90%	= 54	
Power 215 KVA @ 55%	= 118	
Misc. Power 95 KVA @ 60%	= 57	
Total Demand Load		229.0 KVA

OVERALL PROJECT DEMAND

Total Lighting Demand = 81 + 54 = 135 KVA @ 95% =	128 KVA
Total Power Demand = 28.8 + 118 = 146.8 " @ 85% =	125 "
Total Misc. Power Demand = 77 + 57 = 132 " @ 80% =	106 "
TOTAL PROJECT DEMAND	359 KVA