KARNAFULI PROJECT
DESIGN REPORT

VOLUME II

IECO INTERNATIONAL ENGINEERING COMPANY INC

KARNAFULI PROJECT
DESIGN REPORT

VOLUME II

PREPARED FOR THE

GOVERNMENT OF PAKISTAN

AND THE

AGENCY FOR INTERNATIONAL DEVELOPMENT

BY

INTERNATIONAL ENGINEERING COMPANY, INC.

74 NEW MONTGOMERY STREET
SAN FRANCISCO, CALIFORNIA

MAY 1962

KARNAFULI PROJECT

DESIGN REPORT

PART III

STRUCTURES

CHAPTER I GENERAL

CHAPTER II SPILLWAY

2.01 General
2.02 Hydraulic Design
2.03 Structural Design

CHAPTER III DIVERSION AND OUTLET TUNNEL

3.01 General
3.02 Design Criteria and Assumptions
3.03 Design of Tunnel Components

CHAPTER IV POWER INTAKE

4.01 General
4.02 Hydraulic Design
4.03 Structural Design - Intake Structure
4.04 Structural Design - Penstock Tunnels

CHAPTER V POWERHOUSE

5.01 General
5.02 Arrangement
5.03 Structural Design

CHAPTER VI SWITCHYARD AND SINGLE-CIRCUIT TOWER

6.01 General
6.02 Design Criteria and Assumptions
6.03 Types of Loading
6.04 Methods of Analysis
6.05 Results

CHAPTER VII CARGO TRANSFER SYSTEM

7.01 General
7.02 Operational Requirements
7.03 Superstructure Design
7.04 Substructure Design
7.05 Terminal Facilities

PART III

CHAPTER I

GENERAL

This part of the Design Report pertains to the design of the structures of the major project features. The purpose, considerations of site selection, descriptions, loads, stresses, stability, and methods of analysis are discussed for each of the features.

The structural design of the various concrete structures was in accordance with "Building Code Requirements for Reinforced Concrete" (ACI-318-56) by the American Concrete Institute. Steel structures were designed in accordance with the American Institute of Steel Construction "Standard Specification for the Design, Fabrication and Erection of Structural Steel for Buildings."

In addition to structural design, the hydraulic designs of the spillway, diversion and outlet tunnel, and the power intake structure are covered.

PART III

CHAPTER II

SPILLWAY

2.01 GENERAL

A. Purpose

The purpose of the spillway is to ensure the safety of the dam by the passage of periodic floods and the regulation of the level of the reservoir. Project hydrology and hydraulic requisites of the spillway are discussed in PART I, PROJECT SCOPE AND CRITERIA.

B. Description

The spillway, as shown on Exhibit II-1, is located in a saddle near the left abutment of the main dam. The principal features are the inlet walls, gated crest, chute, and stilling basin, which have a total length of 540 feet and a width of 745 feet. The inlet and outlet channels, which are unlined, have approximate lengths of 1300 feet and 4000 feet, respectively.

C. Design

The design of the spillway falls into two major divisions, hydraulic and structural. Good hydraulic performance was the prime consideration of the spillway design with economic and structural limitations imposed to give a practical structure.

The spillway was designed to be constructed in two stages (see Section 4.04, Chapter V, PART I.). Stage I consists of a flat crest structure 745 feet wide at Elevation 51.00 feet, with side walls to Elevation 76.00 feet, and the complete chute, stilling basin, inlet and outlet channels as required for the finished structure. Stage II consists of completion of the crest structure, side walls, piers, and bridge, and installation of the hoists, gates, and miscellaneous metalwork.

2.02 HYDRAULIC DESIGN

A. General

The spillway was designed to discharge 560 000 cubic feet per second when the surface of the surcharged reservoir is at a maximum elevation of 119.00 feet. In addition, the spillway was designed to be constructed so that a diversion design flood of 250,000 cubic feet per second during construction could be passed without the reservoir level overtopping the upstream cofferdam at Elevation 80.0 feet.

Model studies were performed by the Hydraulic Research Laboratory, State College of Washington, Pullman, Washington, U. S. A. The spillway model did not conform entirely with the final design because the decision to change the normal maximum reservoir level from Elevation 113.0 feet to Elevation 118.0 feet was made after the model study was completed. (See paragraph G). Also the abutment training walls had to be revised due to changed foundation conditions which became apparent after excavation had been completed.

B. Inlet Channel

1. Shape - The inlet channel is trapezoidal in section, with side slopes of 1-1/2 horizontal to 1 vertical. The base is at Elevation 51.0 feet, and the base width is approximately equal to the spillway crest width.

2. Plan - The inlet channel was curved in plan to avoid extensive excavation of a ridge which projects into the spillway approach area from the left bank.

3. Left Bank - Model studies of the inlet channel as originally designed indicated that extensive areas of slack water would appear along the left bank. Shifting of the inlet channel farther to the right eliminated the greater portion of slack water and reduced the quantity of excavation. Several natural bays exist along the left bank but they had no adverse effect on discharge of the model.

4. Right Bank - Due to natural topography, the right bank of the inlet channel is very short. During the model study it was noted that water approaching the inlet channel from the right side was directed into the main approach stream at right angles, adversely affecting the flow distribution in the 16 spillway bays. The major cause of this appeared to be a shallow bay, adjacent to and behind the right bank, extending to the toe of the end fill section of the dam. Smooth flow on the model was attained by filling this bay. A similar fill was placed on the prototype as shown on Exhibit II-1.

The maximum prototype velocity of flow, measured on the model, at the right bank, was 9.0 feet per second at a discharge of 300,000 cubic feet per second. The maximum prototype velocity of flow measured on the model, along the upstream toe of the dam, was 3.0 feet per second.

The final design of the inlet channel closely resembles the most satisfactory model shape tested.

5. Diversion Floods - The spillway shown on the contract drawings was not designed for two stage construction. The bottom of the inlet channel was set at Elevation 69.0 feet so that the total depth of water in the channel was 50 feet (to Elevation 119.0 feet) which produced an average inlet channel velocity of approximately 13.0 feet per

second for the peak flow during the design flood. This velocity is relatively high, and 70 feet of reinforced concrete apron was used to protect the bottom of the inlet channel from erosion immediately upstream from the spillway crest.

In the final design, the depth of water in the inlet channel is 68 feet (to Elevation 119.0 feet) which produces an average channel velocity of 9.7 feet per second at the upstream end of the channel walls for the peak flow during the design flood. For "normal" floods, velocities will be much lower and a protective apron for scour was no longer considered necessary. However, to reduce seepage because of jointing in the shale of the inlet channel that became apparent after excavation had progressed, the entire area between the inlet walls was paved with a 5-inch thick asphaltic concrete blanket. A 3-foot deep concrete cutoff was placed at the upstream end of the blanket. In the event of damage to the blanket during diversion through the Stage I Spillway, it could be repaired during the Stage II construction period.

C. Crest

Based on preliminary studies, the length of the spillway crest was set at 640 feet. Considering (1) crest length; (2) design flood; (3) reservoir storage capacity; and (4) maximum surcharged reservoir surface elevation, the spillway crest for the final design was set at Elevation 80.25 feet.

The upstream and downstream slopes of the crest shape were chosen on the basis of stability limitations and are also discussed in paragraph D2, Section 2.03. The curved portion near the crest is a circular arc which approximates the lower nappe of free discharge.

The shape of the lower nappe was determined using "Boulder Canyon Project Final Reports, PART VI, Hydraulic Investigation Bulletin 3, Studies of Crests for Overfall Dams" - United States Bureau of Reclamation, (USBR). Downstream from the gate sill the crest shape is defined by a parabolic transition from the circular curve to the uniform slope.

The gate sill was located downstream from the crest so that the release with partial gate openings would be directed downward. The gate sill was placed at Elevation 79.75 feet with the top of the gates at the normal maximum surface of the reservoir, Elevation 118.00 feet.

The width of the piers, based on structural requirements, is 7 feet. The combination of sixteen 40-feet wide gates and fifteen 7-foot piers required an overall width of 745 feet between spillway sidewalls. This width was used for the inlet channel, crest structure, chute and stilling basin.

The discharge capacity of the spillway was checked on the model of the original design. The measured discharges were plotted against corresponding reservoir elevations and are shown on Figure A. The discharges for partial gate openings were also checked on the model and the results shown on Figure B. Curves of discharge coefficients versus reservoir surface elevations were prepared for both free crest flow and for discharge with gates partially open. (See Figures C and D). The discharge coefficient, C_d for free crest flow was obtained using the equation:

$$C_d = \frac{Q}{LH^{3/2}}$$

The discharge coefficient, C_d, for discharge through partial gate opening was obtained using the equation:

$$C_d = \frac{Q}{0.667\ L\ \sqrt{2g}\ (H_1^{\ 3/2} - H_2^{\ 3/2})}$$ where:

Q = Discharge in cubic feet per second.

L = Effective length of the spillway crest (640 feet).

H = H_1 = Difference in elevation between the reservoir water surface and the spillway crest.

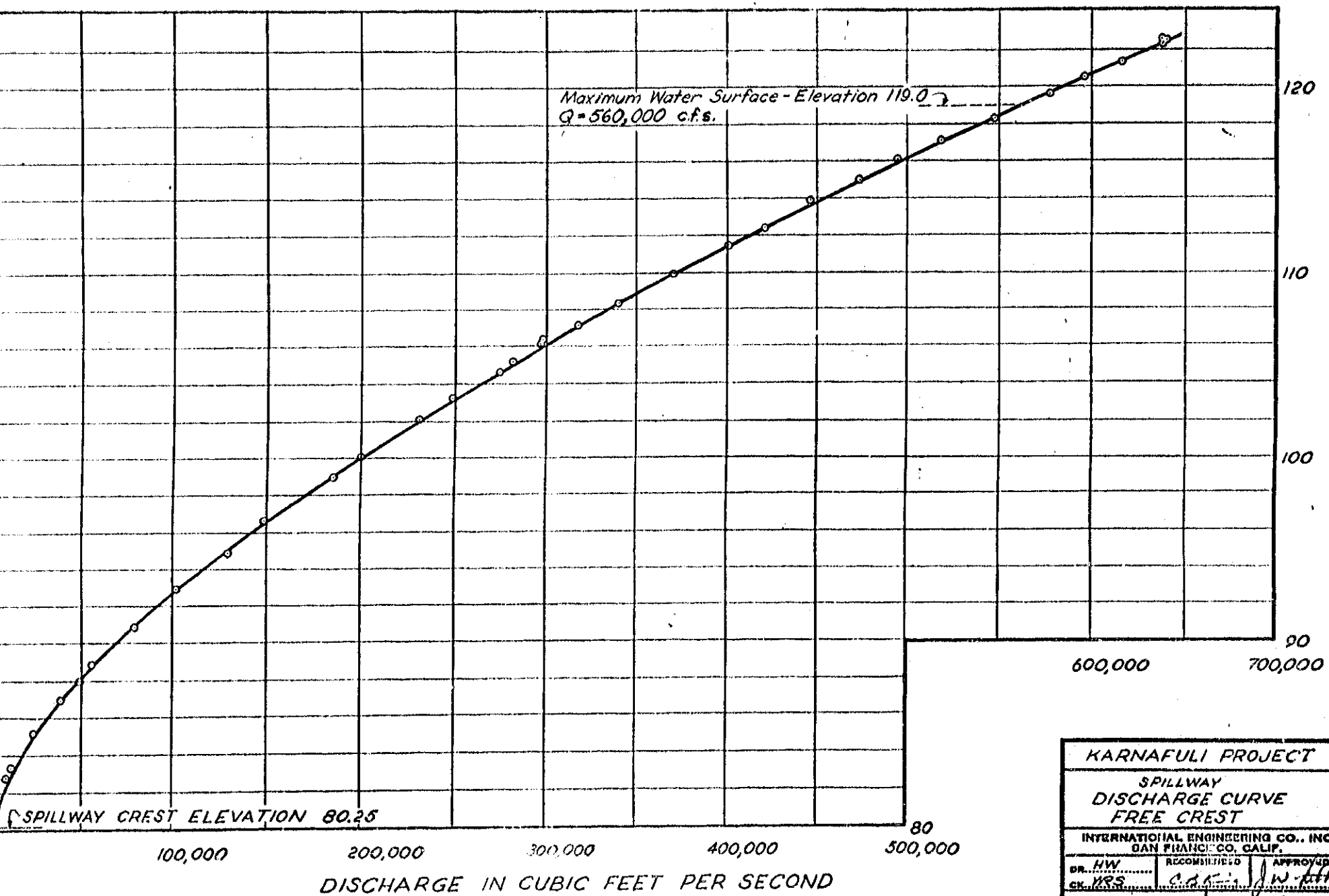
H_2 = Difference in elevation between the reservoir surface elevation and the bottom of the gate.

g = Acceleration of gravity, 32.2 feet per second per second.

Discharge capacity of Stage 1 of the spillway was also checked on the model, and the discharge curve is shown in Figure E.

Flow patterns through the spillway model were satisfactory for all ranges of discharge. The maximum tested was 640,000 cubic feet per second, at Elevation 123.25 feet which is well above the maximum flow for the design flood. This flow of 640,000 cubic feet per second should not be interpreted as being a basic capacity requirement or a design criteria, but was selected for model use because it accentuated the flow tendencies that existed with lesser discharges.

The usual drawdown was observed around the nose of the piers and a typical "fish tail" appeared where the flows merge down the chute from the end of the pier. The "fish tail" reached a maximum height of about 20 feet on the prototype and exhibited no apparent ill effects. There was no separation between the lower nappe and the crest profile and no sub-atmospheric pressures on the crest for either fully-open or partially-open gate operation, for all rates of discharge.



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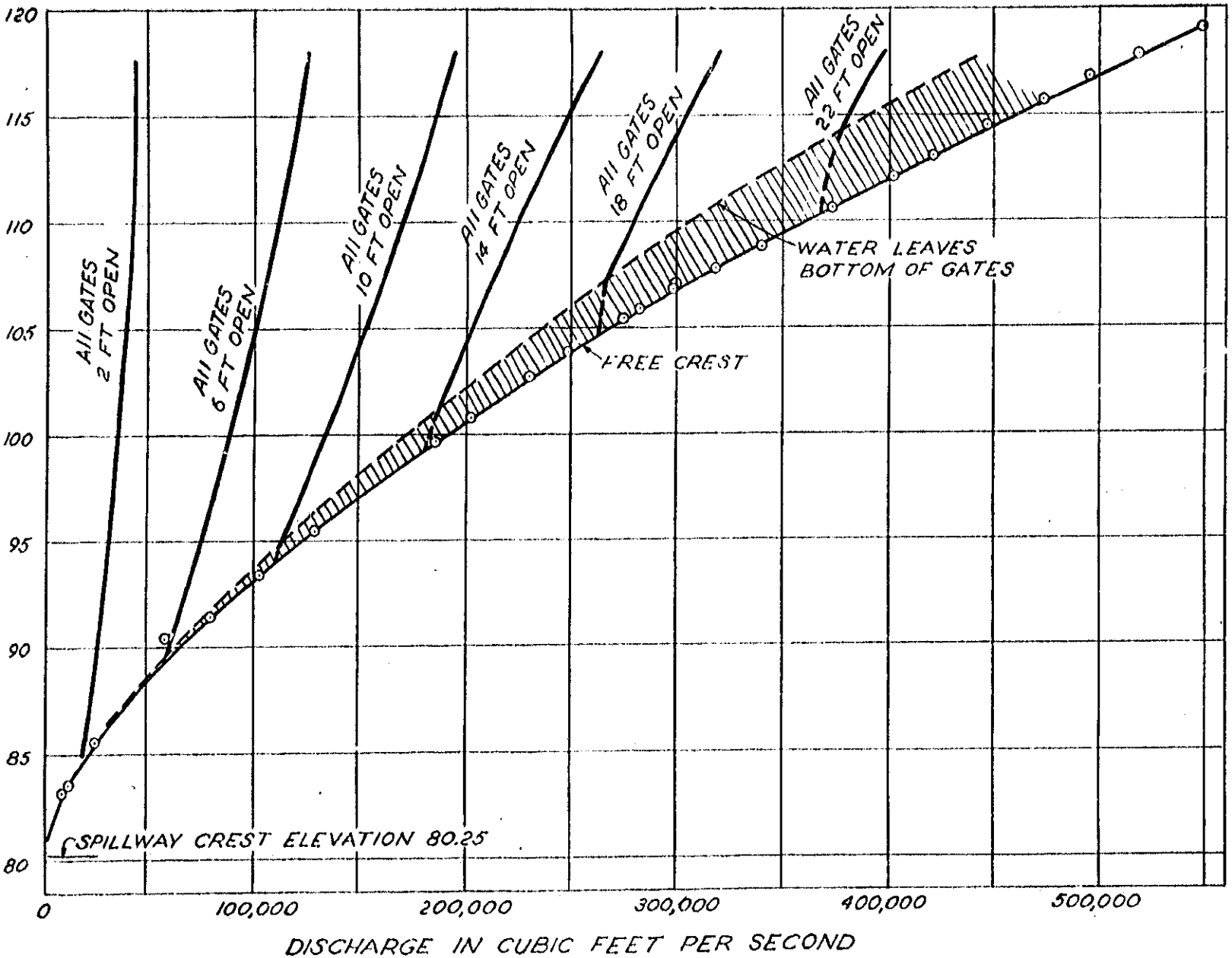
SPILLWAY
DISCHARGE CURVE
FREE CREST

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. HW	RECOMMENDED	APPROVED
CK. HRS		
DATE 2-4-59	FIG. A	



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SPILLWAY
PARTIAL GATE OPENINGS
DISCHARGE CURVES

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. HW | RECOMMENDED | APPROVED
CK. WPS

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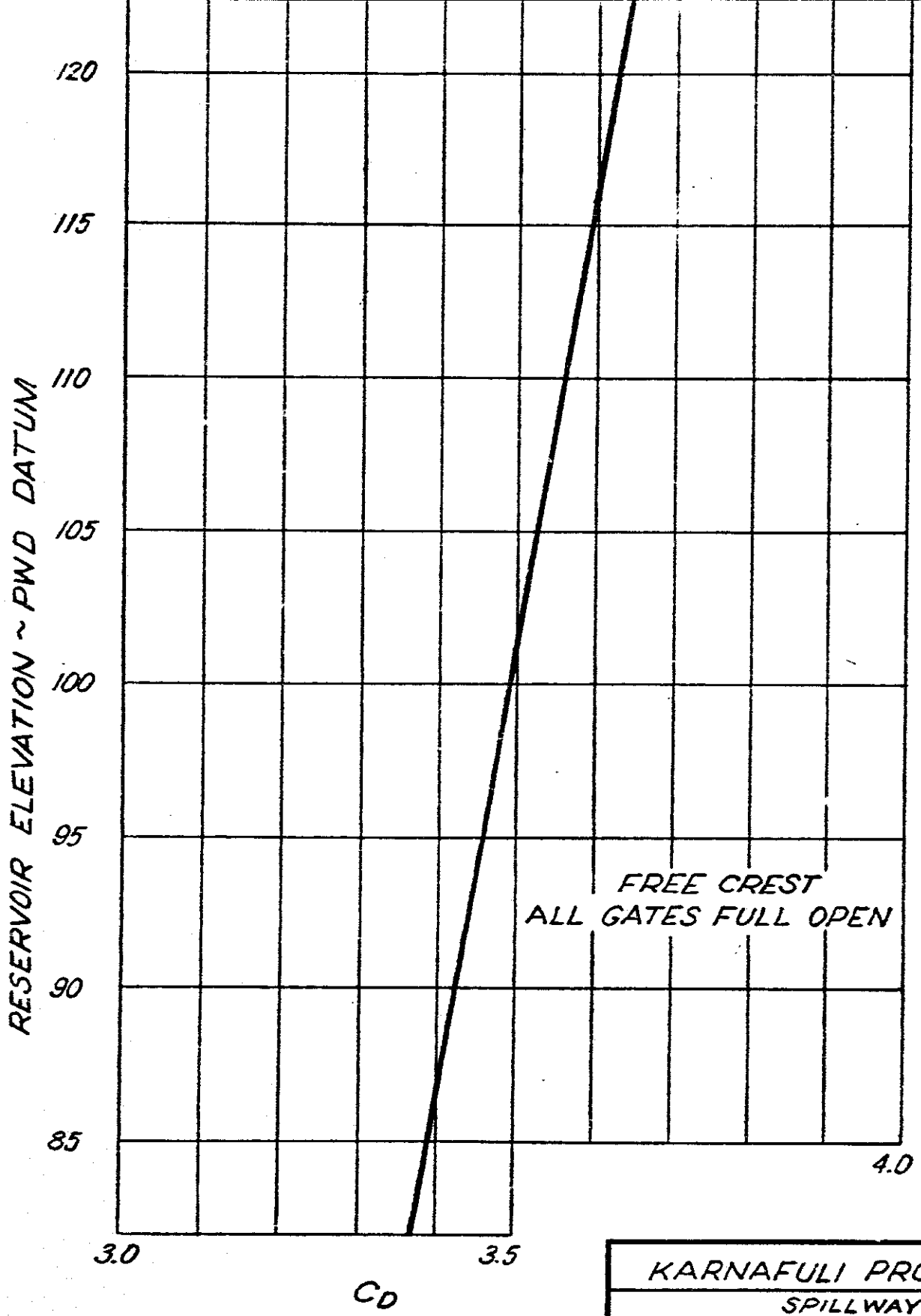

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SPILLWAY

DISCHARGE COEFFICIENTS

SHEET 1

INTERNATIONAL ENGINEERING CO., INC.
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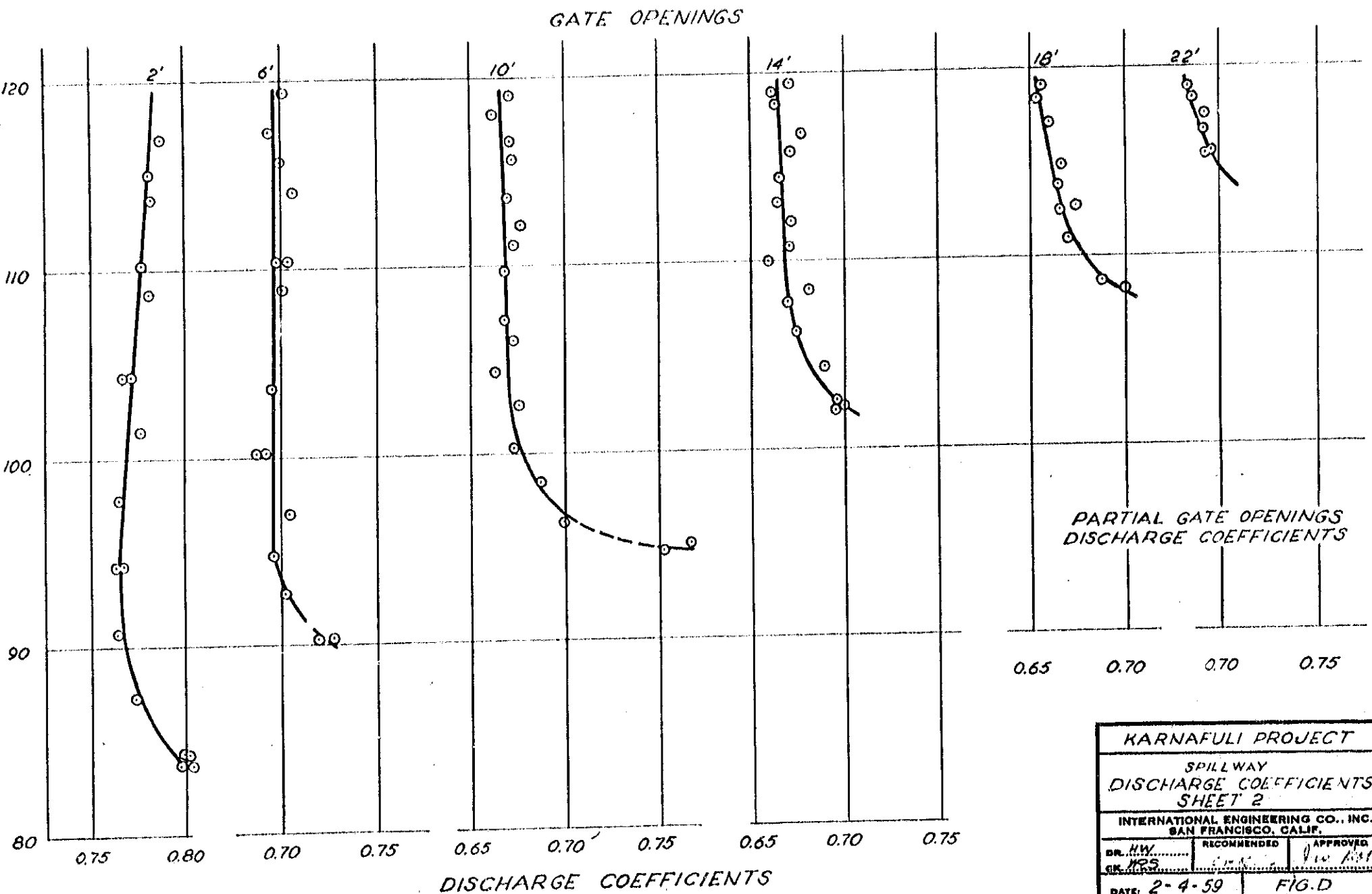
DR. HW	RECOMMENDED	APPROVED
CK. WRS		J.W. [illegible]
DATE: 2-4-59	FIG. C	

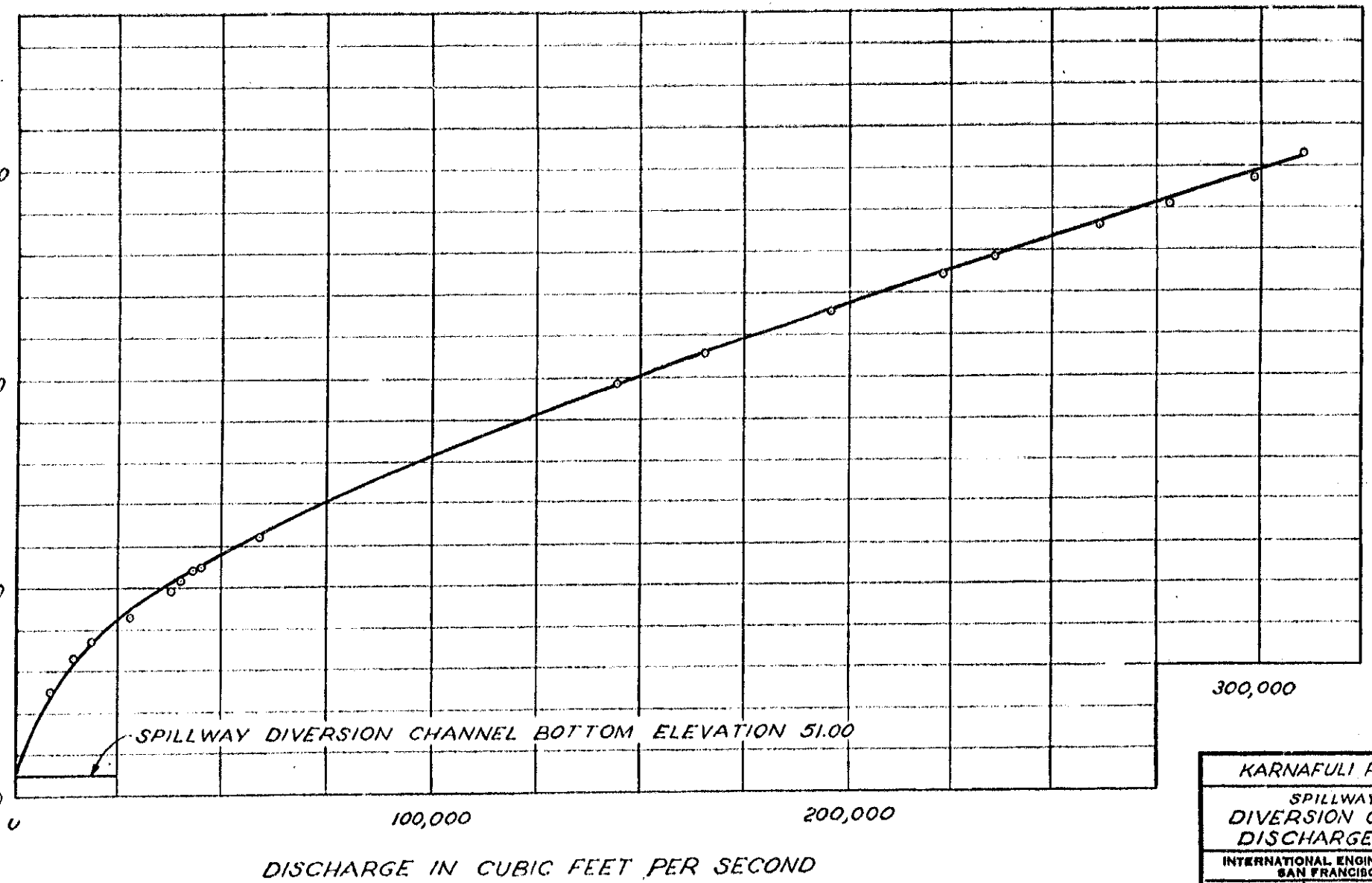


KARNAFULI PROJECT

SPILLWAY
DISCHARGE COEFFICIENTS
SHEET 2

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. H.W.	RECOMMENDED	APPROVED
CK. MPS		
DATE: 2-4-59	FIG. D	



KARNAFULI PROJECT

SPILLWAY
DIVERSION CHANNEL
DISCHARGE CURVE

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. HW	RECOMMENDED	APPROVED
CK. WRS		
DATE: 2-4-59	FIG. E	

During the model study of the original design, the upper nappe touched the gate pin at a discharge of 560,000 cubic feet per second. In the final design, the radius of the tainter gate was increased from 30 feet to 35 feet, and the gate pin is located above the upper nappe for all flows.

D. Chute

The slope used for the chute is 2 horizontal to 1 vertical. The chute walls required to maintain the backfill are about twice as high as the depth of maximum flows, as measured on the model.

E. Stilling Basin

The stilling basin was designed using the United States Bureau of Reclamation (USBR) "Progress Report III - Research Study on Stilling Basins, Energy Dissipators and Associated Appurtenances." The floor of the stilling basin was set at Elevation -3.00 feet, and the length of the apron at 180.00 feet. Chute blocks were used to shorten the hydraulic jump. Study of the model showed that the jump characteristics were satisfactory for all discharges, including diversion flows. (See Figures F and G). A dentated sill was used at the end of the stilling basin apron. Model studies indicated that it functioned properly.

F. Outlet Channel

The outlet channel is required to return spillway releases to the river channel through a distance of approximately 4000 feet. The outlet channel is aligned with the spillway. The channel narrows to a width of 400 feet in the first 1100 feet, and the channel bottom slopes up from Elevation -3.0 feet to Elevation 14.0 feet, at 5 horizontal to 1 vertical immediately downstream from the structure.

A study of scour in the outlet channel was made on the spillway model. (See Figures F and G). The scout patterns appearing on the model test were satisfactory. Removal of the dentated sill on the model of the stilling basin greatly increased scout at the end of the basin and is a good indication of the efficiency of the sill as designed. The foundation sand in the outlet channel was protected by riprap for a distance of 200 feet downstream from the dentated sill.

For a large percentage of the time, releases will be made with the gates partially open rather than with free flow over the crest. The scour studies for various combinations of partial gate opening indicated less erosion with all 16 gates at equal openings than for any other combination.

G. Model Study

The spillway model was constructed following the basic pattern of the design as shown in the contract drawings.

Tests of the final spillway model were satisfactory in all respects. Several changes were made, however, in the final design of the prototype, which were not tested on the model. It is necessary, therefore, to discuss these changes and to analyze their effect on the model results.

1. Inlet Walls - These walls retain the backfill just upstream from the crest structure. In the model, they curved away from the spillway centerline and extended into the inlet channel side slopes. In the final design, the inlet walls are straight and aligned with the crest structure side walls. The height of the wall varies with the depth of the backfill. The original alignment was not used because of structural considerations. The present alignment will interrupt the smooth flow along each bank of the inlet channel and will reduce the discharge of the end bays somewhat. Considering the total width of spillway involved, the percent decrease will be very small. This alignment change was built into the model for study of diversion flows, so that its effect during diversion was correctly shown by the model.

2. Crest Shape - The crest shape of the final design differs in several respects from the shape tested on the model. The final crest is at Elevation 80.25 feet, 2.25 feet above the original design. The crest profile is somewhat sharper than the original design. The change in crest elevation has no effect on the discharge coefficient. The adoption of a sharper crest profile will increase the discharge coefficient somewhat. It is expected that the resulting increase in discharge will more than offset the losses due to the inlet wall change, since this increase will affect the entire spillway.

3. Pier Shape - The piers of the final design differ from the model in several respects.

The piers on the final design are 6 inches thicker. This change will have negligible effect on the discharge.

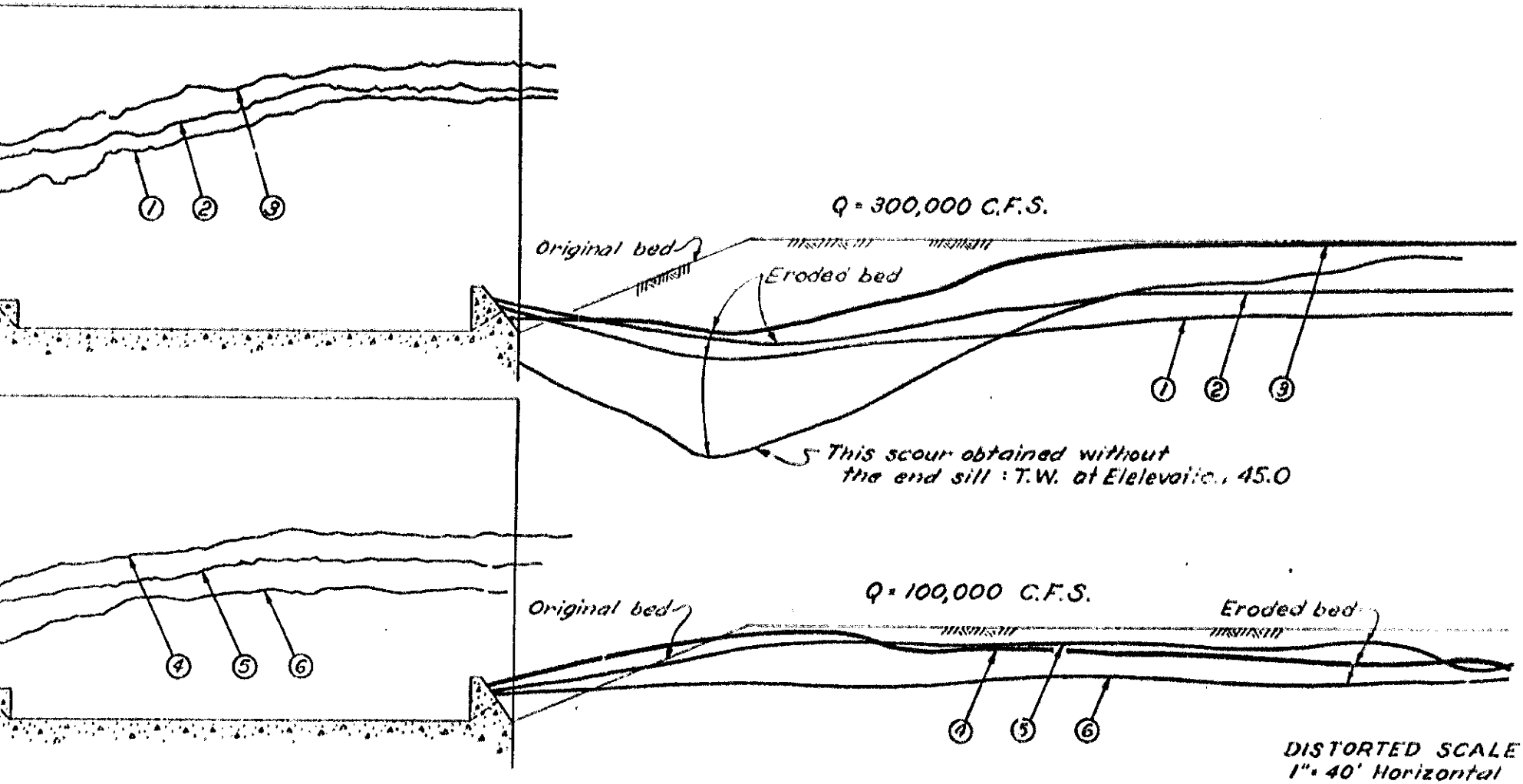
The upstream end of the piers on the final design are vertical, while the model piers were inclined. This change will also have no effect on the discharge.

4. Chute and Stilling Basin - The only difference between the original design and the final design of the chute and stilling basin is an increase of 7.5 feet in width. The hydraulic effect of this increase will be negligible.

2.03 STRUCTURAL DESIGN

A. Design Criteria and Assumptions

The reinforced concrete structures in the spillway were designed using the following criteria and assumptions:



NOTE:

A) Bed screeded to original contour before each test.
B) Each test run until scour stability is reached.
C) Profiles of hydraulic jump are mean approximations from visual observations.

LEGEND:

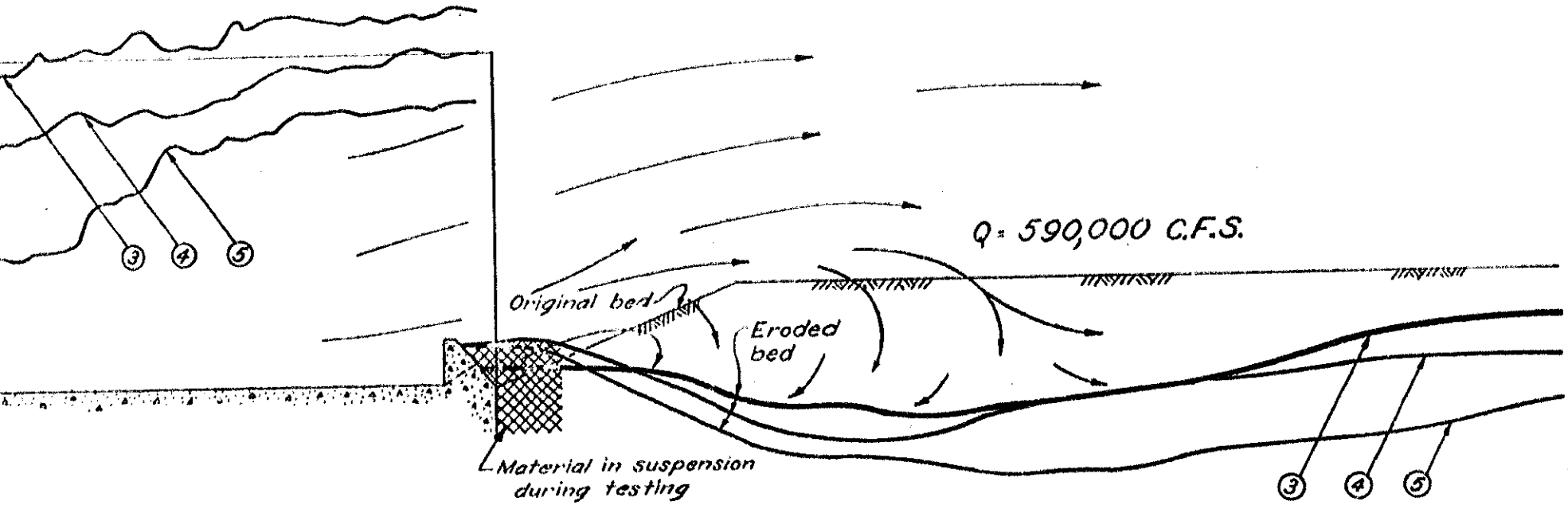
1. T.W. at Elevation 39.00
2. T.W. at Elevation 42.00
3. T.W. at Elevation 45.00
4. T.W. at Elevation 30.00
5. T.W. at Elevation 26.00
6. T.W. at Elevation 22.00

KARNAFULI PROJECT

SPILLWAY
CHANNEL PERFORMANCE WITH VARIABLE TAILWATER

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. HW	RECOMMENDED	APPROVED
CK. [illegible]	[illegible]	[illegible]
DATE: 2-4-59	FIG. F	



NOTE:

A) Bed screeded to original contour before each test.
B) Each test run until scour stability is reached.
C) Profiles of hydraulic jump are mean approximations from visual observations.

LEGEND:

(3) T.W. at Elevation 58.0

(4) T.W. at Elevation 54.0

(5) T.W. at Elevation 52.0

DISTORTED SCALE
1" = 40' Horizontal
1" = 20' Vertical

KARNAFULI PROJECT

SPILLWAY
CHANNEL PERFORMANCE
WITH VARIABLE TAILWATER

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

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DATE: 2-4-59	FIG. 6	

1. Loads

a. Dead Loads - The dead loads shall consist of the weight of all the permanent stationary construction entering into or becoming a part of the final structure.

b. Live Loads - Assumed live loads are as follows:

Location	Live Load
Gate Hoist Acess Bridge	AASHO H20 truck loading
Gate Hoist Deck	Hoist Load (discussed in paragraph D). plus 100 pounds per square foot of deck area.
Gate Anchorage	Prestressing loads and gate arm loads as discussed in paragraphs E and F

c. Seismic Loads - Seismic loads shall be equal to 10% of the dead weight of the structure and any backfill resting on the structure or adjacent and in a position to affect the structure applied horizontally in any direction at the center of gravity of the mass.

Seismic loads on the structures from water shall be calculated using Westergaard's parabola: $x = \frac{7}{8}\sqrt{Hy}$ (See Figure J) where:

x = distance of water away from the structure, which can be assumed to act with the structure during horizontal seismic acceleration.

H = depth of water.

y = distance from the surface.

When $y = H$, the total volume of water enclosed by the parabola can be expressed as $V = \frac{7}{12} H^2$ whose weight, $W = (62.4 \frac{7}{12} H^2)$, or

$W = 36.5 H^2$. Applying a seismic acceleration of α expressed with respect to gravitational acceleration, the force due to earthquake exerted by the water on the structure is $Pe = (36.5 H^2)\alpha$. This force acts at 0.4H, the center of gravity of the parabola quadrant.

For short reservoirs, between the piers for example, the total force exerted by water on the structure can be written $Pe = \beta (36.5H^2)\alpha$, where β is given on the chart in Figure J.

d. Combined Loadings - The various components of the spillway shall be designed to sustain at normal stresses the maximum dead load and normal live load to which they may be subjected. Where stresses due to seismic loads are combined with those due to static loads, the permissible working stresses may be increased 33-1/3 percent. For structural loads induced at stall torque of the hoist motor, working stresses may be increased by 50 percent.

2. Allowable Stresses

a.

Concrete Class	Unconfined Compressive Strength in pounds per square inch at 28 days
A	3000
B	2500
High Strength	4500

b. Reinforcement - Reinforcement shall be intermediate grade billet steel conforming to ASTM Designation A-15. Deformations shall conform to ASTM Designation A-305. Normal working stresses shall not exceed 20,000 pounds per square inch.

c. Foundation Pressures - Allowable bearing pressure on shale = 15,000 pounds per square foot.

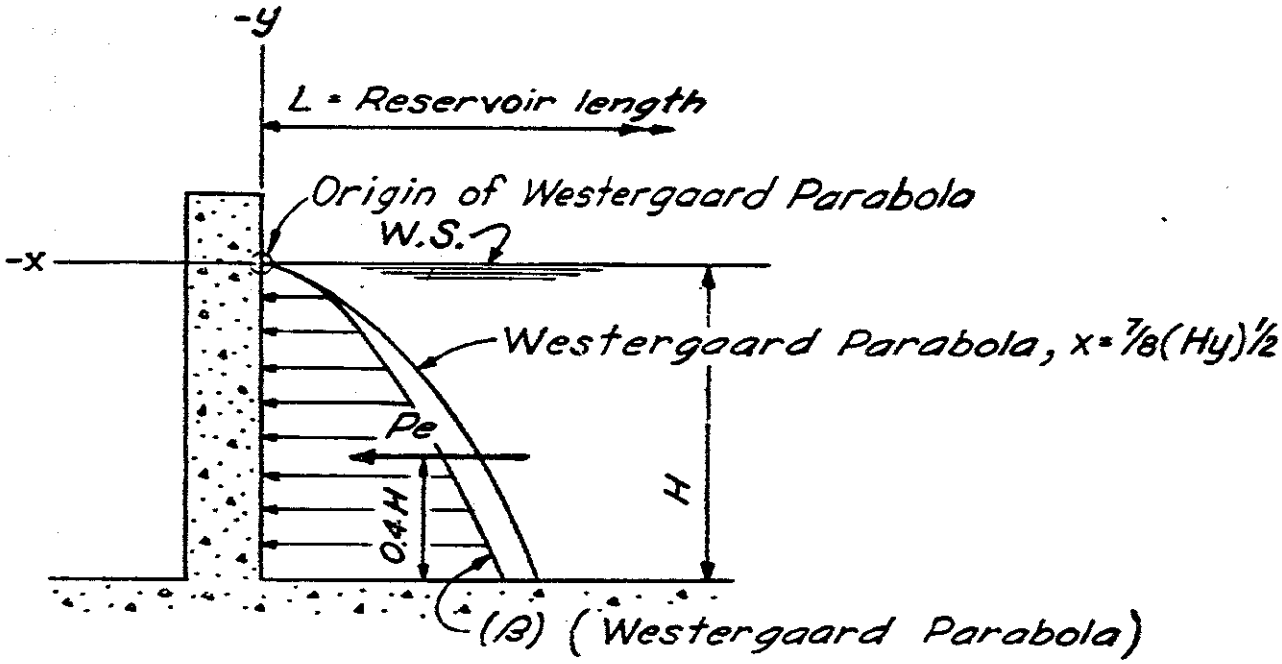
Allowable bearing pressure on confined sand = 12,000 pounds per square foot.

Allowable foundation bearing pressures may be increased by 1/3 for seismic loads.

d. Stability

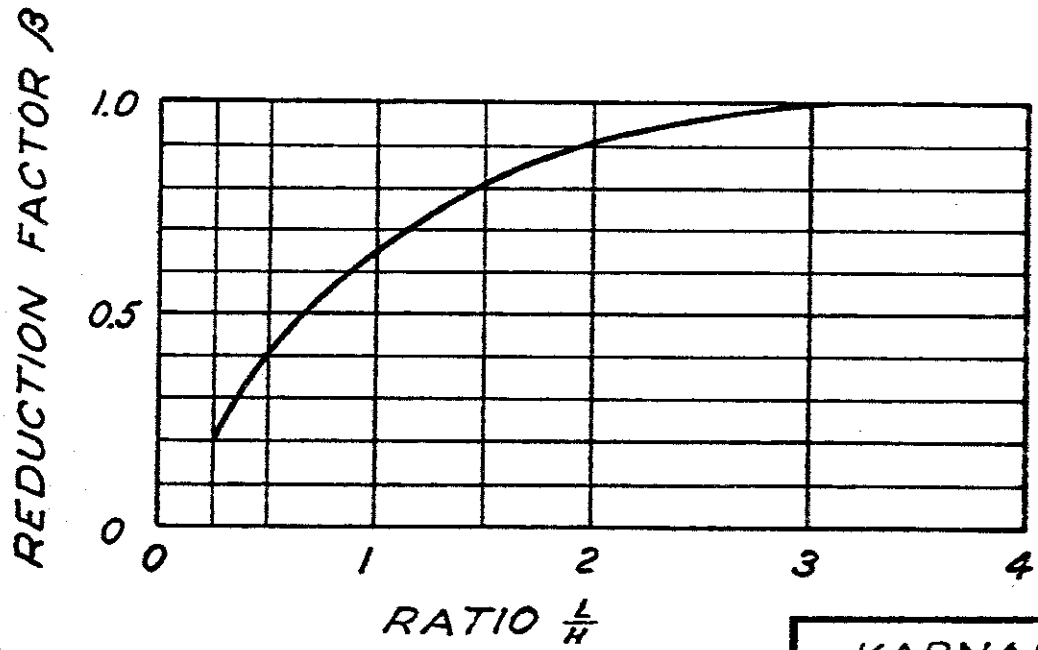
(1)	Maximum sliding factor	0.35
(2)	Maximum sliding factor for seismic loads	0.45
	The sliding factor is defined as the ratio of the net horizontal forces on the structure to the net vertical forces on the structure.	
(3)	Minimum safety factor against overturning	2.0
(4)	Minimum safety factor against overturning for seismic loads	1.5

DYNAMIC WATER PRESSURES DURING EARTHQUAKE



NOTE:
Pe can act either direction from the wall.

REFERENCE:
Paper No. 1835 and discussions,
Trans. ASCE, Vol. 98



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DR. HW / CK. W.R.S.	RECOMMENDED	APPROVED
DATE 2-4-59	Figure 1	

The safety factor against overturning is defined as the ratio of the resisting moment to the overturning moment.

(5) The resultant of all loads must fall within the middle 1/3 of the base.

3. Weights of Materials

Plain and reinforced concrete	150 #/ft^3
Water	62.4 #/ft^3
Dry backfill, except impervious core	110 #/ft^3
Saturated backfill, except impervious core	130 #/ft^3
Dry backfill, impervious core	115 #/ft^3
Saturated backfill, impervious core	135 #/ft^3

4. Friction Angles

Backfill Internal friction (ϕ)	30°
Backfill to excavated shale	15°

5. Standards - The design of the reinforced concrete structures shall conform to the following standards: "Building Code Requirements for Reinforced Concrete (ACI 318-56)" of the American Concrete Institute, and "Standard Specifications for Highway Bridges," Sixth Edition, of the American Association of State Highway Officials.

B. Methods of Analysis

1. General - Foundation pressures were calculated by finding the resultant of the weights and horizontal loads on a 1-foot wide strip, and applying the formula $p = \frac{P}{A} \pm \frac{m}{s}$, where:

p = foundation pressure at either edge of the base slab, in pounds per square foot.

P = net weight, in pounds.

A = area of a 1-foot strip of the base width, in square feet.

m = moment of the resultant of all forces acting on the structure about the midpoint of the base width, in foot pounds.

s = section modulus, $\frac{bd^2}{6}$, in cubic feet, where b equals 1 foot, and d equals the width of the base, in feet.

Intermediate foundation pressures were assumed to vary as a straight line between the edge pressures.

2. Retaining Walls Stability - For retaining walls, the backfill loads were determined by dividing the backfill into two wedges (see Figure K), the active wedge and the inert wedge. The inert wedge is described as the triangle of backfill whose base is the heel slab and height is the back face of the retaining wall. The active wedge is the remaining wedge, one side of which is the top surfaces of the backfill, and the other is the excavated shale slope. For all of the walls investigated, the shale slope was assumed to extend up from the edge of the heel slab at a slope of 1 to 1. This was a conservative design assumption.

The weight of the inert wedge was applied on the heel slab. The method of applying the loads from the active wedge is shown diagrammatically on Figure K. Two surfaces, the excavated shale slope and the assumed sliding plane between active and inert wedges, were assumed to support the weight of the active wedge. The direction of the supporting forces is a function of the angles of the supporting surfaces and the appropriate friction angles. The amount of the supporting force is such that the force diagram forms a closed triangle. Only the force acting on the inert wedge was assumed to affect the retaining wall. This force was resolved into horizontal and vertical components and applied to the inert wedge at a point 0.4 of the wall height above the base. For seismic loading, a force polygon was similarly constructed, with the addition of a horizontal load equal to 0.1 weight of the active wedge. The difference between the seismic and normal loadings, which was considered to be the seismic contribution from the active wedge, was applied at a point 2/3 of the wall height above the base.

3. Internal Structural Loads - For design of the toe slab, the difference between foundation pressures and weight of the structure was taken as a load to be resisted in shear and bending. The counterforts were designed as T-beams, cantilevered from the base slab, to resist moment and shear caused by horizontal loads on the stip of wall supported by the counterfort. Web reinforcement was not considered to contribute to the moment reinforcement requirements.

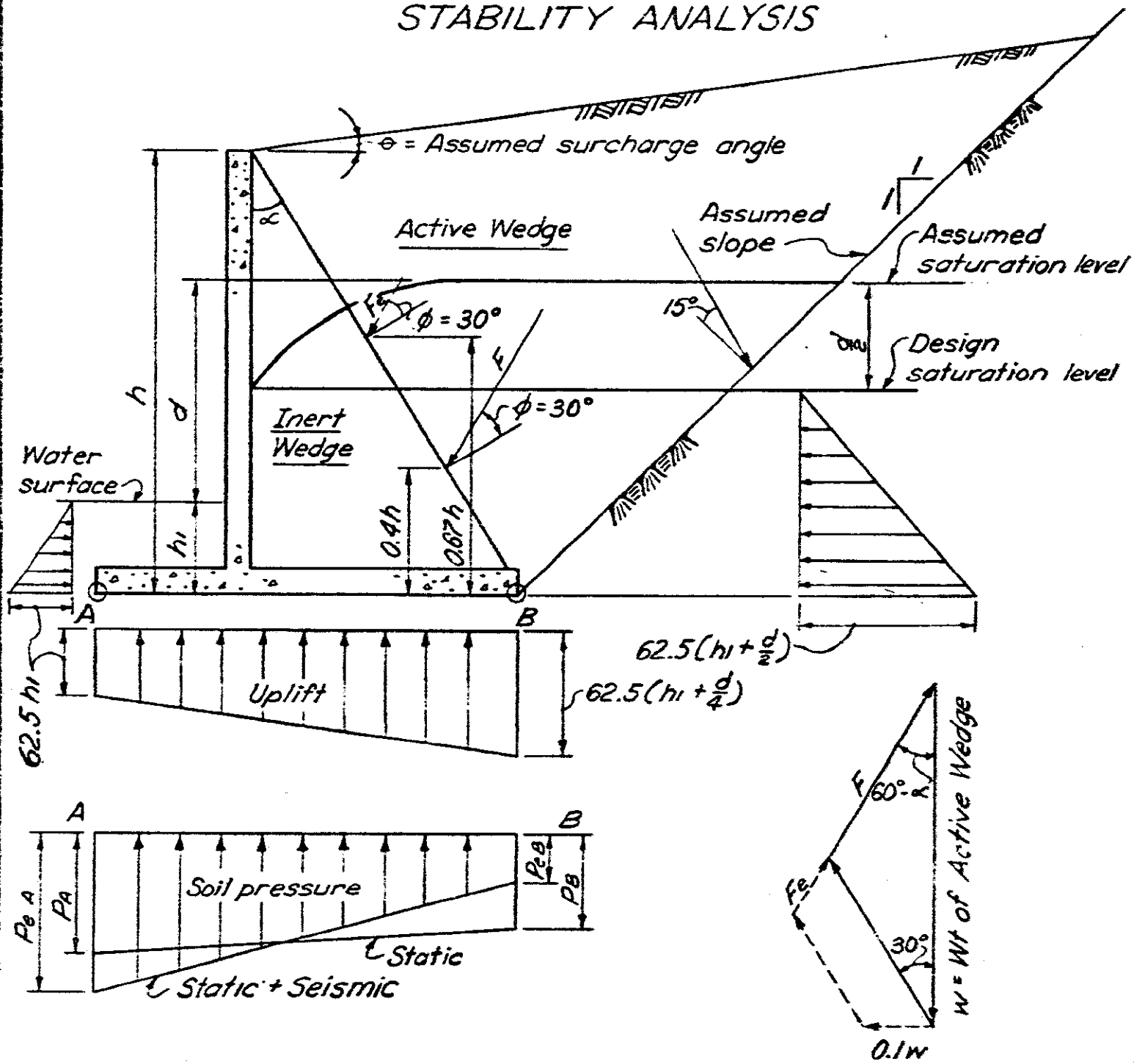
The wall and heel slab were designed as fixed-end beams supported at the counterforts. For local design loads, horizontal and vertical components from the active wedge were assumed to be equivalent triangular loads.

As nearly as practicable, counterforts were spaced so that the moments in the walls and slabs on each side of the counterforts were balanced.

C. Inlet Wall

The purpose of the inlet wall is to retain the sloping backfill upstream from the abutment walls. The base of the inlet wall is level at Elevation 51.00 feet, the elevation of the inlet channel floor. The top of the wall lies on a slope of 2 horizontal to 1 vertical and is flush with the surface of the backfill. The height of the wall varies uniformly from

STABILITY ANALYSIS



FORCE POLYGON

Note:
P — pressure
F — force from active wedge
e — due to earthquake

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DR. R.H.G. CK. NRS	RECOMMENDED	APPROVED
DATE: 2-4-59	FIG. K	

0 feet to 68 feet over a length of 136 feet. The base width was varied from a 2-foot toe and 2-foot heel at the upstream end to a 10-foot toe and 40-foot heel at the downstream end adjacent to the abutment wall.

The inlet wall was separated by contraction joints into three sections of approximately equal length. The upstream section was designed as a cantilevered wall. The other two sections were designed as counterforted walls. Each section of wall was analyzed for stability for the dry condition, (immediately after construction) and for partially unbalanced hydrostatic lateral loading. Results of stability analysis for the dry condition, which were found to be critical, are given in Figure L.

A three-foot deep cutoff wall was placed below the toe of the slab to safeguard the foundation of the inlet walls from undermining.

D. Crest Structure - Pier Blocks

1. Pier Blocks - Fifteen identical pier blocks are each 47 feet wide with the pier located on the centerline. The functions of the pier blocks are:

Stage I

a. To protect the foundation from erosion, and provide a crest at Elevation 51.00 feet for diversion flows.

Stage II

a. To provide a crest at Elevation 80.25 feet, equipped with gates for regulation of spillway discharge.

b. To provide for gate trunnions and anchorages and gate hoists.

c. To provide stoplog slots.

d. To provide a seat for the gate hoist access bridge and for a future bridge.

2. Crest - The shape chosen for the spillway crest was determined, as a function of hydraulic and stability requirements, to be the most economical. Shapes with flatter and steeper upstream slopes and with an ogee downstream curve were also investigated. The design of the curved surface at the crest and gate sill is discussed in paragraph C, Section 2.02 of this chapter. A fifteen foot deep cutoff wall was placed at the upstream edge of the pier block to protect against undercutting, increase the path of percolation, and also act as a positive key against sliding on the foundation. The downstream edge of the crest block was thickened to provide a minimum depth of five feet for the Stage I slab and also leave room for five feet of Stage II concrete.

3. Pier - The design of the pier was based on the following considerations:

a. A minimum pier width of 7 feet was required to encase the gate anchorage and withstand the prestressing loads. The gate anchorage is discussed in Chapter II, PART V.

b. With the location of the gate sill known and the radius of the gate selected at 35 feet, the gate trunnion was set at Elevation 92.50 feet to minimize vertical forces on the gate trunnion and keep it above the upper nappe during maximum discharge.

c. The hoist decks were located so that a vertical line from the hoist drum approximately bisects the angle between the positions of the hoist cable at gate full open and at gate closed. This ensures that horizontal loads on the hoist will be minor.

The surface of the hoist decks was set at Elevation 127.00 feet, which is convenient for access, and high enough to clear the gates in any raised position.

d. The stoplog slot was located 3 feet upstream from the spillway gate, where the surface of the crest is fairly level.

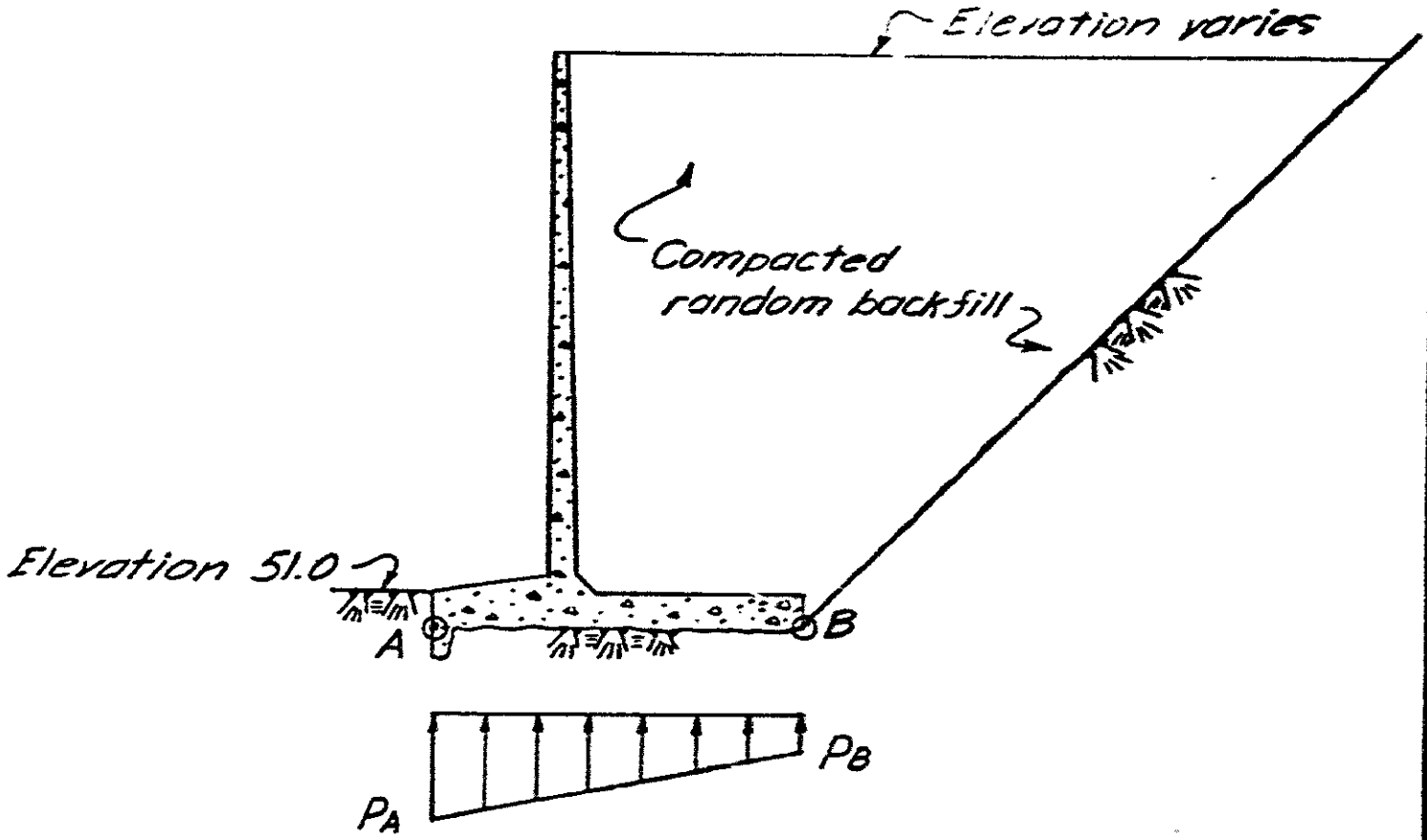
e. The bridge was placed just upstream from the stoplog slot, leaving 1 foot of clearance. This is sufficient clearance for handling the stoplogs, which must be brought along the bridge, lifted over the handrails, and lowered into position. The piers were designed to accommodate two 10-foot wide bridges, the upstream bridge to be constructed in the future. The upstream 4.5 feet of bridge seat was designed as an overhang to reduce the volume of concrete required for the piers. The bridge seat was set at Elevation 123.37 feet, with 1.5-inch deep slots provided to receive the bridge girders.

4. Stability Analysis - Loads investigated and results obtained in stability analysis of the spillway crest block are given in Figures M, N, and O.

The structure was analyzed for stability considering two possible sliding planes, the first being the plane between shale and concrete, the second being an inclined plane from the bottom of the cutoff wall to the downstream toe of the structure. Analysis of the structure for either sliding plane gave results within the limits of the design criteria.

A complete stability analysis was made for the pier block when adjacent gates are open and water discharges freely on each side of the pier, (see Figure 0). Overturning safety factors were found to decrease but are still well within the allowable. For this analysis, hydrostatic pressures on the crest surface were calculated using pressure measurements from models tested by the United States Bureau of Reclamation (USBR) and given in Boulder Canyon Project Report, PART IV, Bulletin 3.

RESULTS OF STABILITY ANALYSES
INLET WALLS



Station	Loading Condition	Foundation Pressure in Kips/ft² P_A	P_B	Sliding Factor	Overturning Safety Factor
7+35	Static	3.4	1.4	0.35	2.9
	Static + Seismic	4.8	0.3	0.45	2.2
7+80	Static	6.8	3.4	0.32	3.5
	Static + Seismic	9.6	1.4	0.39	2.6
8+25	Static	10.4	5.3	0.30	3.9
	Static + Seismic	13.5	3.3	0.36	2.8

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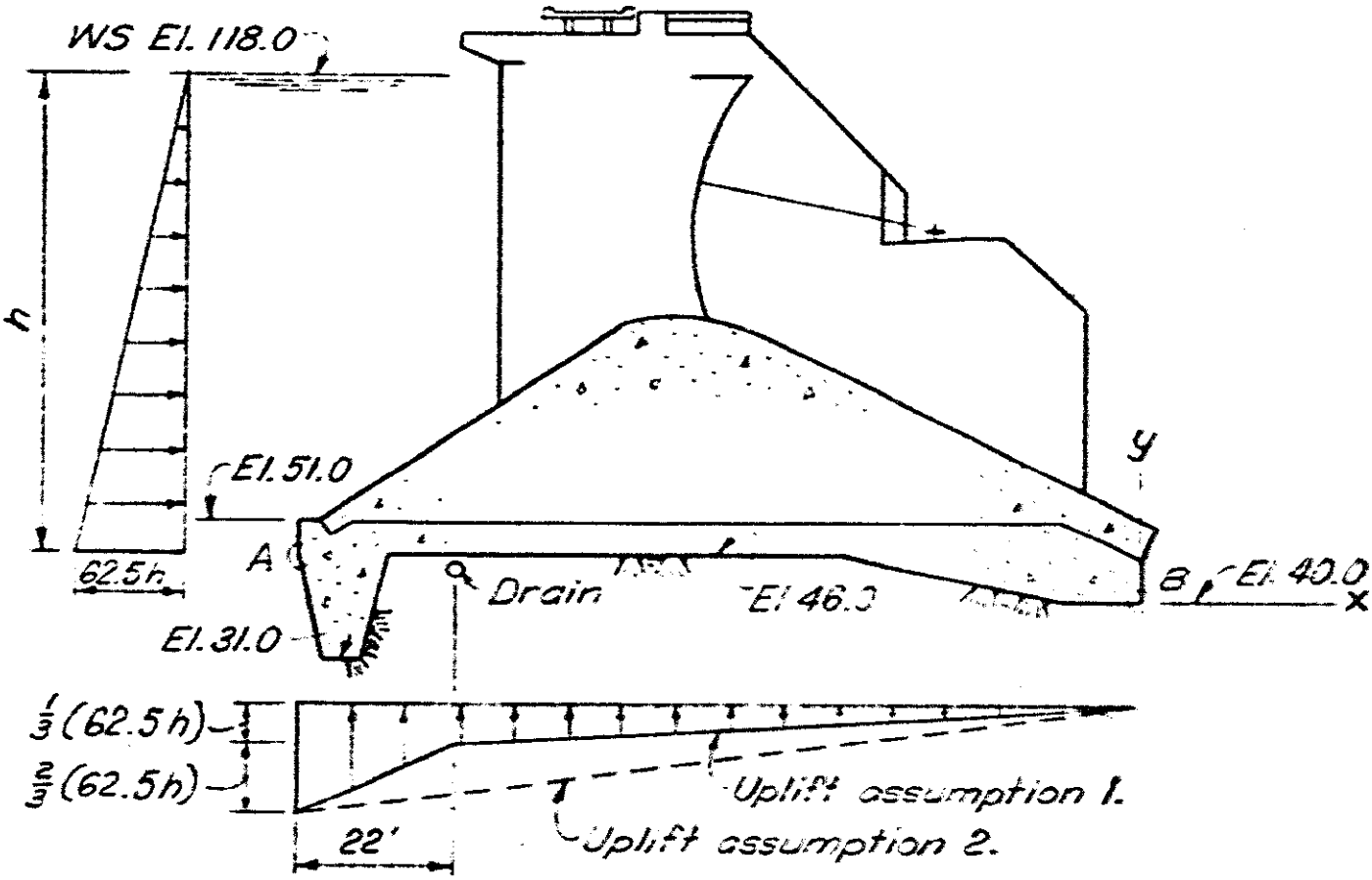
DATE: 2-4-59 | FIG. L

LOADS AND RESULTS
SPILLWAY CREST STRUCTURE-PIER BLOCK STABILITY ANALYSIS



Uplift assump.	Case	Sliding factor	Foundation Pressure, kips/ft² A	B	Overturning Safety Factor
1.	I	0	4.1	4.1	—
	II	.10	3.7	4.6	—
	III	.32	2.9	5.5	2.67
	IV	.44	2.5	6.3	2.41
2.	III	.42	1.5	5.1	1.73
	IV	.58	0.8	5.9	1.63

Case
- I Dry
- II Dry & Earthquake 0.1g
- III Res. W.S. at El. 118
- IV Water + Earthquake 0.1g

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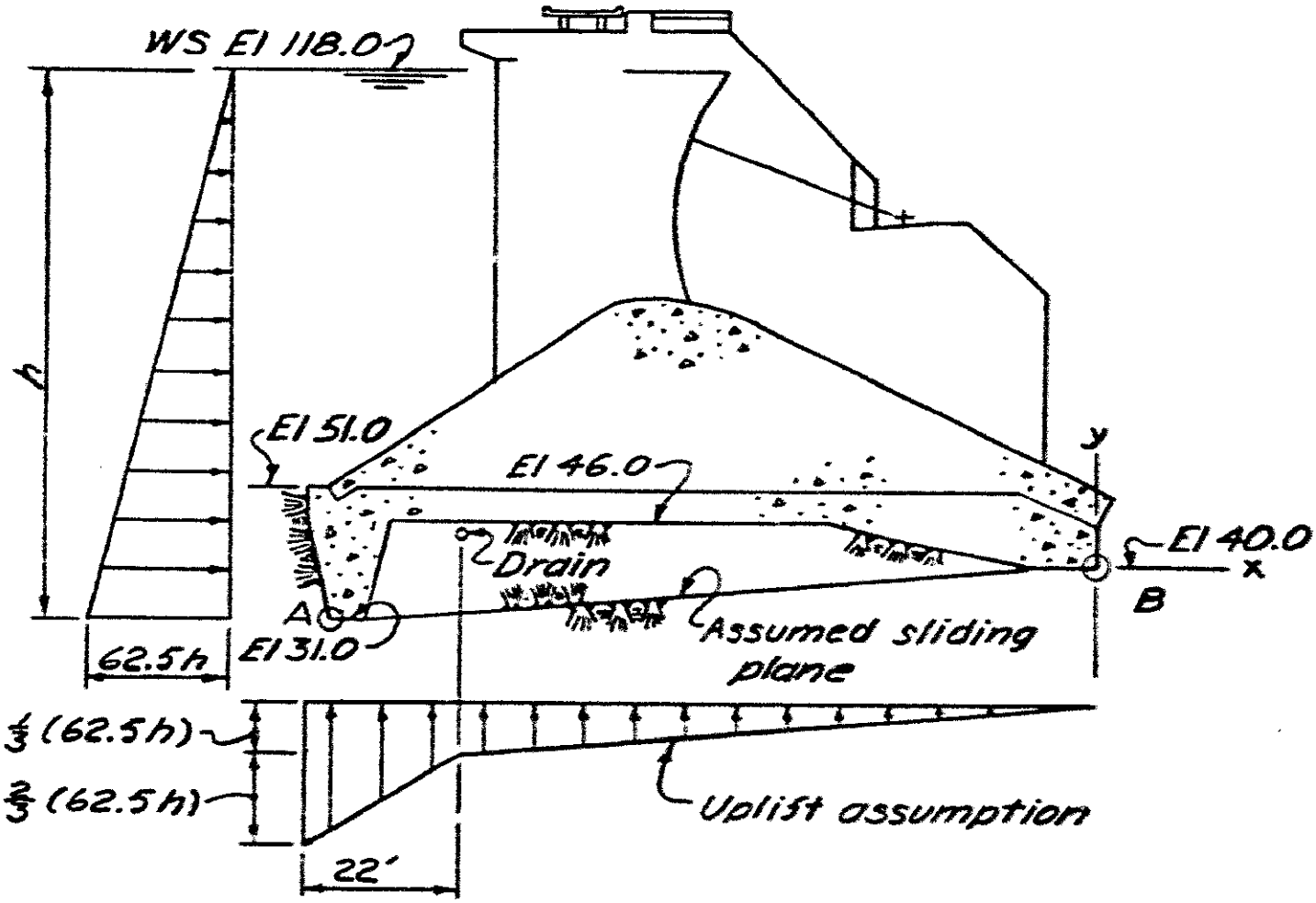
CK. W.R.S.		
DATE 2-4-59	FIG. M	

SPILLWAY CREST STRUCTURE - PIER BLOCK STABILITY ANALYSIS



Case	Sliding Factor	Foundation Pressure, kips/ft² A	B	Overturning Safety Factor
I	0	5.5	4.5	—
II	0.10	6.0	4.0	—
III	0.28	3.8	6.6	2.74
IV	0.39	2.9	7.6	2.51

Case
I - Dry
II - Dry + Earthquake (0.1g)
III - Res. W.S. at El 118.0
IV - Water + Earthquake (0.1g)

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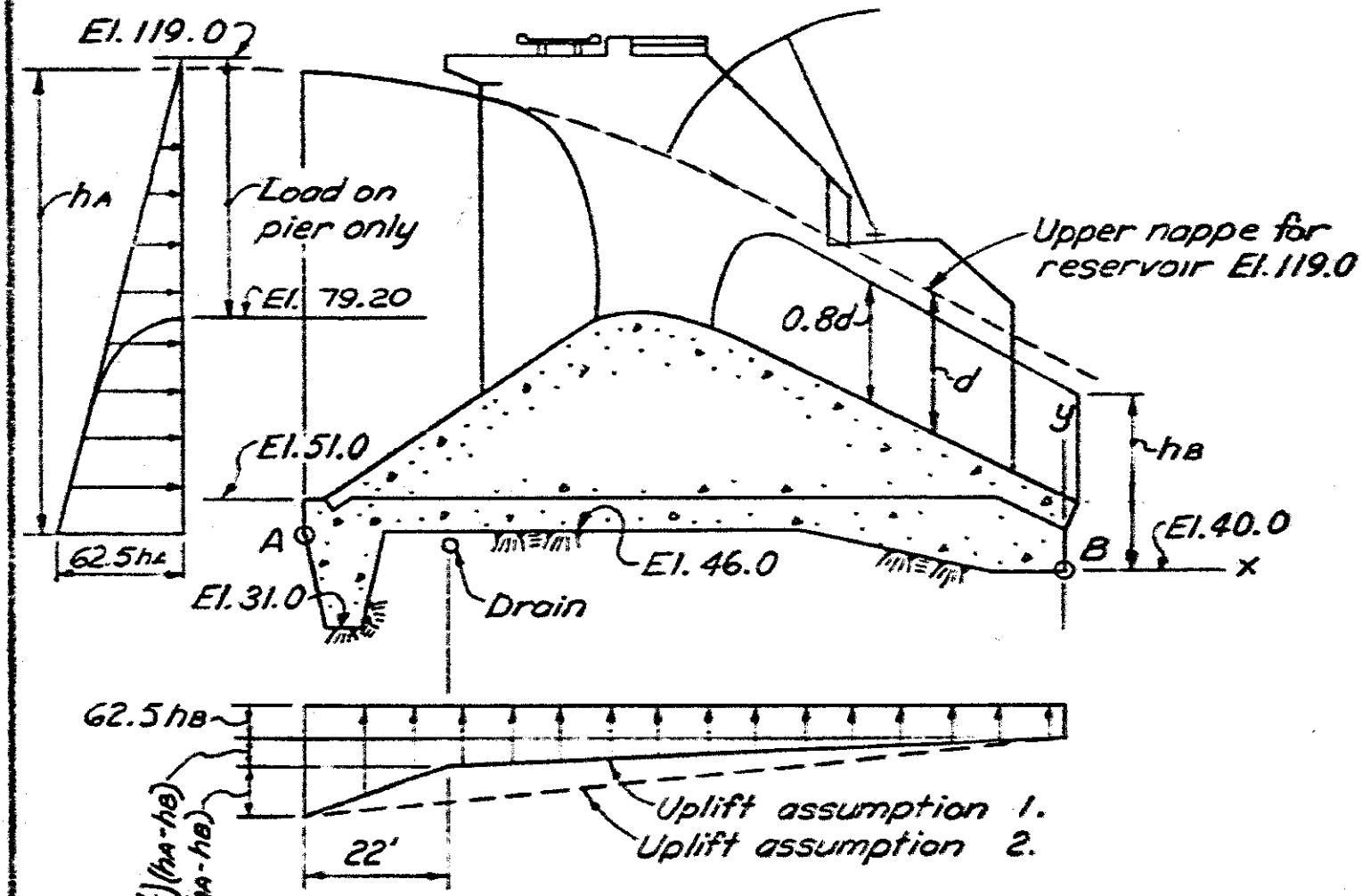
DATE: 2-4-59 | FIG. N

STABILITY ANALYSIS



Uplift assump.	Case	Sliding factor	Foundation Pressure, Kips/ft² A	B	Overturning Safety Factor
1.	III	.21	3.1	3.6	2.05
	IV	.35	2.5	4.2	1.92
2.	III	.25	2.0	3.3	1.64
	IV	.44	1.4	3.9	1.56

Case

I Dry
II Dry + Earthquake (0.1g)
III Res. W.S. at El. 119.0
IV Water + Earthquake (0.1g)

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DATE: 2-4-59 | FIG. 0

a. Horizontal loads on the upstream slope of the crest section were calculated using the pressures above, applied on a vertical projection of the slope.

b. Horizontal hydrostatic loads on the upstream end of the pier were taken as equal to static pressure of water standing to the depth of the hydraulic gradient, in this case assumed to be at Elevation 119.0 feet.

c. Downstream from the crest, vertical pressure equal to 80 percent of the depth of the water was applied on the downstream slope.

d. Horizontal pressures on the downstream slope were not considered.

5. Pier Analysis - The piers are subject to lateral bending due to seismic loads and unbalanced gate openings. Hoist loads also cause bending but their effect is so small at the base of the pier that they are negligible. The loading conditions investigated are given on Figure P. Seismic loads were critical in all of the cases investigated.

The pier was checked for shear on a plane along the downstream slope of the crest. The shear was found to be less than 40 pounds per square inch, and therefore safe.

6. Crest - The crest is quite massive. Class B concrete was used for the core of the crest, inside a protective shell of Class A concrete.

The crest was investigated for stability and shear at each end of the pier, using the foundation loads obtained in the stability analysis, (see Figures M, N, and O).

The crest was checked for overturning and horizontal shear at the construction joint at Elevation 51.00 feet, (see Figure Q). Since this construction joint was to be exposed for two monsoon seasons before Stage II construction, lateral keyways were used to help resist horizontal shear. Keyways near the downstream end of slab were run longitudinally, perpendicular to the direction of principal shear stresses in that area.

7. Gate Anchorage Loads - Gate anchorage loads are discussed in Chapter II, PART V. Loads due to prestressing the gate anchorage steel affect the pier in several ways. The pier was checked and found to be safe against buckling. The tendency of the prestressing load to crack the pier at the prestressed steel upstream support was investigated. Reinforcement was used to tie the nose of the pier firmly across the support plane so that the pier would act as a unit while subject to the prestressing load.

At the downstream support, which is the box girder, the bearing area is small, resulting in high compressive stress in the pier concrete. High stress due to eccentric loading will occur for some distance upstream from the box girder. High-strength (4500 pounds per square inch) concrete was used in this area to prevent crushing. The concrete surrounding the box girder was designed to withstand a torsional moment of 320 kip-feet, because of the high loadings during the prestressing operation, full 28 day strength was required before prestressing.

8. Hoist Deck - The hoist deck is 13 feet long and cantilevers out from each side of the pier a distance of 8 feet, to allow sufficient space for the gate hoists. The hoist deck was designed to sustain the breakdown hoist lifting load of 412 kips entirely on one hoist deck.

9. Miscellaneous - Temperature reinforcement was used over the entire exposed surface and part of the lower surface, at the cutoff wall and at the downstream end, of the Stage I slab. Temperature reinforcement was used on the slopes of the crest at each end. Stage II temperature reinforcement was made continuous with Stage I reinforcement.

E. Crest Structure - Abutment Walls

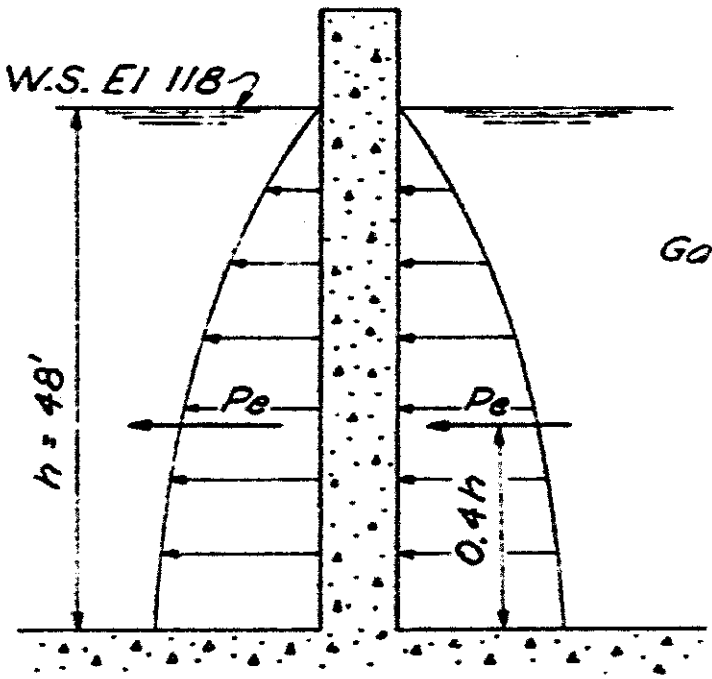
1. General - The abutment walls are located at each side of the spillway crest structure and are basically retaining walls. In addition, each must perform the functions of one-half of a pier block described in paragraph D. The locations of the spillway gate, stoplog slot and bridge seats are the same with respect to the axis as the location of these respective features on the pier. The abutment wall toe length was fixed at 20 feet, one-half the clear distance between piers, so that all pier blocks would be identical.

Since the toe of the abutment wall is a part of the spillway crest, its shape, including cutoff wall, is the same as the pier block crest shape. For this reason, the length of the abutment wall was made equal to the length of the pier block. The abutment wall thickness was set at 4 feet to provide sufficient thickness to encase prestressing steel for one gate anchor and to resist prestressing loads and backfill loads.

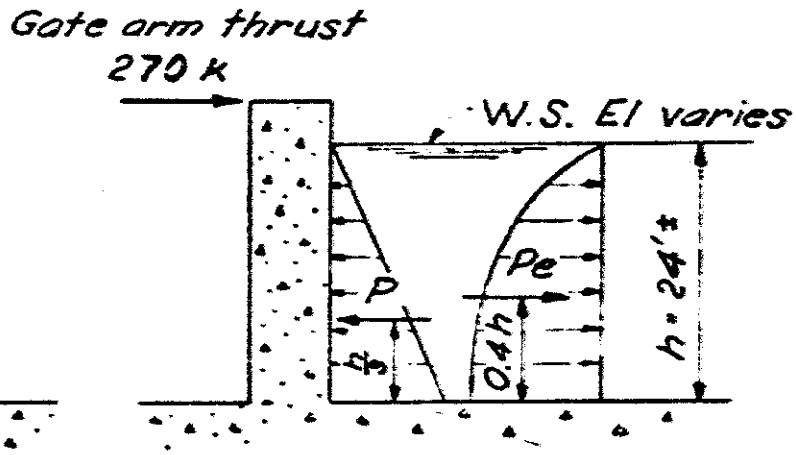
The abutment wall was designed to be constructed in two stages. Stage I consists of a 5-foot thick toe and heel at Elevation 51.00 feet, with the top of the wall at Elevation 76.00 feet. The backfill was also placed to Elevation 76.00 feet. During Stage II construction the abutment wall and backfill was placed to Elevation 127.00 feet at the roadway. The abutment wall was required, therefore, to be stable and structurally sound under two different sets of loading conditions.

2. Stability - With the toe length and height of wall established, the heel length of the abutment wall slab was selected to obtain suitable stability results. For the maximum height portion of the wall a heel length of 40 feet was found to be satisfactory. Since the wall height varies, it was possible to reduce the heel length at the downstream end of the

LOADS
SPILLWAY PIER



STATION 8+57
BOTH GATES CLOSED



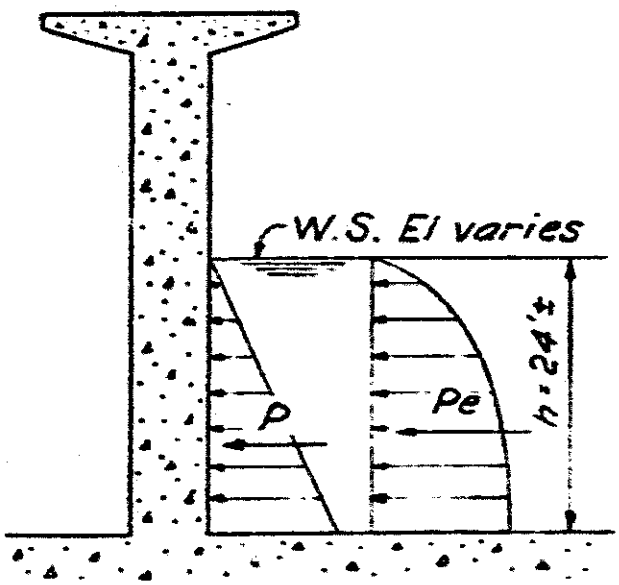
STATION 9+15
ONE GATE OPEN

NOTE:

Seismic condition was critical for all lateral pier loads.

Pe equals resultant of hydrostatic pressures due to earthquake. See Figure J.

P equals resultant of unbalanced hydrostatic pressure.



STATION 8+85
ONE GATE OPEN

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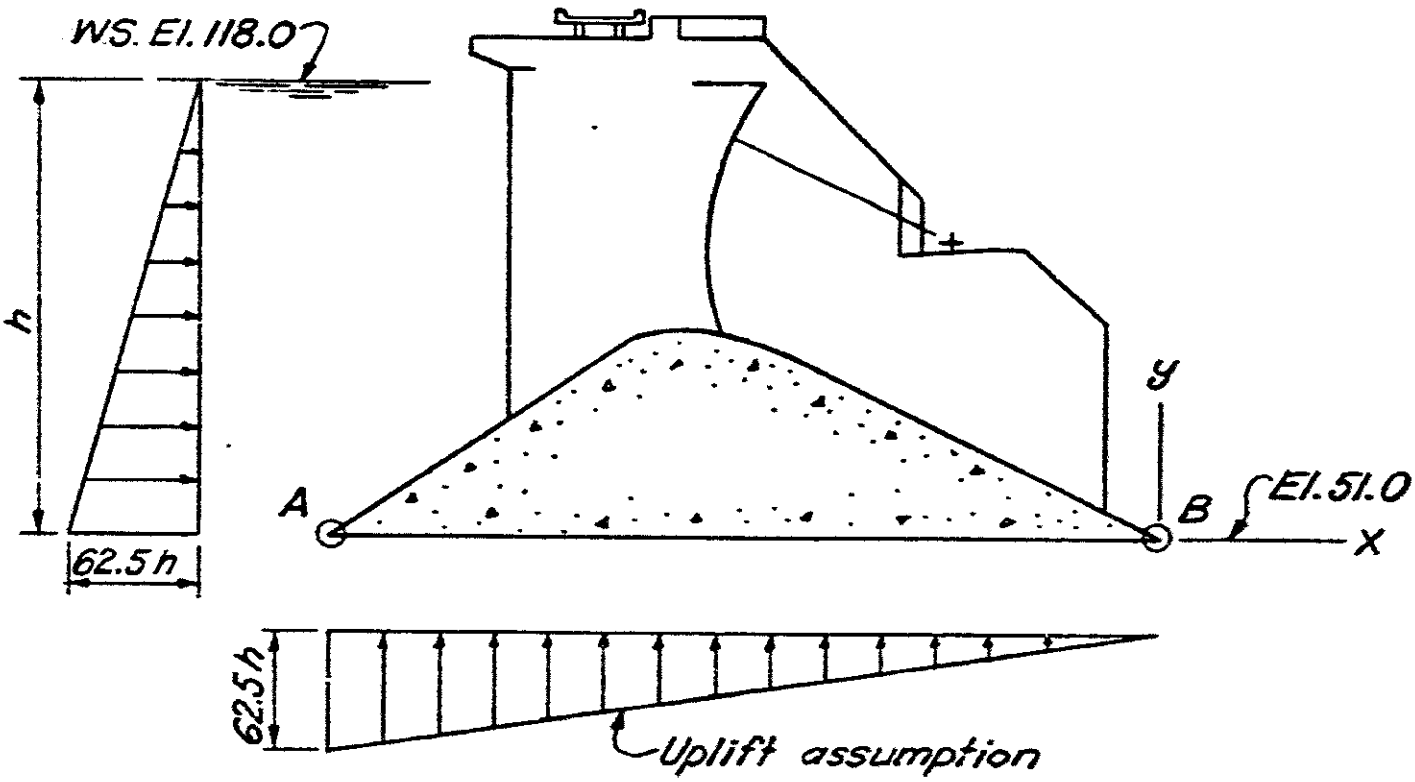
LOADS AND RESULTS
SPILLWAY CREST STRUCTURE-PIER BLOCK
STABILITY ANALYSIS



Case	Sliding factor	Compressive Stress, psi A	Compressive Stress, psi B
I	0	23.5	19.5
II	.10	25.6	17.3
III	.52	10.7	22.1
IV	.71	7.3	25.5

Case

I – Dry
II – Dry + Earthquake (0.1g)
III – Res. W.S. at El. 118.0
IV – Water + Earthquake (0.1g)

KARNAFULI PROJECT

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wall to 30 feet. The results of stability analysis at several stations are given in Figure R.

The impervious core and the upstream backfill were considered saturated to Elevation 118.0 feet. Downstream from the impervious core the backfill was considered to be free draining. Uplift pressure under the base slab at any station was considered to be equal to the uplift pressure under the pier blocks at the corresponding station.

The Stage I structure was checked for stability and found to be safe, see Figure S. A 15-degree backfill surcharge was used in the analysis but hydrostatic uplift was not considered.

3. Structural Design - After a partial investigation of a gravity wall, and thorough investigations of both cantilevered and counterforted walls, the counterforted walls were found to be most feasible. The counterforted wall required more reinforcement but much less concrete than the other types and did not require the construction of special cutoff walls into the impervious core. The wall was designed using the methods outlined in paragraph B of this section. A counterfort spacing of 18 feet center to center was selected. The toe shape, which was fixed by crest shape requirements, was found to be adequate for shear and bending under both Stage I and Stage II loadings. Reinforcement requirements in the central part was found to be governed by Stage I loadings, and near each end by Stage II loadings. Stage II loadings did not affect the central part near the crest because of the great depth of slab, up to 34 feet.

For the abutment wall counterforts, the point of maximum load from backfill was assumed to be at the elevation of the crest surface. The amount of counterfort reinforcement required at this level was continued down to the base slab. Hoist and gate arm loads were applied to the appropriate counterforts for design.

4. Prestress Loadings - Although the abutment wall gate anchorages are different in several respects from those in the piers, the effects of prestressing loads were similar. Only one gate anchor is located in the abutment wall, so that there is 4 feet of concrete per anchor instead of 3.5 feet as in the pier. The prestressing steel is longer in the abutment in order to reach the upstream end so the entire wall will act as a unit. The abutment wall was found to be safe from buckling for the prestressing load combined with the backfill load. Reinforcement was used at the upstream support to prevent cracking, and at the box girder anchorage to resist torsion. High-strength concrete (4500 pounds per square inch) was placed at the box girder, similar to the design of the pier.

5. Miscellaneous

a. Hoist Deck - The abutment wall hoist deck is identical to the pier hoist deck except that a 2-foot cantilever was used at the back of the wall instead of the regular 8-foot cantilever.

b. Construction Joints - The construction joints were spaced at vertical intervals of 5 feet and intercept the finished concrete surface at right angles. Special precautions were taken with the construction joint which separates Stage I and Stage II concrete since it was to be exposed for two monsoon seasons before continuing construction. Keyways were placed along the top of the Stage I slab parallel to flow and perpendicular to the direction of principal shear stresses. A 3-foot wide by 4-inch deep keyway was provided along the profile of the crest on the vertical face of the abutment wall. Below the keyway, 3-foot square waffles 4 inches deep were provided. The keyway and waffles were added to provide shear strength because it is essential that the Stage II crest concrete act monolithically with the Stage I slab.

F. Chute

1. General - The function of the chute is to carry spillway releases from the crest structure to the stilling basin. The chute is 74 feet long and 745 feet wide and lies on a slope of 2 horizontal to 1 vertical. The chute walls are 42.5 feet high as dictated by the backfill slope at the downstream side of the crest. The chute actually extends to Elevation -3.00 feet. The discussion of the lower portion is given in the following paragraph G. The chute floor was divided by longitudinal and transverse contraction joints into slabs 48.33 feet by 41.38 feet. The longitudinal contraction joints were aligned with those of the stilling basin.

2. Chute Slabs - The chute slabs were not designed to resist uplift pressures since an underlying pervious blanket and drain pipe system were installed for drainage. A cutoff at the upstream end of each slab was added to prevent the slab from sliding down the slope. A slab thickness of 1.5 feet was used. These slabs were nominally reinforced.

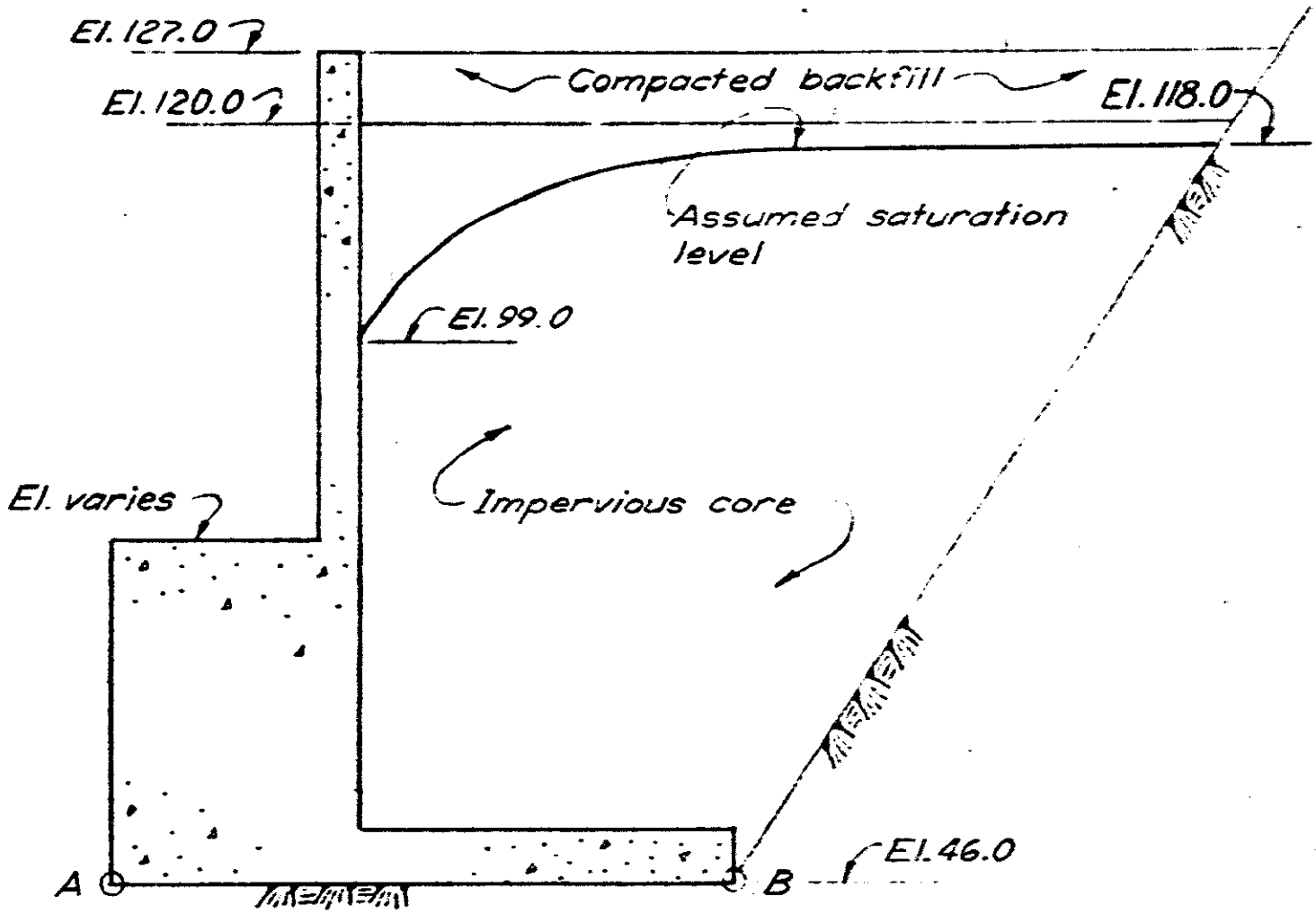
3. Chute Walls - The chute walls were designed using the methods outlined in paragraph B. The results of stability analysis for the chute walls are given on Figure T. No surchage was considered because the backfill surface is sloping and therefore will not readily be surcharged by sliding material. For the chute wall design, backfill saturation was conservatively assumed to be 18 feet above the water surface in the chute.

Cutoffs were placed along the upstream end of each wall slab, as on the chute slabs. A cutoff was also placed along the end of the toe as a stopping point for the pervious blanket and drain system. The drainage system was not extended under the toe of the chute wall.

G. Stilling Basin

1. General - The stilling basin is 745 feet wide between side walls and 211 feet long. This length includes 31 feet of the chute.

RESULTS OF STABILITY ANALYSIS
SPILLWAY ABUTMENT WALLS



Station	Loading Condition	Sliding factor	Foundation Pressure, kips/ft² A	B	Overturning Safety Factor
8+36	Static	.31	10.7	5.5	2.5
	Static+Seismic	.41	16.8	2.3	2.0
8+77	Static	.38	16.6	5.4	3.0
	Static+Seismic	.46	21.6	2.0	2.3
9+05	Static	.22	7.0	7.0	6.4
	Static+Seismic	.30	9.3	5.7	4.4
9+37	Static	.19	4.3	6.4	6.4
	Static+Seismic	.27	5.9	5.4	4.5

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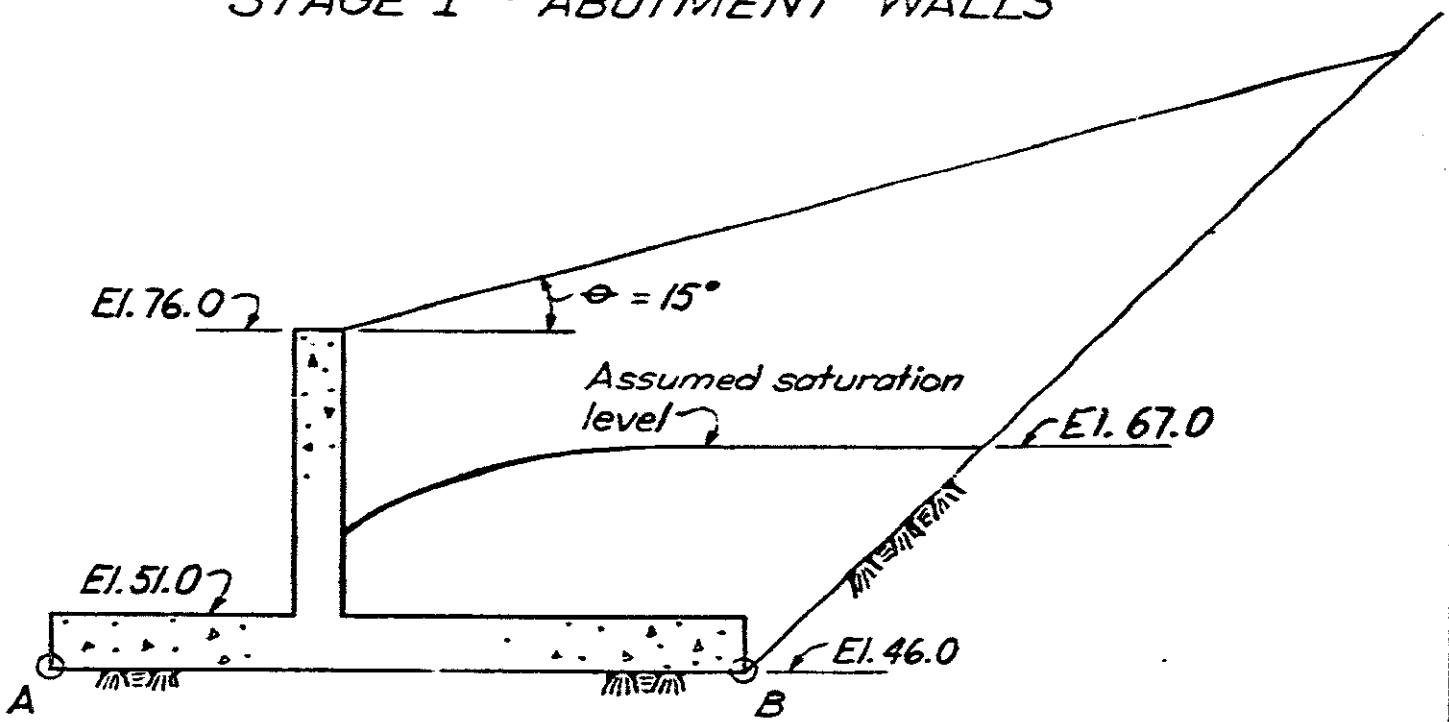
RESULTS OF STABILITY ANALYSIS
STAGE I - ABUTMENT WALLS



Loading Condition	Sliding Factor	Foundation Pressure, kips/ft.² A	B
Static	0.05	0.5	8.8
Static + Seismic	0.1	0.9	9.3

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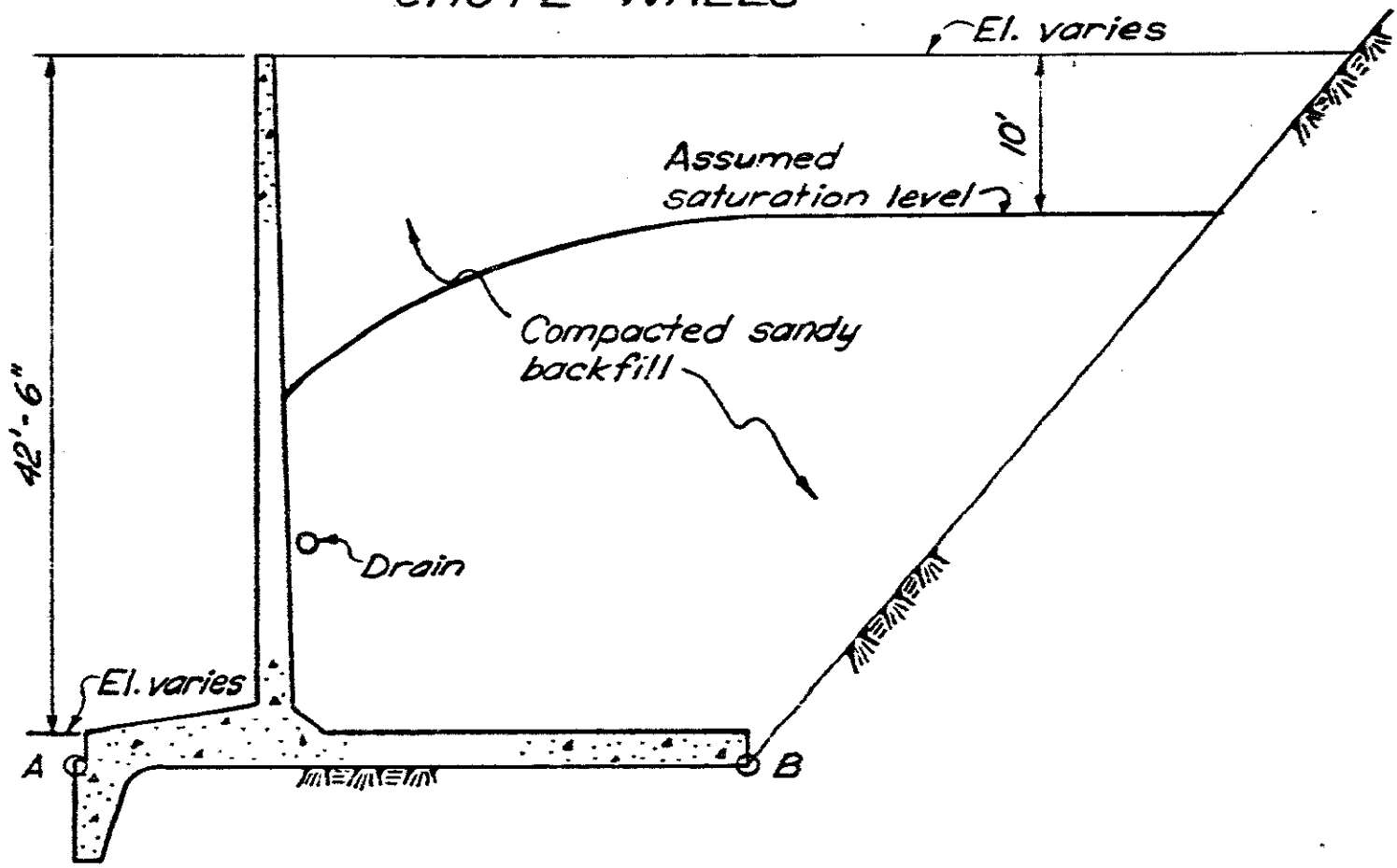
RESULTS OF STABILITY ANALYSIS
CHUTE WALLS



Loading	Sliding factor	Foundation Pressure, kips/ft² A	B	Overturning Safety Factor
Static	0.32	5.3	3.9	3.8
Static+Seismic	0.39	7.1	2.9	3.1

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In the direction of flow, the stilling basin walls and slabs have been separated by contraction joints into four parts, each 52.75 feet long. In the lateral direction, the toe of the retaining walls occupies 10 feet on each side. The remaining 725 feet of width has been divided by contraction joints into 15 separate slabs, each 48.33 feet wide. This spacing was selected to adhere as closely as possible to the ideal dimensions of the chute blocks and dentates selected in the hydraulic design of the stilling basin. The thickness of the slabs was increased from 1.5 feet at the upstream end to 4.5 feet at the chute blocks and decreased to 2.5 feet at the downstream end.

2. Chute Block Slab - At the start of the hydraulic jump, which occurs on the chute block slab, uplift pressures due to tailwater may exceed static pressure of water in the chute. The chute block slab was checked for stability using the profile of the water surface of the hydraulic jump observed on the spillway model. Uplift beneath the slab was assumed equal to the hydrostatic head of tailwater. Two analyses were made, one with the drains assumed to be 50 percent efficient and one with the drains 100 percent efficient. Under the loads investigated, the slab is subject to bending with the maximum moment occurring at the bottom of the slope.

Two cutoffs have been used on the underside of the chute-block slabs. One at the upstream end acts as an anchor. The other, at the foot of the chute, acts as an anchor and also separates the slab drainage systems. The stress induced by dynamic action of water on the chute blocks at the toe of the slope is insignificant.

3. Other Slabs - The other slabs downstream from the chute-block slabs were not designed to resist bending. They were nominally reinforced.

The end slab has a 22.5-foot deep cutoff wall at the downstream end. This cutoff wall extends completely across the spillway. Its purpose is to prevent undermining of the structure at the end of the stilling basin. The dentated end sill is also located on the end slab. Stress in the sill blocks due to dynamic forces by water were investigated and were found to be small. Two designs were required for the 22.5-foot cutoff wall, one where it extends into shale, and the other where it extends into sand.

4. Walls - The tops of the stilling basin walls were set at Elevation 55.00 feet. The walls are 58 feet high with a heel width of 40 feet, except for the 31-foot long transition section at the foot of the chute. The height of the transition section increases from 42.5 feet to 58 feet. The width of the heel slab increases proportionally from 30 feet to 40 feet.

The walls were designed using the methods outlined in paragraph B, except for special cases of the transition wall and end wall. The backfill was assumed to be at the top of the wall at Elevation 55.00 feet. A 15 degree surcharge was added, since the stilling basin backfill could retain slide material. Backfill saturation was assumed to be at Elevation 45.0 feet while the water surface in the basin was at Elevation 14.0 feet. The stability analysis results are shown on Figure U.

For the design of the transition wall, investigations were made at several points along the wall. Intermediate loads on the structure were interpolated from computed loads.

The end wall confines backfill in two directions. It was therefore designed to resist stresses induced by two-way loading. Regular counterforts stiffen the face and base slab in the direction perpendicular to the spillway axis. Backfill loads in the other direction are smaller, due to confinement by the regular counterforts. For this reason it was possible to use small depth counterforts and ribs. See Figure V for results of stability analysis.

An alternate design using a thick slab to provide two-way stiffness was studied for comparison and was found to be less desirable because of the additional concrete required.

Cutoffs were used on the foundation slabs of the spillway retaining walls at several places. One is located at the upstream edge of the transition wall base to anchor the sloping base slab to the foundation. A cutoff extends along the toe of each wall to act as a cutoff for the stilling basin drainage system, similar to the chute walls. A 22.5-foot deep cutoff is located on the slab of the end wall section, similar to the end chute slab.

H. Wing Wall

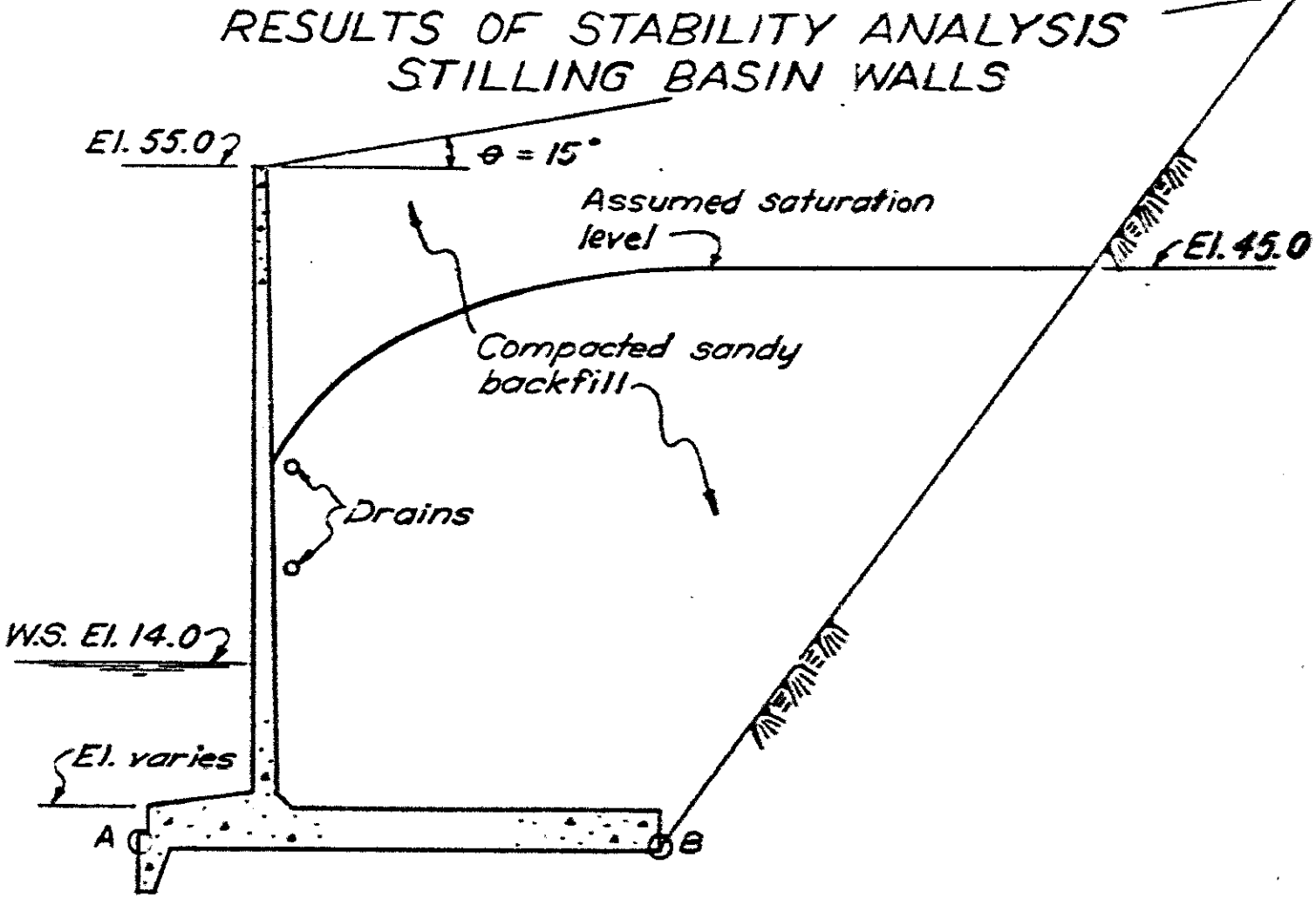
A wing wall is required on the left side of the spillway only, where the side slope of the outlet channel falls in a natural sand formation. The wall has a length of 125.50 feet. It is divided into three sections by contraction joints. The first section, which is adjacent to the end stilling basin wall, has a length of 55.00 feet. The second section has a length of 40.00 feet and the third a length of 30.00 feet. The top of the first two sections of wall is level at Elevation 55.00 feet. The height above the base slab varies from 58 feet to 9 feet. The base slab lies on a slope of 1.75 horizontal to 1 vertical and varies in width from 40 feet at the lower end to 10 feet at the upper end. The wall has no toe slab. It has a 7-foot deep cutoff located at the downstream end. The first section of the wall was designed as a counterforted wall; the second was designed as a cantilevered wall. The third section of the wall slopes up from Elevation 55.00 feet to Elevation 75.00 feet. It has a depth of 7 feet and a heel width of 10 feet. It has a 7-foot deep cutoff similar to the first two sections.

For design of these wall sections the backfill was assumed to be level at the top. The backfill on the downstream face of the wall was assumed to have been removed by erosion. For results of stability analysis at several points along the wall, see Figure W.

I. Gate Hoist Access Bridge

The gate hoist acess bridge provides a means of access to the piers for the operation and maintenance of the spillway tainter gate hoists. The bridge consists of 16 simply supported spans, each 10 feet wide between

RESULTS OF STABILITY ANALYSIS
STILLING BASIN WALLS



Station	Loading Condition	Sliding Factor	Foundation Pressure, Kips/ft² A	B	Overturning Safety Factor
10+18.52	Static	0.37	7.0	5.8	3.6
	Static+Seismic	0.42	9.9	4.5	3.0
10+49.52 to Sta 12+29.52	Static	0.40	11.2	4.5	2.57
	Static+Seismic	0.47	15.0	2.3	2.23

Note:

Load Condition	Allowable S.F.
Static + Surcharge	.45
Static + Seismic + Surcharge	.50

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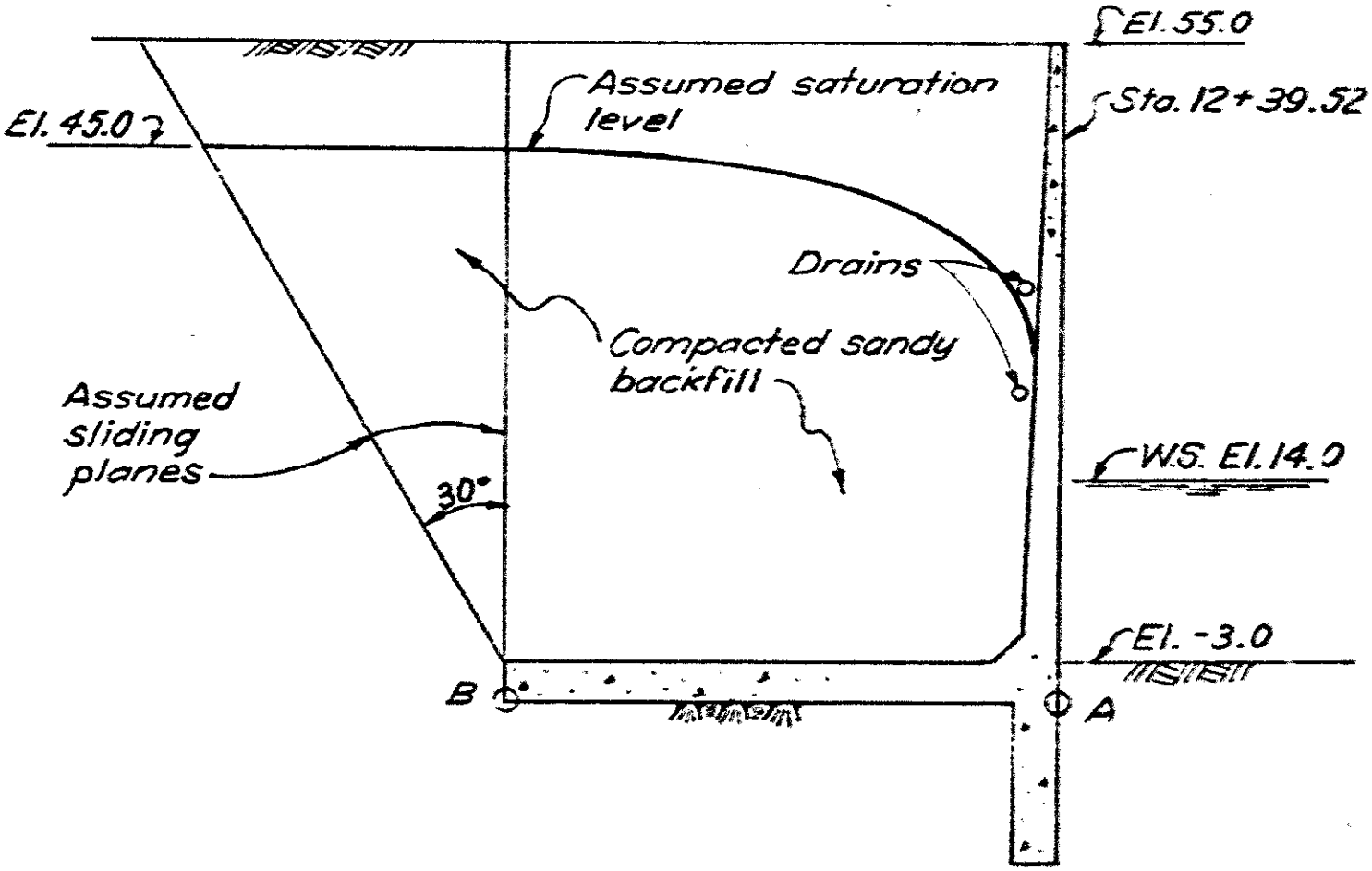
CK. W.R.S.

RESULTS OF STABILITY ANALYSIS
END WALL - STILLING BASIN



Loading Condition	Sliding Factor	Foundation Pressure, kips/ft² A	B	Overturning Safety Factor
Static	.40	12.0	4.8	—
Static + Seismic	.47	15.0	2.5	—

Notes:

Stability results shown here are a combination of the results of the above loading and the results of Sta. 10+49.52 given on Fig. U. Only the extreme values are given.

Loading condition	Allowable S.F.
Static + surcharge	.40
Static + Seismic + surcharge	.50

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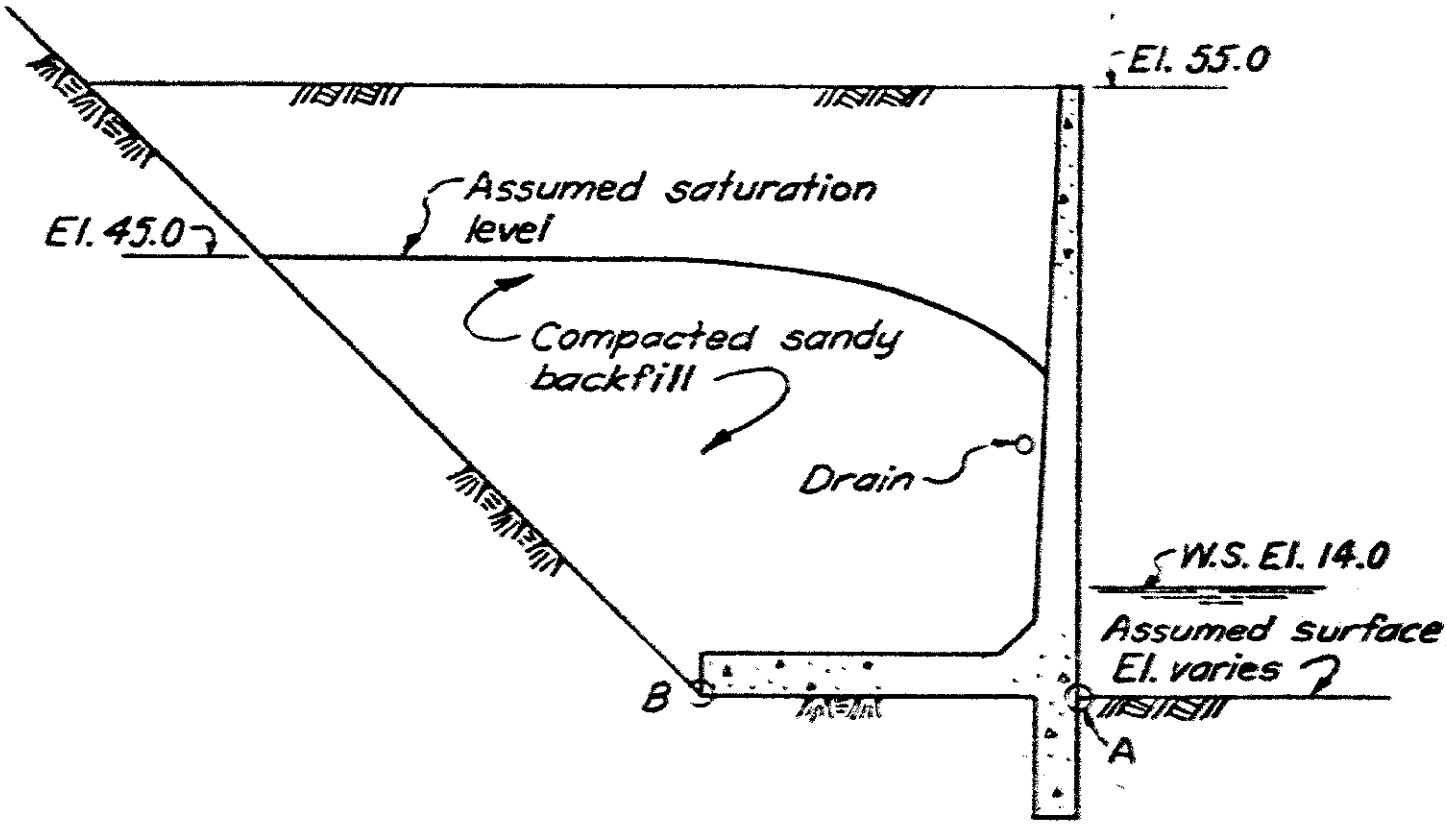
CK. W.R.S.		



Distance from Left End-Wall	Loading Condition	Sliding factor	Foundation Pressure in kips/ft² A	B	Overturning Safety Factor
0'	Static	0.35	12.0	0	2.1
	Static+Seismic	0.44	15.0	0	1.7
55.5'	Static	0.34	7.1	0.3	3.1
	Static+Seismic	0.41	9.7	0	2.2
95.5	Static	0.32	2.0	0	2.5
	Static+Seismic	0.42	2.7	0	1.9

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CK. W.R.S. | C.S.K. | J.J.

DATE: 2-4-59 | FIG. W

curbs. The design span length was 43.6 feet, which includes the clear span plus one-half of each support. The bridge was designed in accordance with the AASHO "Standard Specifications for Highway Bridges", using one lane H-20 truck loading. The design was investigated for stresses resulting from the stoplogging operation and found to be safe.

The bridge was designed to be constructed using precast girders and a cast-in-place deck. Precast girders were used to avoid erection of supporting falsework above the spillway crest.

The precast girders were designed to carry the total dead weight of the bridge and were investigated for erection stresses and found to be safe. High strength concrete (3750 pounds per square inch) was used to minimize the size and weight of the girders, making them easier to handle. The precast girders were cambered to offset deflections resulting from the dead weight of the bridge. Keyways were provided at the construction joint between girder and bridge slab near the ends of the span where shear stress is highest. Holes were provided near each end of the girders to provide convenient lifting points.

The bridge girders rest in slots on the pier and abutment wall bridge seats. The slots were chamfered at the edge to protect the face of the piers and abutment from spalling. At one end of each span the bridge girders were bolted to clip angles anchored to the pier.

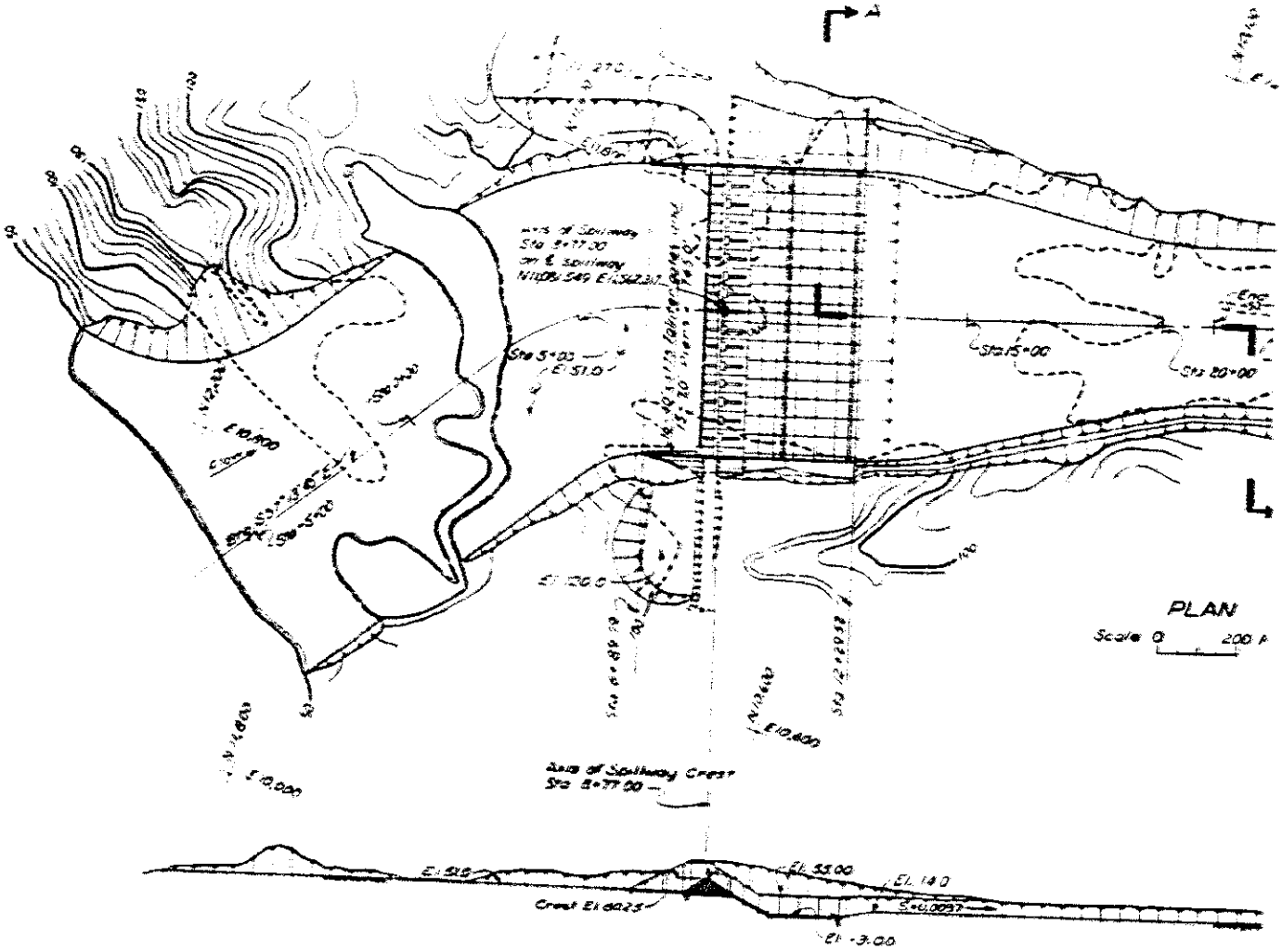
To reduce friction between the free end of the girder and the seat, two layers of tar paper were placed beneath the girders. A heavy coat of emulsified asphalt was applied to the surfaces of tar paper in contact with concrete.

Expansion deck plates were installed at the joint between spans. Drainage of the deck was provided by formed openings in the slab. Ten-inch high concrete curbs and 2.5-foot high pipe handrails were provided on both sides of the bridge.

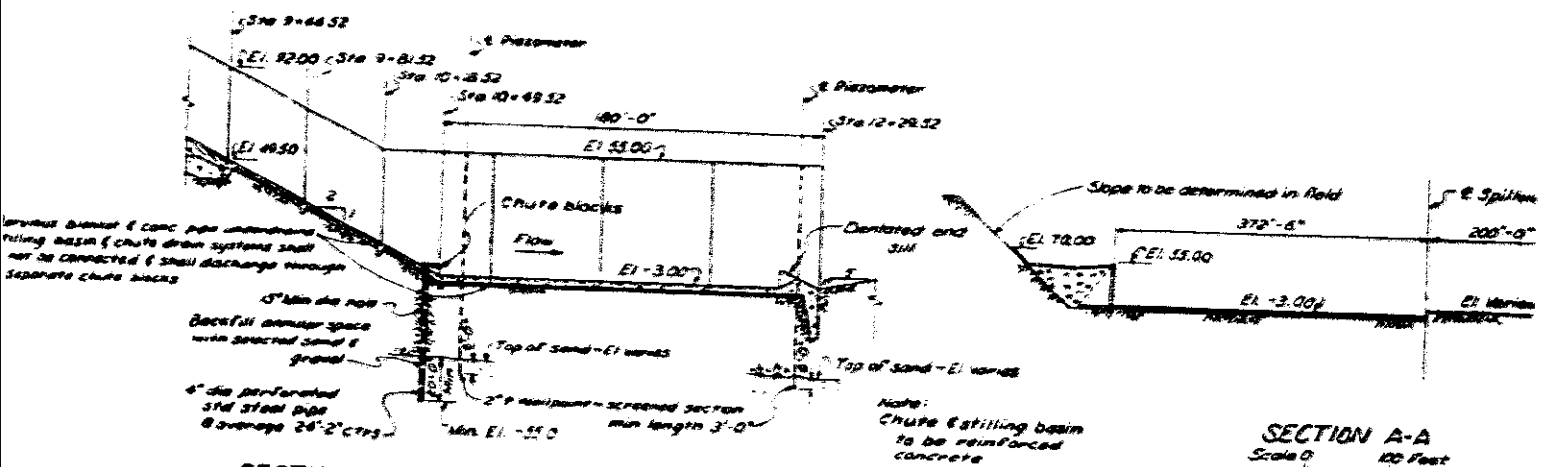
PART III - CHAPTER II

LIST OF EXHIBITS

Exhibit	Title
II-1	Spillway - General Layout and Sections

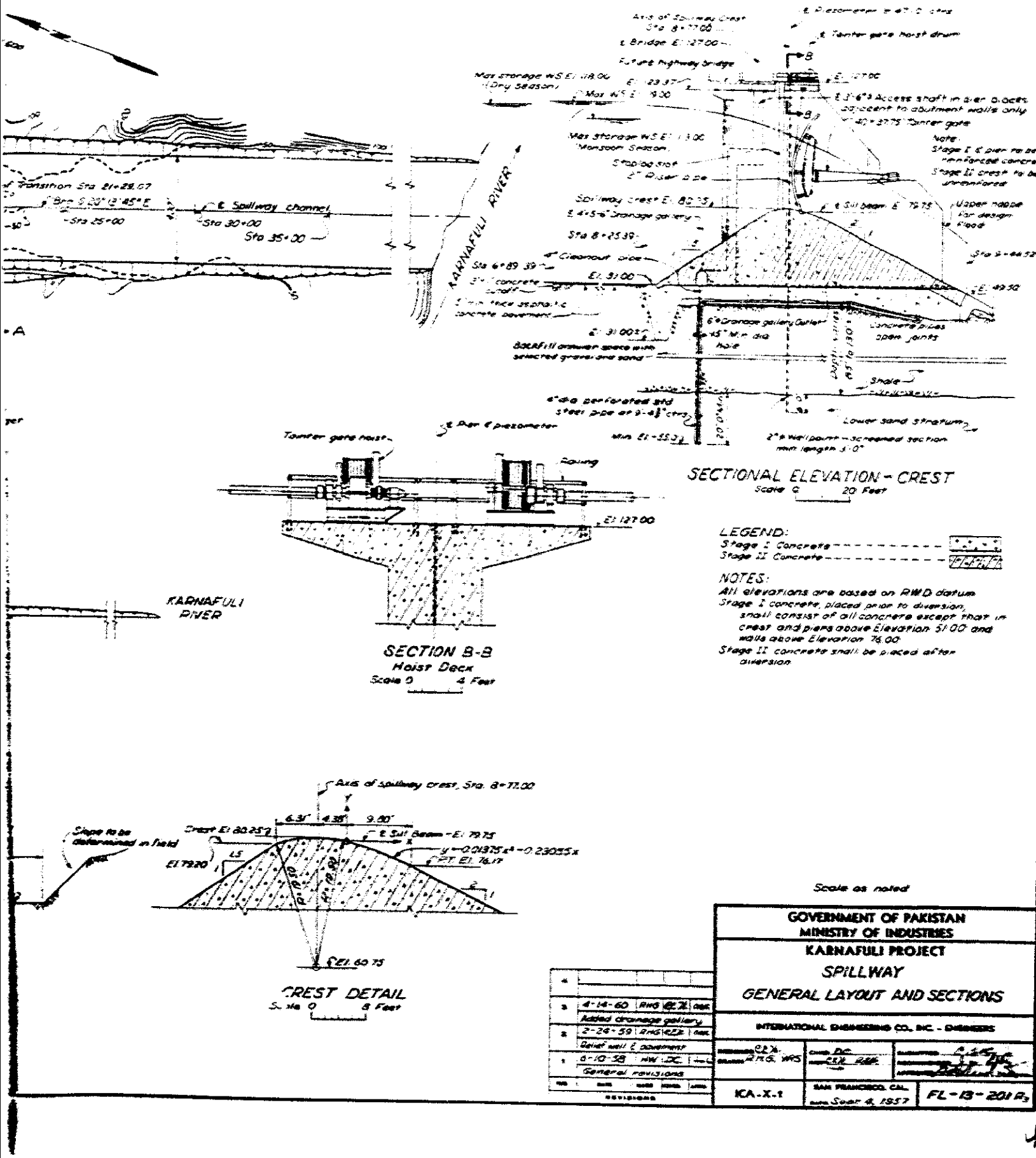


SECTION ON CENTER LINE
Scale 0 200 Feet



SECTIONAL ELEVATION - CHUTE AND STILLING BASIN
Scale 0 40 Feet

SECTION A-A
Scale 0 100 Feet


KARNAFULI RIVER
℄ Spillway channel
Sta 25+00
Sta 30+00
Sta 35+00
Axis of Spillway Crest Sta 8+77.00
℄ Bridge El 127.00
Future highway bridge
Max storage WS El 118.00 (Dry Season)
Max WS El 110.00
Max storage WS El 113.00 (Monsoon Season)
Spillway crest El 80.25
℄ 4'-5" Drainage gallery
Sta 8+25.39
Clean-out pipe
El 31.00
Backfill around space with selected gravel and sand
6" dia perforated std steel pipe at 9'-6" ctrs
Min El -55.3
℄ Tainter gate hoist drum
℄ 3'-6" Access shaft in pier blocks adjacent to abutment walls only
Tainter gate
Note:
Stage I: ℄ pier to be reinforced concrete
Stage II: crest to be unreinforced
℄ Sill beam El 79.75
Upper nappe for design flood
Sta 9+44.52
El 49.50
6" Drainage gallery outlet
15" Min dia hole
Concrete blocks open joints
Shale
Lower sand stratum
2" ϕ Well point - screened section min length 3'-0"
SECTIONAL ELEVATION - CREST
Scale 0 20 Feet
Tainter gate hoist
℄ Pier & piezometer
Casing
El 127.00
SECTION B-B
Hoist Deck
Scale 0 4 Feet
LEGEND:
Stage I Concrete
Stage II Concrete
NOTES:
All elevations are based on RWD datum
Stage I concrete, placed prior to diversion, shall consist of all concrete except that in crest and piers above Elevation 51.00 and walls above Elevation 76.00.
Stage II concrete shall be placed after diversion
Axis of spillway crest, Sta. 8+77.00
Shape to be determined in field
Crest El 80.25
6.35' 4.38' 9.80'
℄ Sill Beam - El 79.75
y = -0.01375x² - 0.23055x
℄ PT. El. 76.17
El 79.20
1.5
R = 10.00
R = 8.00
℄ El. 60.75
CREST DETAIL
Scale 0 8 Feet
Scale as noted
GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
SPILLWAY
GENERAL LAYOUT AND SECTIONS
INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS
4-14-60 RHG
Added drainage gallery
2-24-59 RHG
Relief well & pavement
6-10-58 HW
General revisions
REVISIONS
KA-X-1
SAN FRANCISCO, CAL.
Sept 4, 1957
FL-13-201 R3

PART III

CHAPTER III

DIVERSION AND OUTLET TUNNEL

3.01 GENERAL

The diversion and outlet tunnel is located through the right abutment on a line approximately normal to the projected axis of the main dam.

In establishing the alignment of the tunnel the following requirements for safe and practical construction were considered:

1. The upstream and downstream portals must be sufficiently removed from the dam construction area to leave ample space for the cofferdams.

2. The tunnel must be carried sufficiently far into the abutment to provide ample horizontal and vertical rock cover.

3. The portal locations must result in a minimum length of tunnel consistent with the other requirements.

4. Discharge from the tunnel must be directed to avoid scour at the toe of the main dam embankment.

The final tunnel location and alignment, as shown on Exhibit III-1, adequately meets the above requirements.

Since the initial function of the tunnel is to divert the river around the foundation area during construction of the main dam, the size and shape of the tunnel were based on the following requirements and criteria:

1. The tunnel should be of sufficient size to discharge dry season flows without endangering dam and spillway construction.

2. Diversion through the tunnel should be made at the earliest possible date after the monsoon season to allow ample time for construction of the dam.

3. After the monsoon season, the reservoir should drain in a reasonable length of time to permit access to upstream borrow areas.

4. The maximum water surface during initial closure should not exceed Elevation 26.0 feet. This elevation was established as the maximum water level at which the initial closure cofferdam could be constructed without undue difficulty.

From the available hydrographs, an initial diversion capacity of 2800 cubic feet per second was selected for the design. Studies indicated that, with this capacity, closure could have been made by December 22nd for 17 out of 18 years without overtopping the cofferdam.

It was determined that a concrete-lined, twenty-foot diameter horseshoe-shaped tunnel would most economically meet all requirements.

After the diversion period, the tunnel will function as a permanent emergency outlet to release water for navigation purposes when there is no flow over the spillway or through the power plant. This release is controlled by a slide gate installed in the tunnel plug. Discharge rating curves for both diversion and outlet uses are shown on Exhibit III-1.

The diversion and outlet tunnel consists of the following components:

1. Approach Channel
2. Tunnel Intake Structure
3. Tunnel Proper
4. Gate Shaft
5. Tunnel Transition and Plug
6. Slide Gate Chamber
7. Outlet Portal Structure
8. Discharge Channel

This chapter deals with the principal features of the design of these components. The designs of the mechanical and electrical equipment, intake portal trashrack, bulkhead, fixed-wheel gate, and the slide gate are given in PART V, Chapter III.

3.02 DESIGN CRITERIA AND ASSUMPTIONS

Basic criteria and assumptions established for the design of the various components of the diversion and outlet tunnel are as follows:

A. Loads

1. Dead Loads - The dead loads consisted of the weight of all permanent construction entering into and becoming a part of the final structure.

2. Live Loads - The live loads assumed for final design were dictated by the conditions involved or as recommended by the recognized codes for the design of similar structures.

Live loads assumed were as follows:

Location	Live Load Pounds per Square Foot
Roof of Gate Shaft House	35
Hoist Deck at Elevation 135.0 feet	200
Inspection Deck at Elevation 126.0 feet	150
Gate Support Deck at Elevation 120.0 feet	150
Slab at Base of Spiral Stairway, Elevation 30.5 feet	200

Assumed loadings from construction equipment were as follows:

Location	Loading
Cut and Cover Section	AASHO H20-S16 Truck Loading
Emergency Fixed-Wheel Gate Storage Area	Surcharge equal to 2.26 feet of fill.

3. Wind Load - Wind pressure was assumed to be 20 pounds per square foot on the vertical projection of exposed building area.

4. Earthquake Forces - Seismic forces were assumed as 10 percent of the sum of dead and permanent loads applied at the center of mass in any horizontal direction.

5. Hydrostatic Pressure and Uplift - The various components were designed to sustain the hydrostatic pressures and full uplift pressures arising from reservoir or tailwater conditions involved in the function of the structure.

6. Earth Pressures - Equivalent fluid pressures were assumed to be as follows:

Material	Equivalent Fluid Pressure Pounds per Cubic Foot
Saturated backfill, water and earth combined	75
Submerged backfill, earth only	40
Dry backfill	30

7. Combined Loadings - The various components were designed to sustain at allowable stresses the combined loadings of full dead load and normal live load. For temporary live loads from wind, earthquake, hydrostatic pressures of short duration, or breakdown loads combined individually with full dead load and normal live load, the allowable stresses were increased 33-1/3 percent. Simultaneous combinations of temporary loads were not considered.

B. Allowable Stresses

1. Concrete - Three thousand pounds per square inch concrete at 28 days, except for the unreinforced tunnel lining where 2500 pounds per square inch concrete was used.

2. Reinforcement - Reinforcing steel shall be intermediate grade billet steel conforming to ASTM Designation A-15. Deformations shall conform to ASTM Designation A-305. Normal working stress used was 20,000 pounds per square inch.

C. Standards

The standards used for the design of concrete and reinforcement are in accordance with "Building Code Requirements for Reinforced Concrete (ACI 318-56)" of the American Concrete Institute.

D. Unit Weights

Unit weights assumed were:

Material	Unit Weight Pounds per Cubic Foot
Saturated Earth	130.0
Submerged Earth	68.0
Dry Earth	110.0
Concrete	150.0
Submerged Concrete	88.0
Water	62.5

E. Hydraulic Assumptions

	Value
"n" for Concrete-Lined Tunnel	0.016
"n" for Channels	0.025
Slopes:	
Approach Channel	0.00
Tunnel	0.00289
Outlet Channel	0.002

E. Hydraulic Assumptions (cont'd)

Tunnel Losses:	Value
Entrance	0.3 h_v
Gate Transition	0.6 h_v
Friction	SL (where S = slope of the hydraulic gradient)
Exit	1.0 h_v

Tunnel Capacity:	Capacity Cubic Feet per Second	Reservoir Elevation
For Initial Diversion	2800	26.0
For Permanent Outlet	2000	60.0

3.03 DESIGN OF TUNNEL COMPONENTS

The designs of the various components of the diversion and outlet tunnel were based on practical considerations to ensure safety and economy consistent with the function of the particular feature.

A. Approach Channel

The approach channel is 2300 feet long and has a trapezoidal cross section. The bottom width is 20 feet and side slopes are 1.0 to 1.0. The invert of the channel for its entire length is at Elevation 10.0 feet, which is the same elevation as the invert of the intake portal.

B. Tunnel Intake Structure

The tunnel intake structure consists of three parts: upstream training walls, a cut and cover section, and a section of reinforced tunnel lining. Final designs of the various elements of these parts evolved from the investigations discussed in the paragraphs which follow.

1. Training Walls - The reinforced concrete vertical upstream training walls are 45 feet long overall, and vary in height from 5 feet at the upstream ends to 25 feet at the cut and cover section.

The walls are constructed monolithically with the floor slab and were designed as cantilever beams fixed at the base. The slab was designed as a partially fixed beam.

The combined loadings for final design as illustrated in Figure A were based on the following assumptions:

a. Structure submerged.

b. Equivalent fluid pressure of the submerged backfill equal to 40 pounds per cubic foot. (This is a conservative assumption.)

c. Uniform base pressure from structure dead load only.

Other conditions were investigated but were not found to be critical.

2. Cut and Cover Section - The cut and cover section is a 20-foot diameter horseshoe-shaped interior section with horizontal exterior base and with a minimum thickness of 20 inches of reinforced concrete. The overall length of the section is 40 feet with the invert at Elevation 10.00 feet along its entire length. Since the intake structure will be permanently submerged, the function of the cut and cover section is to prevent blocking of the tunnel by any sloughing of material from the exposed portal face. A trashrack is also provided.

The structure was analyzed as a closed ring with symmetrical loading by the Elastic-Weight principles.

Design studies ensured that the section was adequate for the combined loadings, as illustrated in Figure B, based on the following assumptions:

CASE I

a. Twenty feet of fill over barrel.

b. Fill and barrel submerged.

c. Equivalent fluid pressure of the submerged backfill equals 40 pounds per cubic foot from earth only.

d. Uniform base pressure from structure dead load plus submerged weight of fill.

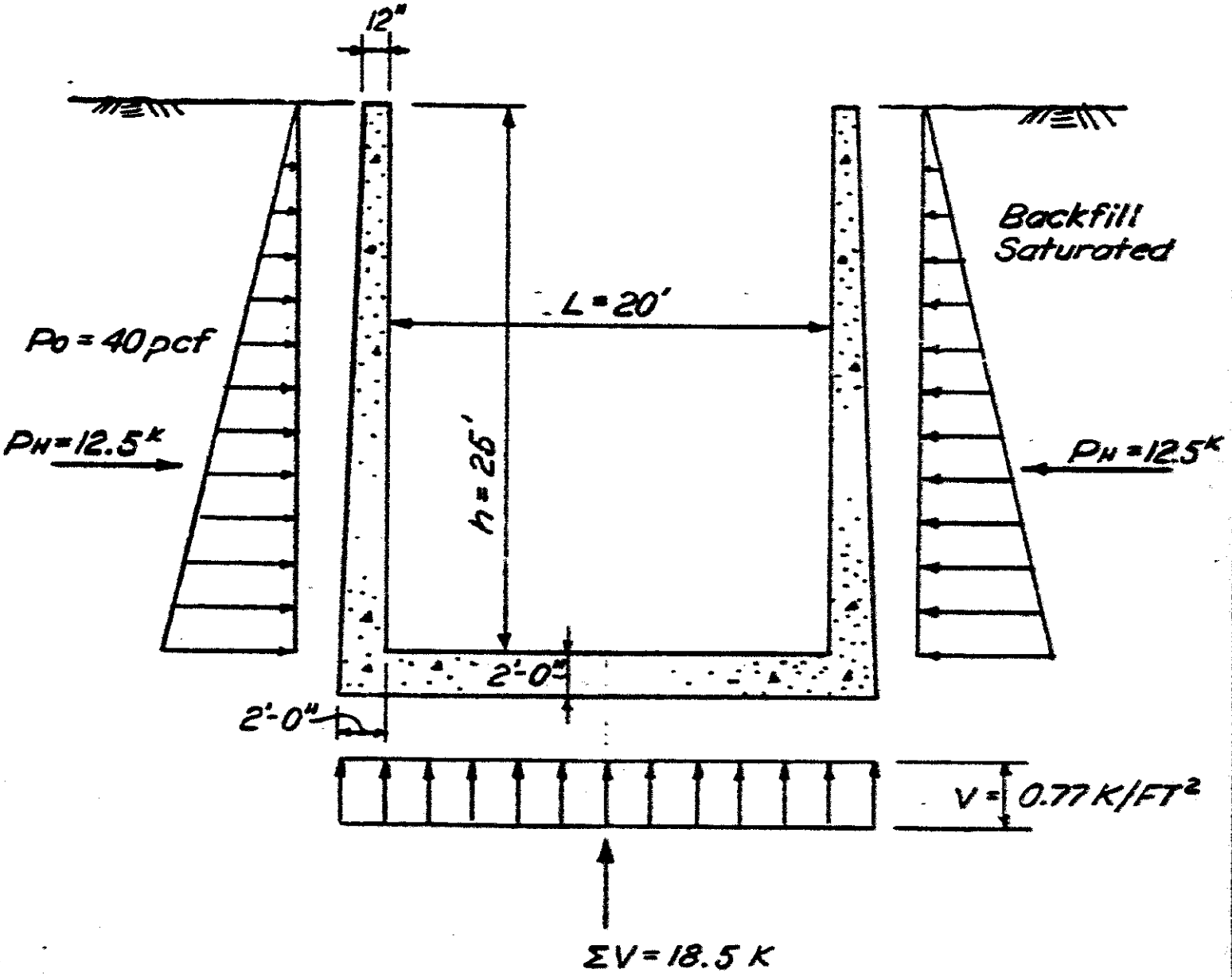
CASE II

The structure considered empty only immediately after construction, and an H20-S16 loading introduced to represent any additional loading imposed by heavy construction equipment.

Based on judgment, reinforcement in addition to that required by the above cases was provided to take care of other possible loadings.

3. Section of Reinforced Tunnel Lining - This section of the structure is a 20-foot diameter horseshoe-shaped tunnel section 120 feet long, with a 20-inch uniform thickness of reinforced concrete lining for the first 40 feet of length and an 18-inch uniform thickness of reinforced concrete lining for the remaining 80 feet of length. The invert of this

TYPES OF LOADINGS INVESTIGATED
INTAKE STRUCTURE - TRAINING WALLS



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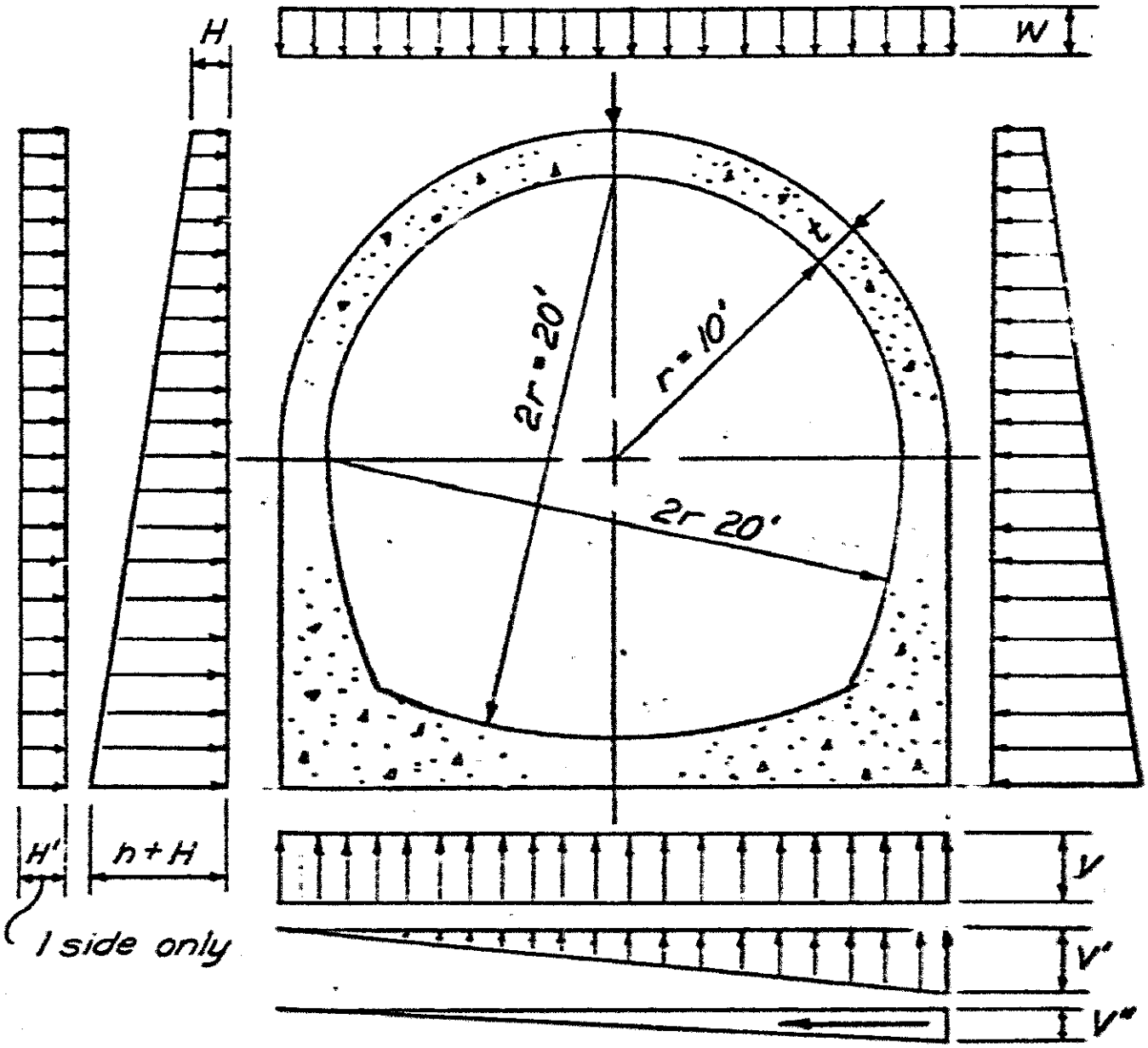
DIVERSION AND OUTLET TUNNEL DESIGN REPORT
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.
DR. PC / RECOMMENDED / APPROVED

TYPES OF LOADINGS INVESTIGATED
CUT AND COVER SECTION



Final design checked for these loadings:

Case I (Submerged)

Structure DL = 13.5 K

$w = 1.36\ K/FT^2$ $H = 0.80\ K/F$

$h + H = 1.73\ K/FT^2$ $V = 1.94\ K/FT^2$

Case II (Empty plus H20-S16 Loading)

Structure DL = 23.0 K

$w = 0.76\ K/FT^2$ $P = 4.0\ K$

$H = 0.07\ K/FT^2$ $h + H = 0.77\ K/FT^2$

$H' = 0.17\ K/FT^2$ $V = 1.43\ K/FT^2$

$V' = 1.0\ K/FT^2$ $V''\ 0.33\ K/FT^2$

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INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. PC	RECOMMENDED	APPROVED

section has a slope of 0.00289. The original excavation of the tunnel was such that after excavating to the design grade the tunnel section was of irregular and variable height. This section was designed to support the concrete backfill necessitated by the original excavation.

The structure was analyzed as a closed-ring with symmetrical loading by the Elastic-Weight principles.

The combined loadings for final design as illustrated in Figure C were based on the following assumptions:

a. Weight of the backfill equal to 150 pounds per cubic foot.

b. Structure empty immediately after construction and before diversion.

c. Uniform foundation pressure from weight of the backfill and the structure dead load.

C. Tunnel Proper

The tunnel proper consists of a 20-foot diameter horseshoe-shaped section 1317 feet long. Since exposed shale would not be erosion-resistant, a nominal thickness of unreinforced lining is provided.

D. Gate Shaft Structure

The gate shaft structure consists of three parts: The vertical gate shaft, the emergency fixed-wheel gate storage area, and the gate shaft house. Final design of these elements of the gate shaft structure is made on the basis of the investigations discussed in the paragraphs which follow.

1. Vertical Gate Shaft - The shaft proper is a 20-foot outside diameter circular shaft, approximately 97.5 feet high, as measured from the invert of the tunnel to Elevation 105.0 feet. The shaft, with a minimum thickness of 13-inch reinforced concrete lining, is divided by a 3-foot concrete wall. The upstream side provides space for the emergency fixed-wheel gate and bulkhead. The downstream side of the shaft provides space for a 5.5-foot diameter spiral stairway and ventilation ducts to the slide gate chamber.

The structure was analyzed as a closed ring with symmetrical loading by the Elastic-Weight principles.

The combined loadings for final design as illustrated in Figure D were based on the following assumptions:

a. Uniform pressure equivalent to full hydrostatic head at base of the shaft.

b. The rock is sound and will not exert any horizontal pressure.

c. Seismic forces not considered.

Design investigations other than those upon which final design was based were:

a. On the upstream portion, an unbalanced uniform internal pressure, equal to 20 feet of head, with zero pressure on exterior.

b. On the upstream portion, an unbalanced uniform external pressure, equal to 20 feet of head, with zero pressure on interior.

Reinforcement of the slab at Elevation 30.5 feet in the downstream side of the shaft was provided to withstand pressures of (1) 50 pounds per square inch from contact grouting of the underlying tunnel plug, (2) dead load of the spiral stairway and (3) 200 pounds per square foot live load on the slab. See Figure H.

2. Emergency Fixed-Wheel Gate Storage Area - At Elevation 105.0 feet a 24-foot square gate storage area, with reinforced concrete walls 26 feet high, is constructed over the gate shaft. This provides an area for construction of the fixed-wheel gate bulkhead, and for storage of the fixed-wheel emergency gate. Second stage concrete construction consists of a 24-inch thick reinforced concrete divider wall terminating at Elevation 126.0 feet, seven feet above the maximum water surface elevation; an 8-inch thick reinforced concrete gate-support deck at Elevation 120.0 feet on the upstream side of the divider wall, two feet above the maximum normal water surface elevation; and an 8-inch thick reinforced concrete spiral stairway landing and inspection deck at Elevation 126.0 feet, on the downstream side of the divider wall.

For first stage construction a horizontal section of the walls was analyzed by moment distribution as a square closed section of uniform thickness. For second stage construction the 24-inch divider wall was assumed fixed along the sides and bottom, hinged along the top and analyzed by moment distribution.

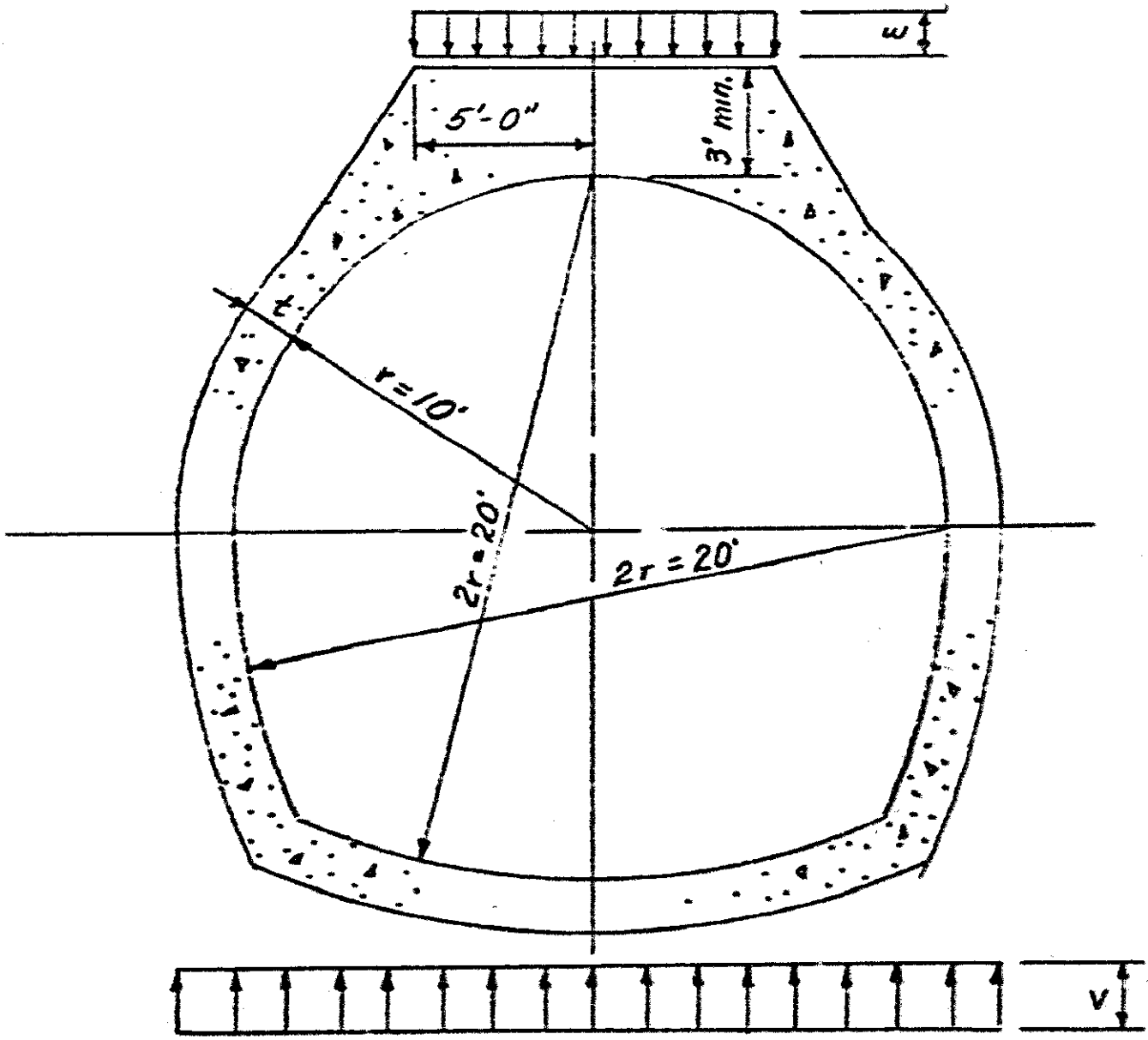
The combined loading for final design of first stage construction, as illustrated in Figure E, was based on the assumption that the equivalent fluid pressure of the backfill and the temporary surcharge loading from construction equipment are equal to 46 pounds per cubic foot.

The combined loadings for final design of second stage construction, as illustrated in Figure F, are based on the following assumptions:

a. The gate-support deck at Elevation 120.0 feet is designed for a loading of 150 pounds per square foot live load, plus 100 pounds per square foot dead load.

b. The divider wall supports the decks at Elevations 120.0 and 126.0 feet, and is subject to a hydrostatic pressure equal to 13 feet of head.

TYPES OF LOADINGS INVESTIGATED
SECTION OF REINFORCED TUNNEL LINING



t = 20 inches ~ Sta. 1+73 to Sta. 2+13
t = 18 inches ~ Sta. 2+13 to Sta. 2+93

Description	Loads			
	1	2	3	4
w #/Ft	2.0	1.28	0.96	0
V #/Ft	1.74 max.	1.42	1.26	0.87 min.

Load	Station
1.	1+73
2.	2+13
3	2+43

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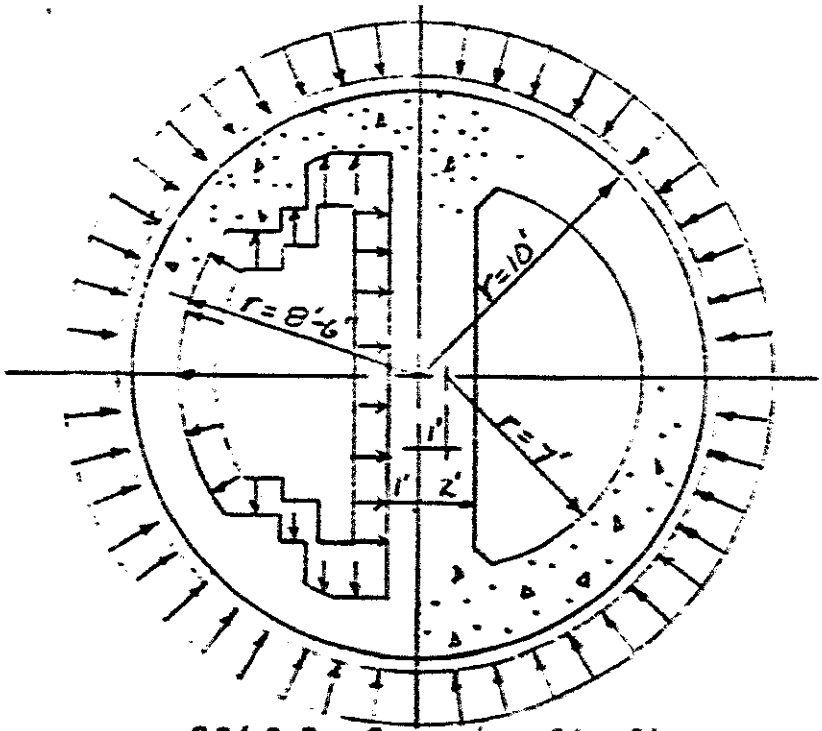
DIVERSION AND OUTLET TUNNEL DESIGN REPORT

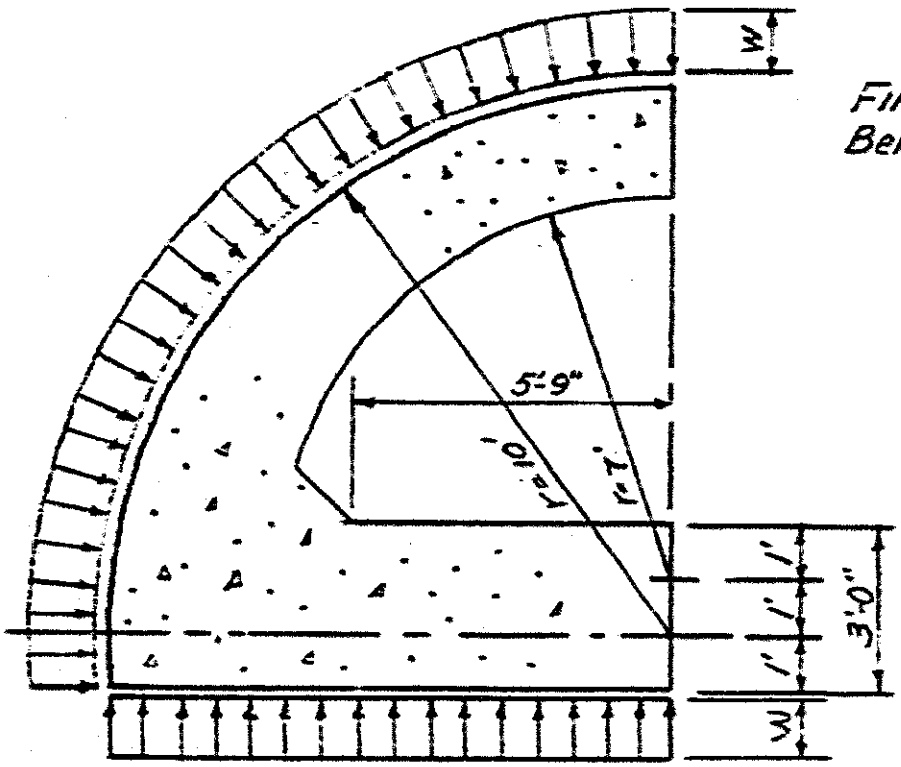
INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. PC | RECOMMENDED | APPROVED

TYPES OF LOADINGS INVESTIGATED
VERTICAL GATE SHAFT



20' O.D. Circular Shaft



SECTION FOR DESIGN

Final Design Loading For:

Below. Shaft Elev.	W #/FT²
105.0	2.19 #/FT²
81.0	3.12 #/FT²
69.0	4.05 #/FT²
57.0	5.00 #/FT²

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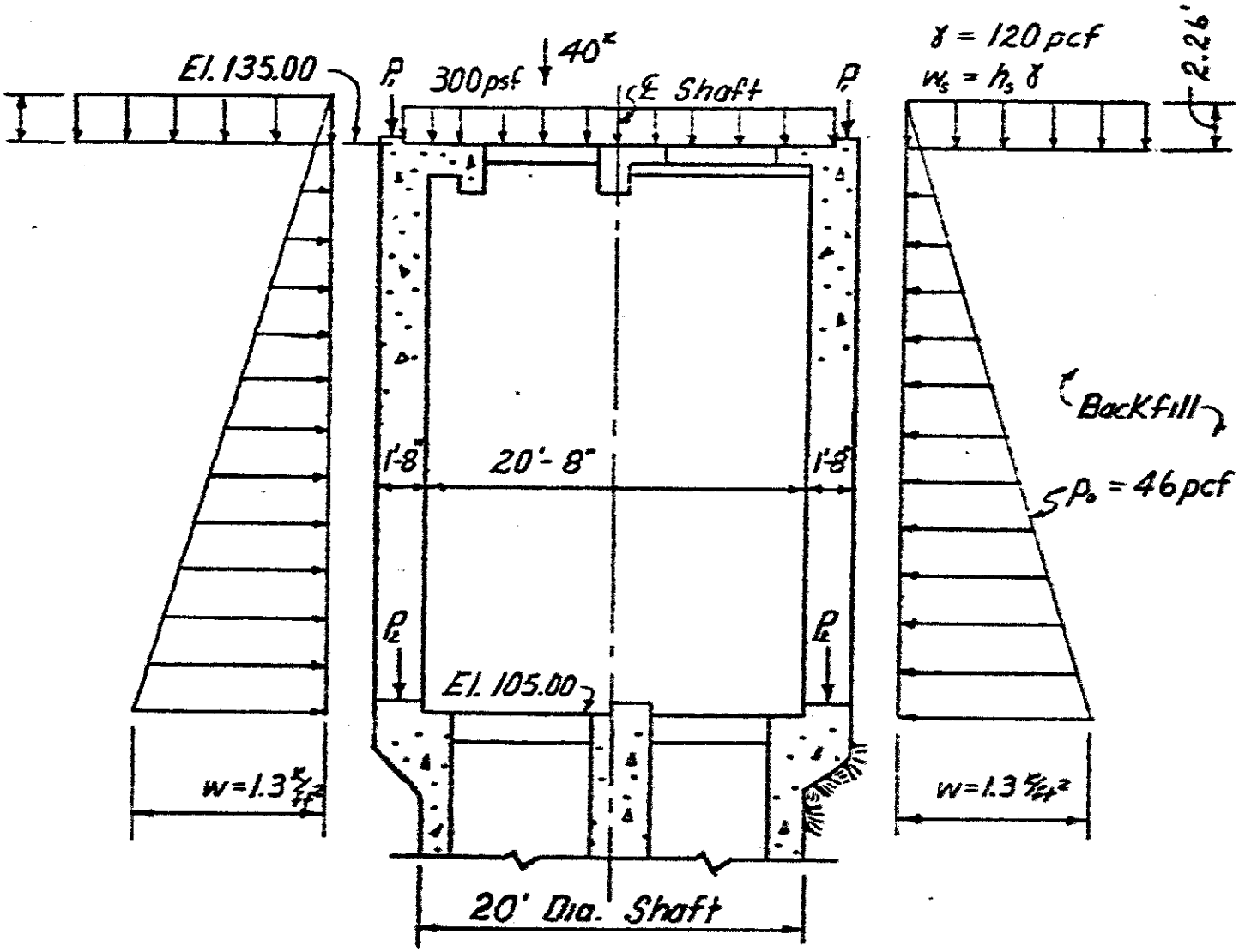
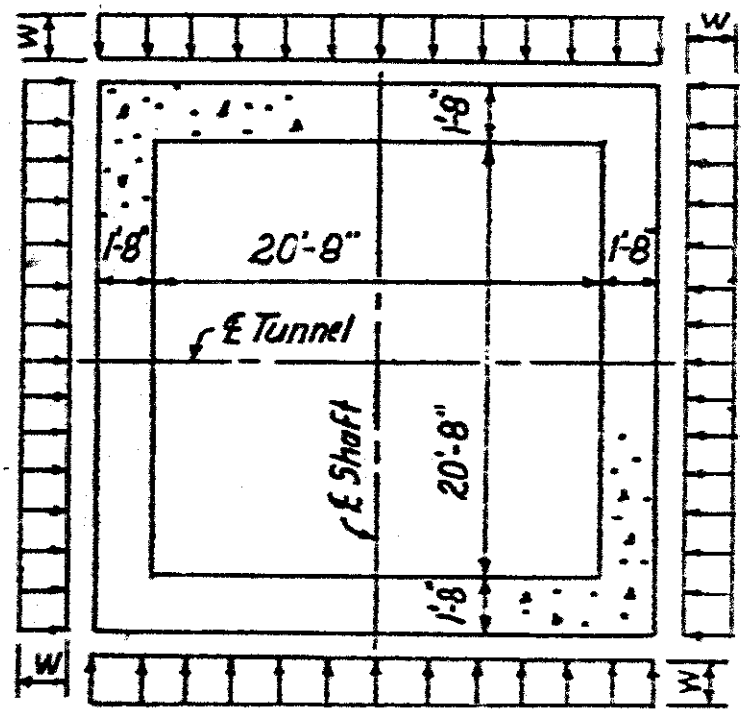
DESIGN REPORT

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

TYPES OF LOADINGS INVESTIGATED
EMERGENCY FIXED-WHEEL GATE STORAGE AREA



$P_1 = 2.9^k$
$P_2 = 11.9^k$



SECTION FOR DESIGN

KARNAFULI PROJECT		
DIVERSION AND OUTLET TUNNEL DESIGN REPORT		
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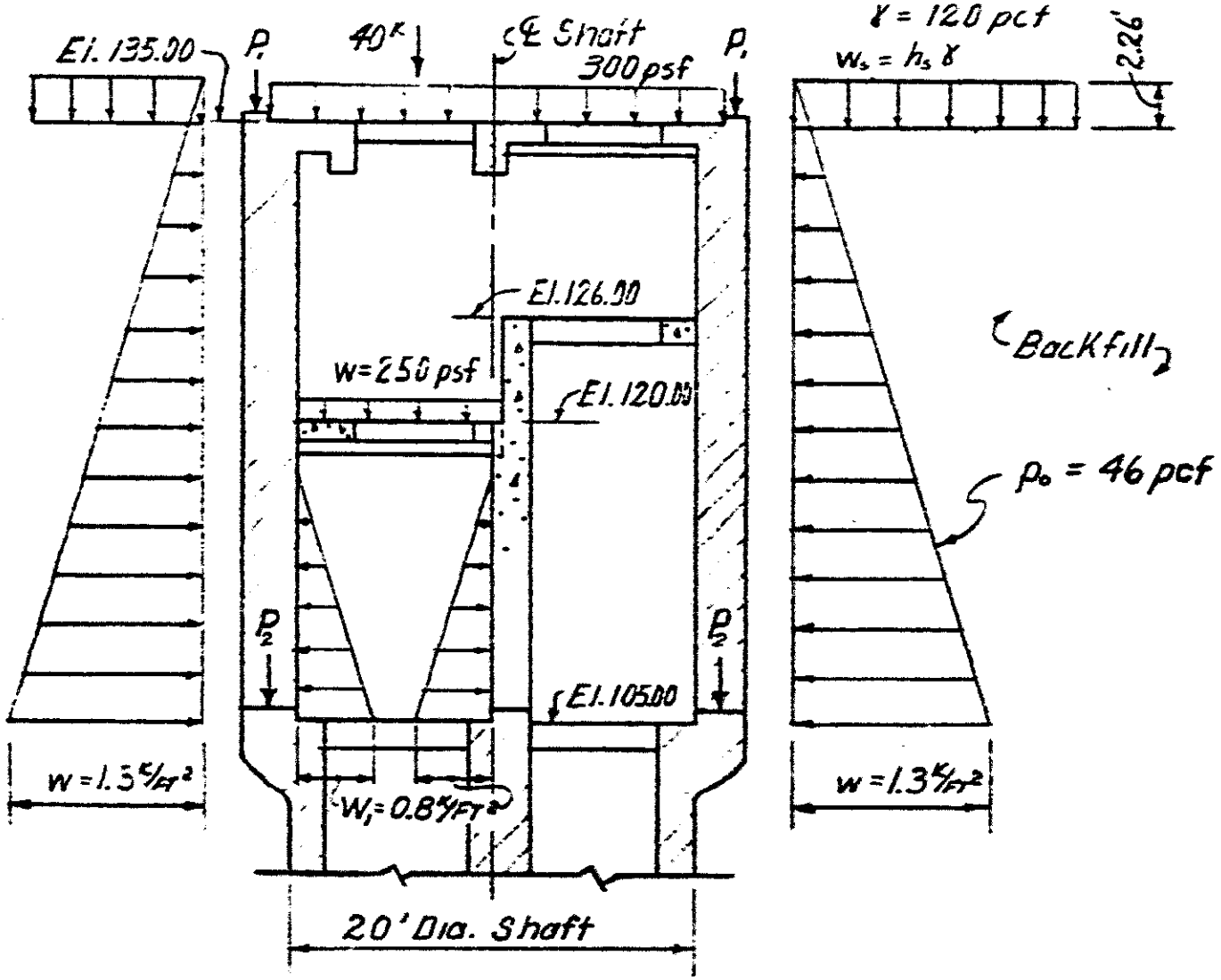
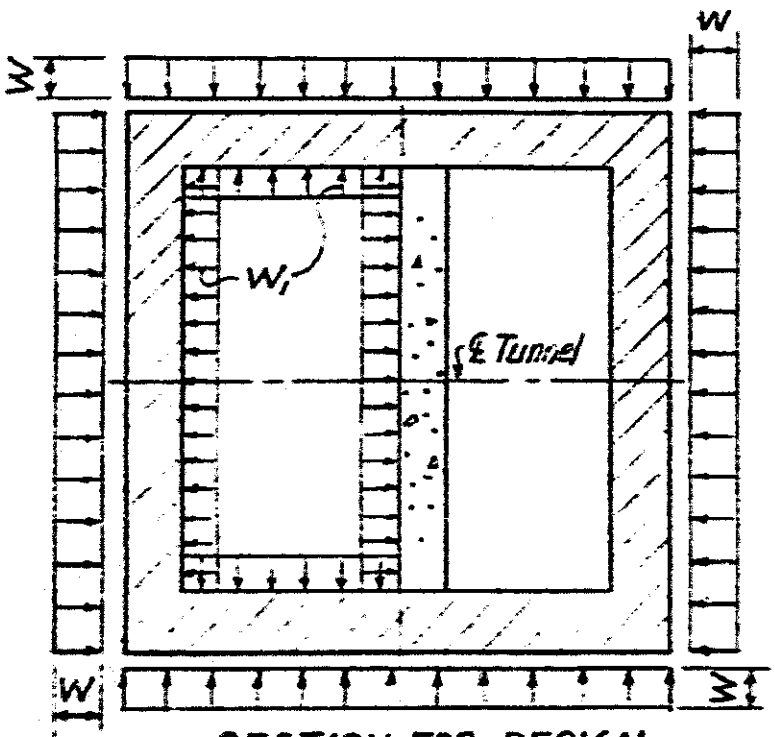
DR. P.C.	RECOMMENDED	APPROVED

TYPES OF LOADINGS INVESTIGATED
EMERGENCY FIXED-WHEEL GATE STORAGE AREA





SECTION FOR DESIGN

P_1 = 2.9K
P_2 = 12.8K

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DR. PC	RECOMMENDED	APPROVED

3. Gate Shaft House - The gate shaft house, located on the centerline of the tunnel at the top of the gate shaft, is a 24-foot by 24-foot structure with reinforced concrete walls 10 inches thick and a reinforced concrete roof slab 6 inches thick. The hoist deck at Elevation 135.0 feet provides support for the stationary fixed-wheel gate hoist. A manually-operated monorail hoist is provided for removal of the fixed-wheel gate from the gate storage area and equipment from the slide gate chamber. Electrical controls, ventilation equipment and slide gate hydraulic equipment which are located in the house are described in PART V, Equipment and Systems.

The walls and columns were designed as fixed at the bottom and the deck beams were assumed as partially fixed. The deck and roof slabs were designed as 2-way slabs discontinuous at the edges.

The combined loadings for final design of the hoist deck as illustrated in Figures E and F were based on the following assumptions:

a. Fixed-wheel gate hoist capacity = 30,000 pounds

b. Weight of hoist = 10,000 pounds

c. Uniformly distributed live load = 200 pounds per square foot

d. Uniformly distributed dead load = 100 pounds per square foot

Other combined loadings investigated but which did not govern were as follows:

a. Live load of 100 pounds per square foot, plus 100 pounds per square foot dead load, plus 275 percent of the hoist capacity for downpull, plus dead load of the hoist.

b. Live load of 300 pounds per square foot, plus 100 pounds per square foot dead load, plus dead load of the hoist.

The combined loadings of the roof slab, walls and columns, as illustrated in Figure G, were based on the following assumptions:

a. Wind pressure on the walls = 20 pounds per square foot.

b. Uniformly distributed load on the roof slab = 35 pounds per square foot live load plus 75 pounds per square foot dead load.

c. Column loading of 24,000 pounds included the weight of the fixed-wheel gate, monorail and monorail hoist. The lateral impact load was 20 percent of the vertical loads.

d. Seismic forces on any equipment or structural parts were assumed as 10 percent of the dead and permanent live loads applied at center of mass in any horizontal direction.

E. Tunnel Transition and Plug

The diversion tunnel was designed to be utilized as an outlet works after the diversion period. This was accomplished by two-stage construction in the area of the tunnel plug.

To permit construction of the plug and installation of the slide gate (Stage II), the tunnel is closed by means of the emergency fixed-wheel gate and bulkhead.

1. Tunnel Transition - The tunnel transition is 20 feet in length and is lined with 18 inches minimum thickness of reinforced concrete. It varies in section from horseshoe-shape to rectangular, beginning 28.5 feet upstream from the intersection of the tunnel and gate shaft centerlines.

Structural analysis for rock loads was not considered necessary.

Practical considerations made for final design follow:

a. Nominal lining is provided to prevent scour and reduce frictional resistance.

b. Except for the diversion period, external water pressure on the lining, if any, was considered balanced by water pressure inside the tunnel.

c. Hoop reinforcement is provided sufficient to withstand first stage grouting pressures and longitudinal reinforcement is provided to withstand normal thermal expansion and contraction.

In addition to the above considerations, design studies ensured that the section was safe under the following conditions:

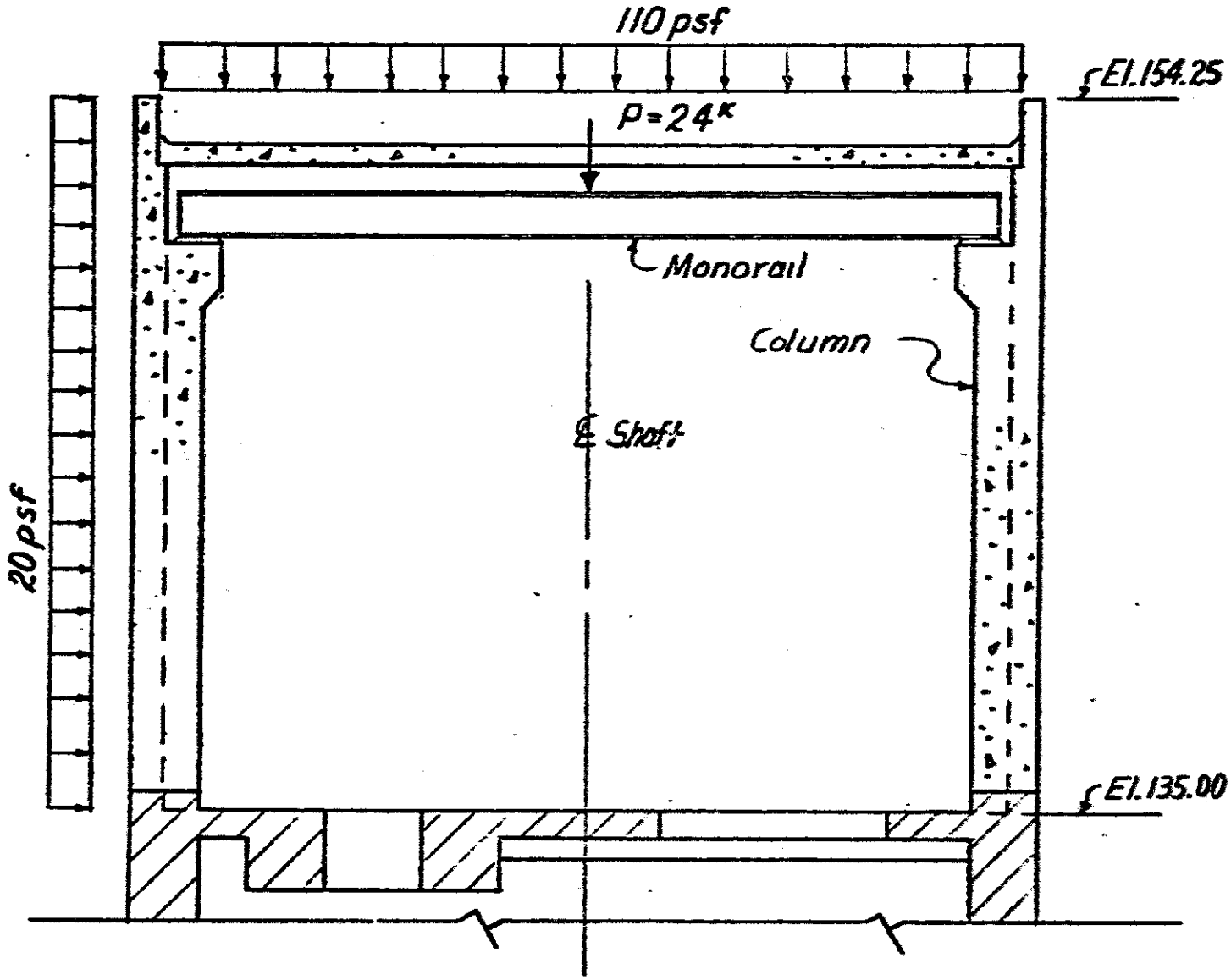
a. Unbalanced uniform internal pressure equal to 20 feet of head, with zero pressure on the exterior.

b. Unbalanced uniform external pressure equal to 20 feet of head, with zero pressure on the interior.

2. Tunnel Plug - Stage I construction in the area of the tunnel plug consisted of a rectangular passage lined with 2 feet minimum of reinforced concrete. The average inside dimensions of this section are 13 feet by 22 feet. It was designed to resist an external water pressure on the lining during Stage II construction.

The structure walls and slab were analyzed as fixed-end beams by moment distribution.

TYPES OF LOADINGS INVESTIGATED
GATE SHAFT HOUSE



KARNAFULI PROJECT

DIVERSION AND OUTLET TUNNEL
DESIGN REPORT

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. P.C.	RECOMMENDED	APPROVED
CK. A.S.Y.		

The combined loadings for final design as illustrated in Figure H were based on the following conditions:

a. An external water pressure on the lining, after closing the tunnel, equal to full hydrostatic head as measured from the invert at Elevation 7.5 feet to the spillway crest at Elevation 80.25 feet.

b. Drains in the tunnel immediately downstream assumed to be 66 percent effective.

Stage II construction consisted of installation of the tunnel plug and slide gate. A portion of the plug becomes the floor of the slide gate chamber.

Reinforcement around the water passage and the slide gate bonnet was provided sufficient to withstand the pressure from full hydrostatic head as measured from the invert of the water passage Elevation 10.52 feet to maximum normal water surface Elevation 118.0 feet.

Reinforcement for normal thermal expansion and contraction was provided for all other exposed surfaces.

F. Slide Gate Chamber

The slide gate chamber, 10 feet by 14 feet by 21 feet high, is constructed adjacent to and downstream from the shaft, above the plug and is symmetrical about the centerline of the tunnel. Access to the chamber is from the gate shaft via a spiral stairway.

Walls of the structure were assumed as fixed-end beams and analyzed by moment distribution.

The combined loadings for final design as illustrated in Figure I were based on the following assumptions:

1. Hydrostatic pressure at the base of chamber is equal to 87.5 feet of head maximum.

2. The drainage system at the downstream side of the chamber is assumed 50 percent effective.

G. Outlet Portal Structure

The outlet portal structure consists of two parts; a section of reinforced tunnel lining and downstream vertical training walls.

Final design of these components of the outlet portal structure evolved from the investigations and studies discussed in the paragraphs which follow.

1. Section of Reinforced Tunnel Lining - This section of the structure is a 20-foot diameter horseshoe-shaped tunnel 40 feet long, with a 20-inch uniform thickness of reinforced concrete lining. The invert of this section has a slope of 0.00289.

Since the actual loads which may come on the structure are variable and indefinite, final designs evolved from practical considerations for safe and economical construction.

2. Downstream Walls - The downstream training walls have a 25-foot maximum height, and are curved for alignment with the discharge channel. The structure has a 25-foot high center pier with stoplog guides, a foot-bridge spanning the training walls and located immediately downstream from the stoplog guides. An intermediate training wall 10 feet high minimizes the scouring and erosive action of undesirable waves in the channel created by centrifugal forces in the curved portion of the structure.

The walls were designed as cantilevered from the base. The slab was designed as a partially fixed-end beam.

The combined loadings for final design as illustrated in Figure J were based on the following assumptions:

a. Saturated backfill to top of walls.

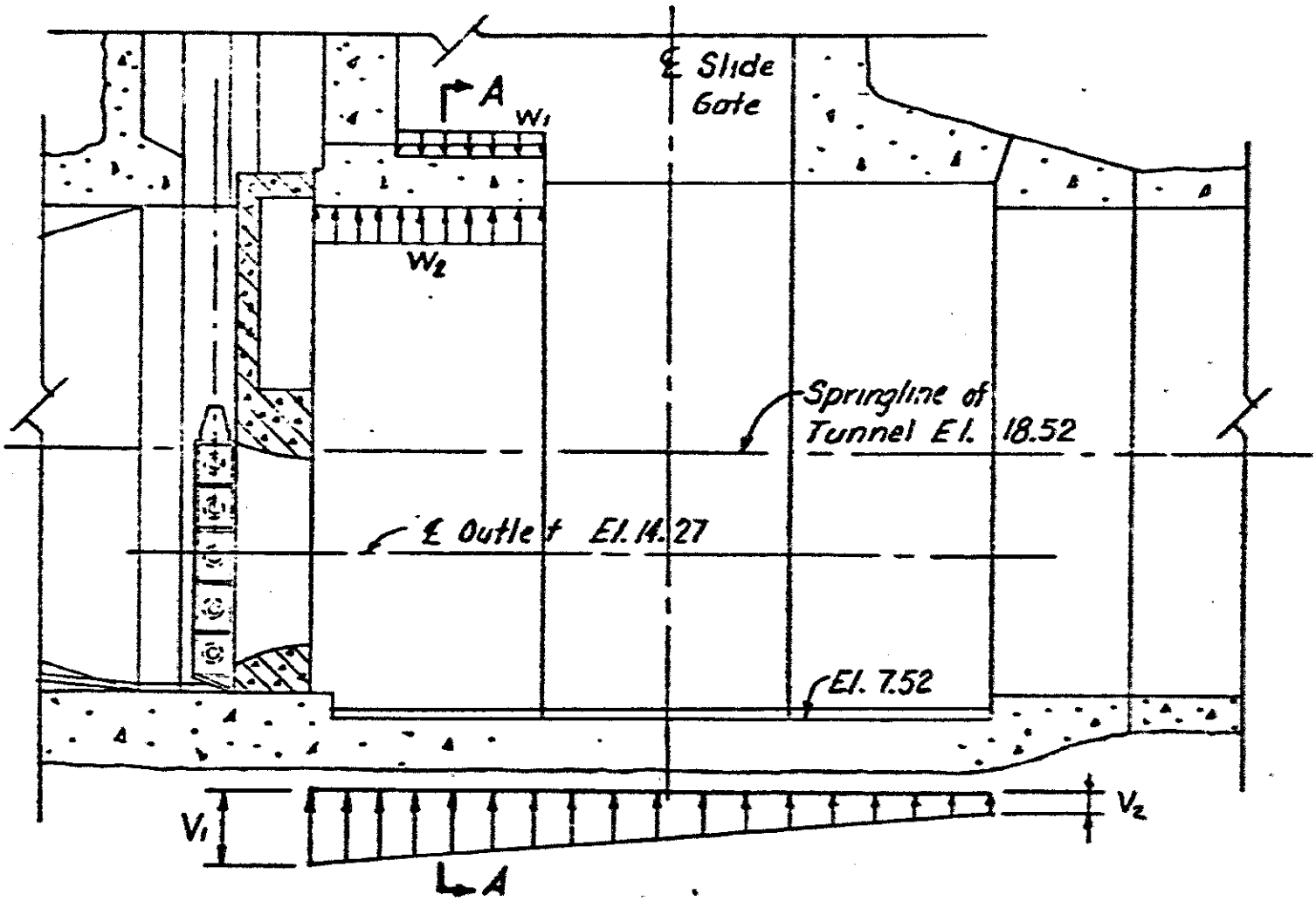
b. Structure submerged downstream from stoplogs.

c. Triangular lateral pressure equal to 40 pounds per cubic foot, from earth only, downstream from stoplogs, and 75 pounds per cubic foot from earth and water combined upstream from stoplogs.

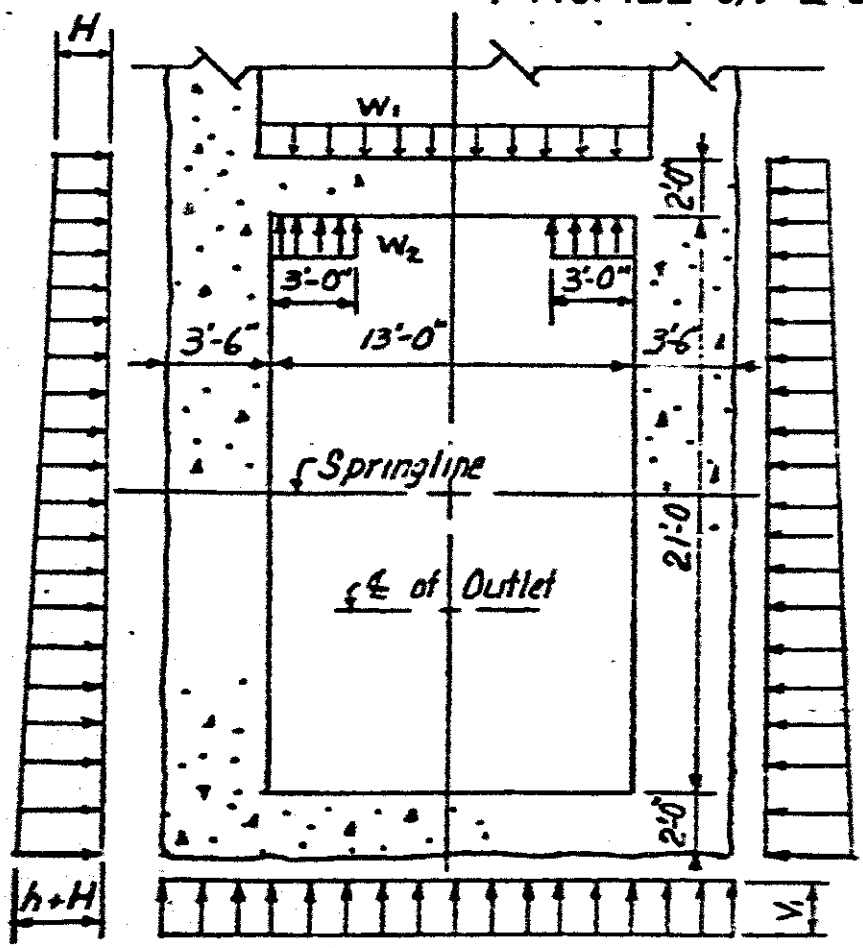
d. Uniform base pressure from structure dead load only.

H. Discharge Channel

The discharge channel is 750 feet long from the outlet portal to the river. It has a trapezoidal cross-section with a bottom width of 20 feet and side slopes of 1.0 to 1.0. The invert at the upstream end of the channel is at Elevation 6.51 feet and has a slope of 0.002 for its entire length.



PROFILE ON ℄ OF TUNNEL



SECTION A-A

Final Design Loading:

w_1 = 200 psf LL

w_2 = 50 psi grouting pressure

H = 3.1 K/FT²

$h+H$ = 4.5 K/FT²

V_1 = 4.5 K/FT²

V_2 = 1.5 K/FT²

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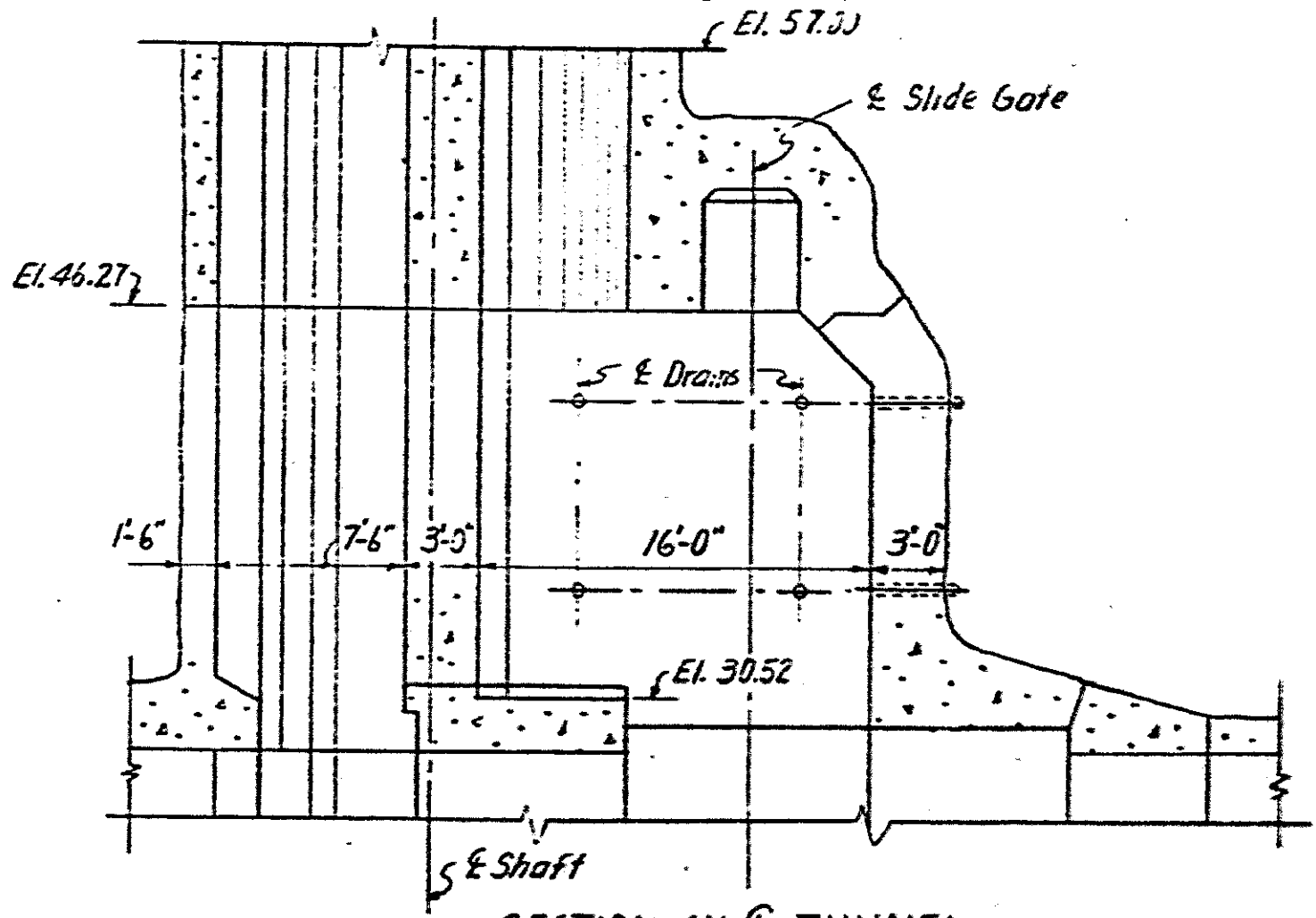
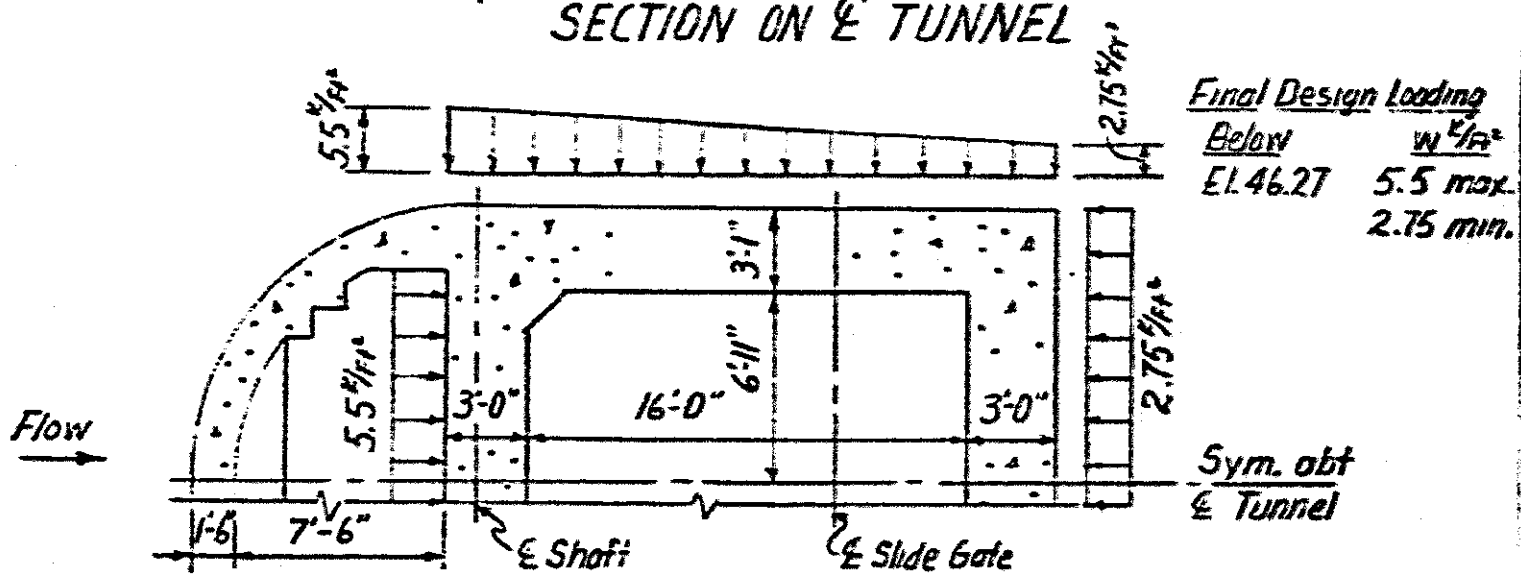
DIVERSION AND OUTLET TUNNEL		
DESIGN REPORT		
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. PC / CK. E.E.H.	RECOMMENDED Pat Enright	APPROVED J W Libby
DATE: June 1958		FIG. H



SECTION ON ℄ TUNNEL



SECTION FOR DESIGN

Assumptions:

1. Loading symmetrical about ℄ tunnel.
2. Hydrostatic pressures on lining are balanced on upstream side of shaft.
3. Drains on downstream end are 50 % effective for varying load as shown.

KARNAFULI PROJECT		
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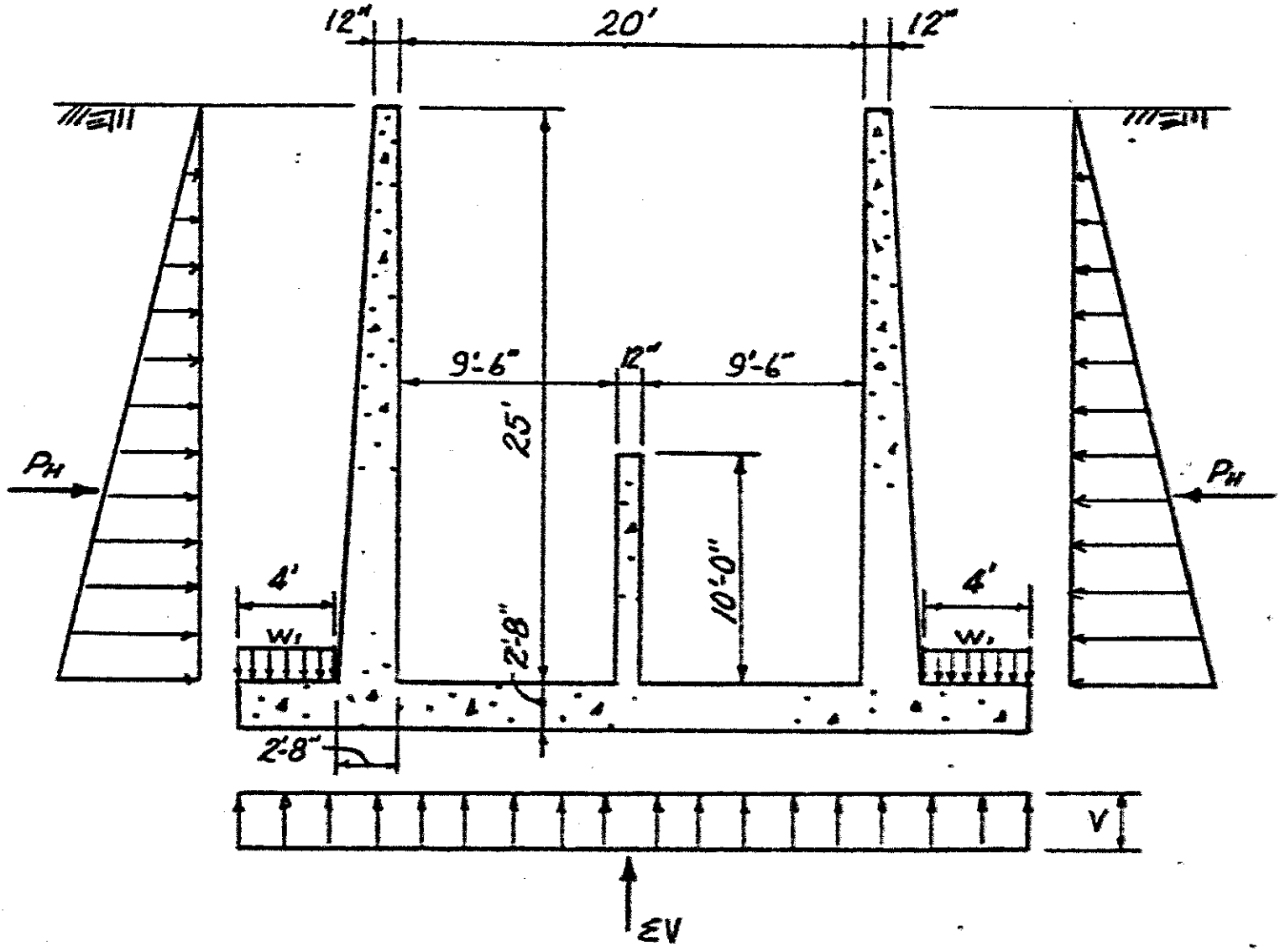
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. PC	RECOMMENDED	APPROVED
CK.		
DATE: June 1958		FIG. I



Final Design Loading:

Downstream from stop-logs

p_o = 40 pcf from earth only

P_H = 12.5 ᵏ

w_1 = 3.25 ᵏ/FT²

Structure DL = 28.6 ᵏ

V ᵏ/FT² = 1.80 ᵏ/FT²

ΣV = 60 ᵏ

Upstream from Stop-logs

p_o = 75 pcf, earth and water combined

P_H = 23.4 ᵏ

w_1 = 3.25 ᵏ/FT²

Structure DL = 27.1 ᵏ

V ᵏ/FT² = 1.75 ᵏ/FT²

ΣV = 58.5 ᵏ

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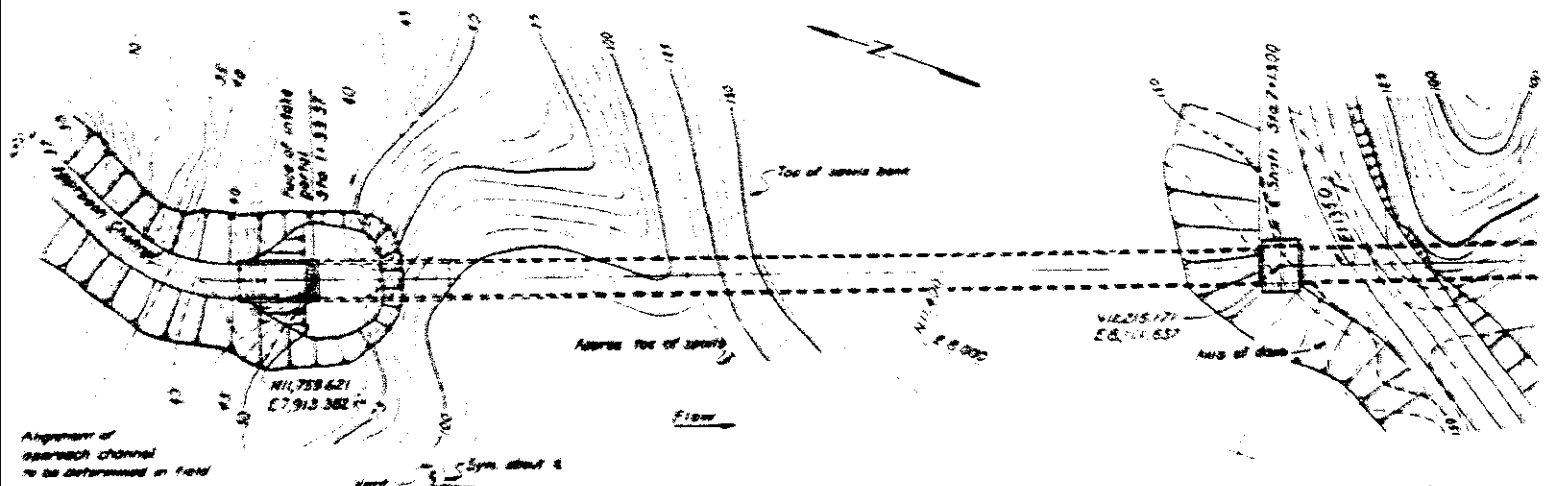
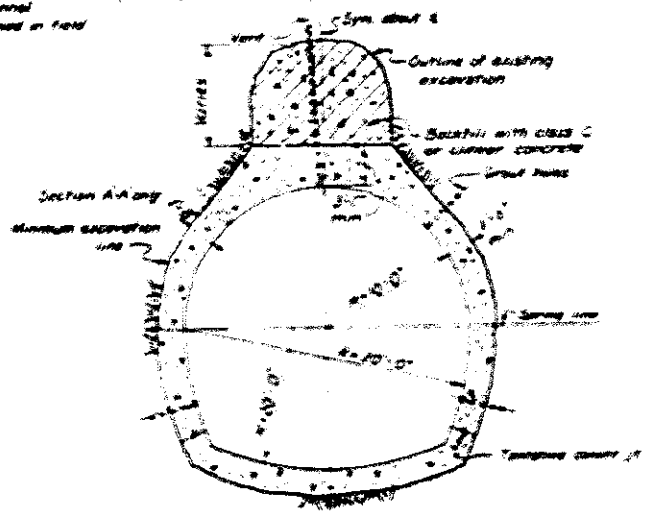
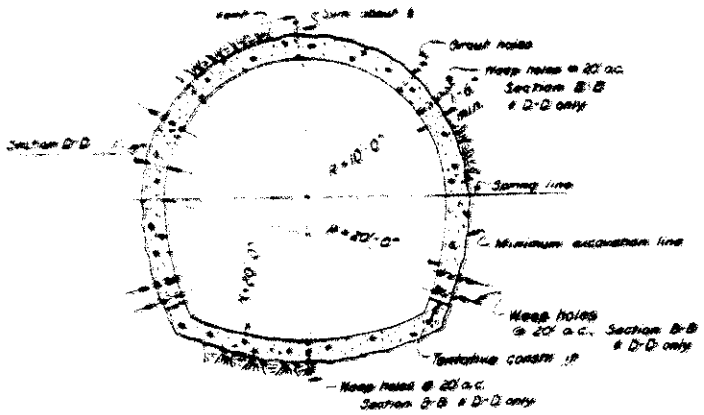
DR. PL / CK. J.E.X.	RECOMMENDED [illegible]	APPROVED [illegible]
DATE: June 1958		FIG. J

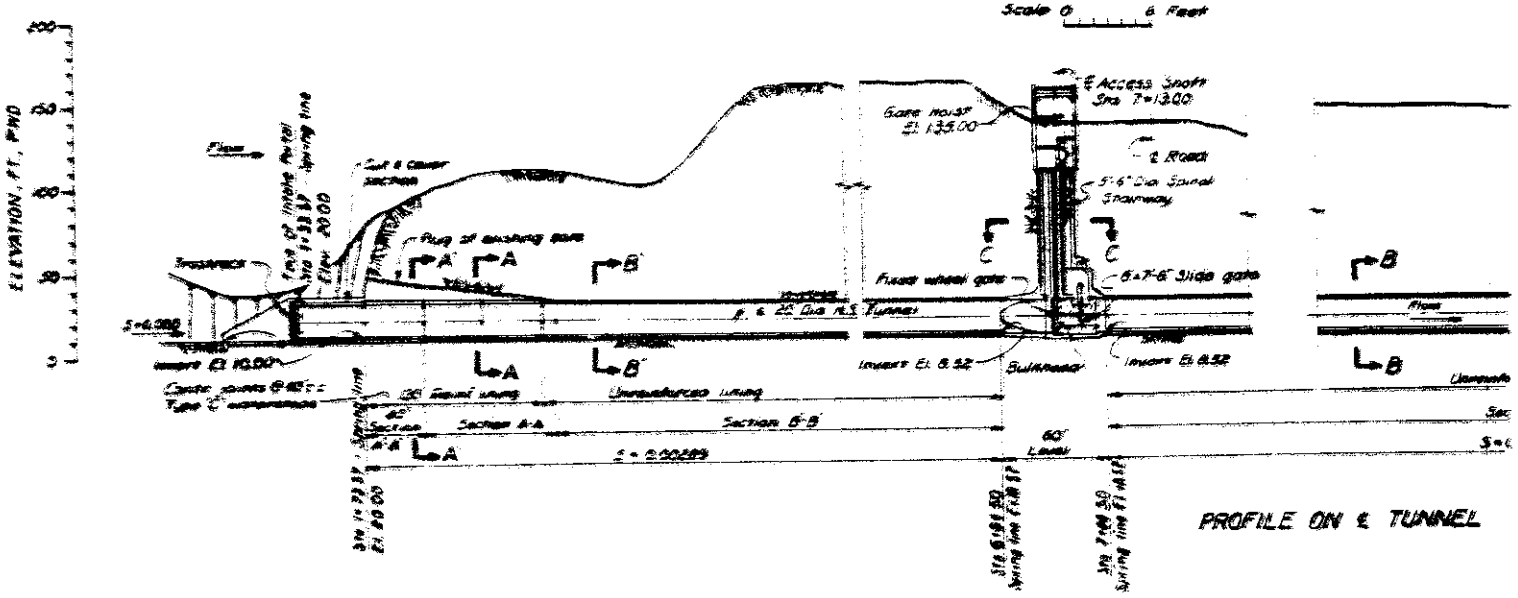
LIST OF EXHIBITS

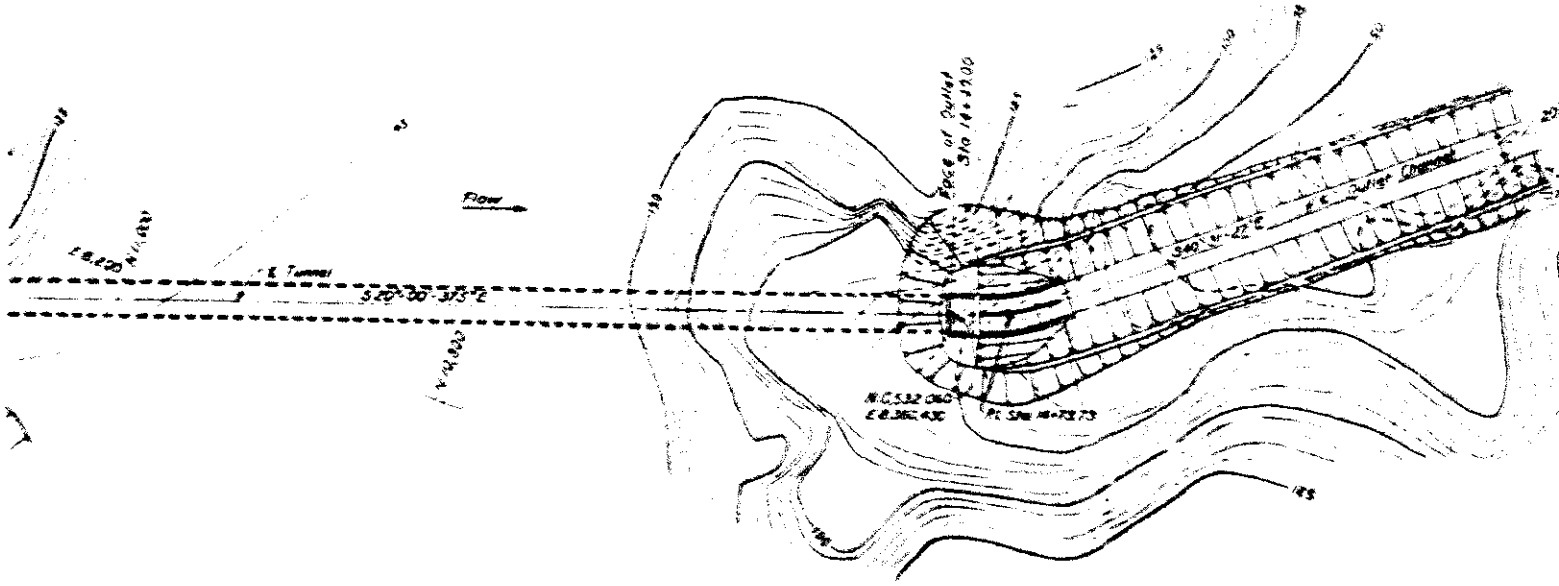
Exhibit	Title
III-1	Diversion - Outlet Tunnel, Alignment, Profile, and Sections

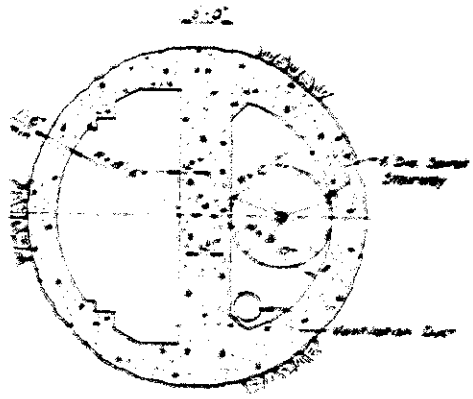

Flow
PLAN


Outline of existing excavation
Grout holes
Section A-A only
Minimum excavation line
Spring line
Tentative contact
SECTION A'-A' as shown
SECTION A-A as noted
Scale 0 6 Feet


Grout holes
Weep holes @ 20' o.c. Section B-B & D-D only
Section D-D
Spring line
Minimum excavation line
Weep holes @ 20' o.c. Section B-B & D-D only
Tentative contact
SECTION B'-B' as shown
SECTION B-B as noted
SECTION D-D as noted
Scale 0 6 Feet


ELEVATION, FT., PWD
Flow
Cut & Cover Section
Plug of existing dam
Gate hoist El. 135.00
℄ Access Shaft Sta. 7+13.00
℄ Road
5'-6" Dia. Spiral Stairway
Fixed wheel gate
6'-7'-6" Slide gate
℄ 22' Dia. R.S. Tunnel
Invert El. 10.00
Invert El. 8.52
Bulkhead
Unreinforced Lining
Section A-A
Section B-B
60' Level
PROFILE ON ℄ TUNNEL



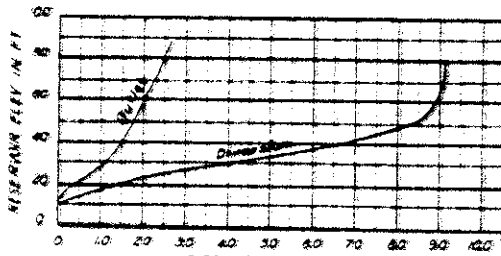


SECTION C-C

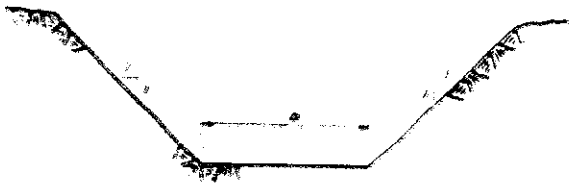
Scale 0 5 Feet

HYDRAULIC PROPERTIES					
	b	S	n	SS	L
Approach Channel	20'	0.000	0.025	n/a	2300'
Outlet Channel	22'	0.002	0.025	n/a	730'

Note:
Hydraulic properties as shown above were used in diversion studies of tunnel. Any changes of section or field must give the same hydraulic results.

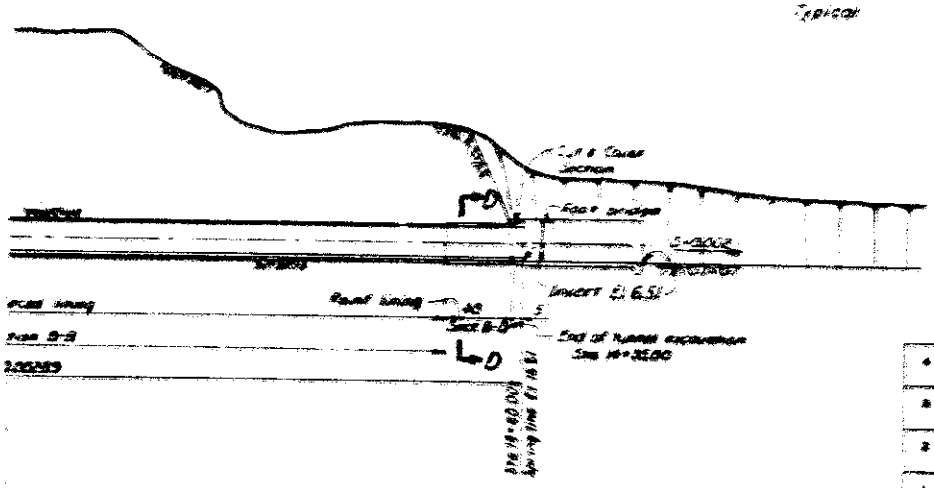


RATING CURVES



APPROACH CHANNEL
OUTLET CHANNEL
(Typical)

NOTES:
Elevations based on PWD datum.
Vent holes to be placed at points of maximum overbre not necessarily on ℄ of tunnel. Spacing of vent h be 15'± on centers.
Grout holes to be drilled 1'± into rock and spaced 15' centers, or as directed by the Engineer.
Construction joints shown are optional and may be on approval of the Engineer to suit construct
Excavation slopes, consistent with the type of mat encountered, to be determined by the engineer.
Install Type "A" waterstops in all transverse construction joints in tunnel lining.



Scale 0 50 100 Feet
except as noted

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
DIVERSION-OUTLET TUNNEL
ALIGNMENT, PROFILE AND SECTIO
INTERNATIONAL ENGINEERING CO., INC. - ENGINEE

Designed: MM	Checked: PC	Submitted:	
Drawn: MM	Approved:		
KA-X-1	San Francisco, Cal. Date: May 6, 1957	FL-32	

PART III

CHAPTER IV

POWER INTAKE

4.01 GENERAL

A. Purpose

The purpose of the power intake is to provide for safe and efficient diversion and control of the flow of water from the reservoir to the hydraulic turbines in the powerhouse.

B. Description

The power intake features consist of the intake structure and three penstocks. PART II, Chapter IV, describes the forebay channel, geologic investigations and foundation and excavation conditions at the site. Trashracks, stoplogs, gates, and hoists are described in PART V, Chapter V.

1. Intake Structure - The intake structure controls and diverts the flows from the forebay channel to the three penstocks leading to the powerhouse turbines. It also prevents debris from entering the penstocks. The principal components of this structure are the trashrack supporting members, water passages, gate shafts, air vents, and operating deck. The appurtenant items are the trashracks, trashrakes, bulkheads, stoplogs, gates, and gate hoists. These are shown on Exhibits IV-1, IV-2, and IV-3.

The intake structure is a free-standing reinforced concrete structure. It is approximately 215 feet wide and 99 feet long and is oriented at right angles to the end of the forebay channel. The overall height is about 95 feet from the top of the foundation slab to the operating deck at Elevation 127 feet.

Trashracks, located at the entrance to the structure, are supported on an inclined concrete frame and extend the full width and height of the structure. Debris which accumulates on the trashracks is removed by a mechanical trashrake. The trashracks are removable for maintenance purposes.

Steel stoplogs sufficient to close one penstock are provided for installation at the entrance to the penstocks for maintenance of the control gates. The stoplogs are positioned by means of a mobile crane which lowers and raises them through openings in the operating deck. Slots in the structure walls and in the dividing piers at the passage entrances guide the stoplogs into place.

Fixed-wheel gates, operated by stationary hoists on the operating deck, are located 12 feet downstream from the stoplog openings. Gate shafts beneath each hoist, and slots in structure walls and dividing piers are provided for the gates. Platforms for servicing the gates are provided at Elevation 92.0 feet. Since the third powerhouse unit will not be installed until some future date, concrete bulkheads are provided for penstock No. 3 in lieu of fixed-wheel gates.

Air vent shafts, which also provide access to the penstocks, are located about 12 feet downstream from each gate shaft. On the deck, above the center air vents, an 18-foot wide by 8-foot long by 8.5-foot high concrete house provides shelter for a reservoir elevation float gage and space for miscellaneous storage.

An 8-inch diameter pipe with valve connects the gate chamber and the reservoir. A 24-inch diameter opening with slide gate connects the gate chamber and the penstock vent shaft. These openings permit bypass of water into the space between the stoplogs and fixed-wheel gates and into the penstock. Bypass of the water enables operation of the stoplogs and fixed-wheel gates under balanced head conditions.

A concrete bridge provides access to the structure.

2. Penstock Tunnels - The three penstocks between the intake structure and the powerhouse are steel lined for a distance of about 307 feet. Each liner consists of:

a. A 32-foot inside diameter section, approximately 259 feet long;

b. A conical section, approximately 32 feet long, which reduces from 32 feet to 25 feet - 3-1/2 inches in diameter;

c. A 25-foot - 3-1/2-inch inside diameter section, approximately 16 feet long, for the scroll case connection.

The portion of the liners within the tunnels is encased in concrete. To intercept seepage and reduce hydrostatic pressure on the penstock liners, a system of drains was provided. This system consists essentially of 19 sets of radial drains (six drains per set) around each liner, and six longitudinal headers which discharge into the coupling chamber.

C. Design

The design of the power intake falls into two major divisions, hydraulic and structural. The two divisions are interdependent. Good hydraulic performance was the prime consideration of design, with economic and structural limitations imposed to result in a practical structure. This chapter discusses both aspects of design.

4.02 HYDRAULIC DESIGN

A. Penstocks

Selection of 32 feet as the diameter of the penstocks was based on an economic comparison of the value of power versus penstock cost. Water hammer effects were considered. Analysis of speed and pressure rises due to load rejection indicated that a surge tank is not required. To meet the peaking requirement at minimum head the velocity in the penstock is approximately 10 feet per second. The velocity at average head is approximately 6.5 feet per second.

The entrance to the penstock is bellmouthed and the transition junctions are streamlined to reduce head losses.

Sudden closure of the fixed-wheel gates tends to create a vacuum behind the gates and in the penstocks unless air is introduced at a sufficient rate to balance the subatmospheric pressure. The air vent shafts just downstream from the gates have been provided for this purpose. These shafts are also used for access. Shaft dimensions were governed by access clearance requirements and are more than adequate for air requirements.

B. Trashracks

A large amount of debris is expected especially during the early phase of plant operation. The straight, sloping trashracks are the most suitable to permit easy trash removal. Velocity through the trashracks is very low and head losses are negligible.

4.03 STRUCTURAL DESIGN - INTAKE STRUCTURE

A. Design Criteria and Assumptions

Basic design criteria and assumptions were as follows:

1. Loads

a. Dead Loads - Dead loads shall consist of the weight of walls, partitions, framing, floors, roof, and all other permanent stationary construction entering into and becoming a part of the structure.

b. Live Loads - Live loads on roof and deck shall be provided for assuming uniform or concentrated loads, or a combination of both, which are likely to occur over the same area. The type of loading causing the higher stress shall govern the design. Assumed live loads are tabulated below:

Location	Uniform Live Load in lbs/sq. ft.	Concentrated Load
Roof	40	300 lbs
Deck		
(1)	300	Trashrake
(2)	300	Mobile crane
(3)	300	Fixed wheel gate hoist
(4) AASHO, H-20 truck loading		
Bridge AASHO, H-20 truck loading		

c. Hydrostatic Pressure and Uplift - The various components shall be designed to sustain the hydrostatic pressures and uplift pressures arising from reservoir conditions involved in the function of the structure.

d. Earthquake Loads - A seismic force equal to 10 percent of the total dead load and fixed live loads shall be assumed to act in any horizontal direction.

e. Wind Loads - Wind pressure shall be assumed as 20 pounds per square foot on all vertical projections above Elevation 75 feet. For open framework, one and one-half times the exposed area shall be used.

f. Earth Pressure

(1) Shale - shall be assumed to exert no horizontal force on vertical walls.

(2) Dry fill - 30 pounds per cubic foot equivalent fluid pressure, plus 300 pounds per square foot surcharge.

(3) Submerged fill - 20 pounds per cubic foot equivalent fluid pressure, plus full hydrostatic pressure.

g. Crane Loads - In addition to the dead and live loads of the crane, a vertical impact of 10 percent of the wheel loads shall be assumed. No lateral or longitudinal force shall be considered.

h. Combined Loadings - The various components shall be designed to sustain at allowable stresses the combined loadings of full dead load and normal live load. For temporary live loads, combined individually with full dead load and normal live load, the allowable stresses shall be increased by 33-1/3 percent. Except for wind and impact, simultaneous occurrence of temporary loads shall not be considered. Allowable stresses resulting from breakdown load on the crane or hoist shall be increased by 50 percent.

2. Allowable Stresses

a. Concrete - Concrete strength for the base slab shall be 2500 pounds per square inch at 28 days. Concrete strength for the remainder of the structure shall be 3000 pounds per square inch at 28 days, except for the trashrack frame which shall be 4,000 pounds per square inch.

b. Reinforcement - Reinforcement shall be intermediate grade billet steel conforming to ASTM Designation A-15. Deformations shall conform to ASTM Designation A305. Normal working stresses shall not exceed 20,000 pounds per square inch.

c. Foundation Pressure - Bearing pressure on shale shall not exceed 20,000 pounds per square foot.

3. Stability

The intake structure shall be stable, under the various loading conditions, with respect to sliding and overturning.

a. Resistance to Sliding - The sliding factor (the ratio of the summation of horizontal forces to the summation of vertical forces) of concrete on shale shall not exceed 0.333.

b. Safety Against Overturning - The factor of safety against overturning (the ratio of total resisting moments to total overturning moments) shall not be less than 1.50.

4. Standards

The design shall conform to the following standards: "Building Code Requirements for Reinforced Concrete (ACI 318-56) of the American Concrete Institute" and "Standard Specifications for Highway Bridges", Sixth Edition, of the American Association of State Highway Officials.

B. Design

1. Stability

The intake structure was investigated for stability with respect to sliding and overturning.

The following table presents the results of stability analyses performed on the intake structure. Other conditions, which were found to be non-critical, are not presented.

No.	Loading Condition	Sliding Factor	Overturning Factor of Safety
I.	At completion of structure, dry. Weight of fill acting on entrance, without arch action. For loading diagram, see Fig. A	0.07	12.3
II.	During operation. No uplift under bottom slab upstream of tunnel entrance. For loading diagram, see Fig. B	0.19	62.0
III.	Immediately after drawdown. 25 percent uplift under bottom slab upstream of tunnel entrance. Water table at Elevation 91.00 feet. For loading diagram, see Fig. C	0.09	12.9

2. Structural Components

a. Entrance and Transition - Each penstock entrance consists of two adjacent bellmouthed rectangular openings. The size of each opening is 11.5 feet wide by 32 feet high and was selected on the bases of a study comparing hydraulic performance with practical limitations of gate and stoplog design.

The transition extends from the downstream side of the intake gate slots to the upstream end of the penstock. In the direction of flow, the section gradually changes from the two rectangular openings to a single 32-foot inside diameter tunnel. The entire component is a cut and cover tunnel 58 feet long on a slope of minus 0.1116 to 1.

Five sections, one at each end and three in the transition, were investigated under the following loading conditions:

Case I - Structure resting dry with backfill in place.

Case II - Saturated backfill to Elevation 91 feet without uplift, penstock empty.

Case IIIa - Water surface at Elevation 119 feet, penstock full and in operation, no uplift.

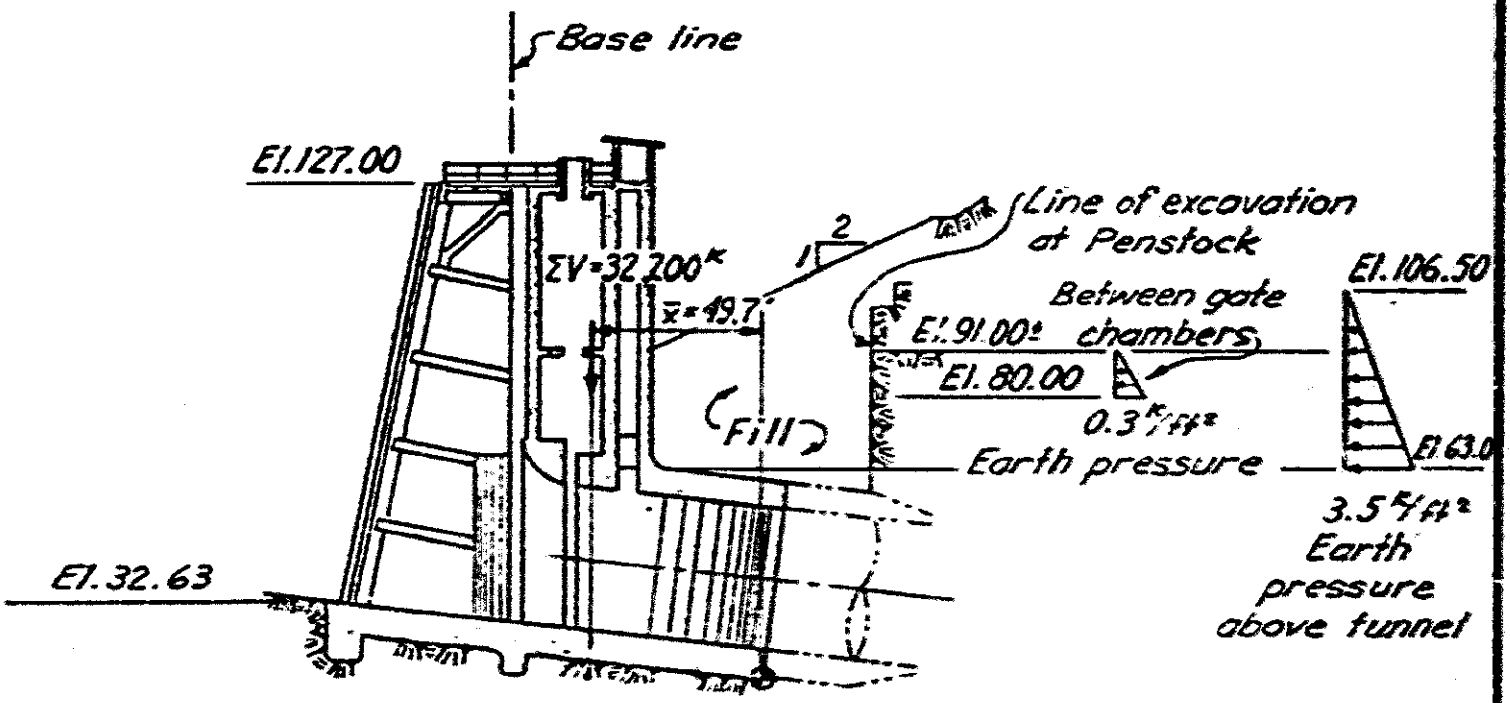
Case IIIb - Water surface at Elevation 119 feet, penstock empty, no uplift.

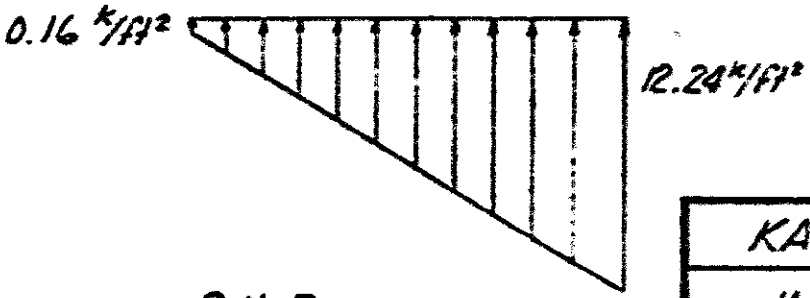
LOADING CONDITION I

Structure at completion.
Forebay channel dry.
ΣV = Wt. of concrete + wt. of fill above tunnel entrance.

Sliding factor _ _ _ _ _ _ _ _ _ _ _ _ _ _ _ _ 0.07
Overturning factor of safety _ _ _ _ _ _ _ _ 12.3

Note: Analysis made for middle 78' of structure.





Soil Pressure

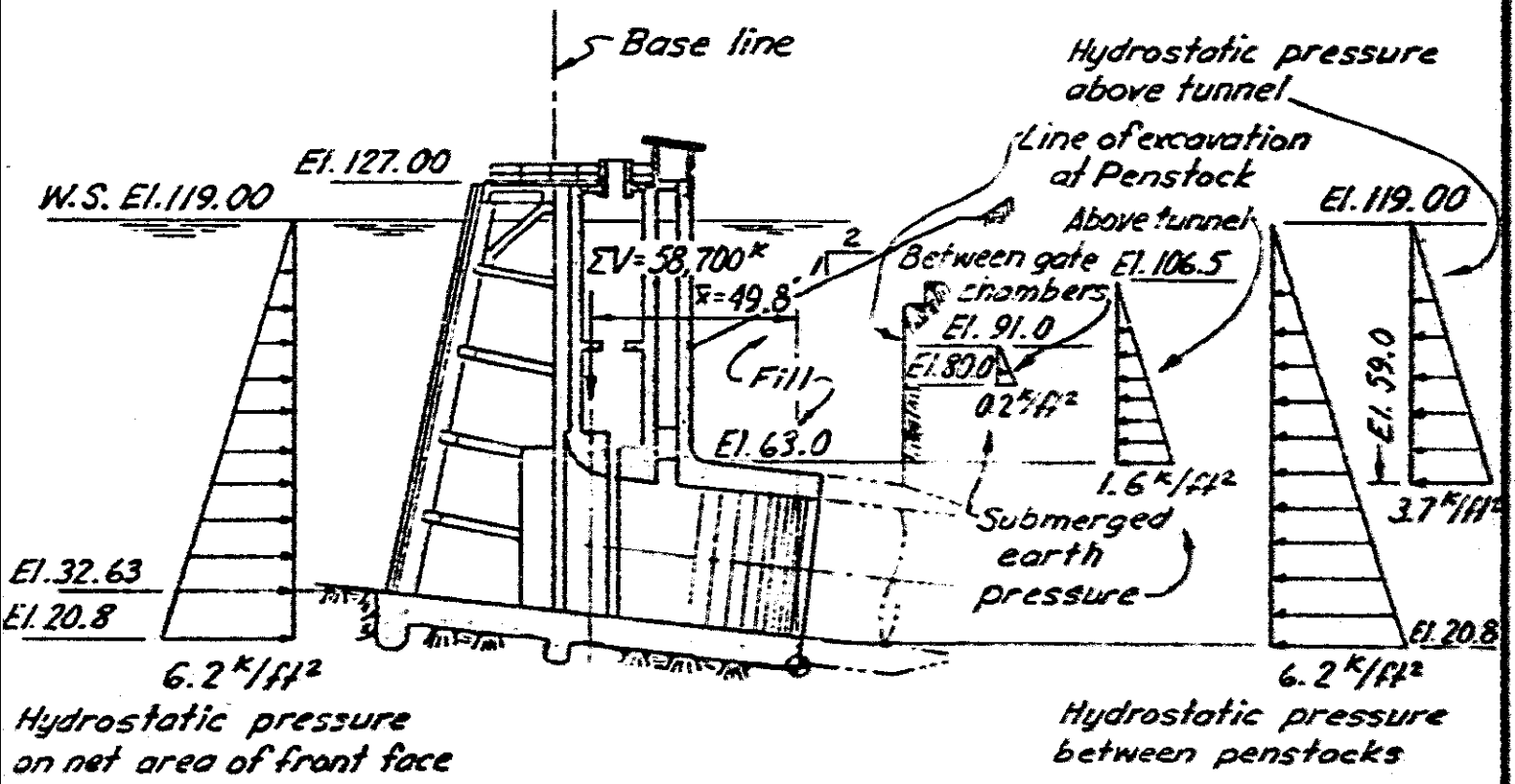
Scale: 1" = 50'-0"

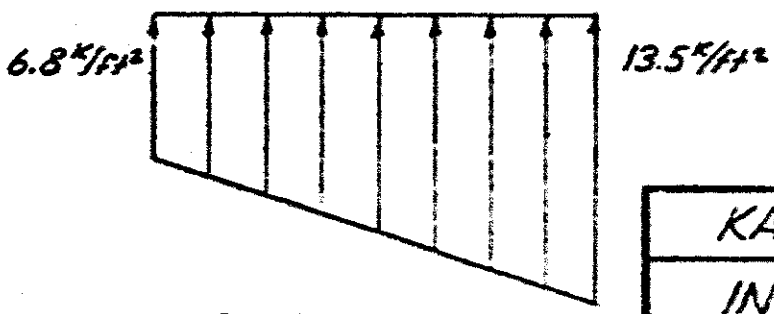
KARNAFULI PROJECT		
INTAKE STRUCTURE LOADING CONDITION I		
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. RHG CK. EPK	RECOMMENDED	APPROVED

Structure in operation.
Water surface El. 119.00.
Static water pressure in penstock.
No uplift.
ΣV = Wt. of concrete + wt. of water + wt. of fill above tunnel entrance.

Sliding factor _____ 0.19
Overturning factor of safety _____ 62.0

Note: Analysis made for middle 78' of structure.





Soil Pressure

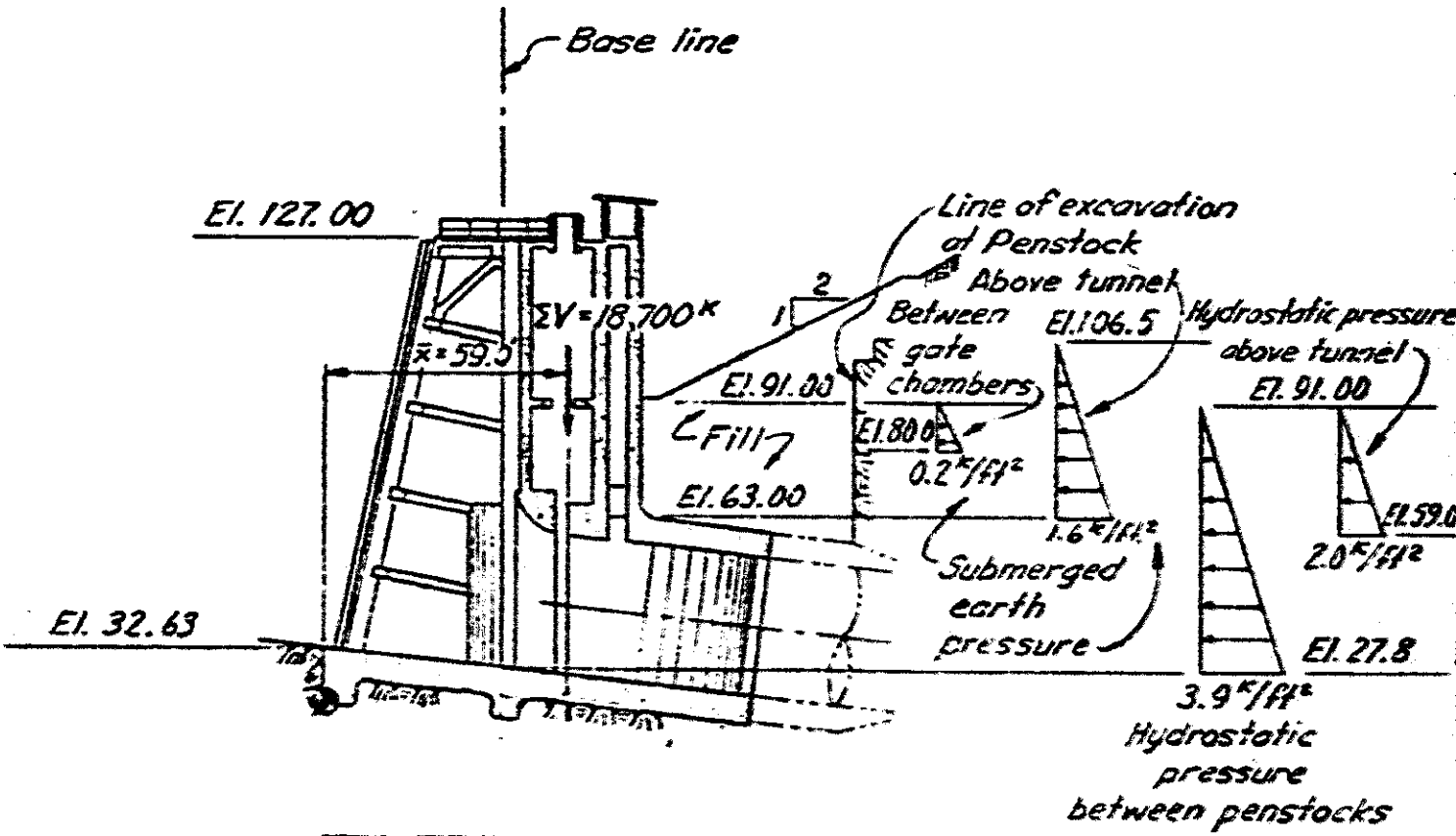
Scale: 1" = 50'-0"

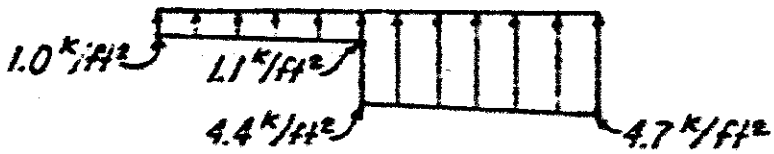
KARNAFULI PROJECT		
INTAKE STRUCTURE LOADING CONDITION VII		
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. RHG / CK. EPK	RECOMMENDED	APPROVED
DATE: 1-15-59	FIGURE B	

Forbay channel drawn down.
Full uplift with water table at El. 91.00
(25% of full uplift on approach slab)
ΣV = Wt. of concrete + wt. of fill above tunnel entrance - uplift.

Sliding factor _____ 0.09
Overturning factor of safety _____ 12.9

Note: Analysis made for middle 78' of structure.





Uplift

Scale: 1" = 50'-0"

Soil Pressure

KARNAFULI PROJECT

INTAKE STRUCTURE
LOADING CONDITION II

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. RHG	RECOMMENDED	APPROVED
CK. EPK		
DATE: 1-15-59	FIGURE C	

Loadings at Station 62 + 18.00 are shown on Figure E. Loadings at intermediate and downstream sections are similar.

b. Base Slab - The base slab serves as a foundation for the structure and as an apron for the forebay at the entrance to the penstock. It is 6 feet thick, 39 feet long and extends the full width of the intake structure. In the direction of flow it lies on a slope of minus 0.1116 to 1.0. Six-foot deep cutoff walls are located both at the upstream and downstream ends. The base slab was reinforced as a one-way slab to resist the foundation pressures in beam action.

c. Piers, Beams and Struts - The piers support the trashrack. Alternate piers are supported by struts. Intermediate piers are supported by horizontal beams which also rest on the struts. A diagonal strut is used at the top to stiffen the structure in the direction of flow.

The system of piers, struts and crossbeams was designed to resist probable combinations of the primary and secondary loads.

The primary loads used for the design of these members were a 20-foot head differential across the trashrack and dead weight. Because of the large width of the intake structure, secondary temperature and shrinkage stresses required special attention. Compressive stresses in bending induced by temperature changes were high and the use of high strength (4,000 pounds per square inch) concrete was justified. A temperature differential of 30 degrees Fahrenheit was assumed adequate in view of the massiveness of the crossbeams. Shrinkage stresses were controlled by the arrangement of the lifts and pours.

In the vertical direction stresses were indeterminate but not critical since the structure is freestanding with a relatively small horizontal loading. All members were reinforced with a minimum of one percent of their cross-sectional area.

d. Walls - The structure has three principal walls which retain the fill at the sides and above the cut and cover tunnels. Together with the piers they support the operating deck.

The gate chamber and air-vent are separate walled enclosures attached to the downstream wall. Above the surface of the backfill the walls were designed to withstand 20 feet of head. Below, they were designed for 20 feet of head plus the saturated backfill. The side walls were also designed to withstand stresses induced by temperature deflection of the crossbeams. The gate chambers and air-vents can be drained independently and were designed for full reservoir head in addition to backfill loads.

e. Deck Slab - The deck slab provides space for operation of the trashrack rake, to install stoplogs, and to operate and maintain the fixed wheel gates. The deck and supporting beam system was designed to carry the dead weight and the live loads which are listed in Paragraph 4.03, A., 1., b. of this chapter. Two-way design was used for the slabs.

The deck slab is sloped for drainage. Hatch covers, curbs and hand rails all around are provided for the safety of personnel.

f. Deck House - The deck house is constructed of concrete and serves as office and storage room and also provides shelter for the reservoir recording gage. The walls of the single-story building were designed to carry the roof and resist wind load. They are nominally reinforced. The roof was designed as a two-way slab.

g. Gate Chamber and Vent - The gate chamber is used to store and service the gates. In the full raised position the bottom of the gate is at Elevation 92 feet. A service floor has been provided at Elevation 92 feet and clearance is available all around the gate.

Another platform was provided at Elevation 119.00 feet which is above maximum reservoir elevation, and will permit operation of the slide gate and valve to balance the head prior to opening the gates or removing the stoplogs. Access ladders and railing for the platforms are provided.

An access ladder has been provided in the air vent which can be used as an entrance to the penstock.

In addition to backfill loads, the three conditions which were used for design are as follows:

Case I - 20 foot head outside.

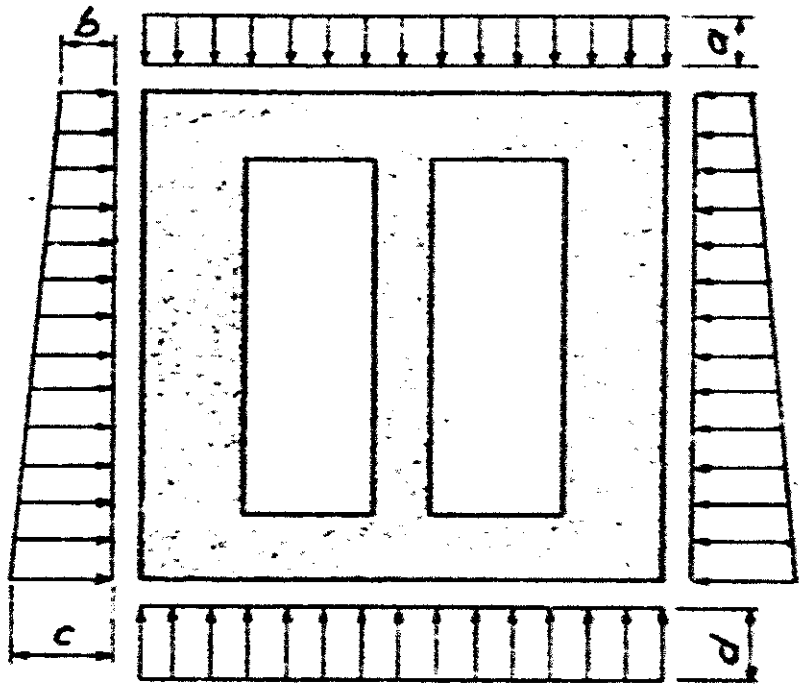
Case II - Reservoir full, wheel gate closed and vent empty.

Case III - Reservoir full, stoplogs in place and both gate chamber and vent empty.

The above loadings are shown on Figure F.

h. Access Bridge - The access bridge was designed to carry an H-20 truck load. It is a 16-foot wide bridge supported by the bank, a central pier and the right side wall of the intake structure in two simple spans. The bridge deck was designed to be poured in place on three precast girders. The girders are similar to the precast girders used for the spillway bridge and were precast in the same forms.

The precast girders were designed to carry the total dead weight of the bridge and were investigated for erection stresses and found to be safe. High strength concrete (3750 pounds per square inch) was used to minimize the size and weight of the girders, making them easier to handle. They are cambered to offset deflection resulting from the dead weight of the bridge. Keyways are provided at the construction joint between girder and bridge slab near the ends of the span where the shear stress is highest. Holes were formed near each end of the girders to provide convenient lifting points.



STA. 62+18.00

Loads in $^k/ft^2$

	a	b	c	d
Case I	4.3	0	0	7.7
Case II	4.9	2.0	4.4	7.4
Case III_a	3.0	0	0	10.5
Case III_b	6.8	3.7	6.1	10.5

Scale: 1" = 20'-0"

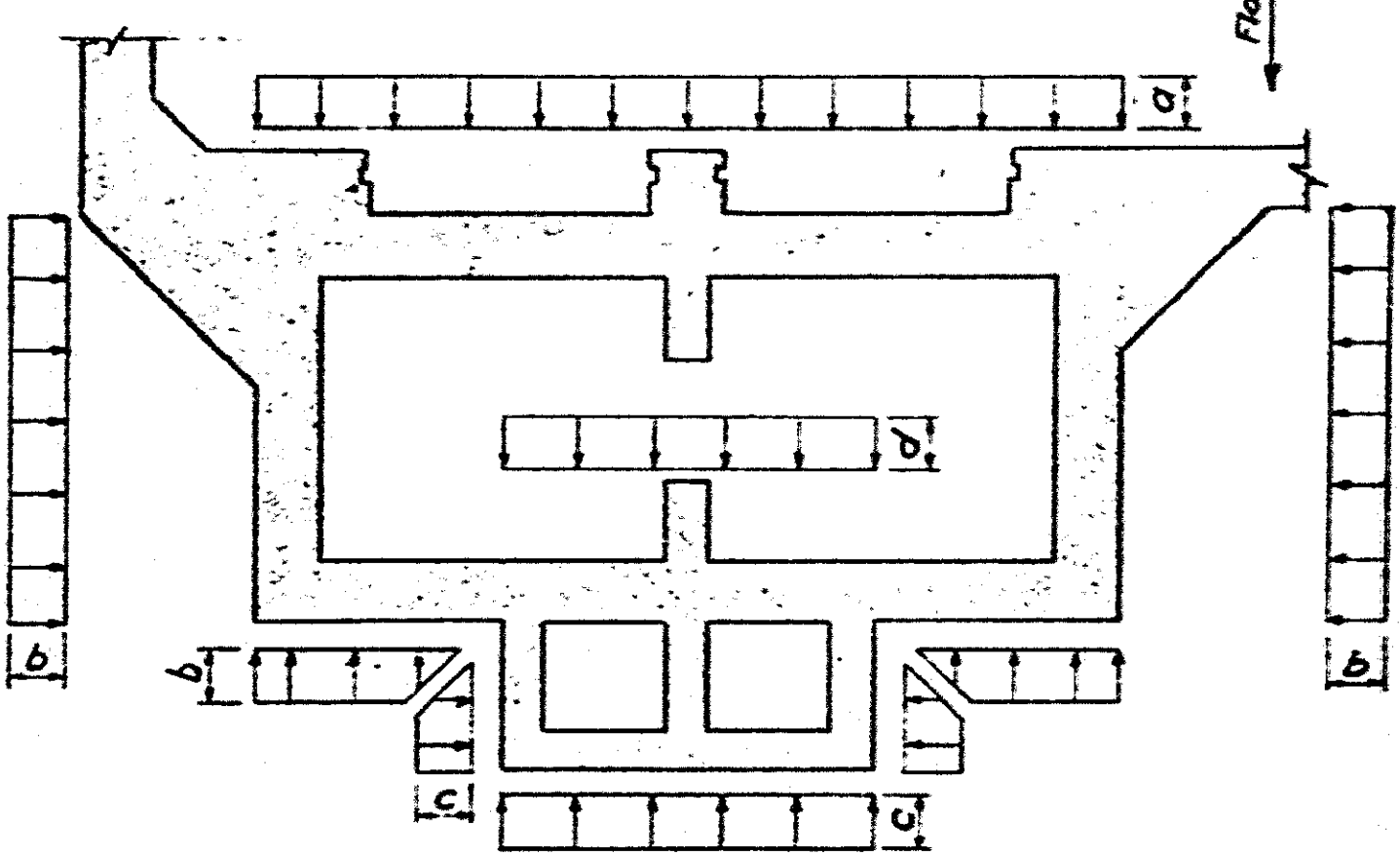
KARNAFULI PROJECT

INTAKE STRUCTURE
PENSTOCK ENTRANCE

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. RHG	RECOMMENDED	APPROVED
CK. WRS	C.E.H.	CAK

DATE: May, 1961 | FIGURE E



SECTIONAL PLAN

Loads in $^{k}/_{ft}^{2}$

	a	b	c	d
Case I	0	$1.25\ ^{k}/_{ft}^{2}$	$1.25\ ^{k}/_{ft}^{2}$	0
Case II	0	0	.0624h	.0624h
Case III	.0624h	.0624h	.0624h	0

h = 119 - Elevation of section in question.

Scale: $\frac{3}{32}$" = 1'-0"

KARNAFULI PROJECT

INTAKE STRUCTURE
GATE CHAMBER

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. RHG	RECOMMENDED	APPROVED
CK. WRS	C. E. H.	[illegible]
DATE: May 1961	FIGURE E	

The central pier is a bent formed by a top beam, two supporting columns and a foundation block. The columns were designed for eccentric loading caused by a truck on one span as well as dead load and earthquake load. Wind load was not a governing design factor.

The foundation block was designed to limit soil pressure to 10 kips per square foot under the eccentric load and also to resist internal stresses caused by the column moments and foundation pressures.

The bridge abutment was designed to support the bridge and retain the roadway. A large bearing area was provided to keep foundation pressure low since the abutment is located near the edge of a cut.

4.04 STRUCTURAL DESIGN - PENSTOCK TUNNELS

A. Design Criteria and Assumptions

Basic design criteria and assumptions were as follows:

1. Penstock Liner - The penstock liners shall be designed to adequately resist (1) deflections and distortion during fabrication and erection, (2) internal hydrostatic pressure (including water hammer) after installation, and (3) external pressure due to grouting or seepage in the concrete encasement.

2. Concrete Encasement - The concrete which encases the liner shall be designed to adequately resist external hydrostatic pressure. The encasement adjacent to the upstream portal shall be designed to additionally resist the load of the overlying material.

3. Allowable Stresses

a. Concrete - The concrete which encases the penstock liners shall have a strength of 2,500 pounds per square inch at 28 days.

b. Steel - the penstock liner steel shall conform to ASTM Designation A-285, Grade B, Firebox Quality. Stresses shall not exceed 11,250 pounds per square inch.

4. Standards

The design of the penstock liner shall conform to the American Society of Mechanical Engineers "Boiler and Pressure Vessel Code", 1956 Edition.

B. Design of Steel Liners

The penstock liners were designed to resist deflection and distortion during installation, as well as internal hydrostatic pressure after installation, and external pressure due to grouting or seepage in the concrete encasement.

Determination of the minimum plate thickness for resistance to deflection and distortion during installation was based on the equation, $t = \frac{d + 20}{400}$, where "t" is the plate thickness in inches, and "d" is the internal diameter in inches.

Determination of the plate thickness for resistance to internal hydrostatic pressure was based on the "free pipe stress", and computed using the equation,

$t = \frac{pr}{s}$, where "t" is the plate thickness in inches, "p" is the internal hydrostatic pressure in pounds per square inch, "r" is the inside radius of the liner in inches, and "s" is the allowable circumferential stress in pounds per square inch. The internal hydrostatic pressure was computed from the pressure heads along the penstock profile, as shown on Figure D. These pressure heads include a maximum 37 percent rise, based on full load rejection at maximum head with a 5-second governor time.

The "free pipe stress" for the entire liner length is shown on the stress diagram of Figure D. Examination of the stress diagram shows that, except for a short length at the downstream transition where internal hydrostatic pressure governed, the necessary plate thickness was governed by the requirement for resistance to deflection and distortion during installation.

The penstock liners were investigated for safety against buckling due to external pressure using the method given by Vaughan in the Proceedings of the American Society of Civil Engineers, Separate No. 949. The results of the investigation indicated that the theoretical critical external pressure on the liner is 183 feet of hydrostatic head. This is well above the maximum reservoir hydrostatic head of 114 feet.

In order to maintain a high margin of safety against buckling due to external pressure the following provisions were made:

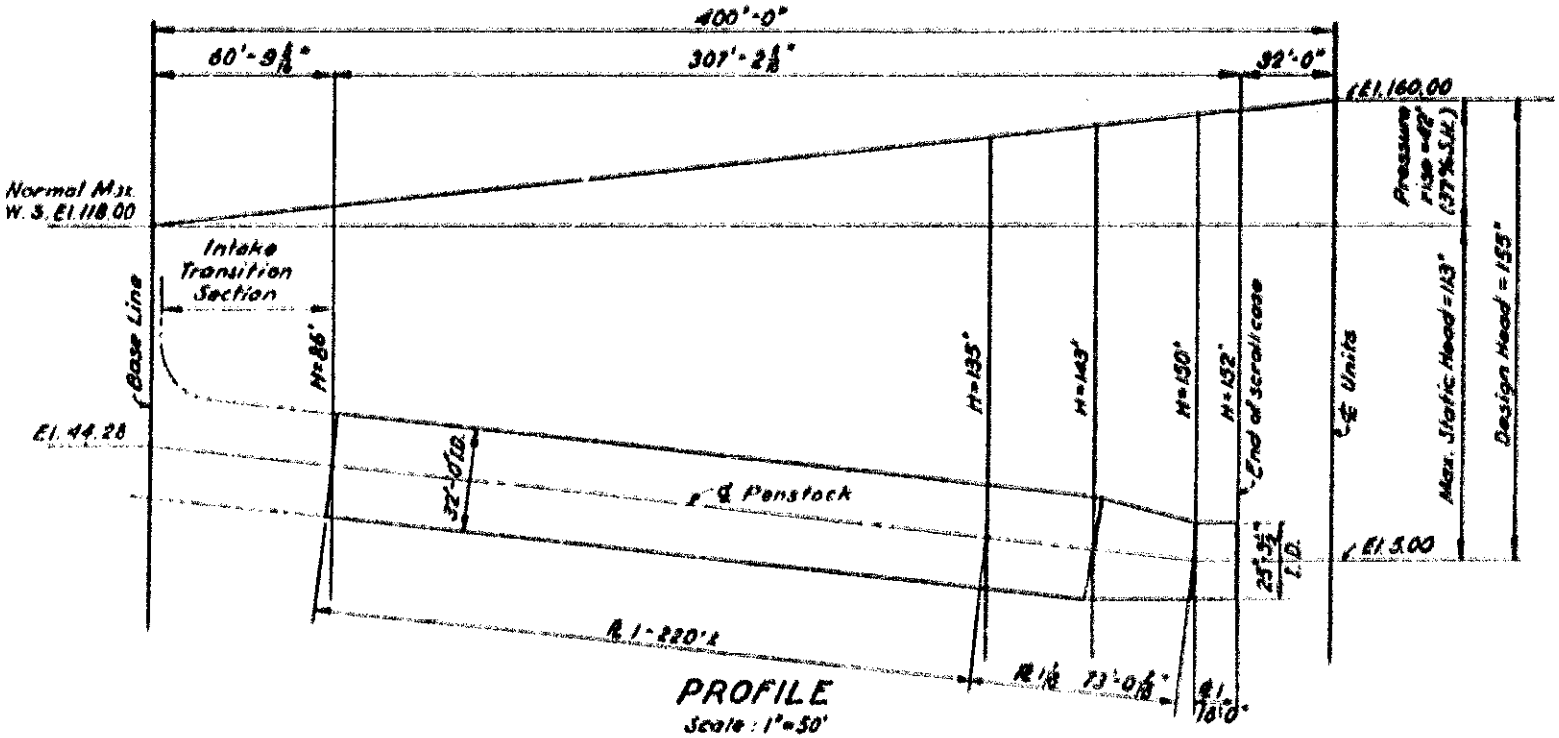
1. Close fabrication tolerances were specified to obtain a circular pipe section.

2. The space between the liner and concrete encasement was grouted to ensure close contact.

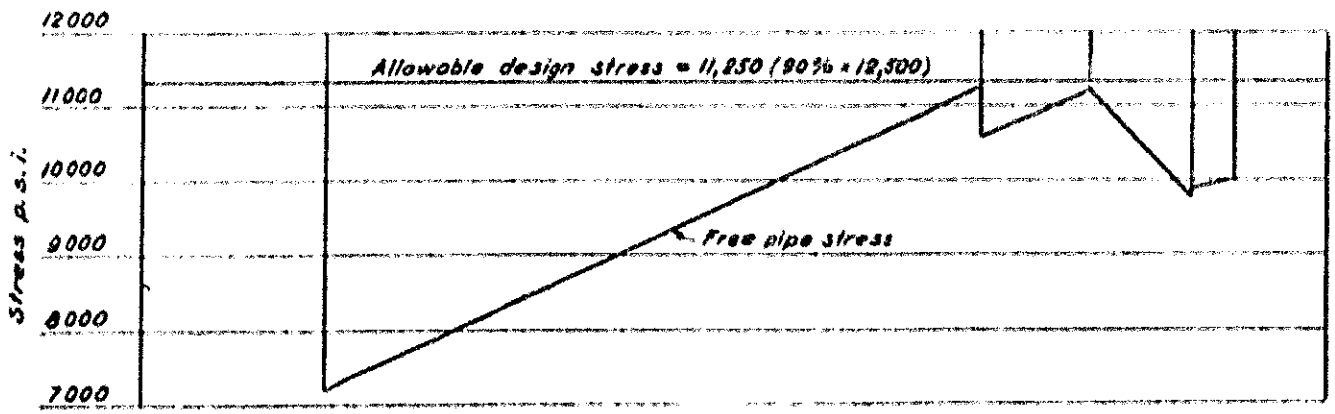
3. A system of drains was designed and installed to limit external hydrostatic pressure on the liner.

C. Design of Concrete Encasement

1. General - The portion of the concrete encasement adjacent to the upstream portal was designed to resist external hydrostatic pressure, as well as the load from the overlying material. The portion of the encasement adjacent to the downstream portal is monolithic with the up-



PROFILE
Scale: 1" = 50'



STRESS DIAGRAM
Scale: 1" = 2000 p.s.i.

NOTES:
Material - Plates ASTM-A 285 Grade
Pipe box.
Min "t" for handling: $\frac{D+20}{400}$ = 1" for 32'
and $\frac{13}{16}$" for 25'-5 1/2" pipe

KARNAFULI PROJE
PENSTOCK LINER PROF
PRESSURE HEADS
STRESS DIAGRAM
INTERNATIONAL ENGINEERING CO.
SAN FRANCISCO, CALIF.
DR. V.S.D.
CK. R.K.W.
RECOMMENDED R.C.Y.
APPR
DATE: 7-2-58
FIGURE

stream wall of the powerhouse coupling chamber. This wall is separated from the rest of the powerhouse structure by a contraction joint. The wall and concrete encasement are subject to external hydrostatic pressure.

2. Reinforced Encasement - The portion of concrete encasement between the portals was designed to resist external hydrostatic pressure, using the equation

$s = \frac{pr}{t}$, where "s" is the circumferential compressive stress in pounds per square inch, "p" is the external hydrostatic pressure in pounds per square inch, "r" is the outside radius of the concrete encasement in inches, and "t" is the encasement thickness in inches. The minimum encasement thickness is 2.5 feet. Compressive stresses computed by the above formula were very low and reinforcement was not required. However, because of possible horizontal movements of the shale, the concrete encasement in the central portion of the tunnels was nominally reinforced in both the longitudinal and transverse directions.

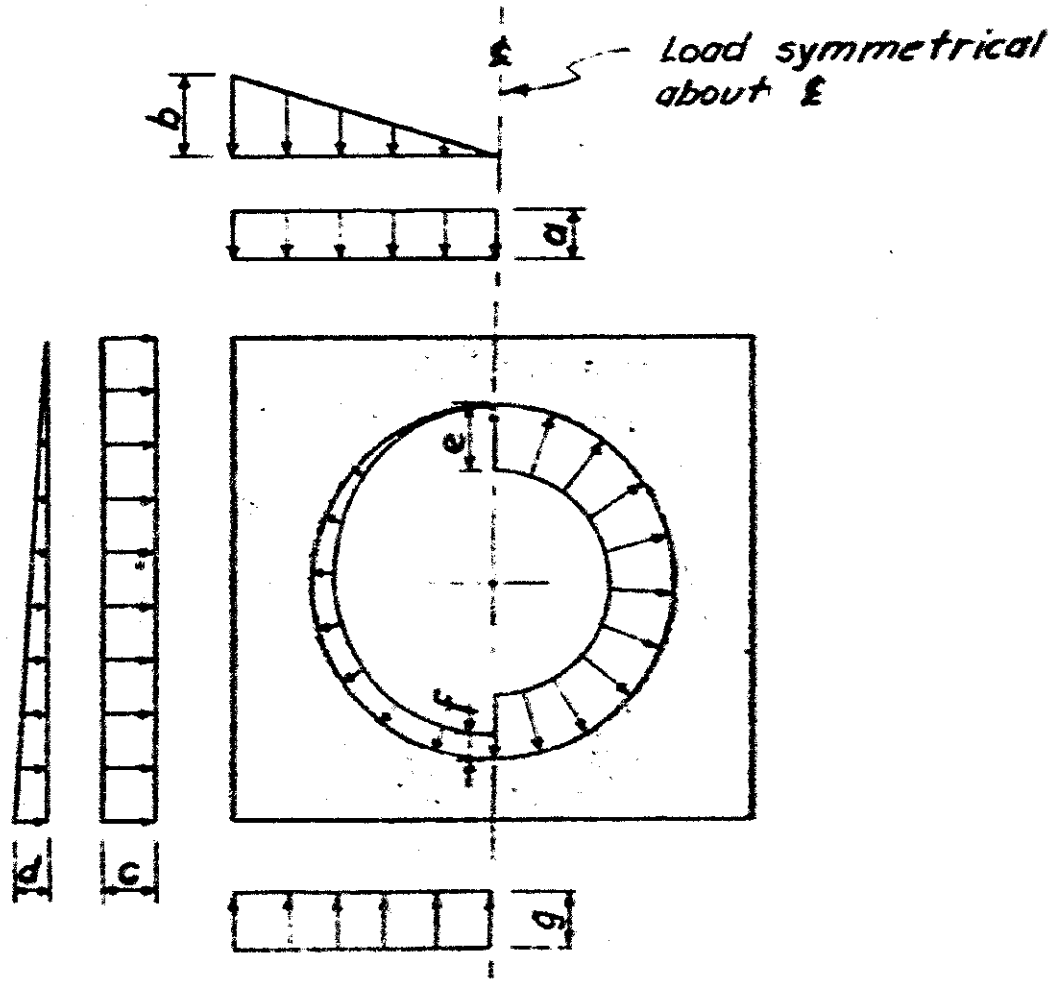
The upstream 16 feet of the penstock is a cut and cover conduit and was designed to support its own weight plus three combinations of external loads. The section was analyzed by the use of coefficients from United States Bureau of Reclamation, Engineering Monograph No. 14, "Beggs Deformeter Stress Analysis of Single-Barrel Conduits." The governing external loading conditions were as follows:

	Outside Vertical	Outside Horizontal	Interior
Case I	52 feet of saturated fill, water surface Elevation 119.0 feet	Hydrostatic pressure from Elevation 119.0 feet	None
Case II	Same as Case I	Same as Case I plus saturated soil at rest	None
Case III	52 feet of dry fill	None	None

See Figure G.

The next 32 feet, proceeding in the direction of flow, is inside the tunnel and tapers in strength from the heavily reinforced cut and cover section to the nominally reinforced central part of the tunnel lining.

The downstream 36 feet of penstock lining is nominally reinforced on each face and is monolithic with the reinforced upstream wall of the coupling chamber.



STA. 62 + 53.00

LOADS IN $^{K}/_{FT^2}$

	a	b	c	d	e	f	g
CASE I	3.78	7.56	3.78	2.76	0	0	7.60
CASE II	3.78	7.56	5.68	4.36	0	0	7.60
CASE III	0	12.00	0	0	0	0	6.00

NOTE:
Dead load of concrete not shown

KARNAFULI PROJECT

UPSTREAM SECTION OF PENSTOCK LINER

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

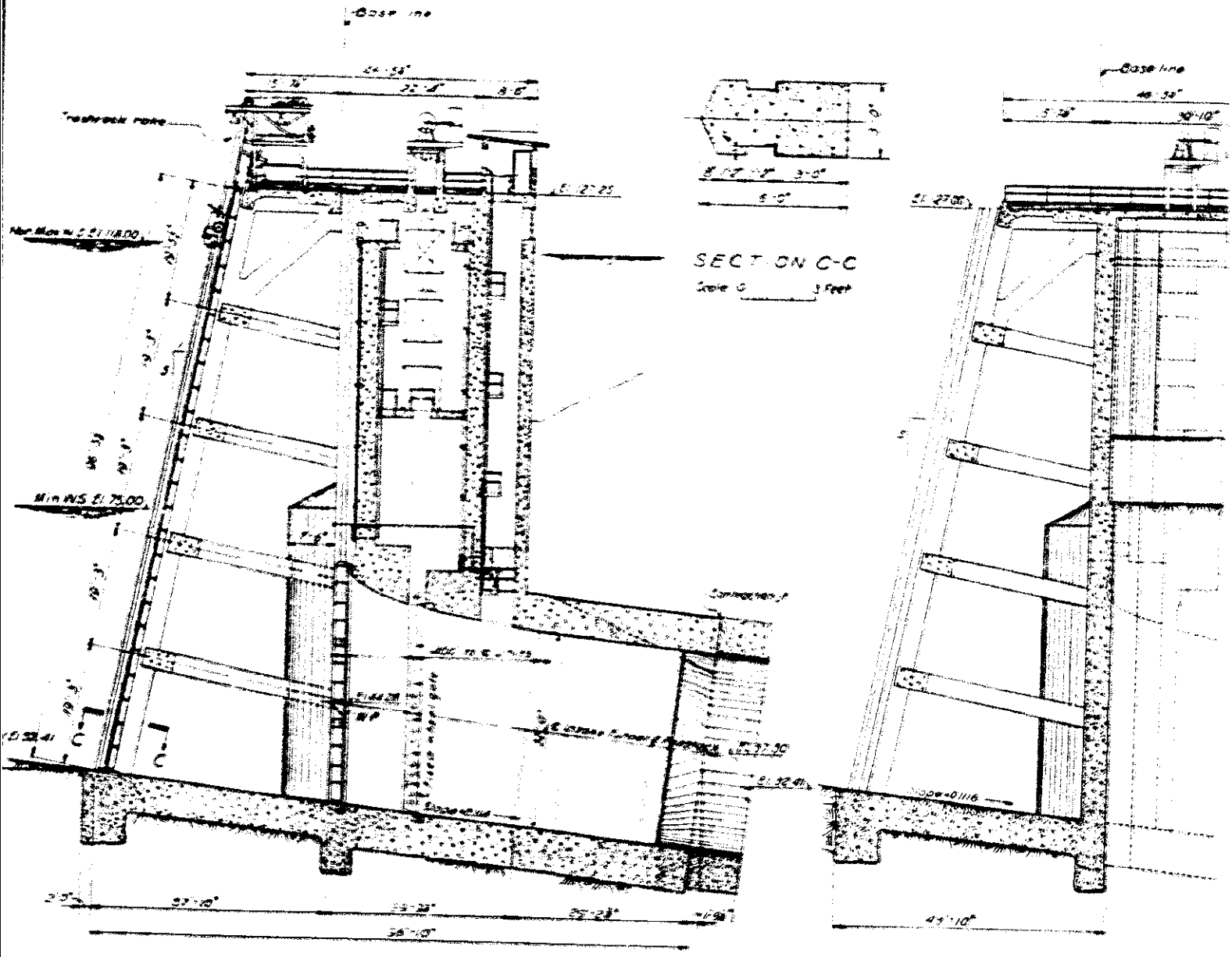
DR. V.G.	RECOMMENDED	APPROVED
CK. HRS	a.E.H.	C.K.K.

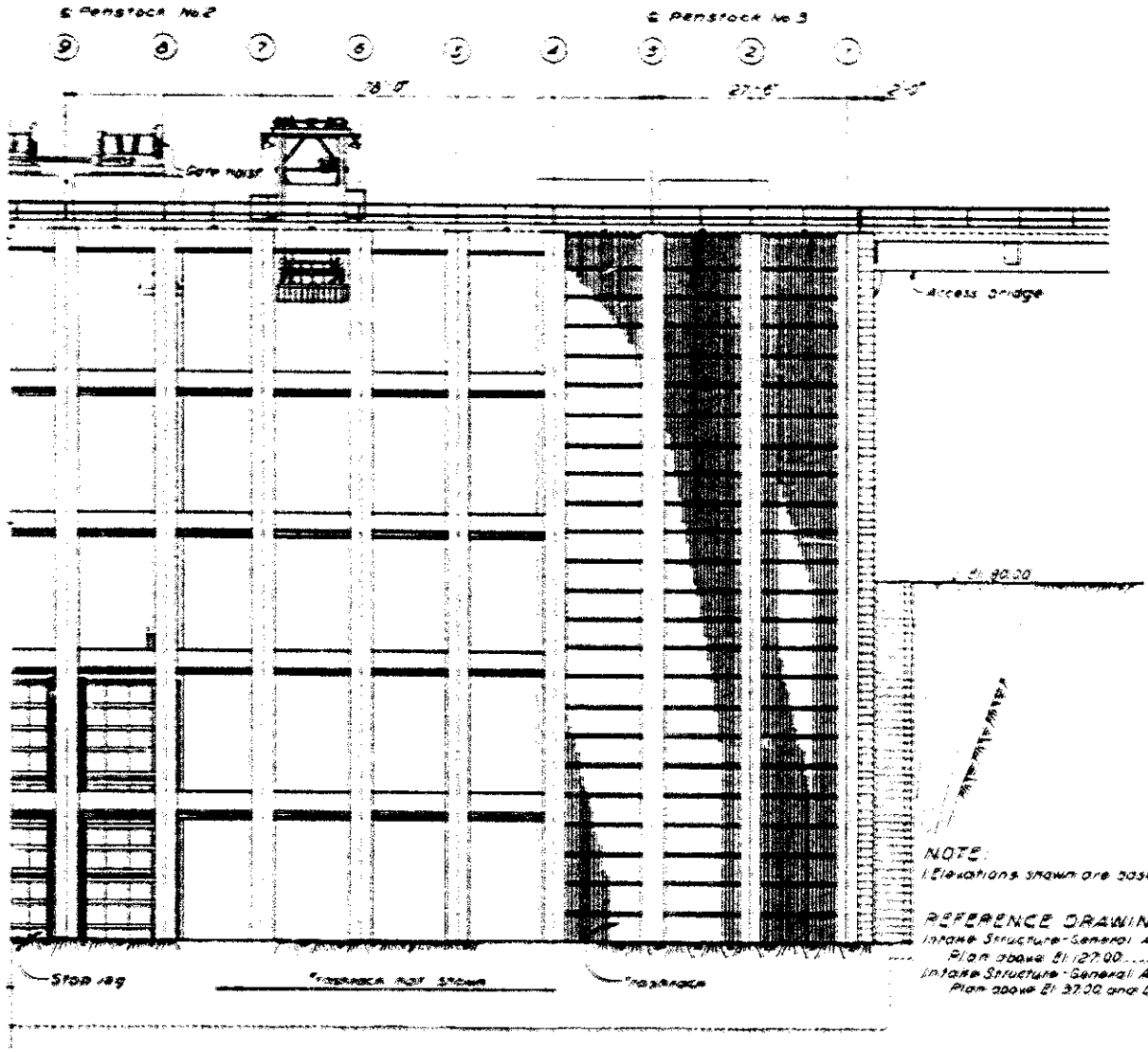
DATE: May, 1961 FIGURE G

PART III - CHAPTER IV

LIST OF EXHIBITS

Exhibit	Title
IV-1	Power Intake - Intake Structure - General Arrangement - Elevations and Sections
IV-2	Power Intake - Intake Structure - General Arrangement - Plan Above Elevation 127.00
IV-3	Power Intake - Intake Structure - General Arrangement - Plan Above Elevation 37.50 and 92.00


Base line
SECTION C-C
Base line
SECTION A-A
(FL-31-301)
SECTION B-B
(FL-31-301)



NOTE:
1. Elevations shown are based on P.W.D. datum.

REFERENCE DRAWINGS:
Intake Structure - General Arrangement
Plan above El. 127.00 ..... FL-31-301
Intake Structure - General Arrangement
Plan above El. 37.00 and El. 28.00 ..... FL-31-302

UPSTREAM ELEVATION

Scale 0 10 20 Feet
Except as noted

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

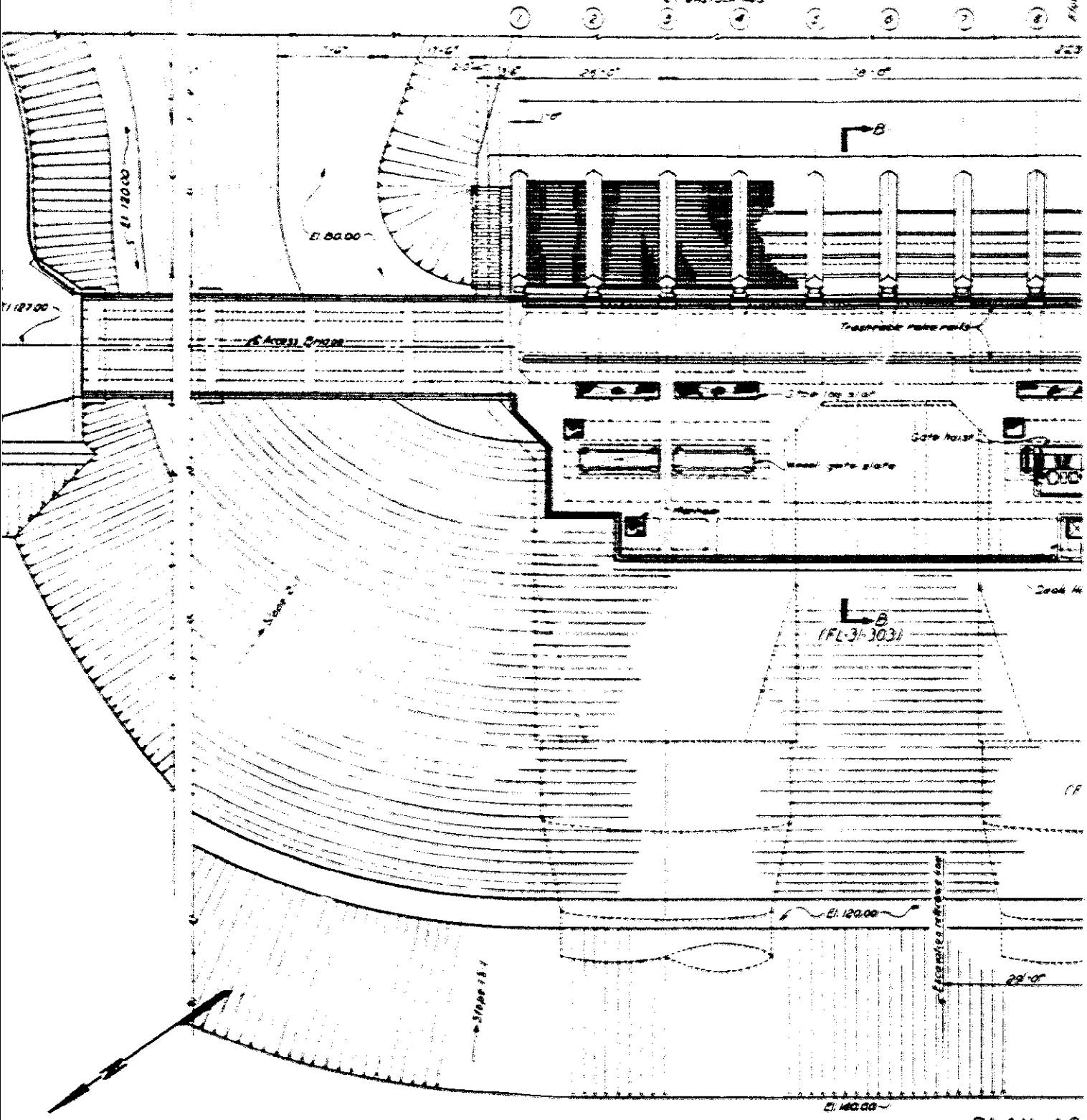
KARNAFULI PROJECT
POWER INTAKE
INTAKE STRUCTURE
GENERAL ARRANGEMENT
ELEVATION AND SECTIONS

INTERNATIONAL ENGINEERING CO. INC. - ENGINEERS

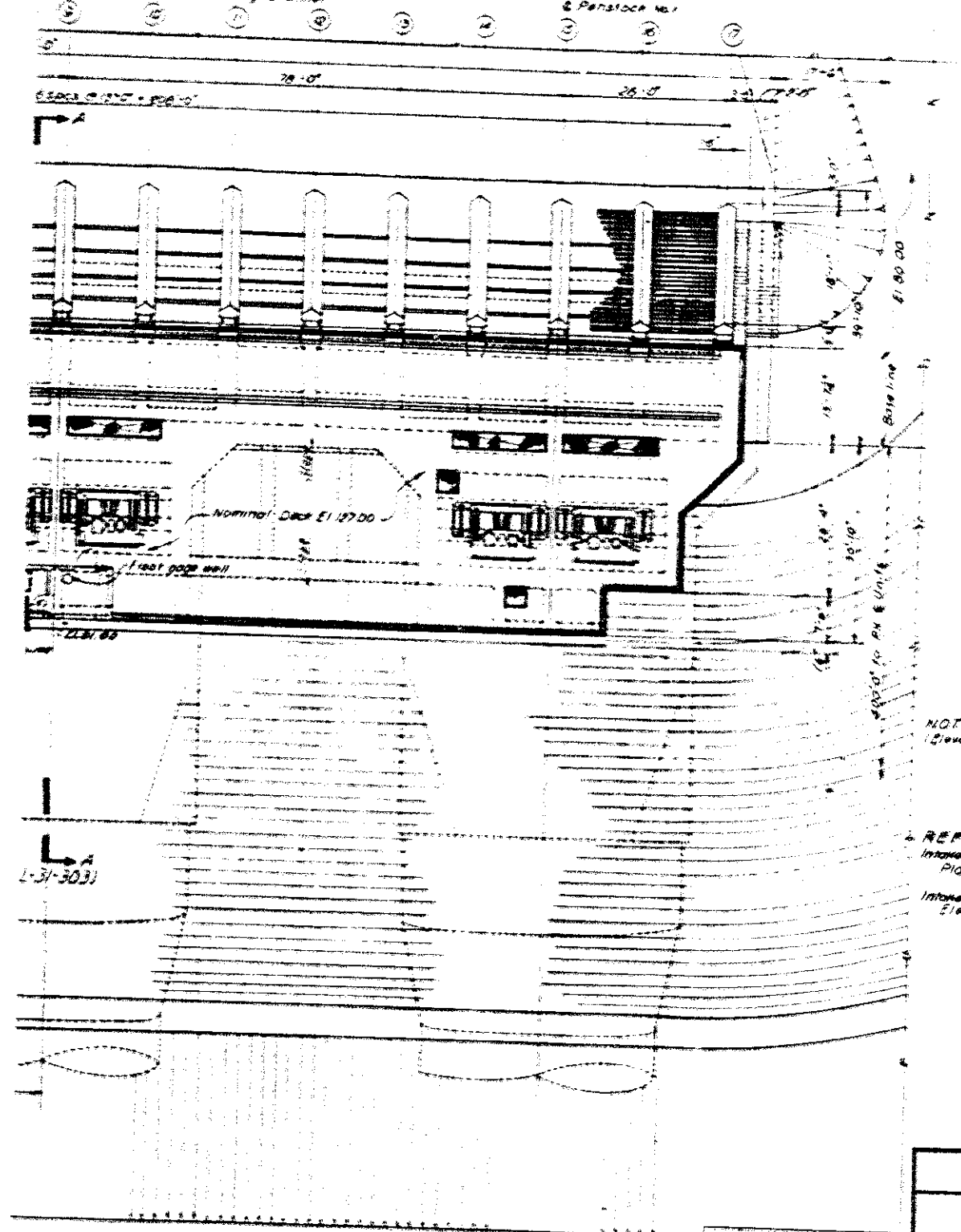
Designed L.E.Co.
Drawn L.M.Z.
Checked
Date
Submitted
Recommended
Approved

KA.-X.-1 | SAN FRANCISCO, CAL. Date Feb. 3, 1959 | FL-31-303

REVISIONS



PLAN AB



NOTE
Elevations shown are based on P.W.D. datum

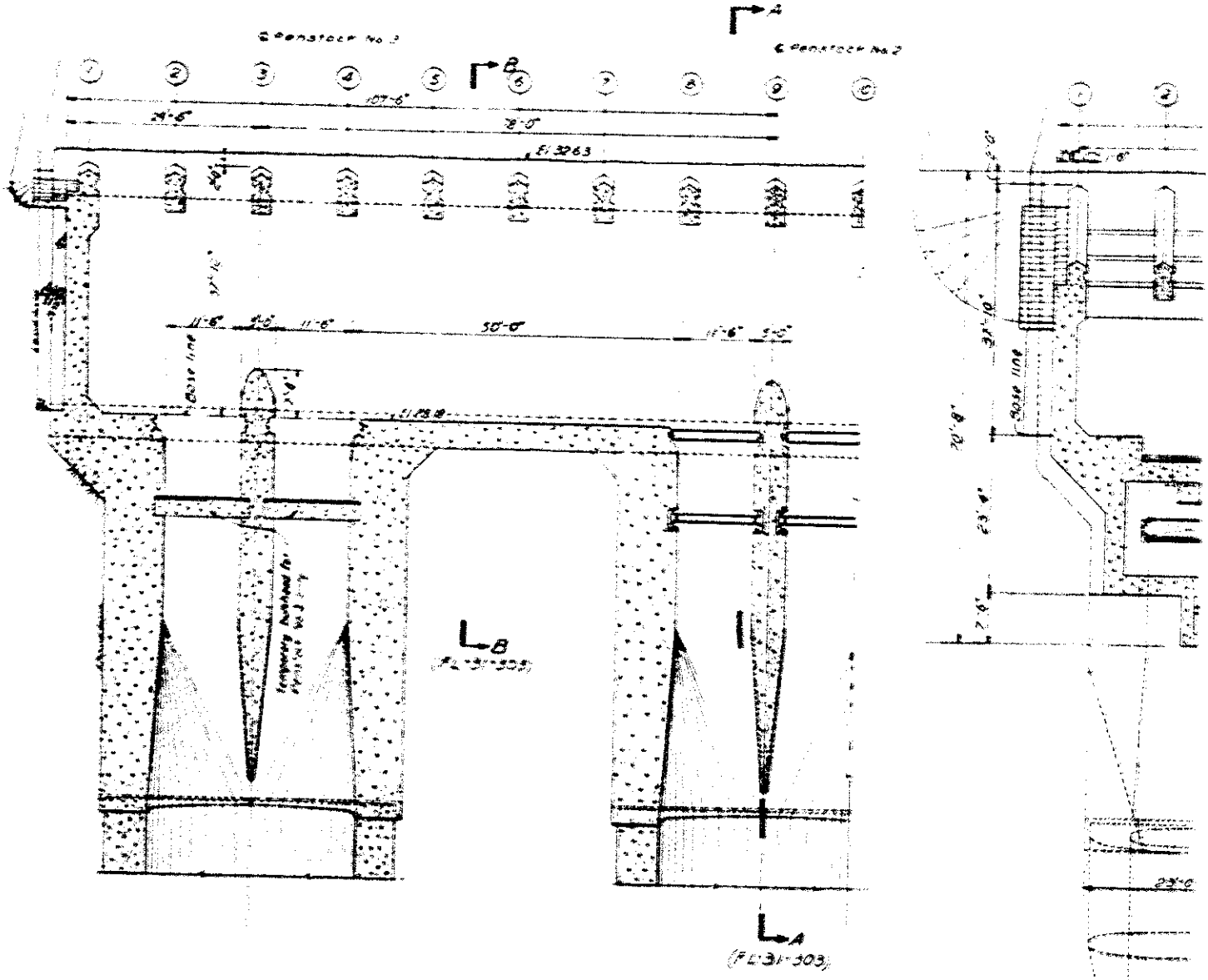
REFERENCE DRAWINGS:
Intake Structure-General Arrangement-
Plan above El. 92.50 and El. 92.00 ..... FL-31
Intake Structure-General Arrangement-
Elevation & Sections FL-31

Scale 0 10 20 Feet

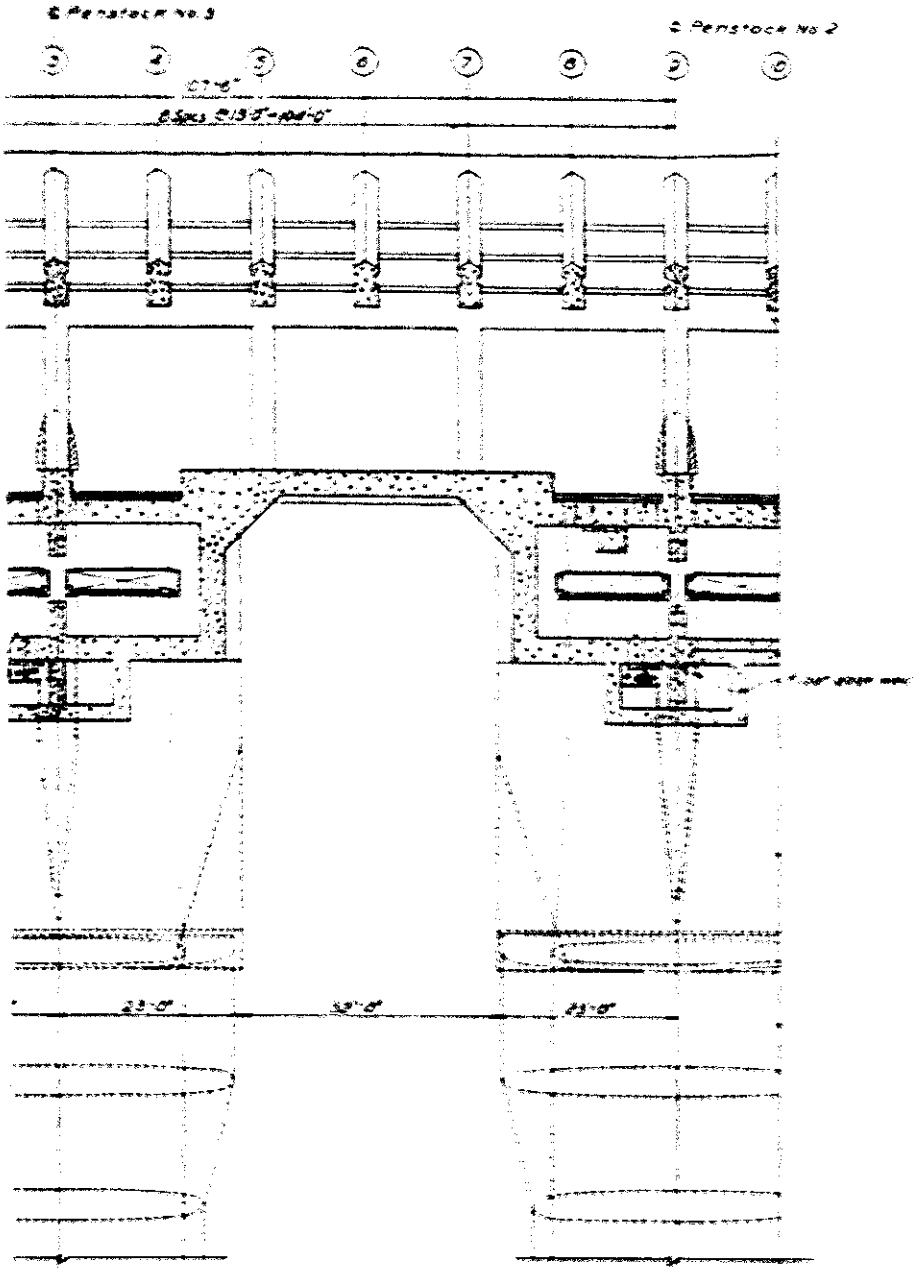
OVE EL 127.00

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
POWER INTAKE
INTAKE STRUCTURE
GENERAL ARRANGEMEN
PLAN ABOVE EL. 127.00
INTERNATIONAL ENGINEERING CO., INC. · ENGINEER

I.E.C. Co.		
K.A.-X.-1	San Francisco, Cal. Date Feb. 3 1959	FL-31-30



SECTIONAL PLAN
ABOVE EL 327.50
Symmetrical about ℄ Penstock No 2
(Except as noted)



SECTIONAL PLAN
ABOVE EL.92.00

NOTE:
1. Elevations shown are based on P.W.D. datum.

REFERENCE DRAWINGS:
Intake Structure - General Arrangement - Plan above El. 127.00 ..... FL-31-301
Intake Structure - General Arrangement - Elevation and Sections FL-31-303

Scale 0 10 20 Feet

GOVERNMENT OF PAKISTAN MINISTRY OF INDUSTRIES			
KARNAFULI PROJECT POWER INTAKE INTAKE STRUCTURE GENERAL ARRANGEMENT PLAN ABOVE EL.37.50 & EL.92.0			
INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS			
L.E.Co.			
KA.X.1	SAN FRANCISCO, CAL. Date Feb. 3, 1959	FL-31-302	

CHAPTER V

POWERHOUSE

5.01 GENERAL

The powerhouse is designed as a conventional indoor-type structure, with a mass concrete substructure and a steel-framed superstructure. This type of building was selected for the Karnafuli powerhouse primarily because of the prevailing climatic conditions. Torrential rains during the monsoon season would make maintenance impractical, if not impossible, unless the plant was enclosed. Economic studies of indoor versus outdoor structures were not in themselves sufficiently conclusive to dictate a choice from the cost standpoint.

Insulated aluminum sandwich-type walls with permanent exterior finish are used for enclosure of the superstructure above Elevation 59.5 feet. Roof design provides for an insulated metal deck, with built-up roofing. Selection of these types of wall and roof was based on their relative superiority over concrete with respect to insulation value, weight, and maintenance. Design was further influenced by the ease of erection and the pleasing appearance inherent in this type of construction.

Above the ground, the design provides for the building to be 75 feet wide, approximately 275 feet long, and 62 feet high. The overall height, as measured from the lowest point of excavation under the draft tubes to the top of the roof is about 164 feet. The principal powerhouse elevations are tabulated as follows:

Powerhouse roof	Elevation	110.50	feet	(Approx.)
Powerhouse crane rails	"	88.00	"	"
Transformer deck	"	49.00	"	"
Erection and maintenance area	"	49.00	"	"
Generator room, machine shop, control room, and office floors	"	36.00	"	"
Turbine room floor	"	22.50	"	"
Centerline of distributors	"	5.00	"	"
Inspection gallery floor	"	-19.00	"	"
Draft tubes-invert at outlet lip	"	-31.97	"	"
Draft tubes-invert at low point	"	-43.19	"	"

Principal tailwater elevations are as follows:

Flood	Elevation	55.00 feet
Normal maximum (28,000 cfs, 3 units)	"	21.10 "
Normal minimum (7,900 cfs, 1 unit)	"	14.50 "

Two complete power-generating units plus provisions for a third unit are to be installed. Concrete walls isolate the substructure of the third unit bay from the rest of the powerhouse to prevent flooding thru the open area of this unit. Each of the three bays measures about 78 feet long by 63 feet wide.

The powerhouse is designed to permit convenient and efficient operation of the plant. This was accomplished by locating major areas, such as the transformer deck, maintenance area, control room, office, and machine shop within easy access of each other. Piping for oil and water systems, electric wiring, and heating and ventilating ducts are designed to permit ready access for maintenance and operation. Descriptions of the powerhouse arrangement with details of the design of the structure are presented in Paragraphs 5.02 and 5.03 of this chapter.

5.02 ARRANGEMENT

The general arrangement of the powerhouse as described in this chapter is shown on Exhibits V-1 through V-6. For convenience of presentation, the arrangement is presented by floor elevations, starting at Elevation 49.00 feet and descending to the lower levels of the structure.

A. Elevation 49.00 Feet

The two principal powerhouse components at Elevation 49.00 feet are the erection area and the transformer deck.

The erection area, measuring about 38 feet by 75 feet, is in the south end of the building. This area has been designed to provide sufficient space for assembly of one complete unit at a time. After the installation of the turbines and generators has been completed, the area will be used for service and maintenance of transformers and major components of the turbines and generators. Access to the area from the outside is through the large rolling door and the entrance door, both in the south wall of the building.

On the outside of the building, the transformer deck extends along the length of the downstream side of the building on the same level as the erection area. Tracks are provided to facilitate transportation of the transformers from the deck to the erection area for servicing. The decision to locate the transformers on the downstream side of the building was made on the basis of (1) the desirability of location of transformers as close as possible to the generators, minimizing both the power losses and the cost of heavy current busses; and (2) the need for

ready accessibility to the service area. Other locations considered were unfeasible with respect to the above criteria, or because of interference with other features. For example, placing of the transformers on the upstream side of the powerhouse would necessitate either excessive excavation of material from the natural cut-off barrier between the powerhouse and the intake structure or lengthening of the unit penstocks.

A gentry crane traveling the length of the deck on the downstream edge, is designed for use in operation of the draft tube bulkhead gates. Discussion of the gantry crane, as well as other powerhouse equipment and systems may be found in PART V, Equipment and Systems.

B. Elevation 36.00 Feet

The generator room floor is 13 feet below the erection area, at Elevation 36.00 feet. Selection of this elevation was governed principally by the turbine manufacturer's selected unit setting requirements. Consequently, it is possible to use a shorter generator shaft, and a saving in the height of the powerhouse superstructure is effected. Location of the floor at any lower level would be impractical because of hydraulic problems, and location at a higher elevation would be less economical.

At the south end of the structure, on the same level and adjoining the generator room floor of Unit Bay No. 1, is the machine shop area, measuring approximately 63 feet by 19 feet. The tool room is located at the east end of the machine shop.

Under the transformer deck, at Elevation 36.00 feet, are the office, reception room, toilets, control room, battery room, emergency generator area, and fan room.

C. Elevation 22.50 Feet and Below

The turbine room floor is at Elevation 22.50 feet. A ramp along the west side of the room slopes to Elevation 21.00 feet and provides ready access to a storage room, the janitor's room, the cable spreading area and the oil purification and storage rooms.

In the southwest corner of the structure at this elevation, is a pump area, beneath which are sumps for drainage and unwatering. The sumps descend to Elevation -50.00 feet.

An inspection gallery, extending the length of the structure at Elevation -19.00 feet, provides access to the draft tubes and the turbine runners.

Individual flexible coupling chambers are provided along the upstream walls of the structure for each penstock. Access to each of the three chambers is principally from the turbine room (Elevation 22.50 feet), and at their lowest level, Elevation -19.00 feet, from the inspection gallery. Those parts requiring a large servicing area can be removed through the hatches at Elevation 49.00 feet.

Until Unit No. 3 is installed, the access door in the north wall, at Elevation 49.00 feet, will not be used. To provide access in the interim, the design calls for a door to be placed in the east wall of Unit Bay No. 2.

To provide access between levels in the powerhouse, two stairways have been included in the building design. Both stairways, one located in the southwest corner of Unit Bay No. 1 and the other in the northeast corner of Unit Bay No. 2, will permit easy access between floors at Elevation 49.00 feet to 22.50 feet.

The tailrace channel, approximately 220 feet wide, runs parallel to the transverse axis of the powerhouse to the river. The invert is level for 10 feet past the ends of the draft tubes to confine possible eroded materials from entering the tubes, and then slopes upward with the same vertical angle as the draft tube inverts. The sides of the channel, excavated vertically in shale, are benched at intervals to confine small amounts of fallen eroded materials. Where loose soil is encountered, appropriate slopes, with suitable transitions are provided to ensure stability.

5.03 STRUCTURAL DESIGN

Basic criteria and assumptions were established for application to the design of the Karnafuli powerhouse. The paragraphs and tabulations which follow give these criteria and assumptions as developed for the various features of design. Results of stability analyses of the structure are also presented.

Criteria for Design

A. Loads

1. Dead Loads - Dead loads shall include the weight of walls, partitions, framing, floors, roof, and all other permanent stationary construction entering into and becoming a part of the building.

2. Live Loads - The following tabulation presents loads assumed uniformly distributed per square foot of roof, floor, or wall area.

Location	Live Loads in Pounds per Sq. Ft.
Roof	40
Floors	
Transformer Deck	200
Erection Area	1000
Generator Room	1000
Office	100
Control Room	100
Emergency Generator Room	300
Machine Shop	150

Location	Live Loads in Pounds per Sq. Ft.
Floors	
Battery Room	200
Toilets	100
Turbine Room	400
Storage Room	400
Oil Storage Room	300
Oil Purification Room	200
Cable Spreading Area	200
Pump Room	300
Stairs and Landings	100

Erection area floor shall be checked for H-20 AASHO standard motor truck, transformer and rotor loads, and the transformer deck for weight of draft tube gate and transformer loads.

3. Wind Load - Wind pressure shall be taken at 20 pounds per square foot on the vertical projection of the gross exposed building areas. For open framework, one and one-half times the exposed area shall be used.

4. Earthquake Forces - Seismic forces on any equipment or structural parts shall be 10 percent of the sum of both the dead and fixed live loads, applied at the center of mass acting in any horizontal direction.

5. Crane Loads - Crane impact loads shall be 10 percent of wheel loads. To provide for the effect of moving crane trolleys, the lateral forces on crane runways shall be 10 percent of the trolley weight, with the full load applied to the runway at the top of each rail. Longitudinal forces on crane runways shall be 10 percent of the maximum wheel loads, applied at the top of each rail.

6. Hydrostatic Pressure and Uplift - The powerhouse shall be designed to sustain any loads arising from hydrostatic pressure and full uplift pressure. A reduction of one-third in uplift pressure may be permitted for maximum tailwater condition, and the uplift pressure due to maximum tailwater shall not exceed the weight of the powerhouse structure.

7. Combined Loading - The powerhouse shall be designed to sustain at normal stresses the maximum dead load, normal live loads, and crane operating loads to which it may be subjected.

Where stresses due to wind, earthquake, construction loadings, or maximum high tailwater condition are combined with those due to static loads, the permissible working stresses may be increased 33-1/3 percent. Except for the assumption of simultaneous occurrence of wind and maximum high tailwater, stresses due to abnormal loading conditions shall not be combined with each other or crane operating loads.

Stresses resulting from break-down load on the crane may exceed the normal allowable stresses by 50 percent.

B. Allowable Stresses

1. Concrete - Three thousand pound per square inch concrete at 28 days shall be used except in the substructure components over 3.5 foot thick, where two thousand-five hundred pound per square inch concrete may be used, unless otherwise specified.

2. Reinforcement - Reinforcement shall be intermediate grade billet steel conforming to the Standard Specification ASTM A-305. Working stresses shall not exceed 20,000 pounds per square inch.

3. Soil Pressure - Horizontal pressure of the excavated vertical shale faces adjacent to substructure walls is assumed to be zero.

Vertical bearing pressure on the shale shall not exceed 20,000 pounds per square foot on the horizontal projection of the area.

C. Standards - All design work shall conform in general to the following standards, unless otherwise specified:

Concrete - "Building Code Requirements for Reinforced Concrete (ACI 318-56)," by American Concrete Institute.

Structural Steel - "Code of Standard Practice for Steel Buildings," 1952, of American Institute of Steel Construction.

In addition to the above codes, the 1955 edition of the "Uniform Building Code", adopted by the Pacific Coast Building Officials Conference is to be generally utilized. This particular code was selected because of similarities in climatic and seismic conditions in East Pakistan and the Pacific Coast.

D. Protection of Reinforcement - Protective covering of reinforcement (including stirrups and ties) from outside of bar to surface of concrete shall be not less than that shown in the following table.

Type of Construction	Dry Interior Faces	Exterior Faces Above High Water and Ground	Faces Permanently Exposed to Water or Ground
Floor and Roof Slabs and Light Walls (12" and less)	1"	1-1/2"	2"
Beams and Girders	1-1/2"	2"	2-1/2"
Columns and Heavy Walls	1-1/2"	2"	2-1/2"
Tunnels and Conduit Barrels	1-1/2"	2"	2-1/2"
Heavy Section	2"	2-1/2"	3-1/2"

E. Stability

The powerhouse, as shown on the general arrangement drawings, (Exhibits V-1 through V-6) was investigated for stability with respect to the uplift, sliding and overturning to which the structure will be subjected. Safe and satisfactory values were obtained, based on the assumptions and criteria set forth in the following tabulation.

1. Flotation - Assuming full uplift due to hydrostatic pressure, the flotation ratio (dead weight divided by total uplift) shall not be less than 1.1 for first stage concrete. After completion of the powerhouse, the ratio shall not be less than 1.25.

2. Resistance to Sliding - The coefficient of sliding friction (the ratio of the summation of horizontal forces to the summation of vertical forces) of concrete on shale shall be equal to or less than 0.333.

3. Safety Against Overturning - The factor of safety against overturning (the ratio of total resisting moments to total overturning moments) shall not be less than 1.50.

The following table presents the results of stability analyses performed on Unit No. 2 of the structure, the critical unit for foundation pressure under the conditions given in the table. In each case the results indicated satisfy the criteria for stability of this unit and Unit No. 1.

No.	Loading Condition - Unit No. 2 Description	Flotation Ratio	Coef. of Sliding Friction	Overturning Factor of Safety
I	First stage concrete to El. 49.0, upstream water surface at El. 48.5 to tailwater at El. -33.19 For loading diagram, see FIG. A	3.00	0.217	2.68
II	Final stage concrete in place. Water surface at El. 55.0. Generator and transformers not in place. Draft tube and scroll case full of water. For loading diagram, see FIG. B	1.54	0.08	1.50
III	Final stage concrete. All equipment in place. Draft tube empty; scroll case full of water. Upstream water surface at El. 48.5 to tailwater at El. 7.5. For loading diagram, see FIG. C	2.16	0.333	1.70

The results of stability analyses for Unit No. 3, the critical unit for lateral pressure parallel to the longitudinal axis of the powerhouse, are presented with the critical loading condition as follows:

No.	Loading Condition - Unit No. 3 Description	Flotation Ratio	Coef. of Sliding Friction	Overturning Factor of Safety
IV	Final stage concrete during erection of Unit No. 3. Draft tube gates in place. Upstream water surface at El. 48.5 to tailwater at El. 21.0. Loading: Weight of concrete and hydrostatic pressure For loading diagram, see FIG. D	1.8	0.333	1.60

As was the case in Unit No. 2 analyses, the results for Unit No. 3 satisfy the stability criteria.

F. Final Design

Based on the results of the studies and investigations conducted, general arrangement drawings of the powerhouse were prepared and are as presented in the accompanying exhibits.

So that transmission of stresses created as the result of possible unequal foundation settlement or deformation are minimized, the unit bays of the structure are isolated from each other. In addition, flexible couplings are used to connect the penstocks to the scroll cases, providing flexibility at those points. The back walls of the coupling chambers use metal water stop and expansion joints to provide flexibility between the powerhouse and the penstocks.

Draft tube dimensions are in accordance with recommendations of the turbine manufacturer, to provide areas and shapes with satisfactory hydraulic characteristics. Uplift pressures caused by water seepage under the draft tubes are relieved by a system of drain holes in the draft tube floors extending into the foundation. These drains are connected to a separate drainage system to relieve uplift pressures caused by artesian head in the sand stratum which lies about forty feet beneath the draft tube floors. This system consists of relief wells penetrating into the sand stratum and discharging whatever water is collected into the draft tubes or into the inspection gallery gutter. Distortion of the foundation, if any, and the resultant effects, are minimized by the use of this relief well system.

LOADING CONDITION I

First stage concrete to El. 49.0. Upstream face water surface at El. 48.5 to tailwater at El. -33.19. Draft tube empty. Loading: Weight of concrete and hydrostatic pressure.

Flotation ratio 3.0
Coef. of sliding friction 0.217
Overturning factor of safety . . 2.68

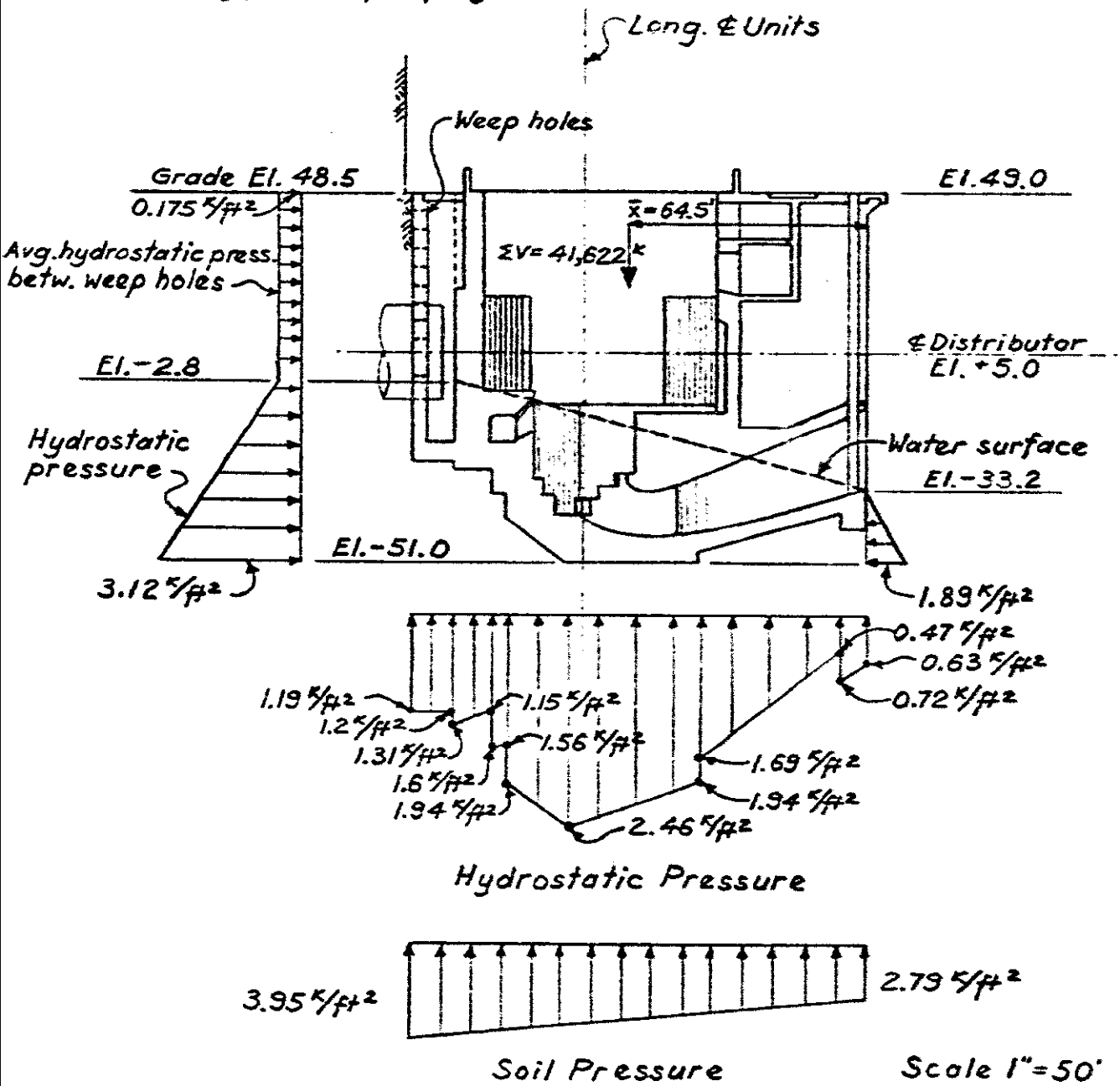


FIG. A

TRANSVERSE SECTION THRU. ℄ UNIT No. 2.

KARNAFULI PROJECT		
POWERHOUSE DESIGN REPORT		
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. V.G. CK. JEL	RECOMMENDED	APPROVED

LOADING CONDITION II

Final stage concrete in place. Water surface at El. 55.0. Generator and transformers not in place. Draft tube and scroll case full of water. Loading: Weight of concrete & turbine, and hydrostatic pressure.

Flotation ratio 1.54
Coef. of sliding friction . . . 0.08
Overturning factor of safety 1.50

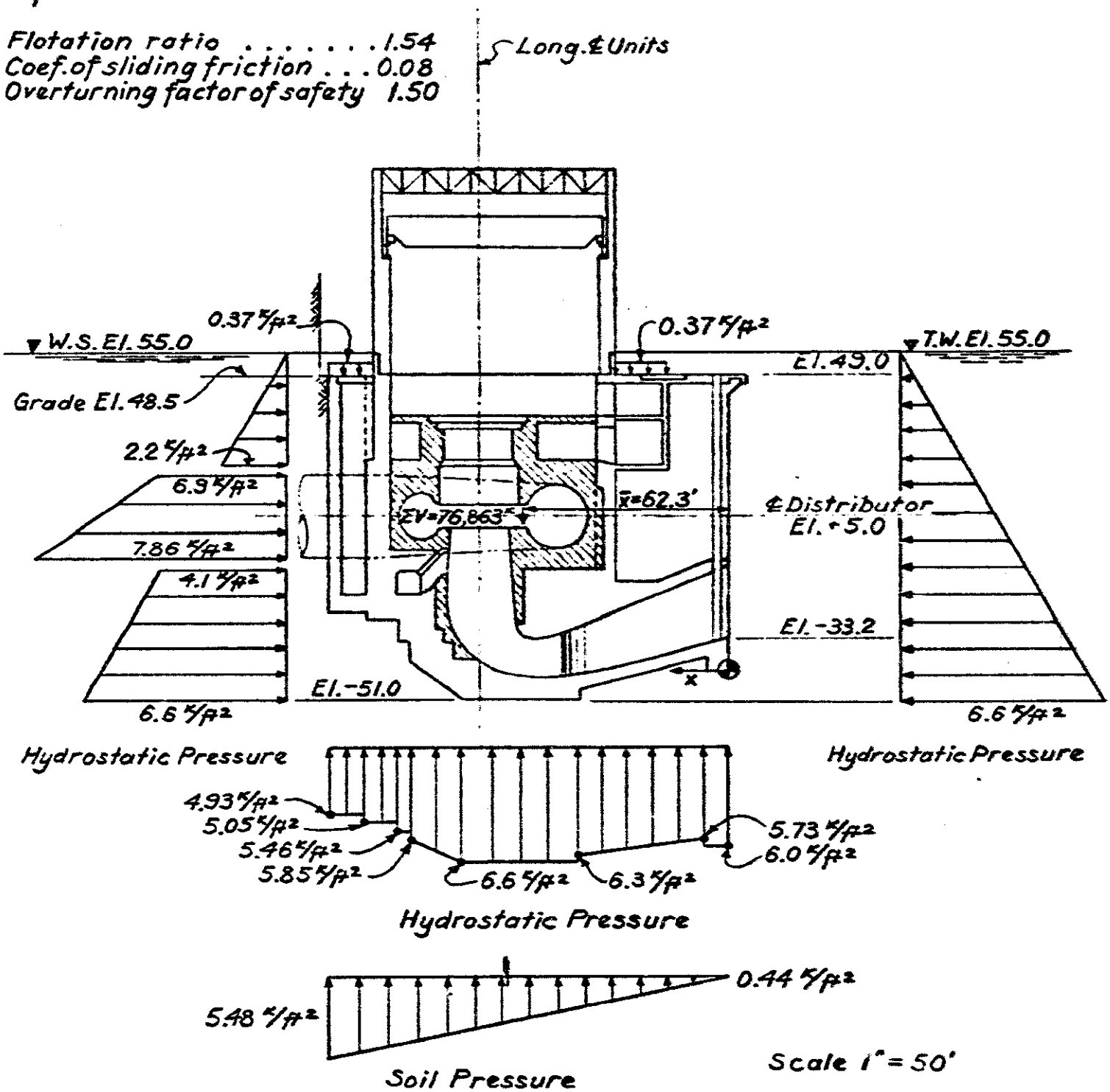


FIG. B
TRANSVERSE SECTION THRU. ℄ UNIT No. 2.

First stage conc.
Final stage conc.

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March 28, 1958		

final stage concrete and all equipment in place (to obtain max. external reaction). Upstream face water surface at El. 48.5 to tailwater at El. +7.5. Draft tube empty; scroll case full of water.

Flotation ratio 2.16
Coef. of sliding friction . . . 0.333
Overturning factor of safety . 1.70

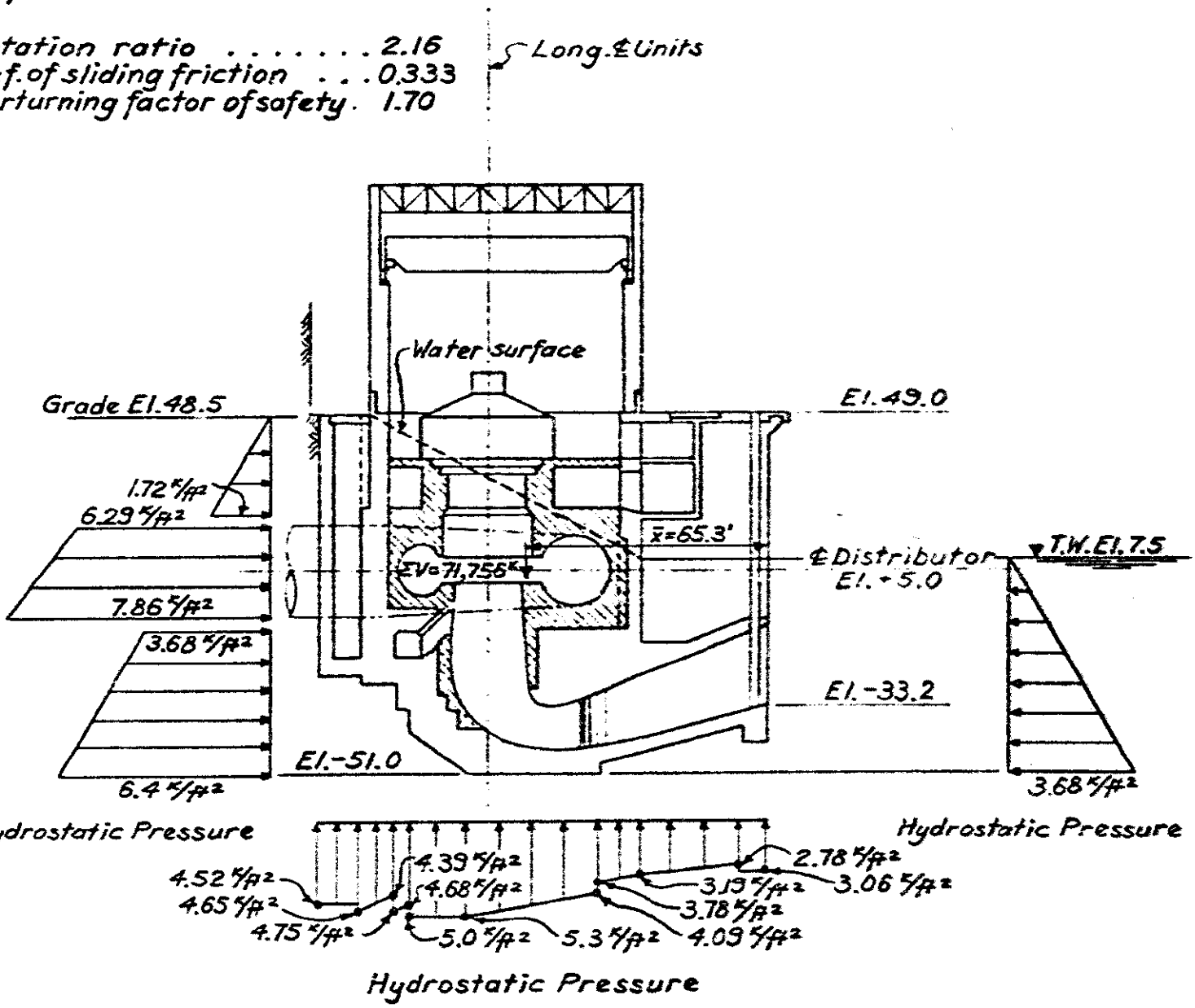

FIG. C

TRANSVERSE SECTION THRU. ℄ UNIT No. 2.

First stage conc.
Final stage conc.

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INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR V.G. CK JEL	RECOMMENDED	APPROVED G.F. Sudman
DATE March 28, 1958.		

Final stage concrete pouring during erection of Unit No. 3: Draft tube gates in place. Upstream face water surface at El. 48.5 with lowest weep holes at El. -5.0, to tailwater surface at El. 21.0. Weep holes to be successively sealed as concrete pouring progresses. Loading: Weight of concrete and hydrostatic pressure.

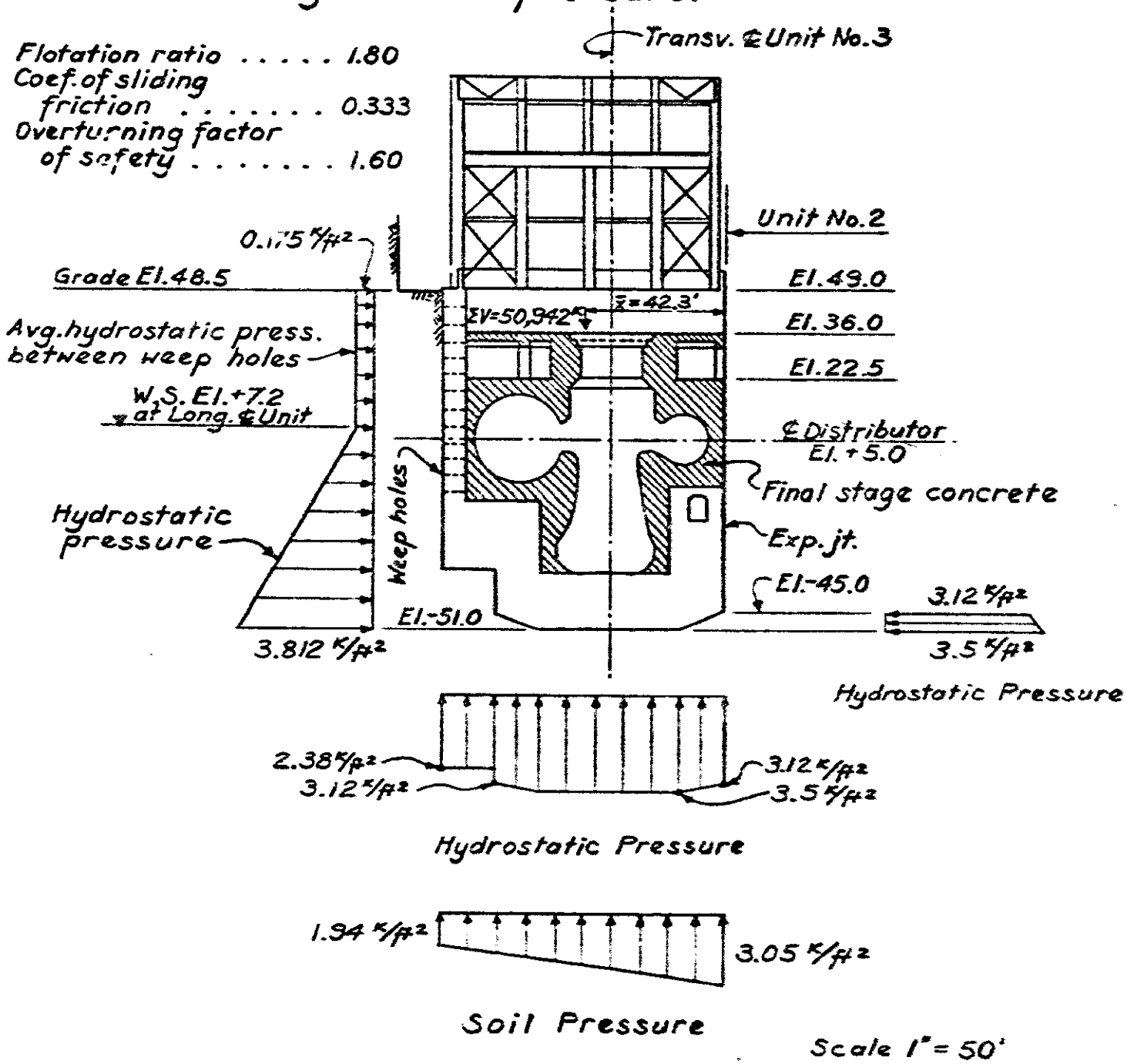

FIG. D

LONGITUDINAL SECTION THRU. ℄ UNIT No. 3.

Note: Pressure diagram shows a condition before final stage conc. is poured.

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G. Structural Components

1. Structural Steel Superstructure - The powerhouse superstructure is steel-framed above Elevation 60.5 feet. In addition to dead, live, and earthquake loads, the superstructure carries the bridge crane load including impact. The superstructure of each unit was designed to act independent of the other two units.

The roof is flat except for a slight pitch for drainage. It is supported by girders which span the powerhouse in the direction of flow. No interior supports are used. Purlins span laterally and brace the girders at the quarter points. Structural roof panels span laterally between the main girders. The panels selected have a loading capacity of 85 pounds per square foot which is ample for the insulation and roofing deadweight plus the live load. These panels also have the ability to transfer wind and seismic loads laterally at the roof level to the supporting columns. Bents formed by the columns and roof girders can safely carry wind and seismic loads in the other direction. These bents also contribute to the support of the bridge crane horizontal loads. These bents are braced laterally by the roof panels described above through structural rigidity and welded connections. Wall panels are supported by girts spanning horizontally between the columns. Wall panels can withstand the design wind loading.

The top of the crane rails is at Elevation 88.0 feet. The rails rest on the crane-rail girders, one each at the upstream and downstream sides of the building. Each crane-rail girder is a built-up member designed to support the crane loads, including impact, applied at any location, and horizontal traction and braking loads applied laterally or longitudinally at the top of the crane rails. Both the impact and longitudinal traction loads were assumed to be 10 per cent of the wheel loads. The lateral loads on each rail were assumed to be 10 per cent of the combined weight of the crane trolleys and live load. The crane-rail girders will also withstand the hoist-stall load of 275 per cent of the rated load with the crane positioned over the generator, in place. The allowable stresses were increased by 50 per cent for this case. See Figure F.

The working loads and breakdown loads which were used for the analysis of the crane-rail girder are tabulated in Table A below:

TABLE A

	Normal Operation Including Impact		Hoist-stall Condition	
	Vertical	Lateral	Vertical	Lateral
* M or V				
a.	764 ft. kips	70.3 ft. kips	1060 ft. kips	none
b.	1027 ft. kips	90.0 ft. kips	1420 ft. kips	none
c.	1366 ft. kips	127.5 ft. kips	1970 ft. kips	none
d.	644 kips	59.8 kips	889 kips	none
e.	412 kips	38.4 kips	570 kips	none
f.	399 kips	35.2 kips	367 kips	none

* M or V due to the bridge crane represent:

a. Maximum positive moment.

b. Maximum negative moment over end support.

c. Maximum negative moment over intermediate support.

d. Maximum reaction at intermediate support.

e. Maximum shear at intermediate support.

f. Maximum shear at end support.

The steel columns extend from base plates at Elevation 60.5 feet to the roof at Elevation 105 feet. All of the columns, except the intermediate columns at the north and south end of the building, are fixed to the roof girders to form bents. These bents were analyzed under normal dead loads and probable combinations of live loads including the lateral crane traction loads as shown on Figure G. Values of loads for Figure G are given in Table B below:

TABLE B

* Loading Conditions	End Bent					Intermediate Bent				
	a k/ft	b k/ft	c kips	d kips	e kips	a k/ft	b k/ft	c kips	d kips	e kips
A	0.65	0	33	275	95	1.25	0	41	588	202
B	0	0	33	275	95	0	0	41	588	202
C	0.65	0	33	95	275	1.25	0	-41	202	588
D	0	0	-33	95	275	0	0	-41	202	588
E	0.65	0.19	0	(not critical)		1.25	0.38	0	(not critical)	
F	(not critical)					(not critical)				

* Load conditions which were investigated are as follows:

A Horizontal thrust, eccentric crane load and roof load.

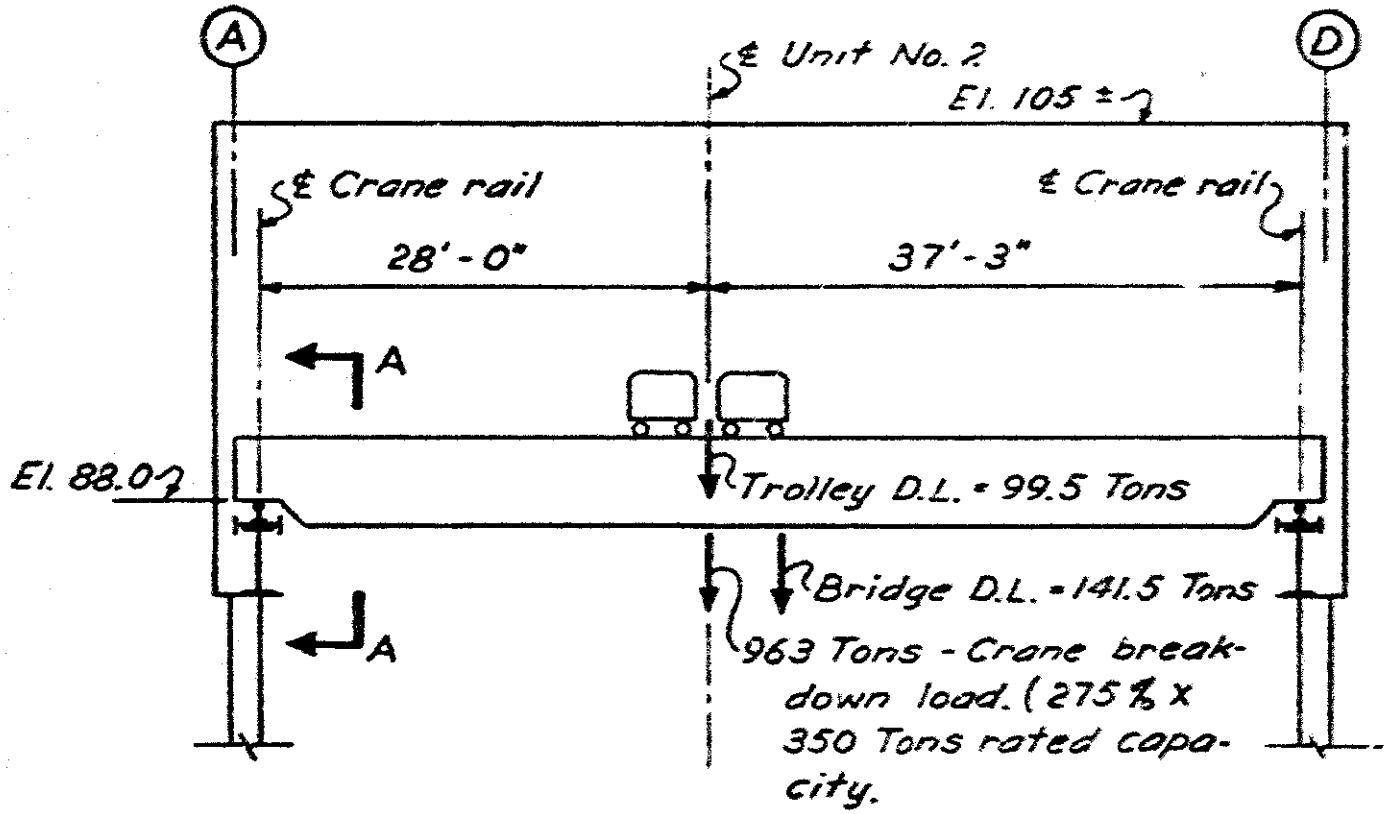
B. Horizontal thrust and eccentric crane load.

C. Horizontal thrust, direction reversed, eccentric crane load and roof load.

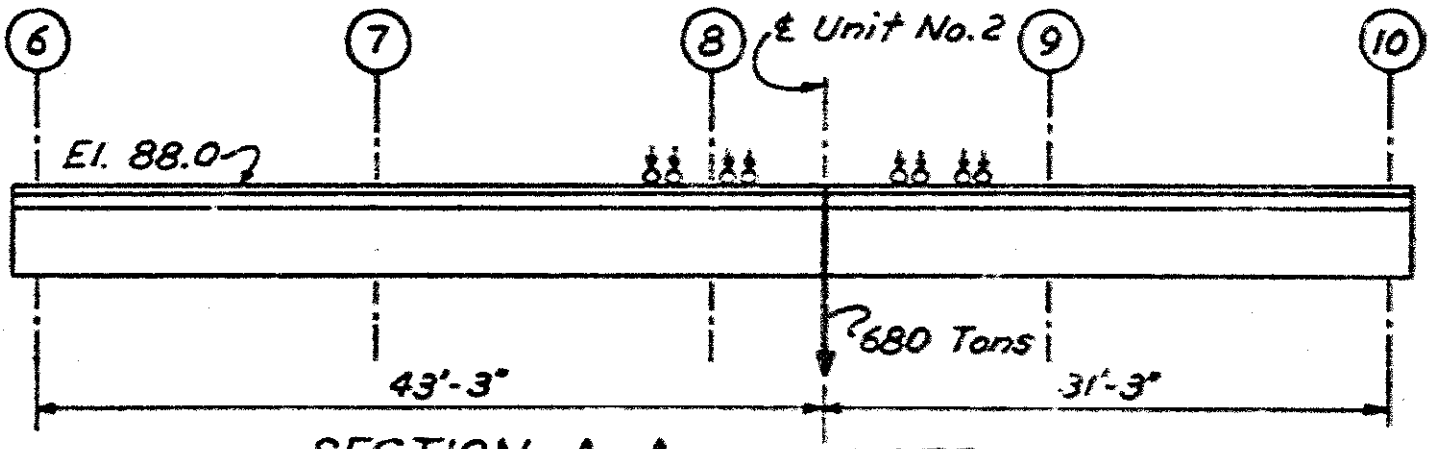
D. Same as C without roof load.

E. Wind load plus roof load.

F. Earthquake load.



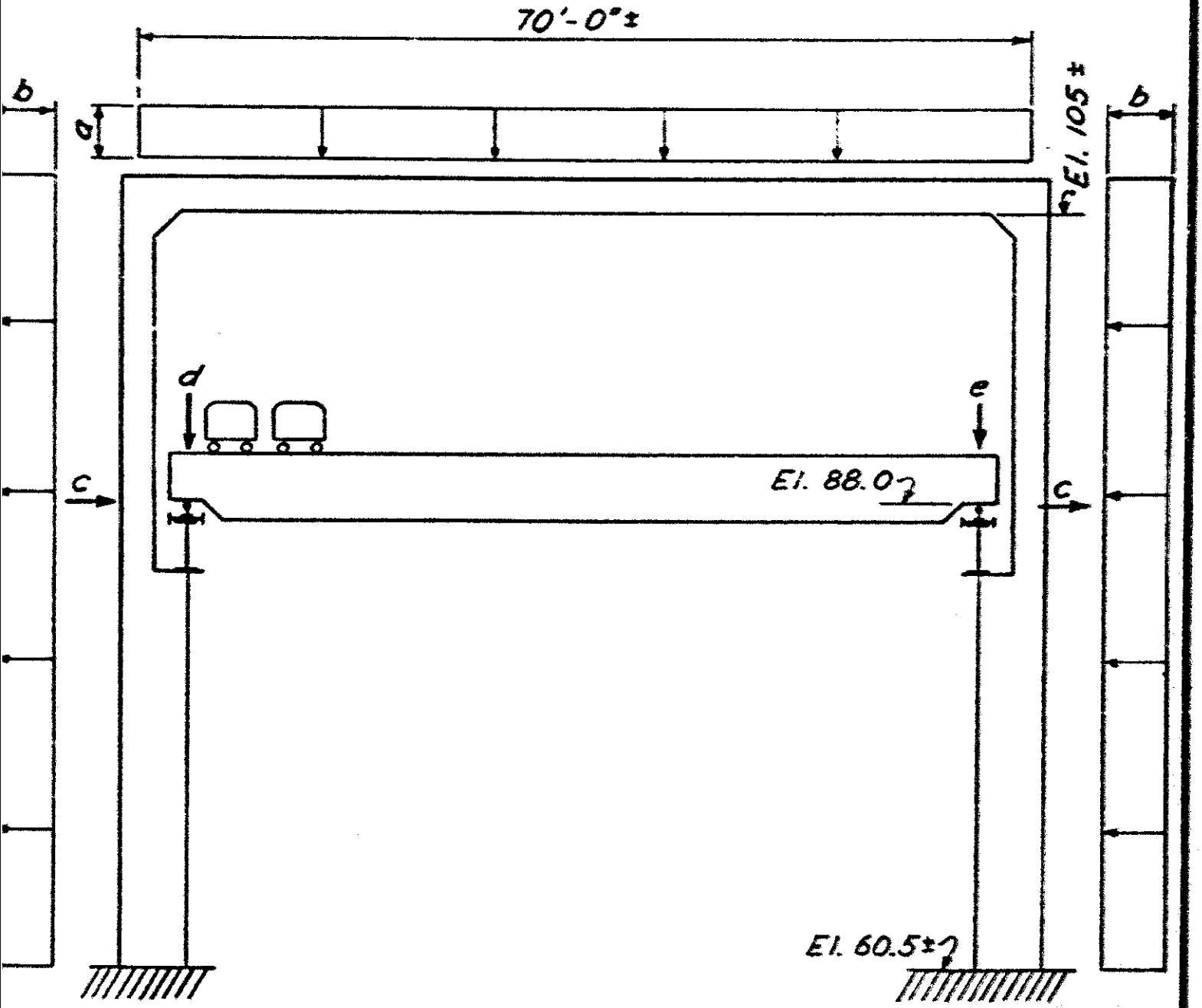
BRIDGE CRANE



SECTION A-A

NOTE:
For breakdown load condition allowable stresses were increased 50%.

KARNAFULI PROJECT		
BRIDGE CRANE BREAKDOWN LOAD		
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. HCJ CK. WRS	RECOMMENDED C.E.J.	APPROVED Ad.K
DATE: May, 1961		FIGURE F



LOADING DIAGRAM

NOTE:
For values of live load see Table B

KARNAFULI PROJECT		
LIVE LOAD LOADING DIAGRAM		
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. HGL CK. VRS	RECOMMENDED C.E.H.	APPROVED C.B.K.
DATE: May, 1961		FIGURE G

Horizontal thrusts on the columns were determined by calculating the reactions from the wheel loads on the crane-rail girder. In calculating the reactions the elastic properties of the girder and columns were considered. The wheel loads were placed at two positions, one to determine the maximum reaction at an intermediate column and the other for an end column.

Bents resisting horizontal thrust are formed by the crane-rail girder and the columns in a building unit. These bents were analyzed assuming the column bases fixed and, again, assuming they were hinged. The columns used are safe for either case. The position of the bridge crane on the bent had a rather small effect and was analyzed at one eccentric position only.

The moments resulting from horizontal thrust (assuming columns fixed at the base) were combined with the wheel load moments to check stresses in the crane-rail girder. Column anchorages were designed to transmit the column moments to the concrete below.

2. Concrete Superstructure - The concrete superstructure consists of the reinforced concrete columns, walls and beams above Elevation 49.0 feet and the columns on Column Line D which extend down to Elevation 22.5 feet. Heavy concrete columns support the steel superstructure columns. A thin, wide beam connects the concrete columns at the top. Thin walls connect the columns below the window level at Elevation 56.0 feet. At the south end of the building the main entrance is framed by concrete columns and beam. A concrete canopy cantilevers from the building to provide shelter for the entrance way. Thin walls connect the columns at the ends of the building.

The main building columns were designed to carry the weight, moments, and horizontal thrust which are transferred through the base plates at Elevation 60.5 feet. The restraining effect of the connecting beams and thin walls was neglected because of their flexibility.

Columns 2 through 14 on Column Line D extend to Elevation 22.5 feet. The beams and floor slabs at Elevation 36.0 feet and Elevation 49.0 feet brace the columns laterally. The beam moments and reactions, which are transferred to the columns at the floor levels, were combined with the superstructure loads for column design. The beams and columns were analyzed as a rigid frame. The beam at Elevation 36.0 feet was also designed to withstand torsion transferred from the generator room floor.

The connecting beams at Elevation 60.5 feet were designed to carry their own dead weight plus a small live load. The connecting walls were designed to withstand water pressure on the outside of the building to Elevation 55.0 feet.

The canopy over the south entrance to the building was designed to carry a nominal live roof load in addition to dead weight. It consists of a concrete slab supported by two cantilever beams.

A reinforced concrete lintel over the building entrance supports the canopy slab and the sliding door. Columns on either side of the door support the lintel, canopy and structural members above.

At the north end of the building, provisions have been made to remove a section of the wall to facilitate the future installation of Unit No. 3. The removable wall section is fixed only at the base and will resist water pressure by cantilever action. After Unit No. 3 has been installed, the wall will be replaced.

3. Walls - At Elevation 49.0 feet the principal loads on the structure come from the main columns. These loads are supported and safely dispersed by the walls below. In addition to the column loads, the walls were designed for two cases as shown on Figure J. Case I is for first stage construction with the weep-holes open and Case II is for the completed powerhouse with the weep-holes closed. A nominal hydrostatic load has been used with Case I because of the possibility of incomplete drainage. Since the walls were poured against vertically excavated faces, soil pressure was assumed to be zero.

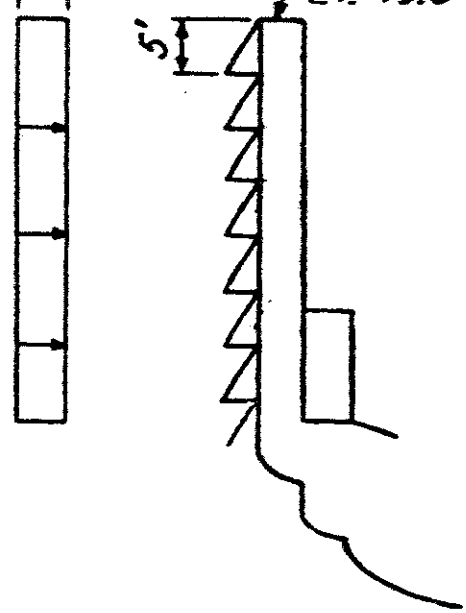
The coupling chamber side walls were designed to act as counterforts for the powerhouse upstream wall and also to resist hydrostatic pressure on one side as shown on Figure K. The coupling chamber removable cover slabs were designed to support an H-20 truck loading.

The downstream wall above the draft tubes and below the gallery was designed to withstand a hydrostatic load from Elevation 22.5 feet. This wall will not be loaded except during the future installation of Unit No. 3. The downstream gallery walls were designed for a hydrostatic load from Elevation 55.0 feet. The downstream walls are supported by the piers. Unit No. 3 is flooded up to tailwater level and the wall between the gallery and the generator room was also designed to carry a hydrostatic load from Elevation 55.0 feet.

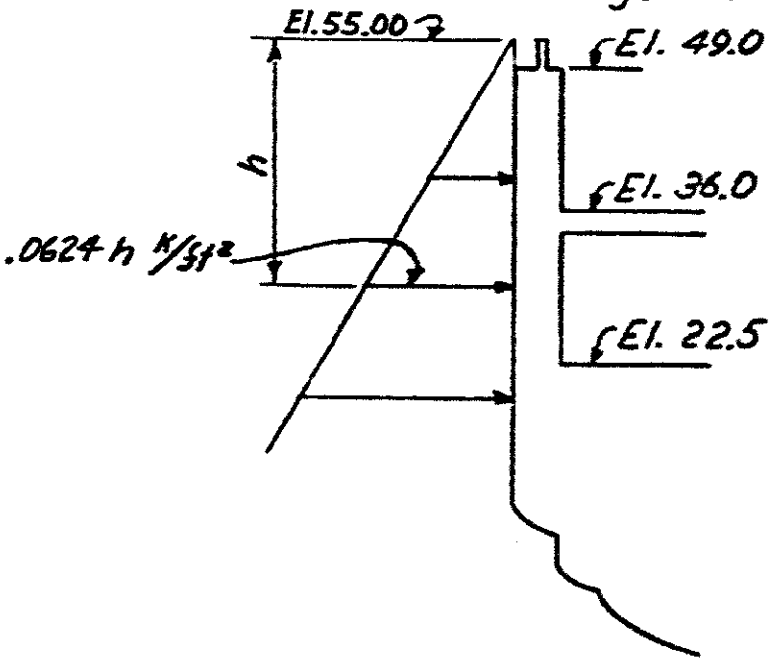
4. Building Foundation - South End - The superstructure at the south end of the building is supported on foundation beams resting directly on the shale. At the downstream corner the foundation shale sloughed during excavation and a portion of the remaining shale was not suitable as a foundation. A cantilever from the sump wall was designed to support the column and safely resist the bending moments. The sump wall was reinforced to carry the additional load.

5. Sump - The sump is divided into two parts, one for drainage and one for unwatering the draft tubes. The walls were designed to resist hydrostatic pressure from Elevation 55 feet on the outside with either or both parts empty. The bottom slab of the sump was designed to resist an uplift pressure of 9.0 kips per square foot. The interior wall was designed to resist water pressure on either side from Elevation 22.50 feet.

6. Piers - The piers wupport the superstructure, gallery, and transformer deck. They will also resist hydrostatic pressure transmitted from the downstream walls, horizontal thrust from the superstructure columns, and local bending moments caused by loads from the transformer deck.



CASE I - First stage, weep holes working.
Nominal hydrostatic load.



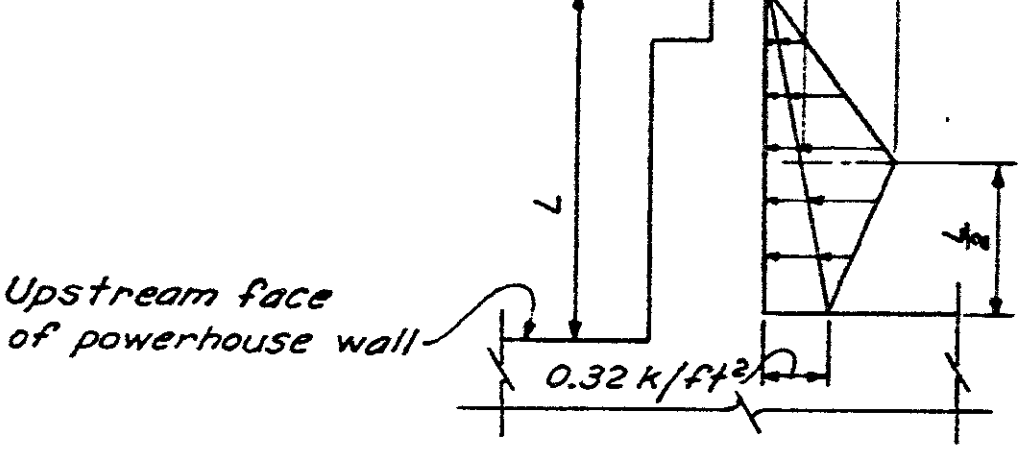
CASE II - Second stage,
weep holes inoperative.
Hydrostatic load.

NOTES:

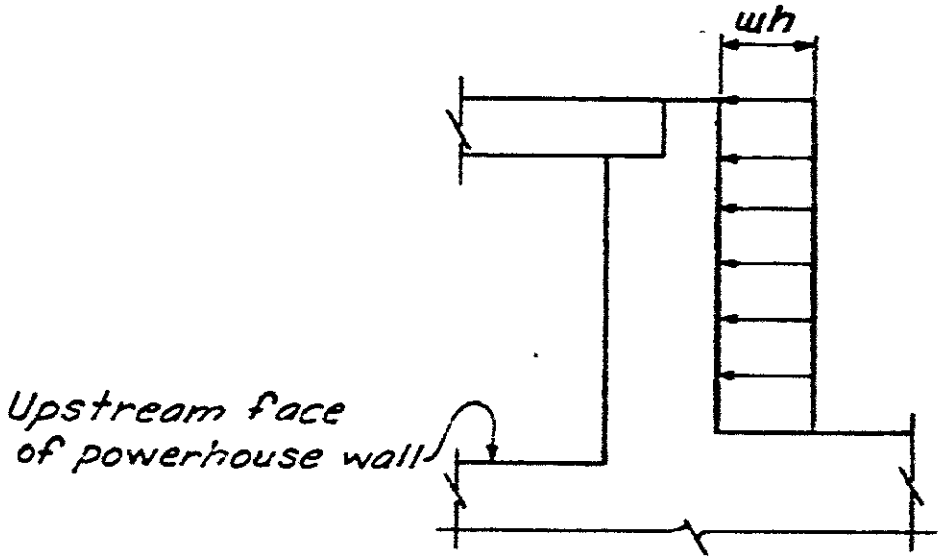
No soil pressure assumed to act.

Upstream wall shown. Loads on sidewalls are identical.

KARNAFULI PROJECT		
POWERHOUSE UPSTREAM & SIDE WALLS		
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. HH / CK. WPS	RECOMMENDED C.E.H.	APPROVED CBK
DATE: May, 1961		FIGURE J



CASE I - As a cantilever



CASE II - Supported by upstream coupling chamber wall

NOTE:

w = 62.4 lbs/cubic feet

Case I - h = El. 49.0 - Elev. of point in question

Case II - h = El. 55.0 - Elev. of point in question

KARNAFULI PROJECT		
COUPLING CHAMBER SIDE WALL - PLAN VIEW		
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. HLH CK. WRS	RECOMMENDED	APPROVED
DATE: May, 1961		FIGURE K

7. Draft Tube and Base Slab - The foundation of the powerhouse is mass concrete except for the base slabs under the draft tubes. The slabs were designed to resist the maximum combination of soil pressure and uplift.

The piers form the sides of the draft tubes. Provision has been made to stoplog the draft tubes at the downstream end for unwatering, inspection and repair. The draft tube roof spans between the piers and was designed for a hydrostatic load from Elevation 22.5 feet. The roof of the draft tube supports the scroll case between piers. It was designed to carry the dead loads plus weight of water in the scroll case. The upper half of the scroll case is lined with 3/4-inch cork-tar mastic open to drains to prevent pressure in case of scroll case leakage.

8. Erection Deck - The erection deck was designed to support a transformer plus a uniform load of 1 kip per square foot.

9. Transformer Deck - The transformer deck is supported on the piers and by beams at the ends of the powerhouse. It was designed to carry the transformers, the stoplog crane, an H-20 truck loading, and above the gallery, a hydrostatic load from Elevation 55.0 feet.

At the sides of the tailrace the shale was considered unsafe to carry large loads so beam and slab systems were designed to span the critical area near the brink. At the north end three beams cantilever out from the powerhouse piers. These beams support deck loads and also one end of each of three smaller simple beams which extend the deck far enough to allow the gantry crane to clear the last transformer. At the south end, three large simple beams support the entire deck area. One end rests on the powerhouse pier while the other rests on concrete piles drilled into the shale about 20 feet back from the brink. These piles will safeguard the structure by spreading the load deeper into solid shale.

10. Galleries - In addition to the various floor loads the gallery floor at Elevation 22.5 feet was designed to resist hydrostatic uplift pressure from Elevation 55.0 feet. Side walls were designed for the same pressure.

11. Second Stage Concrete - Except for the floor at Elevation 36.0 feet, the supporting columns and the turbine block, the second stage is mass concrete. Arch action over the scroll case was investigated. The floors and supporting beams at Elevation 36.0 feet were designed under the applicable live loads. Also considered was the torsional load from the generator which is resisted by the slab at Elevation 36.0 feet.

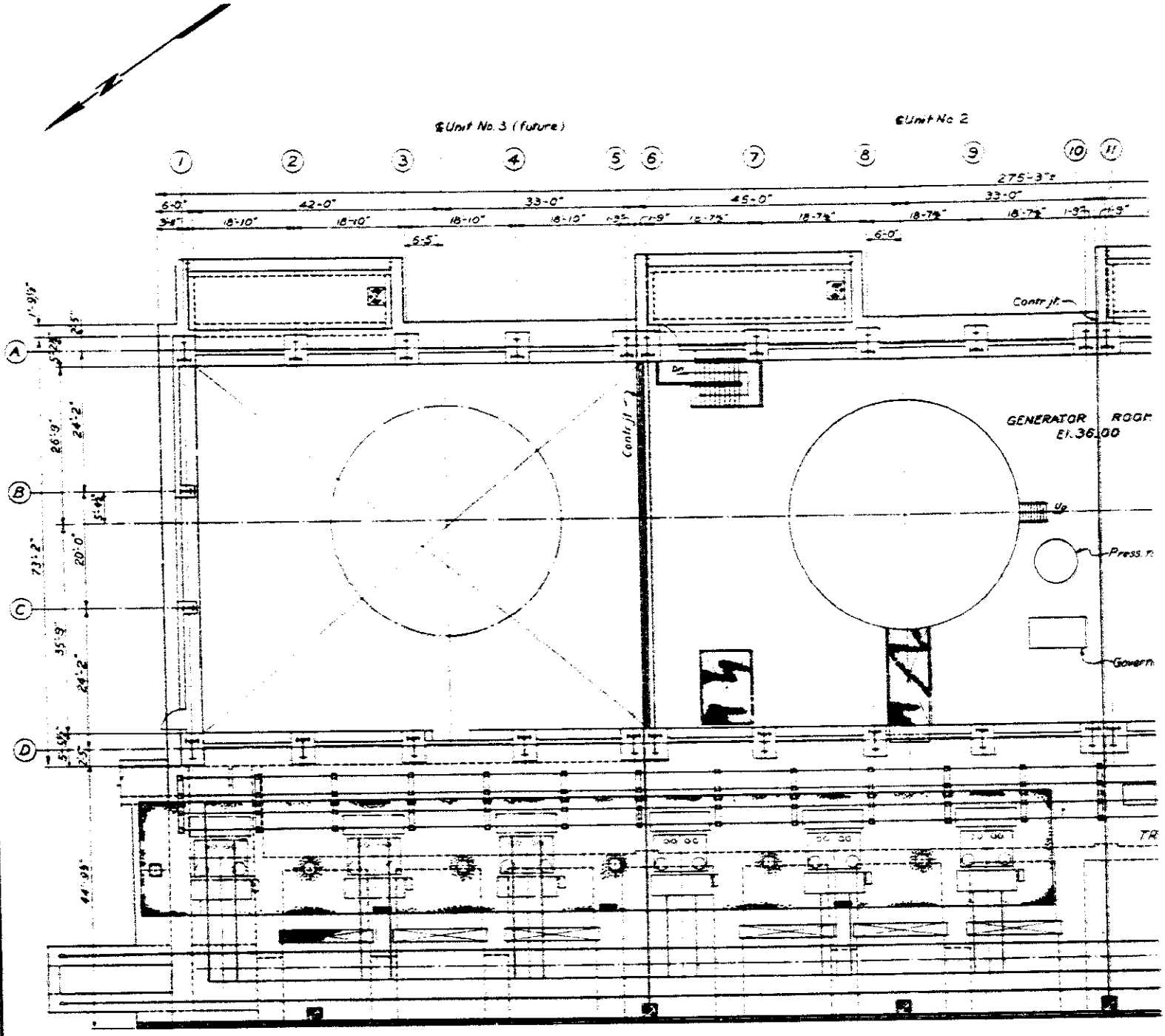
A temporary wall between Units Nos. 2 and 3 was installed to allow flooding of Unit No. 3. This wall is supported at Elevations 22 and 36 feet and will resist a hydrostatic load from Elevation 55.0 feet by cantilever and beam action.

12. Cable Gallery - The cable gallery from the powerhouse to the switchyard consists of a lined tunnel and vertical shaft except for a portion adjacent to the powerhouse. This section is a span which carries its own weight plus deck loads from the transformer deck. The span is simply supported at the powerhouse on a cantilevered beam extending from the north substructure wall and on shale at the other end. The cable gallery lining is unreinforced except at the span section. Access ladders are provided in the shaft. The shaft is sheltered from the weather by a brick house.

PART III - CHAPTER V

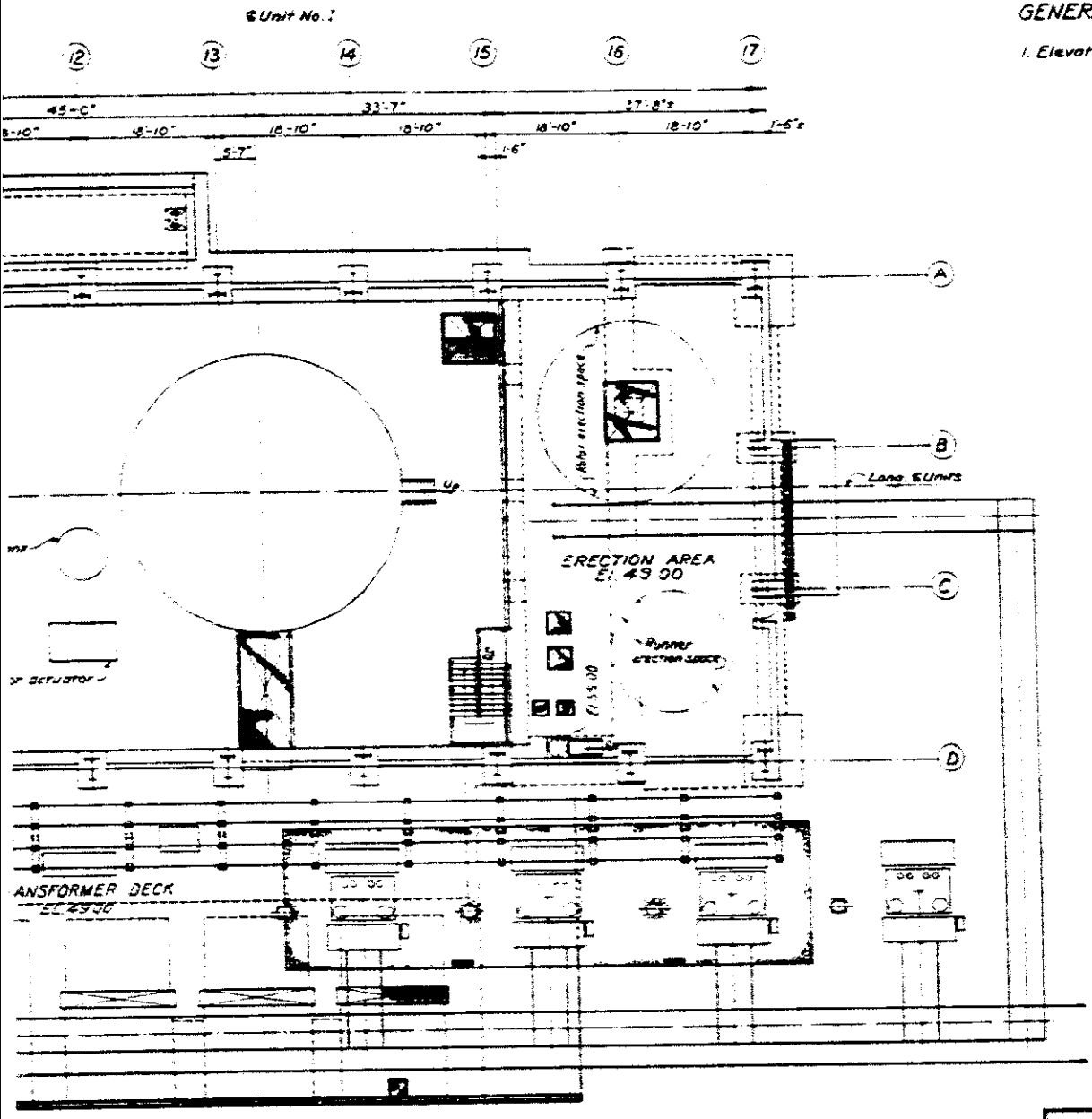
LIST OF EXHIBITS

Exhibit	Title
V-1	Powerhouse Plan - Elevation 49.00
V-2	Powerhouse Plan - Elevation 36.00
V-3	Powerhouse Plan - Elevation 22.50
V-4	Powerhouse Plan - Elevation -19.00
V-5	Powerhouse, Transverse Sections
V-6	Powerhouse, Longitudinal Section


℄ Unit No. 3 (future)
℄ Unit No. 2
1
2
3
4
5
6
7
8
9
10
11
275'-3"±
6'-0"
42'-0"
33'-0"
45'-0"
33'-0"
18'-10"
18'-10"
18'-10"
18'-10"
18'-7½"
18'-7½"
18'-7½"
18'-7½"
5'-5"
6'-0"
Contr. jt.
Contr. jt.
GENERATOR ROOM
El. 36.00
Up
Press.
Govern.
TR
A
B
C
D
26'-9"
24'-2"
73'-2"
20'-0"
35'-9"
24'-2"
44'-9½"

GENERAL NOTE:

1. Elevations shown are based on P.W.D. datum.



Scale 0 10 20 Feet

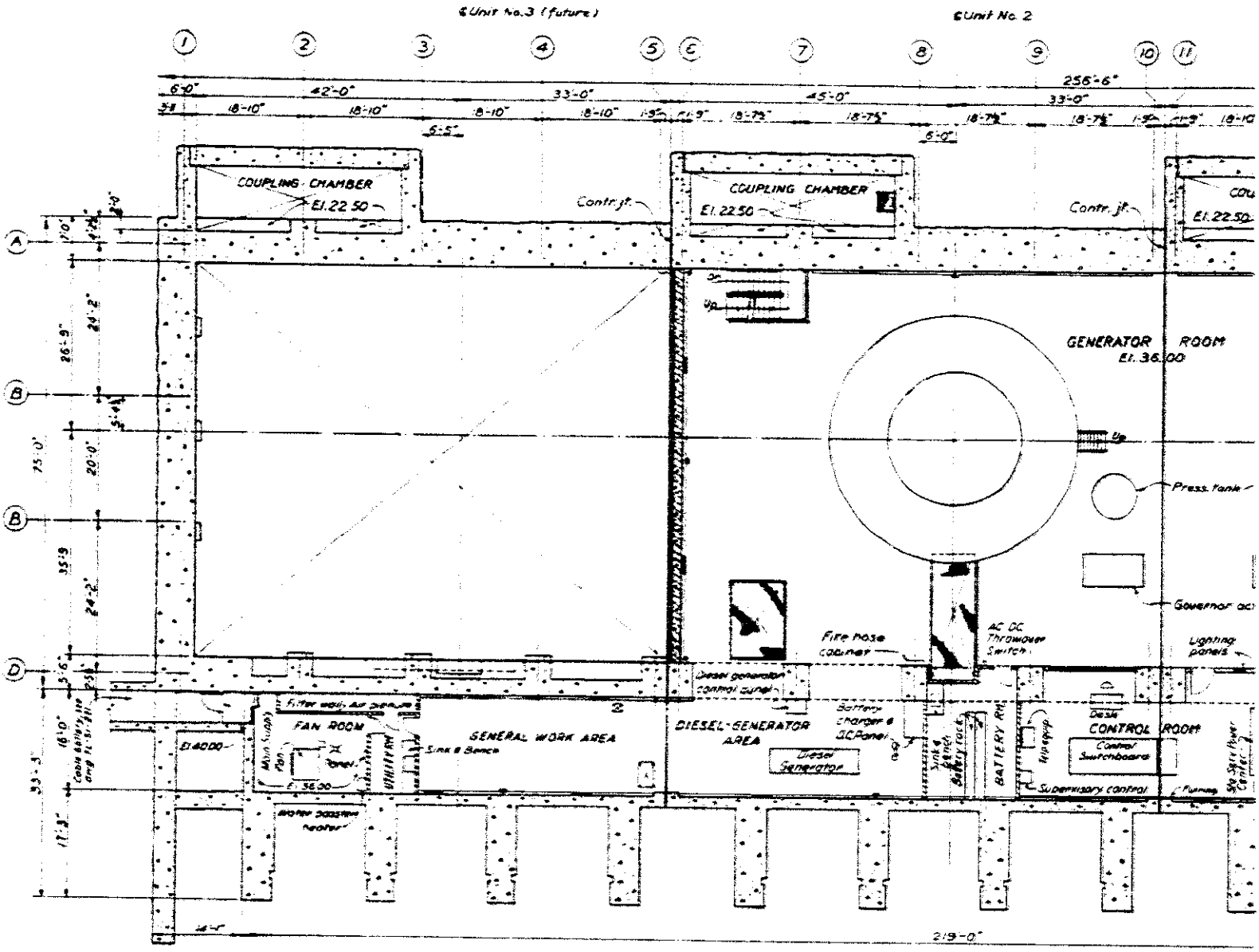
4	
3	
2	
1	1-28-53 V.G. JEL Revised transformer deck
NO.	DATE / REVISIONS

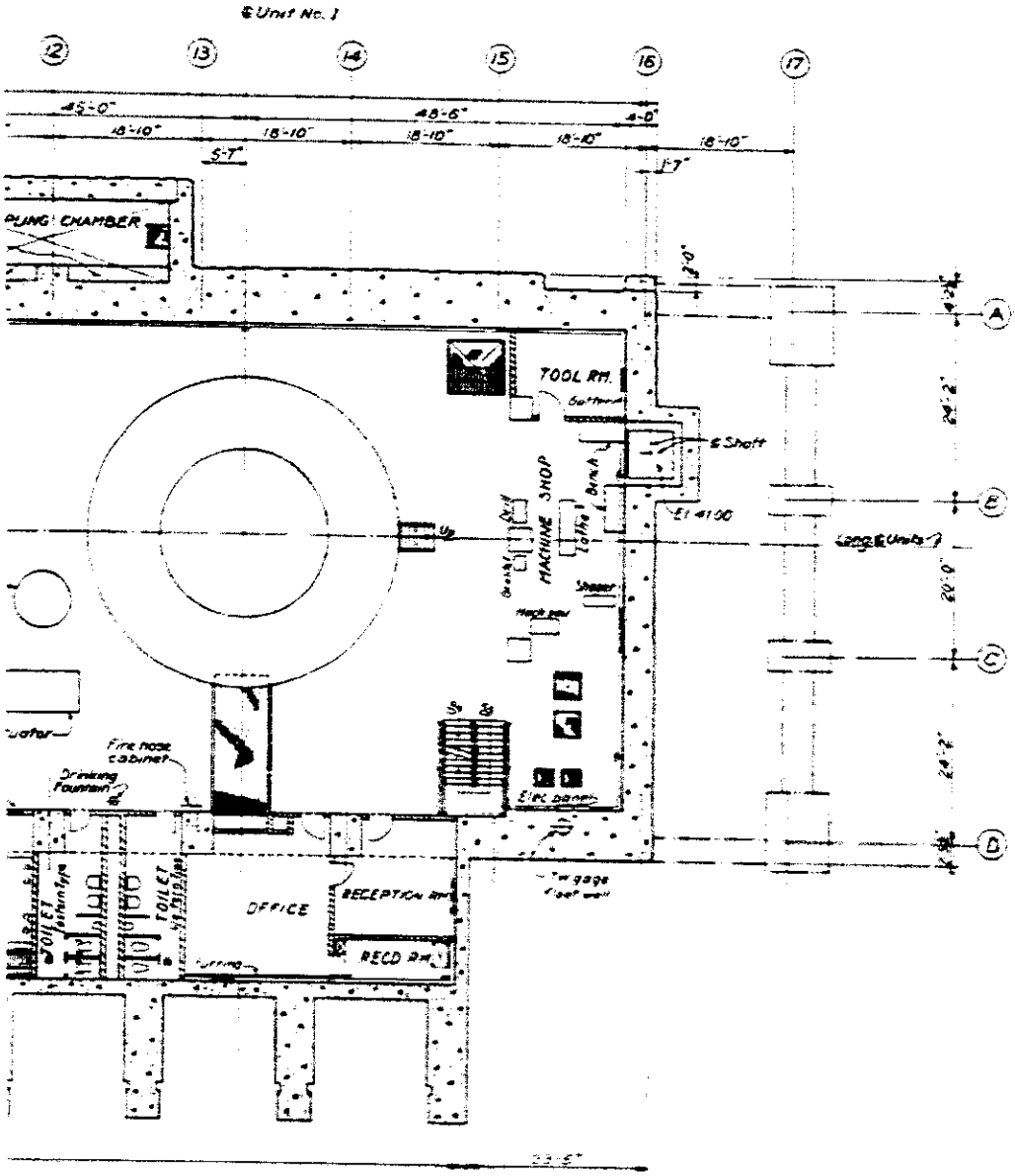
GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

KARNAFULI PROJECT
POWERHOUSE
GENERAL ARRANGEMENT
PLAN - EL. 49.00

INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS

DESIGNED L.E.Co. DRAWN V.G.	CHKD. J.E.L. APPD.	
ICA.-X.-1	SAN FRANCISCO, CAL. DATE March 28, 1952	FL-42-20


℄ Unit No. 3 (future)
℄ Unit No. 2
COUPLING CHAMBER
El. 22.50
Contr. jt.
COUPLING CHAMBER
El. 22.50
Contr. jt.
GENERATOR ROOM
El. 36.00
Press. tank
Governor oil
Fire hose cabinet
AC DC Throwover Switch
Lighting panels
Diesel generator control panel
FAN ROOM
Filter wall, Air pen.
Main Supply Fan
Panel
El. 36.00
Sink & Bench
WASH RM
Water boiling heater
GENERAL WORK AREA
DIESEL-GENERATOR AREA
Battery Charger & DC Panel
Diesel Generator
Sink & Bench
Battery racks
BATTERY RM
Hoist
Diesel CONTROL ROOM
Control Switchboard
Supervisory control
Furnace
Sig Serv Power Center
El. 40.00
Cable Battery Arrg 11-51-211
2'-9"
14'-4"
219'-0"
256'-6"
115



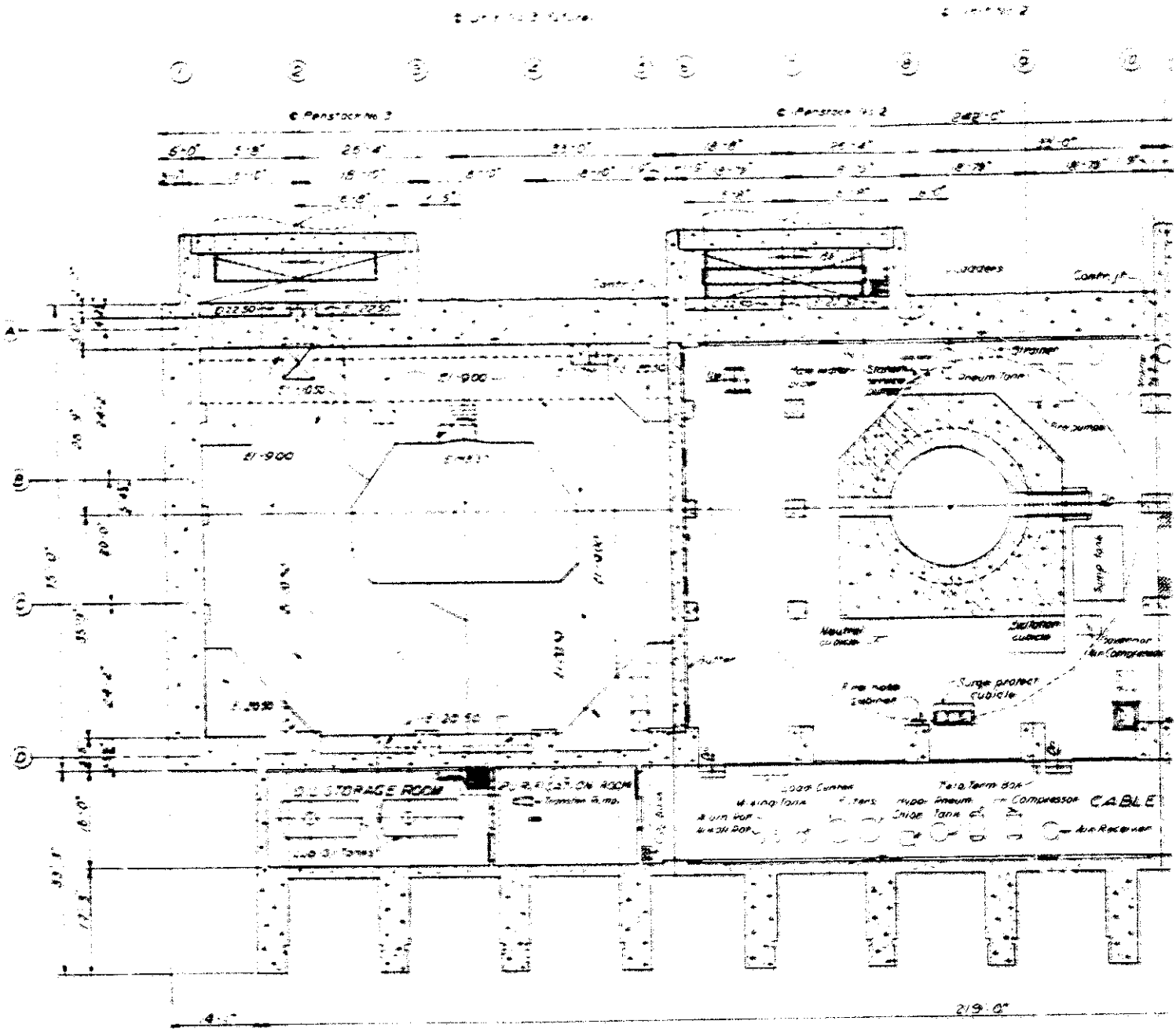
LEGEND:

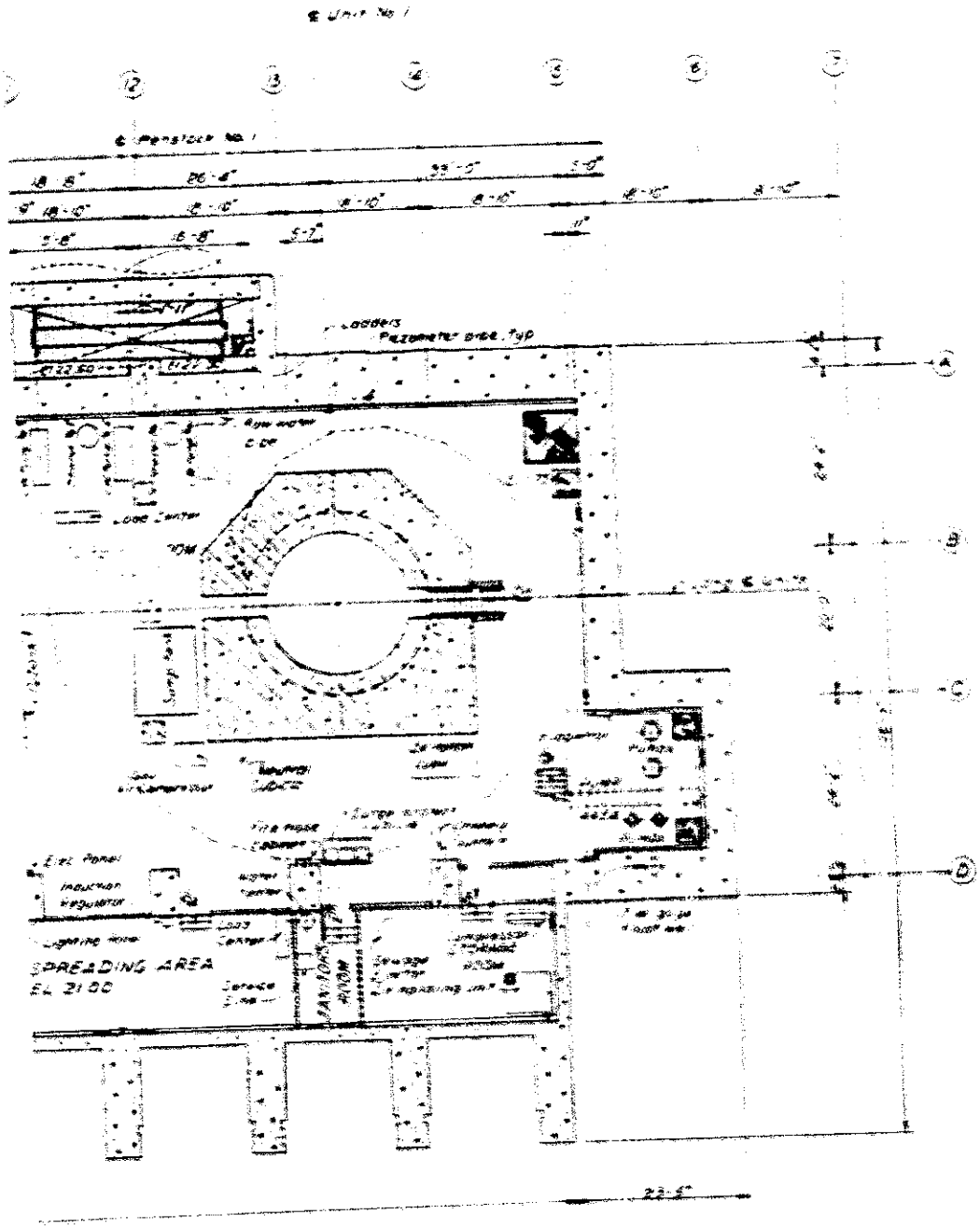
First stage conc

Second stage conc

Scale 0 10 20 Feet

GOVERNMENT OF PAKISTAN MINISTRY OF INDUSTRIES	
KARNAFULI PROJECT POWERHOUSE GENERAL ARRANGEMENT PLAN - EL. 36.00	
INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS	
1 1-28-59 V.G. JEL Minor revision	DESIGNED I.E.Co. DRAWN V.G. CHKD. JEL
REVISIONS	ICA-X-1 SAN FRANCISCO, CAL. DATE March 28, 1958 FL-42-203 R


Penstock No. 3
Penstock No. 2
STORAGE ROOM
Surge protect cubicle
Neutral cubicle
Air Compressor
CABLE
219'-0"



LEGEND:

First stage conc

Second stage conc

Scale 0 10 20 Feet

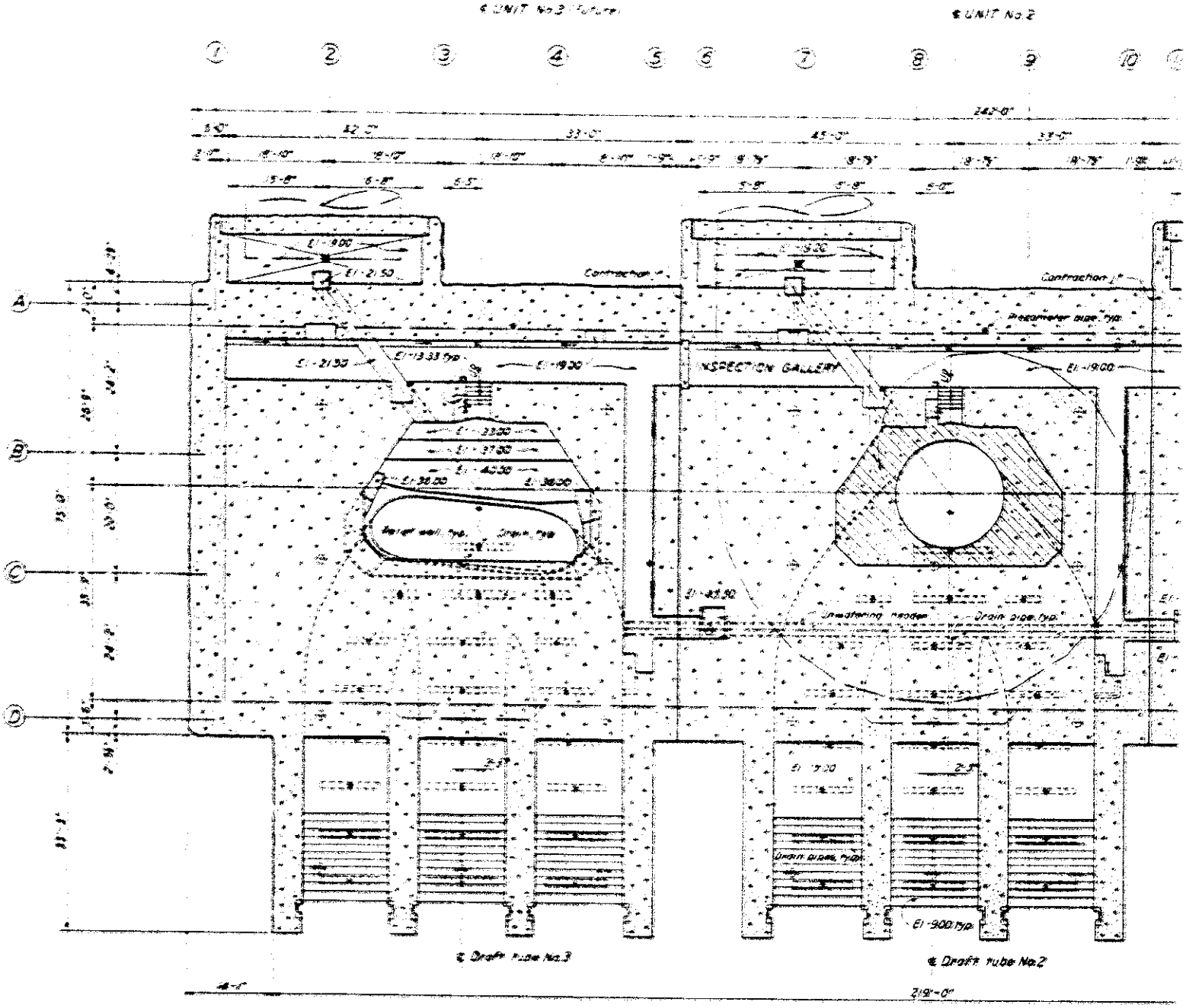
GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAPULI PROJECT
POWERHOUSE
GENERAL ARRANGEMENT
PLAN-EL. 22.50

INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS

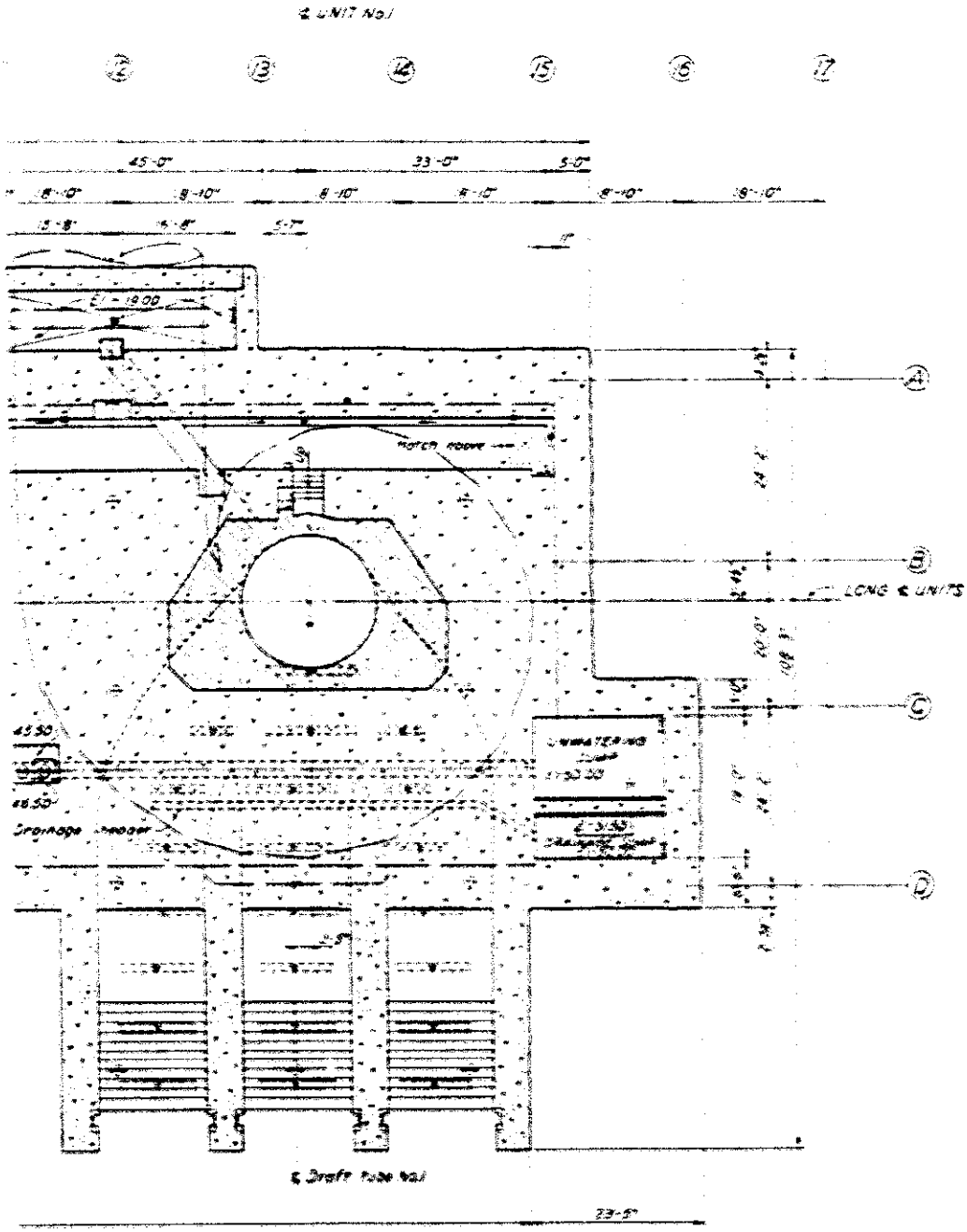
ICA-X-1

SAN FRANCISCO, CAL.
Date March 28, 1958

FL-42-2


℄ UNIT No.3 (Future)
℄ UNIT No.2
INSPECTION GALLERY
Contraction J.
Precoolers pipe, typ.
Drain pipe, typ.
Unwatering header
Drain pipe, typ.
El -900 typ.
℄ Draft tube No.3
℄ Draft tube No.2

- 117



LEGEND

First stage conc

Second stage conc

Scale 0 10 20 Feet

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

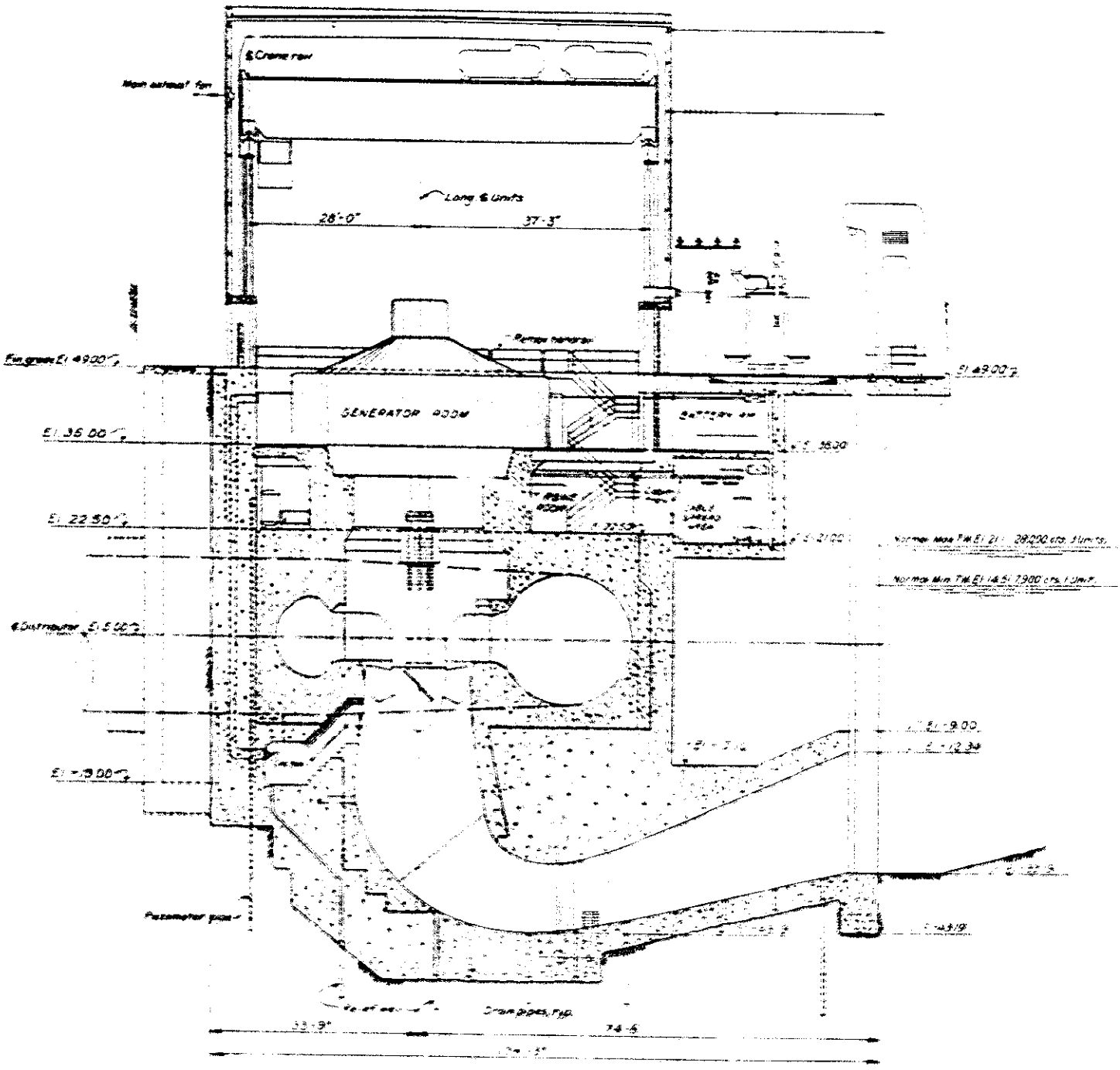
KARNAFULI PROJECT

POWERHOUSE
GENERAL ARRANGEMENT
PLAN-EL-1900

INTERNATIONAL ENGINEERING CO. INC. — ENGINEERS

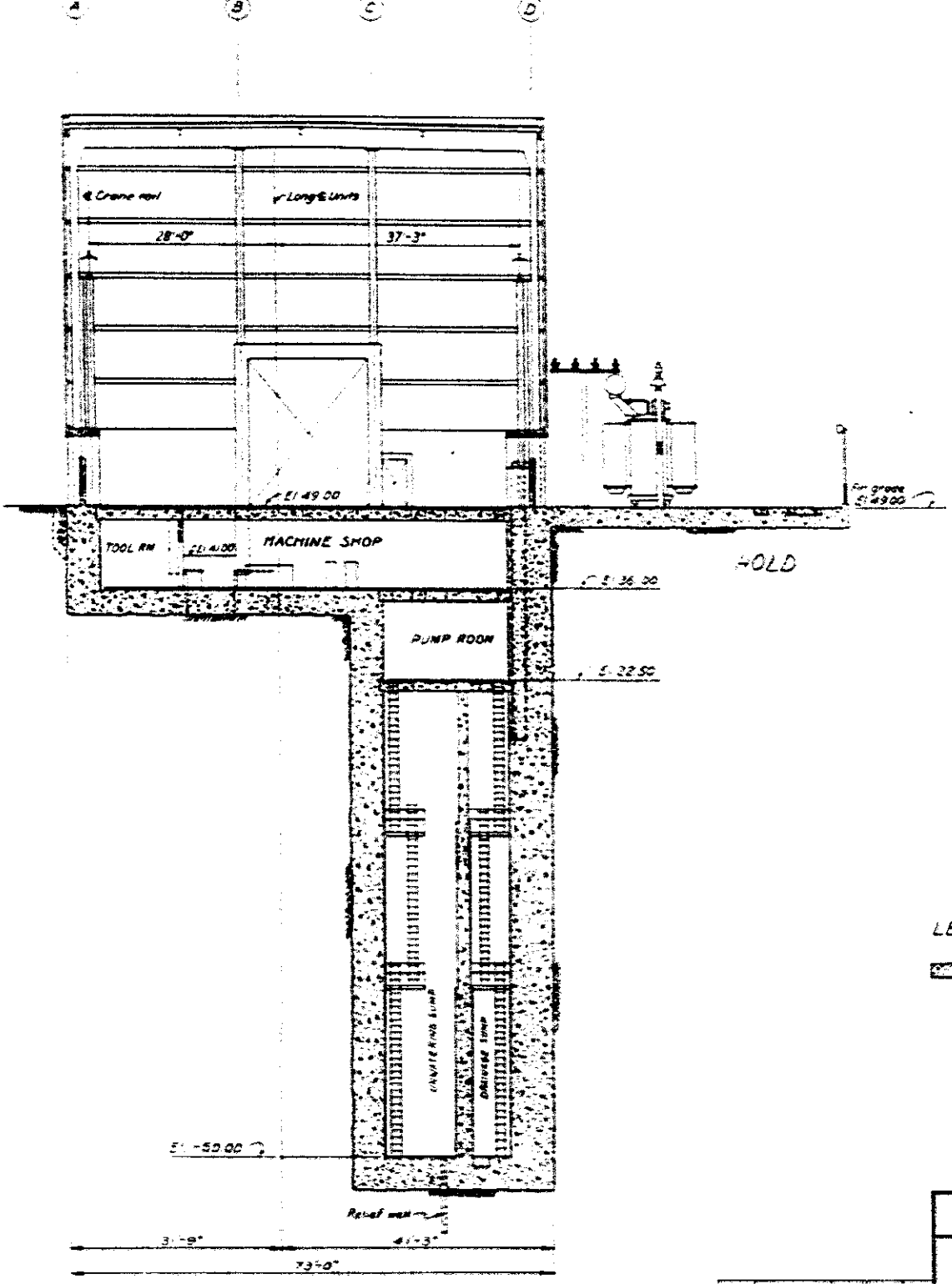
4			
3			
2			
1	12-16-58		
	Minor revision		

KA-X-1	Date March 28, 1958	F-10-205



TRANSVERSE SECTION THRU ℄ UNIT No 2

16



TRANSVERSE SECTION THRU. SUMPS

LEGEND:

First stage conc.

Second stage conc.

Scale 0 10 20 Feet

4				
3				
2				
1	1-28-59	V.G.	J.E.L.	Cal.

REVISIONS

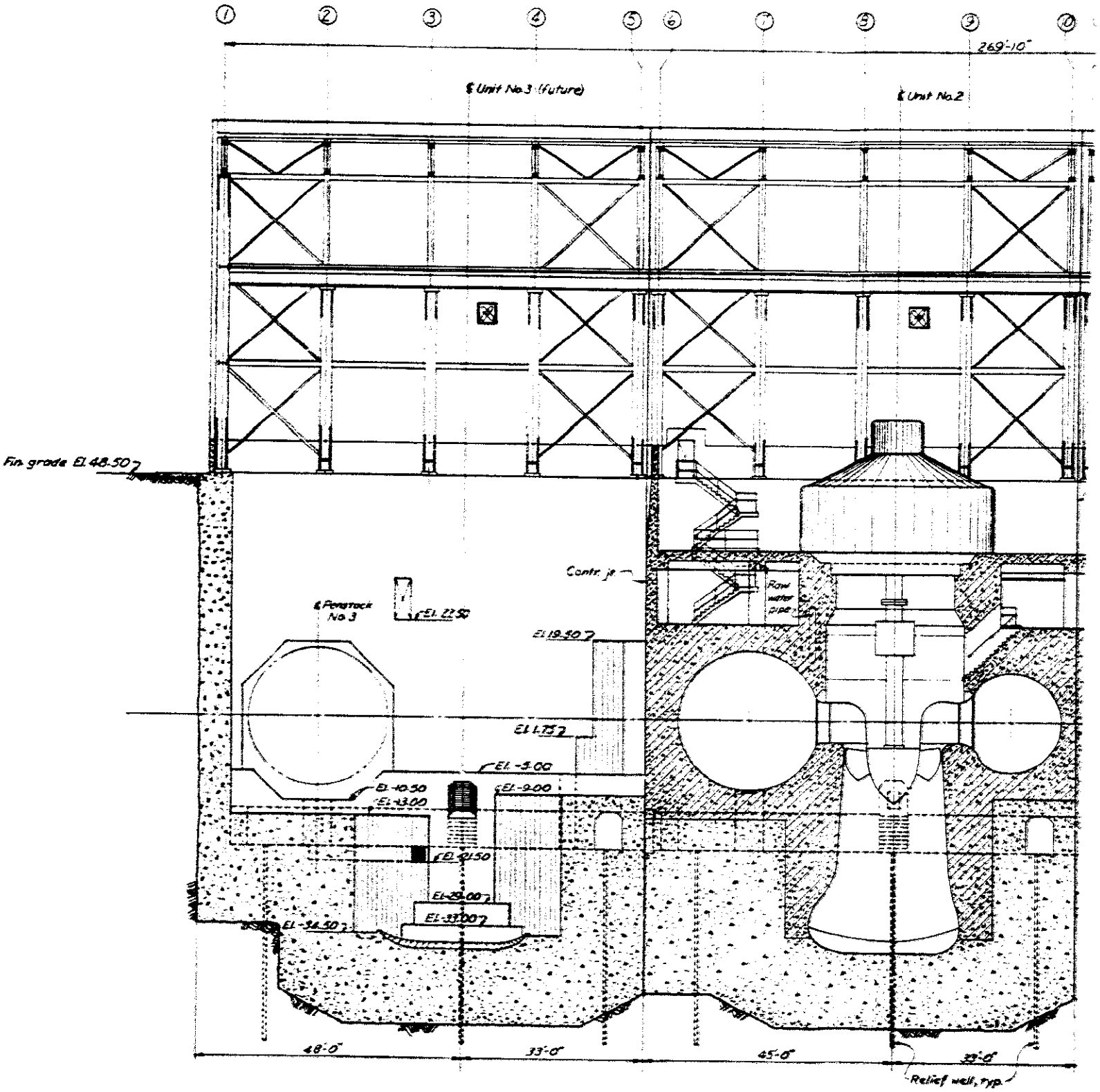
GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

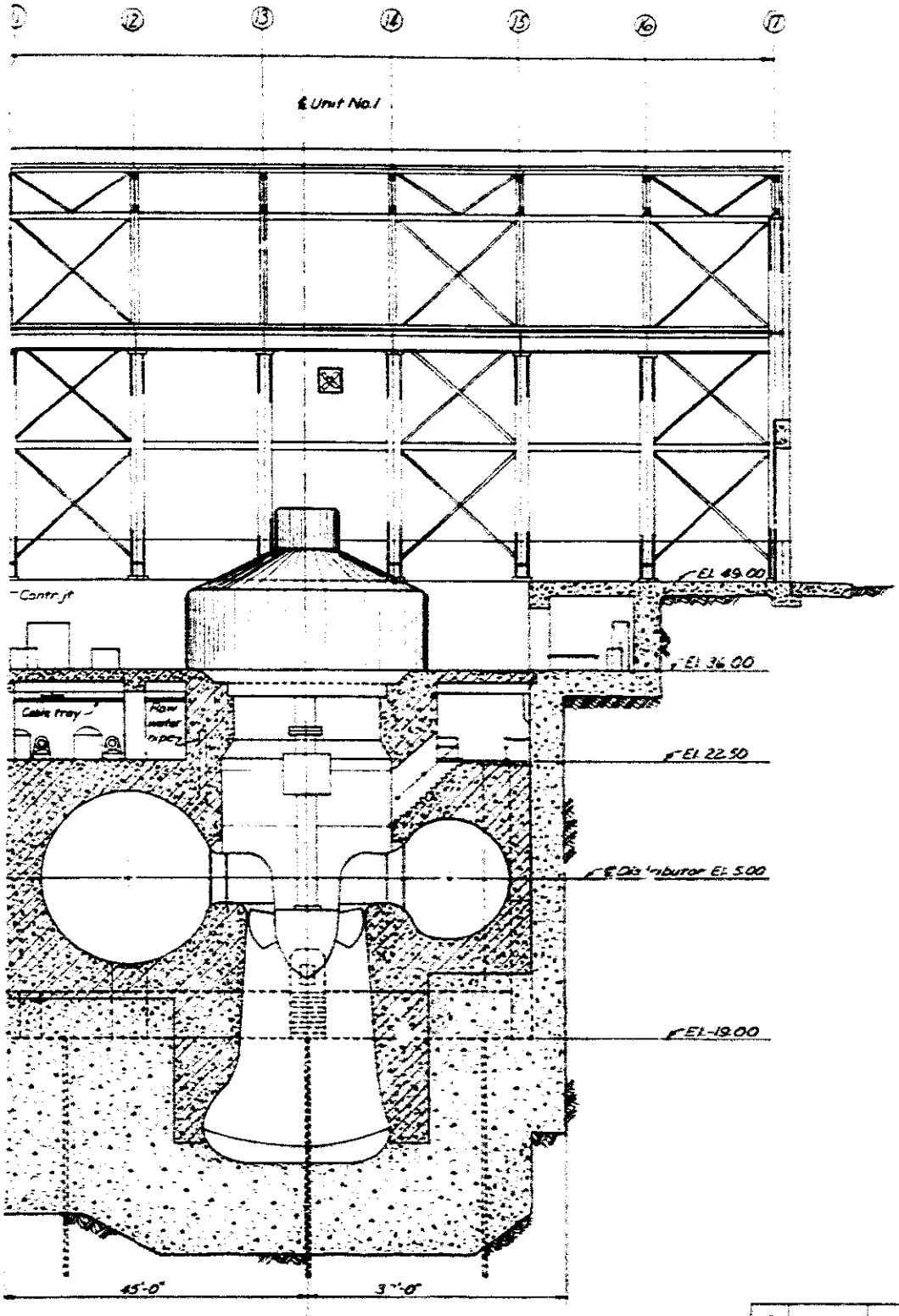
KARNAFULI PROJECT
POWERHOUSE
GENERAL ARRANGEMENT
TRANSVERSE SECTIONS

INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS

Designed I.E. Co. Drawn V.G. Checked J.E. Submitted

ICA-X-1 | San Francisco, Cal. | Date March 28, 1958 | FL-42-206


1
2
3
4
5
6
7
8
9
10
269'-10"
℄ Unit No.3 (future)
℄ Unit No.2
Fin. grade El. 48.50
Contr. jt.
Raw water pipe
℄ Penstock No.3
El. 22.50
El. 19.50
El. 1.75
El. -5.00
El. -10.50
El. -13.00
El. -9.00
El. -21.50
El. -25.00
El. -33.00
El. -36.50
48'-0"
33'-0"
45'-0"
28'-0"
Relief well, typ.



LEGEND:

First stage conc.

Second stage conc.

Scale 0 10 20 Feet

GOVERNMENT OF PAKISTAN MINISTRY OF INDUSTRIES	
KARNAFULI PROJECT POWERHOUSE GENERAL ARRANGEMENT LONGIT. SECTION THRU ℄ UNITS	
INTERNATIONAL ENGINEERING CO. INC. - ENGINEERS	
DESIGNED LE Co. DRAWN D.E.L. V.G.	CHKD. J.E.L.
ICA-X-1	SAN FRANCISCO, CAL. DATE March 28, 1958
	FL-42-207

REVISIONS

CHAPTER VI

SWITCHYARD AND SINGLE-CIRCUIT TOWER

6.01 GENERAL

The center of the switchyard is situated approximately 470 feet downstream from the centerline of the powerhouse generators and 400 feet to the right of the tailrace centerline. It is oriented in such a way as to permit approximately normal conductor take-off.

To establish this location, the following requirements for safe and practical construction were observed:

1. The structure must be surrounded by a protective fence, enclosing a square with lateral dimension of 150 feet.

2. This area must be situated as near as practicable to the powerhouse, angled away from the tailrace centerline only so much as to easily maintain a safe clearance between wires throughout, and at some location which would require a minimum of costly grading and excavation.

These requirements were met by the final location and orientation of the switchyard.

6.02 DESIGN CRITERIA AND ASSUMPTIONS

Basic criteria and assumptions established for the design of the switchyard structure and the single-circuit tower are as follows:

A. Loads

1. Dead Loads - The dead loads consisted of the weight of all permanent construction entering into and becoming a part of the final structures, including structural steel, mounted equipment and insulators.

2. Wire Loads - The wire loads consisted of the maximum stringing tension developed. This tension was assumed to act horizontally without reduction. A vertical component of twenty per cent of the stringing tension was assumed.

3. Wind Loads - Wind pressure was assumed to be 12 pounds per square foot on the projected area of pipes and wires, and 15 pounds per square foot on 150 per cent of the projected area of the boxed lattice structures.

4. Earthquake Forces - Seismic forces were assumed to be 10 per cent of the sum of the dead and permanent loads, which included vertical wire loads, mounted equipment and insulators, applied horizontally along the centerline of the respective beams.

B. Allowable Stresses

All structural steel material for the switchyard shall conform to ASTM Designation A7, and all connection bolts to ASTM Designation A394. High strength members shall conform to ASTM Designation A94.

The allowable safe stresses for the design of the switchyard were assumed to be as follows:

Members in tension - 25,000 pounds per square inch

Members in compression -

a. Slenderness ratio less than 150 25,000 - 110 L/r pounds per square inch with a maximum of 20,000

b. Slenderness ratio more than 150 18,000 - 67 L/r pounds per square inch

Connection bolts in shear - 22,000 pounds per square inch on full shank
Connection bolts in bearing - 44,000 pounds per square inch in single shear.

The maximum slenderness ratio was assumed to be as follows:

Main members	140
Secondary members having calculated stress	200
Redundant members	250

Safe stresses in the high-elastic steel main members of the single-circuit tower were assumed to be as follows:

Members in tension 30,000 psi

Members in compression

a. Slenderness ratio less than 100 32,500 - 145 L/r pounds per square inch with a maximum of 26,000

b. Slenderness ratio less than 150 28,500 - 113 L/r pounds per square inch with a maximum of 26,000

c. Slenderness ratio more than 150 26,000 - 97 L/r pounds per square inch

6.03 TYPES OF LOADING

A. Switchyard

The loading combinations investigated were as follows:

1. Full dead load, all conductors and static wires and full wind from any direction, for both the first-stage two-bay and the future three-bay structure.

2. Full dead load, all conductors and static wire and full earthquake force, again for both first-stage and future structure.

3. Full dead load, all or part of the conductors and static wires considered broken on one face of the structure, and wind in the direction of the maximum resultant wire pull, for both first-stage and future structure.

4. The same as in 3., but with wind in a direction normal to conductors and static wires.

Inspection and partial analysis of the above mentioned combinations indicated that the maximum loading condition to which the switchyard frame would be subjected was the third, with all the conductors and static wires on either the powerhouse or the transmission line side considered broken and wind in the direction of these wires, respectively away from or toward the powerhouse. The maximum horizontal loading condition for the continuous beams which support the conductors, was with all conductors on alternate spans broken as well. To obtain the maximum stresses in the web members, full wind was applied in the direction normal to that of the conductors. The resulting loading combinations were considered to be fully reversible. It was noted that in no case did stress due to earthquake forces exceed those due to wind loads.

The design of the smaller, independent structures was based on a single combination of dead and equipment loads and wind forces.

B. Single-Circuit Tower

The loading combinations investigated were similar to those of the switchyard. In addition, conductors were considered broken in opposite directions in order to obtain torsional stress.

C. Footings

Reinforced concrete footings were designed to support the switchyard towers and equipment. Anchorages and structural members in the footings were designed to transmit all combinations of weight, moments, and thrust.

Foundation pressures in the larger footings (8-foot x 10-foot) were held to 4.5 kips per square foot while pressures for the smaller footings were less, in accordance with good practice for this type of foundation material.

6.04 METHODS OF ANALYSIS

The basic method of analysis was that of moment distribution. The boxed members, both towers and beams, were considered as continuous frames in all the vertical and horizontal planes into which they could be fitted.

Two base conditions were considered:

1. Bases fully fixed, for maximum stresses at the base.

2. Bases 50 per cent fixed, for maximum stresses at the junction of towers and beams.

Any fixity of less than 50 per cent was considered highly unlikely in view of the relatively small amount of rotation inherent in the column bases and footings.

6.05 RESULTS

The results were plotted as shear and moment diagrams for the various loading combinations. All moments were than analyzed as couples acting as direct axial loads on the main members (i.e. tower legs and beam chords) and all shears were distributed as axial loads acting on the web members. The web members of the panels common to both tower and beam were subject to stresses from each. All moments and shears were then divided by two to yield the stresses per face of component member.

Addition of stresses due to loading in directions normal to each other, where such was the critical condition, was effected to obtain maximum tension and compression stresses in all members. The member sizes were then determined on the basis of the design criteria listed previously.

The stiffness factors and dead weights of the selected main member sections were compared with the original assumptions and were found to be in close agreement.

PART III

CHAPTER VII

CARGO TRANSFER SYSTEM

7.01 GENERAL

A. Purpose

The primary purpose of the cargo transfer system is to move bamboo and wood rafts from the reservoir to the lower Karnafuli River. It will also handle small craft and other cargo to and from the reservoir.

B. Location

The cargo transfer system crosses the narrow ridge between the reservoir and the river at saddle dam No. 5. The river terminal is located approximately 800 feet downstream from the end of the powerhouse tailrace channel. For location see Exhibit 2, PART I. This site was selected to minimize the length of the structure and to keep the travel time from the reservoir to the river as short as possible.

C. Description

The cargo transfer system consists of an all-welded structural steel superstructure with looped main track on which electrically-driven monorail trolleys with hoists travel in a continuous circuit between the reservoir and the river. The direction of travel is counter-clockwise as viewed in plan. The main track, side track, work platforms and continuous catwalk are supported by 11 structural steel bents spaced at 45.0-foot centers. At either end, the track overhangs the supporting bent for a distance of 20.0 feet and is flared to permit turning. The height of the substructure is such that suspended loads clear the ground surface a minimum of 6 feet. Access to the catwalk is by way of ladders at bents No. 3, No. 5, and No. 9.

The structural steel bents are supported by reinforced concrete bents. The concrete substructures at either terminal extend above high-water levels. Layout of the cargo transfer system is shown on Fig. A. The design of the trolleys is given in Part V, Chapter VII. Terminal facilities are discussed in Paragraph 7.05.

7.02 OPERATIONAL REQUIREMENTS

The cargo transfer system was designed for the following operational requirements:

1. Capacity shall be 150,000 pounds of cargo per hour.

2. The system shall be capable of operation with reservoir levels from Elevation 75 feet to Elevation 118 feet and river levels from Elevation 10 feet to Elevation 49 feet.

3. The cargo shall consist of:

   a. Bamboo rafts, 20 feet by 6 feet by 1 foot weighing approximately 7,500 pounds.

   b. Small river boats varying in size from 15 feet by 4 feet by 2.5 feet to 50 feet by 7 feet by 7 feet weighing up to 16,000 pounds.

4. Trolleys shall be capable of tandem operation to handle loads up to 16,000 pounds.

7.03 SUPERSTRUCTURE DESIGN

A. Loads

The superstructure was designed for the following combinations of loadings:

1. Dead load, live load and 6 pound per square foot wind load.

2. Dead load and 30 pounds per square foot wind load.

3. Dead load and horizontal seismic load of 10 percent of gravity.

See Fig. B for the river terminal bent loadings.

B. Stresses

Maximum steel stresses were limited to 17,000 pounds per square inch.

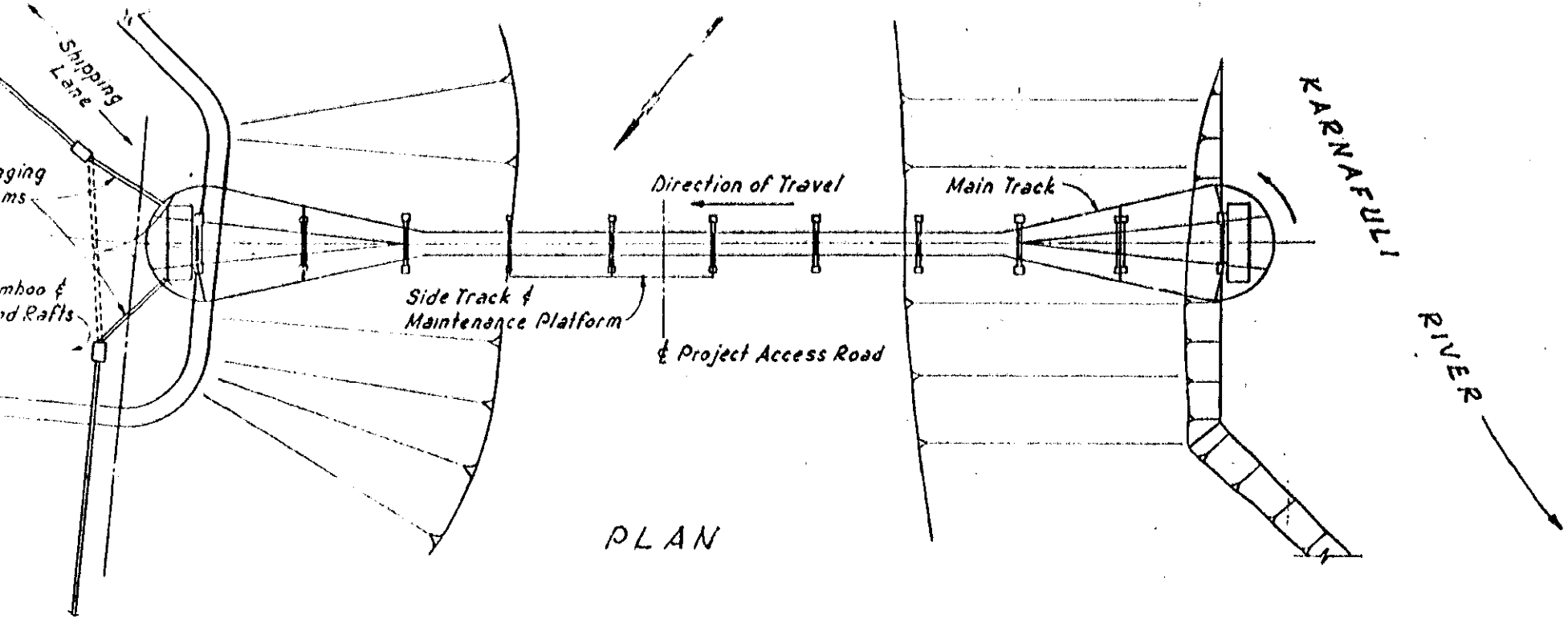
A minimum factor of safety of 6 for wire rope backstays was used.

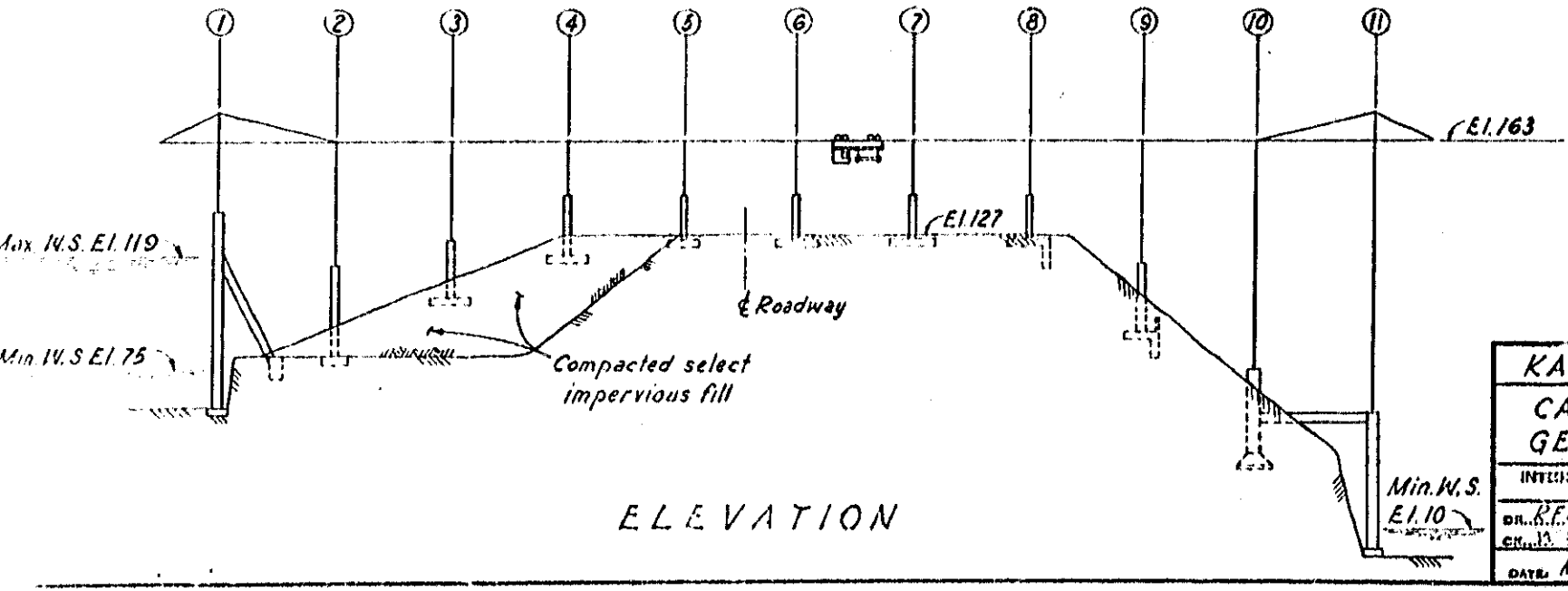
C. Design

All bents were analyzed by moment distribution for the loads given above. Selection of sections were based on stresses from governing loads and within the allowable maximum. No member will deflect more than 0.25 percent of its unsupported length for the design loads. Wire rope longitudinal backstays brace the structure against wind and earthquake forces between bents No. 3 and 9.

Base plates and anchor bolts were designed to transmit all loads to the substructure.

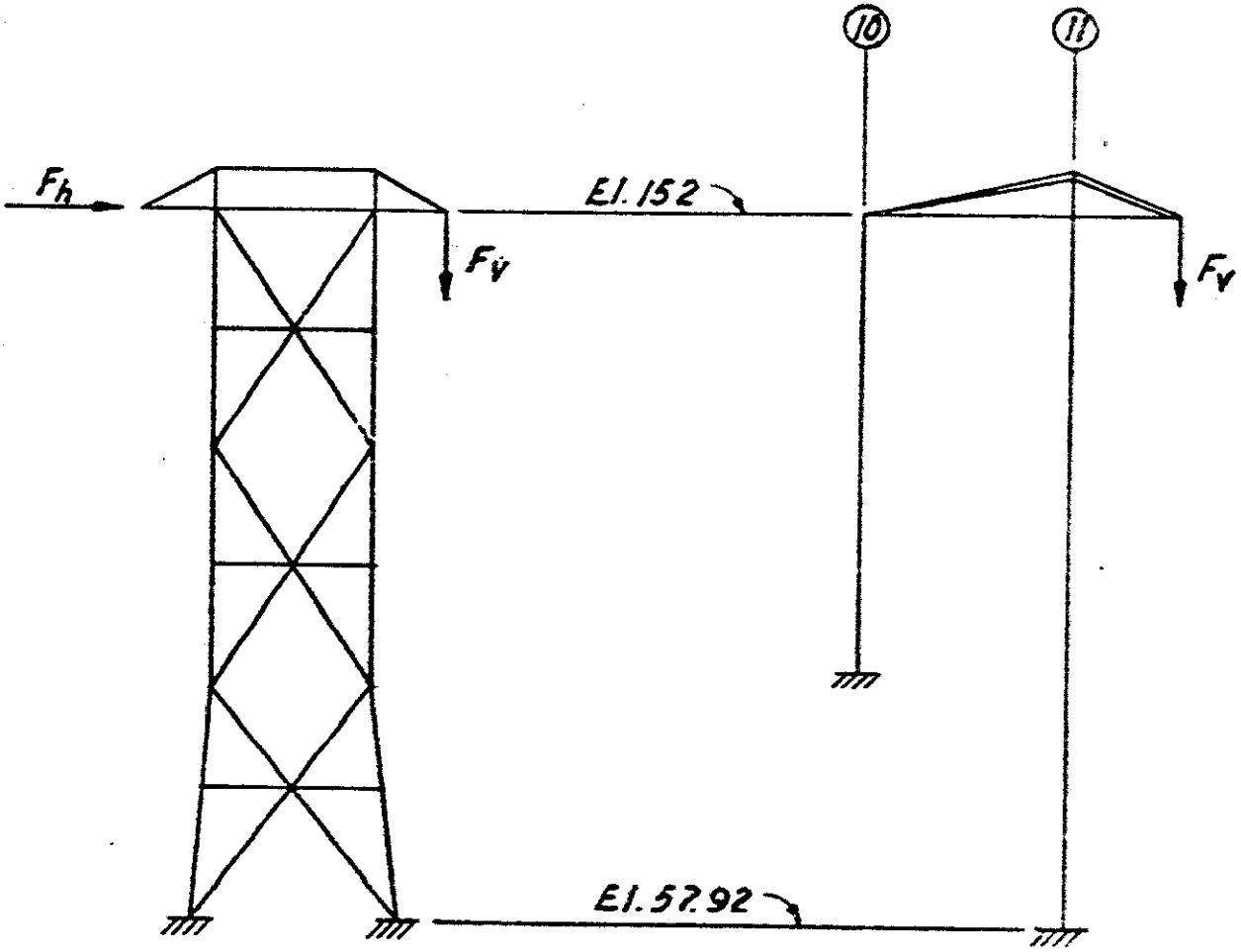
The design of the superstructure was such that field welded joints were held to a minimum.





Scale: 1" = 50'

KARNAFULI PROJECT	
CARGO TRANSFER GENERAL LAYOUT	
INTERNATIONAL [illegible] CO., INC.	
DR. R.F.L. CK. [illegible]	C.E.H. [illegible]
DATE May 1961	FIGURE A



BENT
COLUMN LINE 11

ELEVATION

Design Condition	Live Loads in Tons F_v	F_h
1.	19.1	3.3
2.	8.5	4.2
3.	Seismic Load, 10 % g.	

KARNAFULI PROJECT

CARGO TRANSFER
SUPERSTRUCTURE
RIVER TERMINAL

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. RFL	RECOMMENDED C.E.H.	APPROVED C.B.K.
CK. RES		
DATE: May, 1961		FIGURE B

7.04 SUBSTRUCTURE DESIGN

A. Loads

The substructure was designed for the following loads:

1. Loads from the superstructure plus substructure dead loads and a horizontal seismic load of 10 percent of gravity as applicable.

2. In addition the terminal substructure bents were designed to resist a horizontal impact load of 30 kips at any water surface elevation.

See Fig. C for loadings on the reservoir terminal substructure bent.

B. Stresses

Maximum allowable stresses for the design of the substructure were as follows:

1. Concrete stresses - As specified by the American Concrete Institute for 3,000 pound concrete at 28 days.

2. Reinforcement steel - 20,000 pounds per square inch.

3. Bearing pressure on shale - 4,500 pounds per square foot.

4. Bearing pressure on fill - 1,000 pounds per square foot.

C. Design

The substructure bents were designed as rigid frames to meet the requirements of loads and stresses above. The base slabs for bents No. 3 and 4 rest on compacted fill. All other bents have base slabs resting on solid shale. All base slabs are large enough to transmit the loads to the foundation without exceeding the allowable bearing pressures.

The terminal bents are braced so that the 30 kip impact load causes only minor deflections at the juncture of the substructure and superstructure. This insures that superstructure stresses from impact loads at the water surface will be negligible.

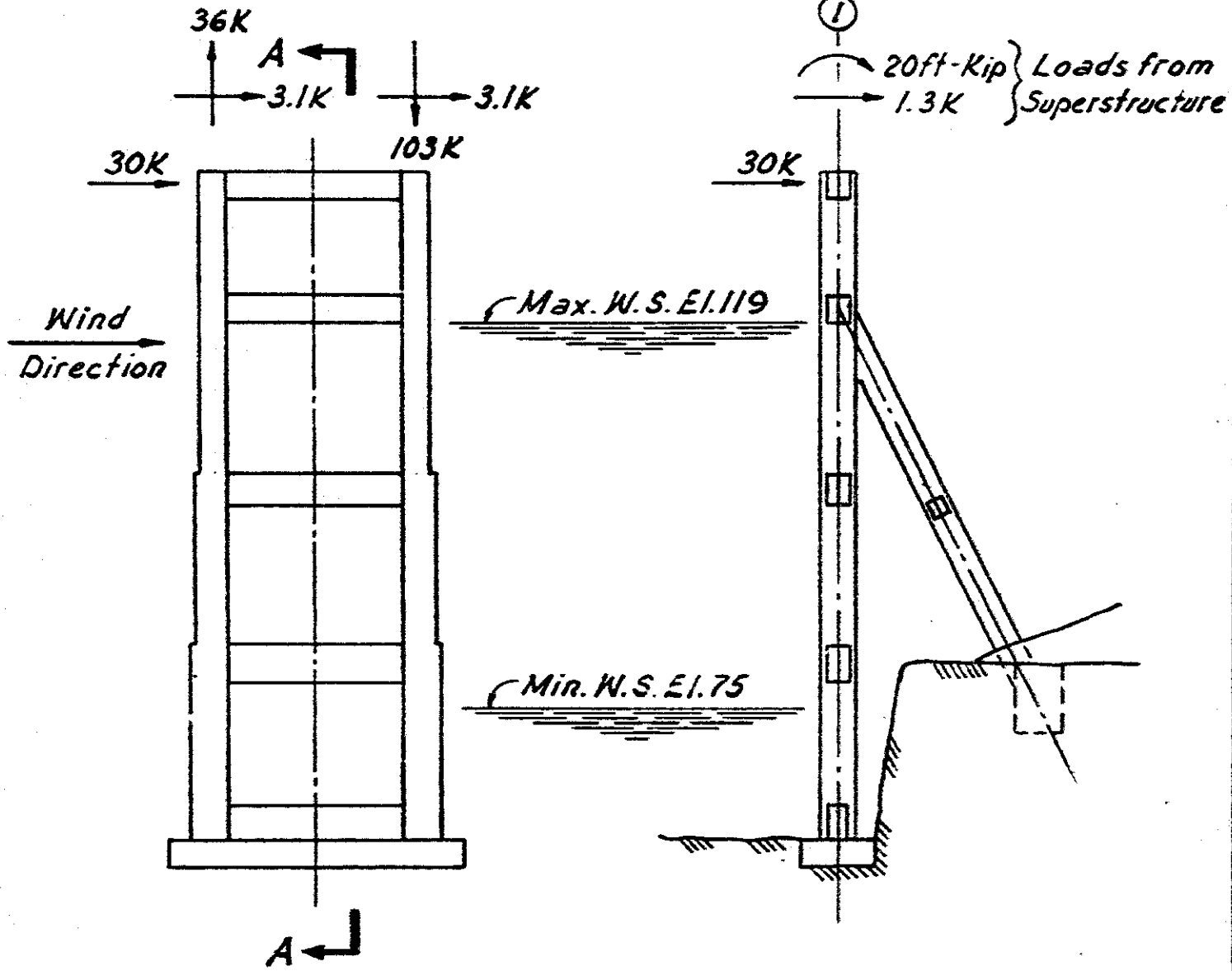
The reservoir terminal substructure bent is braced by reinforced concrete diagonal struts anchored by a reinforced concrete footing. The river terminal substructure bent is braced by reinforced concrete beams which extend from the top of each side column to the side column of bent No. 10.

7.05 TERMINAL FACILITIES

Floating concrete pontoon wharfs at each terminal serve as work platforms for operating personnel. These floating wharfs ride on rails embedded in the terminal substructure bents and are equipped with heavy springs to reduce impact forces transmitted to the structure.

At the reservoir terminal, floating training booms and winches facilitate the routing of cargo. A floating ramp, stairways, and ladders provide easy access for operating personnel.

The floating wharfs are divided into 8 compartments and will float with 3 compartments flooded. The top slab of each compartment has a 4-inch pipe insert and plug so that any leakage may be pumped out. In case of damage to the walls or bottom of the floating wharf it may be hoisted and turned for repair by means of the trolley hoist.



END ELEVATION

SECTION A-A

Scale: 1" = 20'

KARNAFULI PROJECT		
CARGO TRANSFER SUPERSTRUCTURE AT RESERVOIR TERMINAL		
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. RFL CK. WES	RECOMMENDED C. E. H.	APPROVED
DATE: May, 1961		FIGURE C

KARNAFULI PROJECT

DESIGN REPORT

PART IV

ARCHITECTURAL AND FINISH TREATMENT

CHAPTER I GENERAL

CHAPTER II POWERHOUSE

2.01 General

2.02 Powerhouse Approach

2.03 Interior Arrangement

2.04 Superstructure Design and Finishes

2.05 Interior Materials and Finishes

CHAPTER III OTHER STRUCTURES

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

KARNAFULI PROJECT

INTERNATIONAL ENGINEERING CO., INC

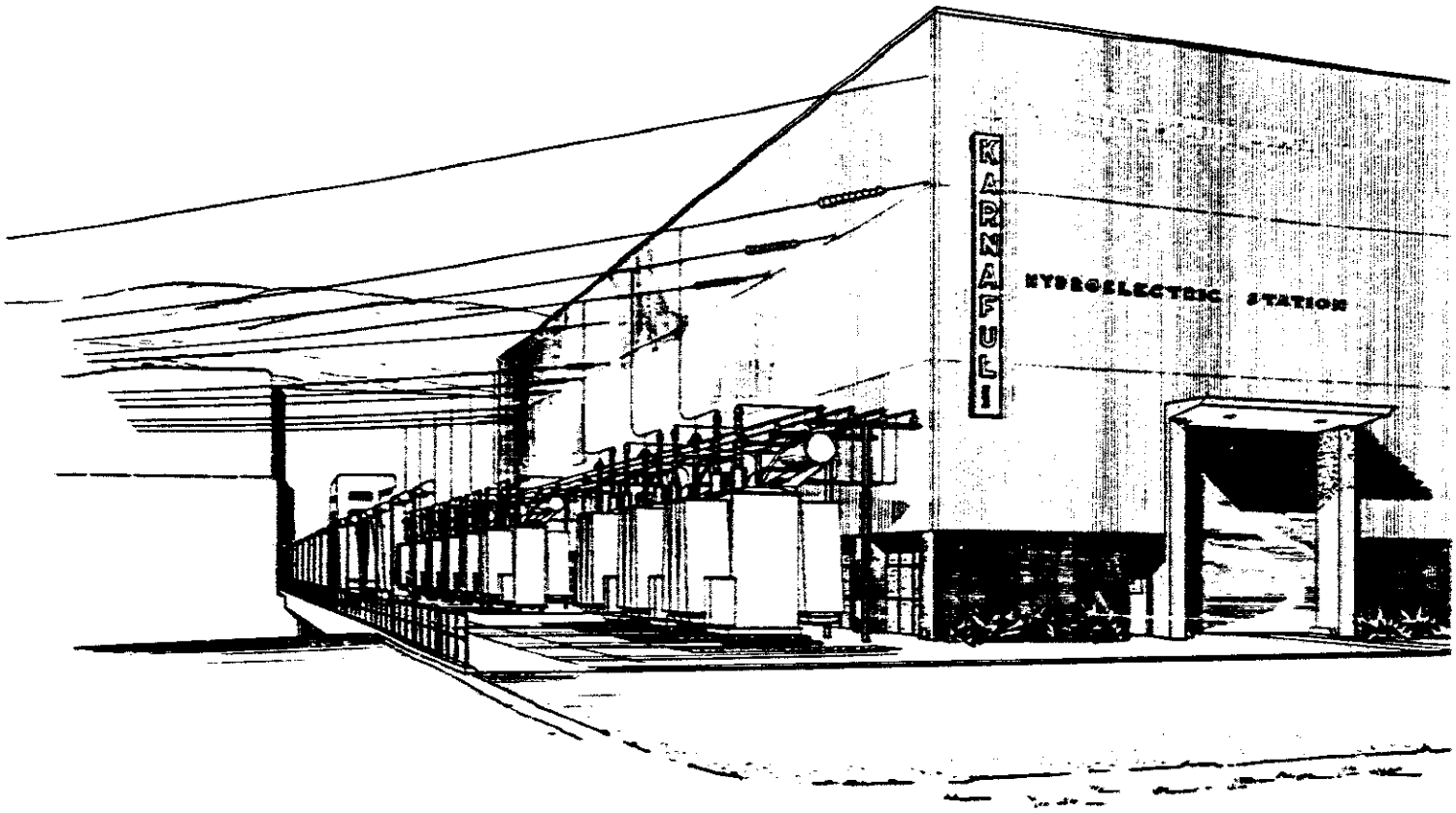
PART IV

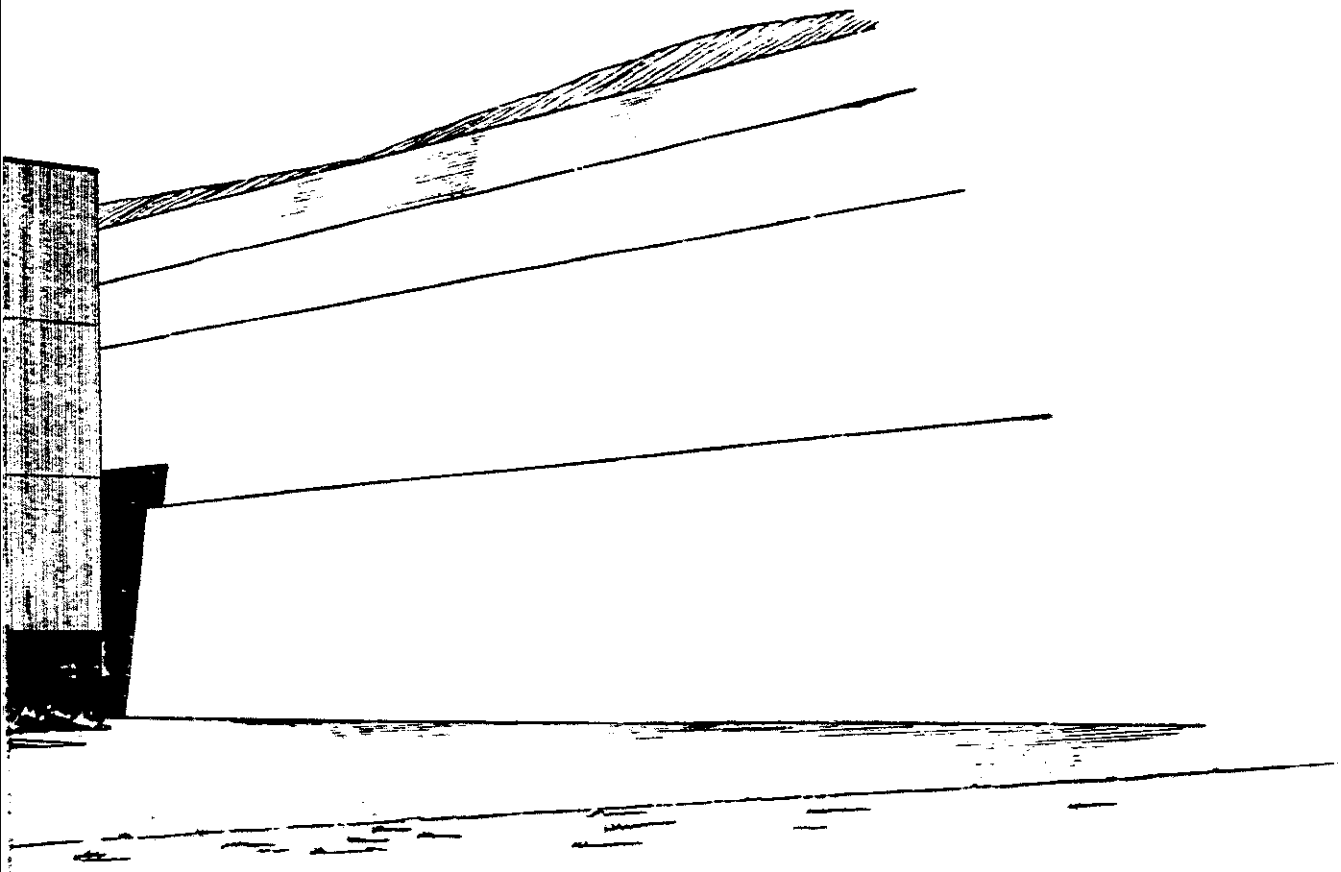
CHAPTER I

GENERAL

The structures in the Karnafuli Project are widely separated; however, they are related to each other not only by their interconnecting functions but also by their materials and finishes. The materials selected for use throughout the project are durable, easy to construct, and easy to maintain, and of quality high enough to justify pride in the appearance of the finished structures. Simplicity of line and shape, and repetition of materials and colors have contributed to the continuity of the project design.

An overall project perspective precedes this page. (Fig. A)

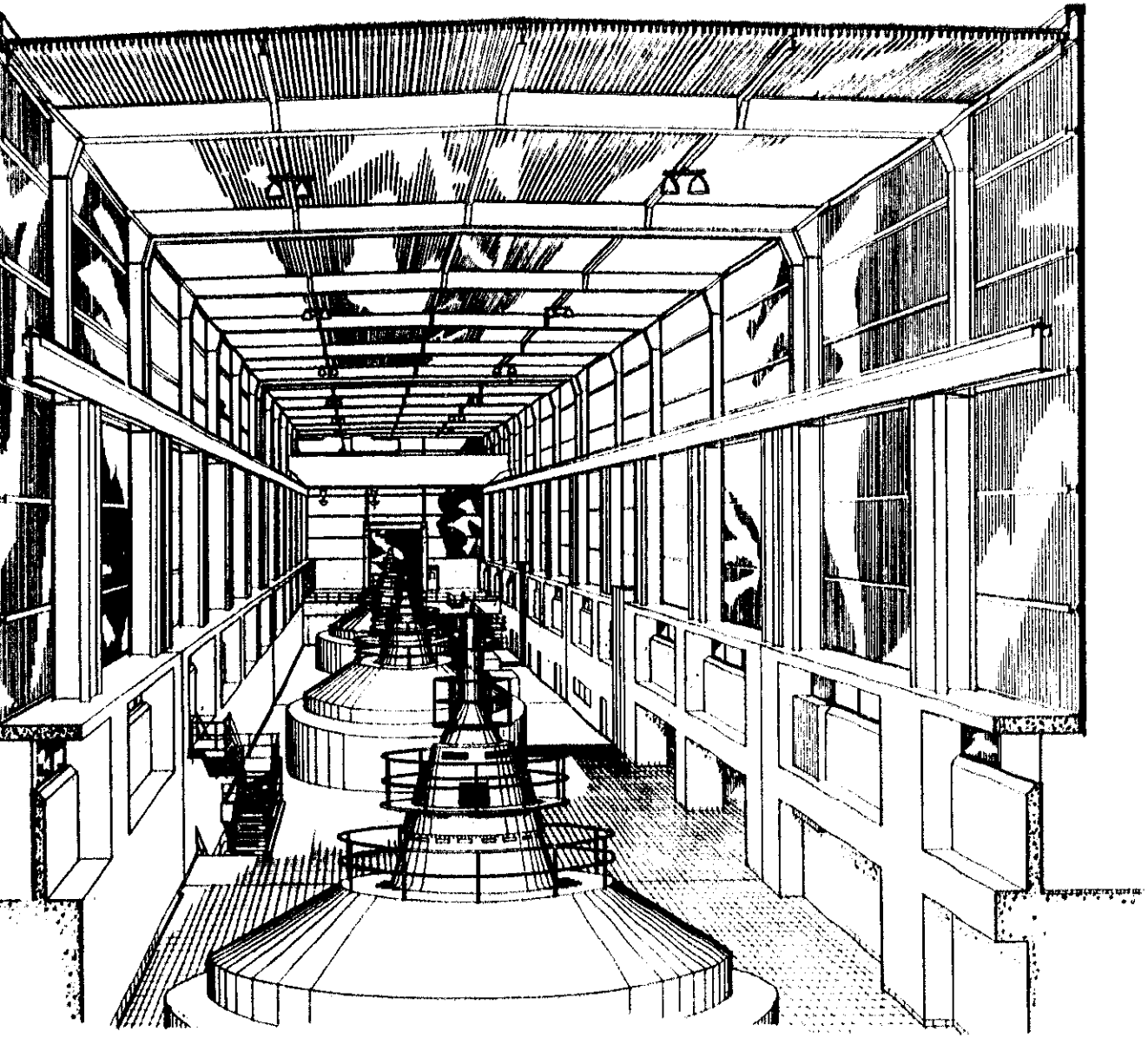

KARNAFULI
HYDROELECTRIC STATION



KARNAFULI POWERHOUSE
EXTERIOR SKETCH

MAY 1959

SKFL-42-225



KARNAFULI POWERHOUSE
INTERIOR SKETCH

SEPTEMBER 1958

SKFL-42-227

PART IV

CHAPTER II

POWERHOUSE

2.01 GENERAL

This chapter of the Design Report presents descriptions of the powerhouse architectural and finish treatment. Plans, elevations, and sections of the powerhouse are shown on Exhibits II-2 through II-8. Perspective sketches of both the interior and exterior of the powerhouse precede this page. (Figures II-A and II-B)

2.02 POWERHOUSE APPROACH

The Karnafuli Project powerhouse is approached via a curved surfaced road which terminates in a paved area in front of the powerhouse entrance. This southeast end elevation of the powerhouse is 73 feet wide and more than 60 feet high. In the center is a roll-up service door 18 feet wide and 22 feet high, with a cantilevered canopy to provide protection from rain and sun as well as to give emphasis and shadow to the elevation. The entrance for personnel is located to the left of the large service door in a recess which gives protection for the entrance without detracting from the massive scale of the structure.

Planting areas filled with hardy native shrubs are located on each side of the powerhouse entrance, to add color and interest to the elevation.

2.03 INTERIOR ARRANGEMENT

All visitors, as well as operating personnel, enter the powerhouse through the same entrance door to the erection deck at Elevation 49.0 feet. From this point the magnitude of the project and the massive scale of the structural steel frame, the generators and the bridge crane can be realized. The contrasting colors of the finished surfaces and the hum of the generators and turbines also add to the dramatic impression for the visitors.

The stairs leading to the generator floor are made of steel, with concrete treads and safety nosings. The clean-cut lines of the stairs are typical examples of the simplicity of design which is stressed throughout the powerhouse.

At the base of the stairs, on the generator floor, visitors can enter the air-conditioned reception room or office. Both of these rooms have acoustical ceilings to help reduce the sound from the generating equipment. Adjacent to these rooms are the toilets, with both eastern and western type fixtures. A refrigerated drinking fountain in the generator room is also easily accessible.

The control room, with its complex switchboards and other control equipment, is another area of special interest to the visitor. This room is also air-conditioned, and the wall between it and the generator room is furred to reduce sound transmission. From within this room the operator can view the generators and governor actuators through double-glazed panels.

Two stairs lead to the turbine room at Elevation 22.50 feet. The walls and ceilings of the turbine room are painted to improve the appearance and lighting in the area. A large quantity of mechanical and electrical equipment, cable trays and piping may be seen, giving this area an interesting appearance as well as functional value.

Galleries below the turbine floor are unpainted and are normally accessible to operating personnel only.

Egress from the building is normally through the door at the erection deck; however, two other exits are provided for emergency or service use.

2.04 SUPERSTRUCTURE DESIGN AND FINISHES

The superstructure of the Karnafuli powerhouse has been designed to fulfill field requirements as well as to present studied simplicity of materials and erection.

A. Concrete Base

The concrete base around the superstructure from Elevation 49.0 feet to Elevation 60.5 feet serves several functional purposes as follows:

1. It prevents water from entering the powerhouse interior, should tailwater ever reach high flood level between Elevation 49.0 feet and 55.0 feet.

2. Vent lines are concealed in the wall and turned out above Elevation 55.0 feet.

3. Air supply and exhaust ducts are supported by the wall with duct louvers worked into the wall design.

4. Electrical conduit and boxes are concealed in the concrete wall.

5. The concrete is of structural value to the steel frame of the superstructure.

The windows, the large air supply louvers and the electrical bus ducts are all worked into the wall design. The exterior faces of the concrete columns and the concrete walls at the ends of the superstructure are faced with ceramic tile, to make a permanently colored finish requiring minimum maintenance. The exterior faces of the panel walls between the concrete columns are faced with stucco, to provide a contrast of color and texture.

Since a mechanical ventilation system is provided for the powerhouse, the windows are made of fixed steel frames without opening vents, except for the windows above the erection area, where hopper vents may be opened by using a short pole. The concrete soffits above the windows shade the window openings from the heat of the sun.

B. Aluminum Wall

Above the concrete base, the superstructure is housed in an uninterrupted surface of fluted V-beam .040 aluminum with a green finish. This material offers an interesting texture of alternating vertical lines of sun and shade. Its green color is shop-applied baked enamel. The finish is resistant to corrosion and stains, and it reduces the reflectivity of the aluminum. The interior of the walls is lined with corrugated aluminum, with the corrugations running in the vertical direction. This type of aluminum sandwich wall, with a center core of one inch of glass fiber insulation, has a "U" value (coefficient of heat transfer) of .155. The lightweight materials are shipped separately and the walls are fabricated in the field, using either weldable studs with aluminum caps or composite aluminum self-tapping screws with neoprene washers. This type of exterior wall was selected because of its outstanding appearance, moderate cost, light weight, ease of erection, speed of erection, insulation value, permanent finish and weather tightness. The total unit weight of the exterior sandwich wall is about two pounds per square foot.

C. Roof Construction

The roof of the powerhouse consists of steel panels covered with two inches of rigid insulation and a 5-ply built-up asphalt and gravel roof. Steel roof decking was selected because it provides an interesting ceiling surface, it can be rapidly installed without shoring, and it is a dry application which does not interfere with erection operations within the structure. The steel decking was estimated to be less costly than a concrete roof slab.

2.05 INTERIOR MATERIALS AND FINISHES

The materials to be used inside the powerhouse were selected for their permanence, appearance, fire resistance, and cost values.

A. Partitions

Good quality brick, manufactured in East Pakistan, was used in construction of partitions. These brick partitions were plastered on both sides as is customary in local practice. Oil Room partitions are of reinforced concrete construction for greater fire resistance.

Where furring was required, steel furring channels and 3.4-pound per square yard metal lath were specified. Where plaster is indicated on the finish schedule, three coats were used on metal lath and two coats on brick surfaces.

The toilet partitions and doors around the western type water-closets are the flush baked-enamel insulated metal type with chrome-plated fittings. The partitions around the eastern type fixtures extend to the floor, and are made of ceramic tile on brick. The fronts of these stalls are made of metal panels and doors.

B. Doors

The hinged doors throughout the powerhouse are a flush hollow metal type with hollow metal frames and good quality hardware. Standard sliding fire doors are provided for the oil rooms. These doors are a type approved by the National Board of Fire Underwriters. The large metal roll-up door at Elevation 49.0 feet is of standard design with hand-chain operation.

C. Floor Materials

As indicated on the Finish Schedule on Exhibit II-1, four types of floor finish are used. Concrete floors have a steel-trowelled finish and a brush-applied concrete hardener to seal the surface and prevent dusting. Quarry tile was selected for the generator floor and erection deck. This material provides a dense surface which is impervious to greasy substances, and easy to keep clean. Its permanent color is also a desirable feature. Quarry tile was also used for a base, on curbs and for lining drainage trenches. Ceramic tile was selected for the toilet room floors and walls. This material was selected because it has an attractive appearance, it is easily cleaned, and its hard surface is not easily marked or damaged. Vinyl-plastic-asbestos tile is used in the reception room, office and control rooms. This 1/8-inch thick material is resistant to greases, alkalis and dilute acids. Its color and sheen are desirable in these rooms. A four-inch high rubber cove base was applied in these rooms to simplify floor cleaning and to stop the wall paint.

D. Ceiling Materials

In rooms where suspended ceilings are required, two types were used. The toilet rooms have plaster suspended on metal channels and metal lath. In other finished areas, metal acoustical tile was selected because of its acoustical values, its shop-applied baked-on white finish, and the ease with which panels may be removed for access to conduit and piping. These metal tiles, with inserted pads, are suspended on No. 8 wires, steel channels and T-bars of special design.

E. Concrete Finishes

Formed concrete surfaces which are exposed, have had all fins removed and holes filled, leaving reasonably smooth and uniform surfaces. Concrete floors in the powerhouse have steel-trowelled finishes on all exposed areas.

The erection area structural floor at Elevation 49.0 feet and all floors on Elevation 36.0 feet were depressed 3 inches below the finished floor elevation. The 3-inch depression provides ample depth for the application of tile where shown on the finish schedule. In other areas the depression provides sufficient depth for placing a uniform well-trowelled concrete finish after the rough construction work is completed.

The transformer deck at Elevation 49.0 feet has a 3-inch concrete topping which is reinforced with steel mesh. Under this topping, a 3-ply asphalt and felt membrane was installed above the control bay rooms at Elevation 36.0 feet. This membrane and concrete topping extends down in the transformer pits to protect them from rain or from oil spillage.

F. Painting

Painting procedures and color selections were carefully analyzed in order to select proper painting systems and to present a cheerful and safe atmosphere in which to work.

All surfaces not having permanent factory finishes were painted as shown on the Finish Schedule. Concrete, plaster and stucco were given a prime coat and two finish coats of paint as required for good quality work. Ferrous metals were given a three-coat paint treatment, whether they are in finished areas or not. Ferrous metals which are submerged in water were given special paint treatment as required in good practice.

PART IV - CHAPTER II

LIST OF EXHIBITS

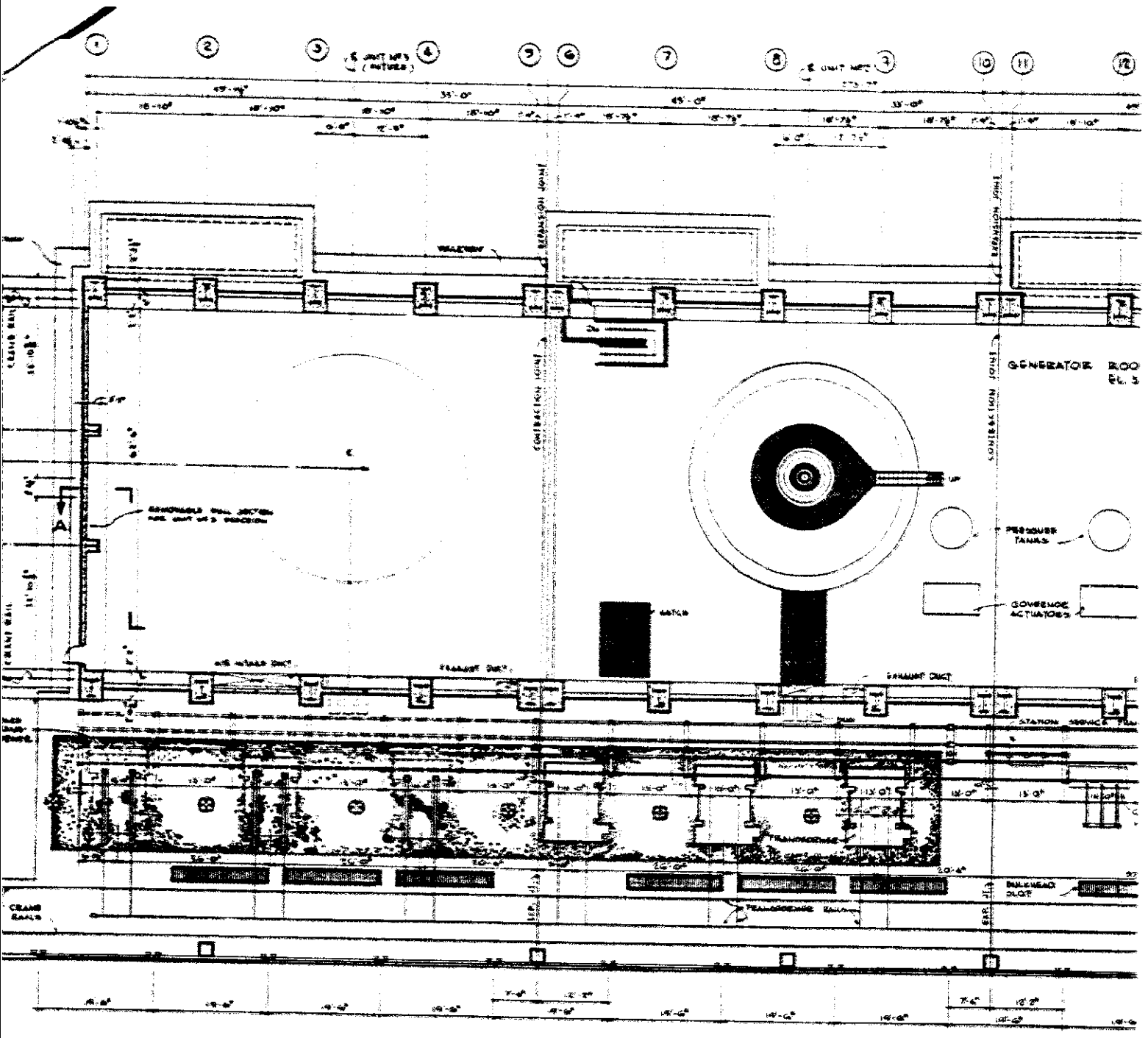
Exhibit	Title
II-1	Interior Finish Schedule
II-2	Powerhouse - Floor Plan, Elevation 49.00
II-3	Powerhouse - Floor Plan, Elevation 36.00
II-4	Powerhouse - Floor Plan, Elevation 22.50
II-5	Powerhouse - Downstream Elevation
II-6	Powerhouse - Elevations
II-7	Powerhouse - Longitudinal Sections
II-8	Powerhouse - Transverse Section

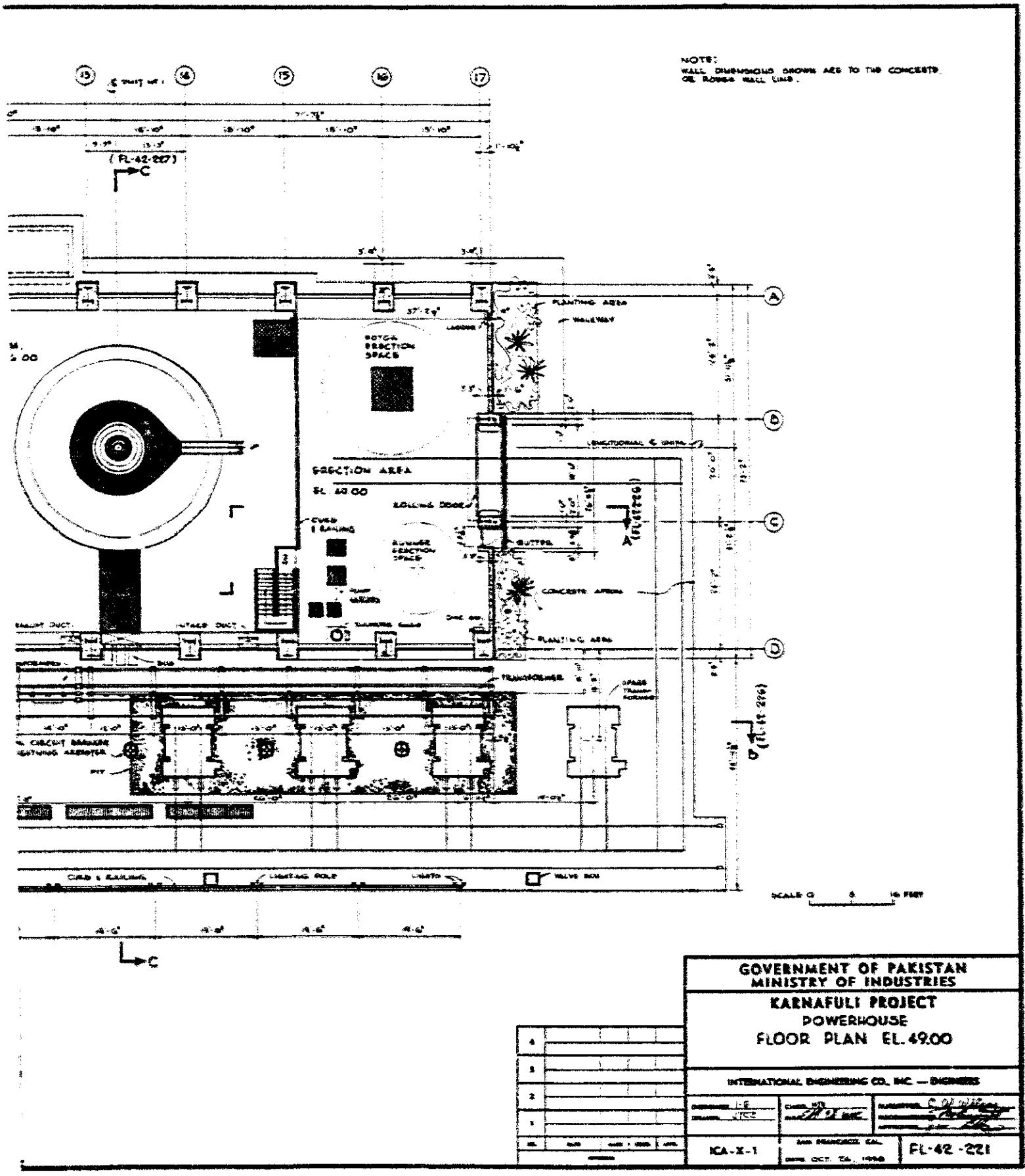
KARNAFULI POWERHOUSE: INTERIOR FINISH SCHEDULE

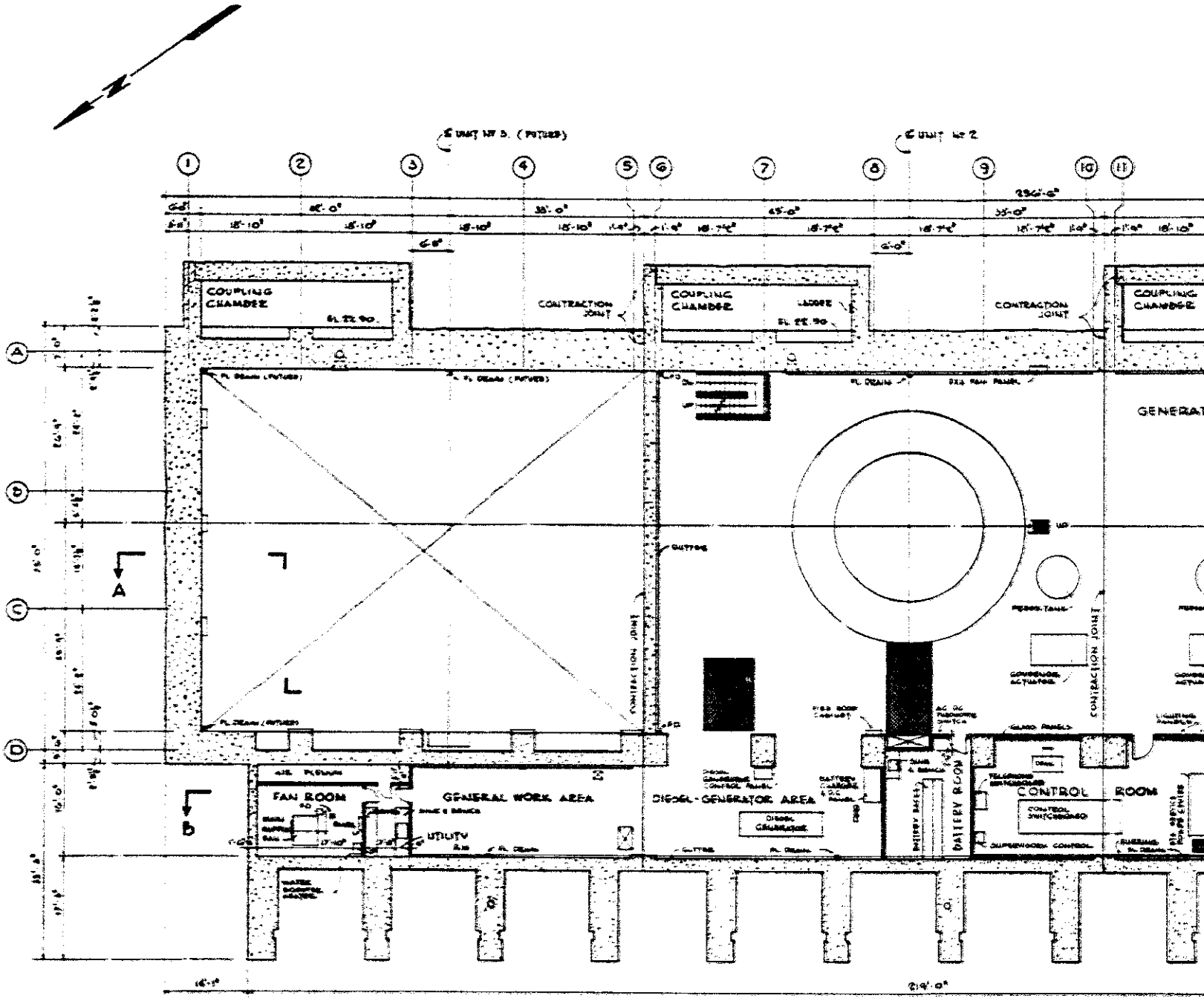
OMS & AREAS	FLOOR		BASE		WALL		CEILING		
ESIGNATION	MATERIAL	FINISH	MATERIAL	FINISH	MATERIAL	FINISH	MATERIAL	FINISH	SUSPEND CLG HGT
RAINAGE SUMP	CONC	HARDENER	—	—	CONC	—	CONC	—	—
WATERING SUMP	CONC	HARDENER	—	—	CONC	—	CONC	—	—
OUPLING CHAMBER	CONC	HARDENER	—	—	CONC	—	CONC	—	—
OPECTION GALLERY	CONC	HARDENER	—	—	CONC	—	CONC	—	—
L STORAGE ROOM	CONC	HARDENER	—	—	CONC	—	CONC	—	—
L PURIFICATION ROOM	CONC	HARDENER	—	—	CONC	—	CONC	—	—
ABLE SPREADING AREA	CONC	HARDENER	RUBBER	—	CONC; PLASTER	PAINT	CONC	PAINT	—
ANITOR'S ROOM	CONC	HARDENER	RUBBER	—	CONC; PLASTER	—	CONC	—	—
TORAGE ROOM	CONC	HARDENER	RUBBER	—	CONC; PLASTER	—	CONC	—	—
UMP AREA	CONC	HARDENER	—	—	CONC	PAINT	CONC	PAINT	—
RBINE ROOM	CONC	HARDENER	RUBBER	—	CONC; PLASTER	PAINT	CONC	PAINT	—
N ROOM	CONC TOPPING	—	—	—	CONC; PLASTER	—	CONC	—	—
ILITY ROOM	CONC TOPPING	—	RUBBER	—	CONC; PLASTER	PAINT	CONC	PAINT	—
ENERAL WORK AREA	QUARRY TILE	FACTORY	QUARRY TILE	FACTORY	CONC; PLASTER	PAINT	CONC	PAINT	—
ESEL-GENERATOR AREA	QUARRY TILE	FACTORY	QUARRY TILE	FACTORY	CONC; PLASTER	PAINT	CONC	PAINT	—
TTERY ROOM	CONC TOPPING	—	—	—	CONC; PLASTER	—	CONC	—	—
ONTROL ROOM	VINYL TILE	WAX	RUBBER	—	PLASTER *	PAINT	MST ACOUST TILE	FACTORY	8'-4" & 9'-4"
OILET, EASTERN TYPE	CERAMIC TILE	FACTORY	CERAMIC TILE	FACTORY	CERAMIC TILE	FACTORY	PLASTER	—	8'-0"
OILET, WESTERN TYPE	CERAMIC TILE	FACTORY	CERAMIC TILE	FACTORY	CERAMIC TILE	FACTORY	PLASTER	—	8'-0"
FICE	VINYL TILE	WAX	RUBBER	—	PLASTER	PAINT	MST ACOUST TILE	FACTORY	8'-4"
CEPTION	VINYL TILE	WAX	RUBBER	—	PLASTER	PAINT	MST ACOUST TILE	FACTORY	8'-4"
ECORDS ROOM	CONC TOPPING	—	—	—	CONC; PLASTER	—	CONC	—	—
CHINE SHOP	QUARRY TILE	FACTORY	QUARRY TILE	FACTORY	CONC; PLASTER	PAINT	CONC	PAINT	—
OL ROOM	CONC TOPPING	—	RUBBER	—	CONC; PLASTER	PAINT	CONC	PAINT	—
ENERATOR ROOM	QUARRY TILE	FACTORY	QUARRY TILE	FACTORY	CONC; PLASTER; ALUM SIDING	PAINT MASONRY & ALL STEEL	STEEL DECK	PAINT	—
ECTION AREA	QUARRY TILE	FACTORY	QUARRY TILE	FACTORY	CONC; ALUM SIDING	PAINT MASONRY & ALL STEEL	STEEL DECK	PAINT	—

TE: 1. OMIT RUBBER BASE AT WALLS PROVIDED WITH CONCRETE GUTTER.
* VINYL WAINSCOT AT CONTROL ROOM DESK.

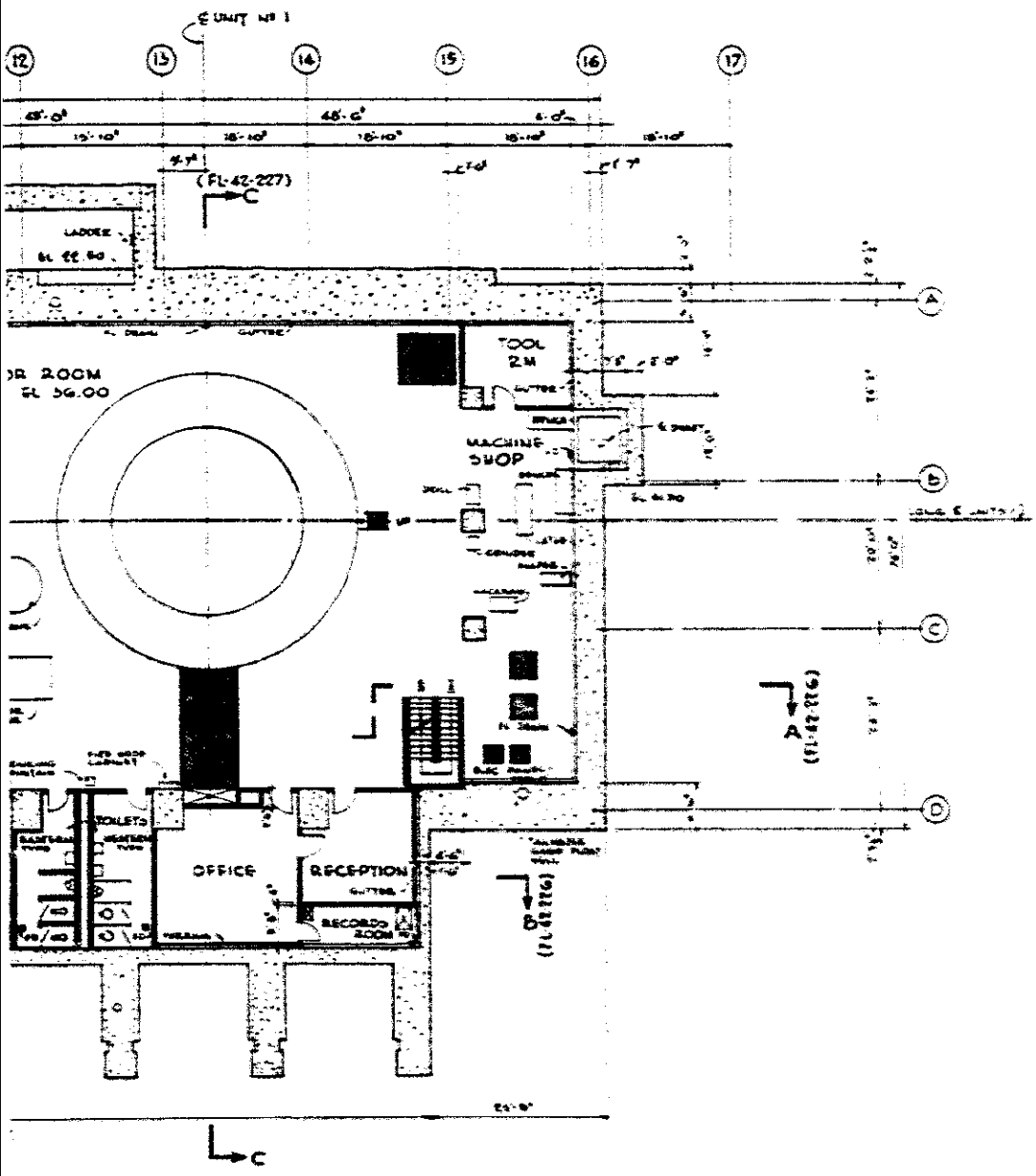
JAN. 8, 1959


EXPANSION JOINT
CONTRACTION JOINT
GENERATOR
PRESSURE TANKS
GOVERNOR ACTUATORS
STATION
RAILROAD SLOT
TRANSFORMER RAILS
CRANE RAILS


NOTE:
WALL DIMENSIONS SHOWN ARE TO THE CONCRETE OR ROUGH WALL LINE.
ERECTION AREA
EL. 49.00
PLANTING AREA
WALKWAY
CONCRETE APRON
TRANSFORMER
VALVE BOX
LIGHTING POLE
SCALE 0 5 10 FEET
GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
POWERHOUSE
FLOOR PLAN EL. 49.00
INTERNATIONAL ENGINEERING CO. INC. — ENGINEERS
KA-X-1
FL-42-221


COUPLING CHAMBER
CONTRACTION JOINT
COUPLING CHAMBER
LADDER
CONTRACTION JOINT
COUPLING CHAMBER
GENERAT
FAN ROOM
GENERAL WORK AREA
UTILITY
DIESEL-GENERATOR AREA
BATTERY ROOM
CONTROL ROOM
CONTRACTION JOINT
CONTRACTION JOINT

NOTE
WALL DIMENSIONS SHOWN ARE TO THE CONCRETE, OR ROUGH WALL LINE.



SCALE 0 4 16 FEET

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

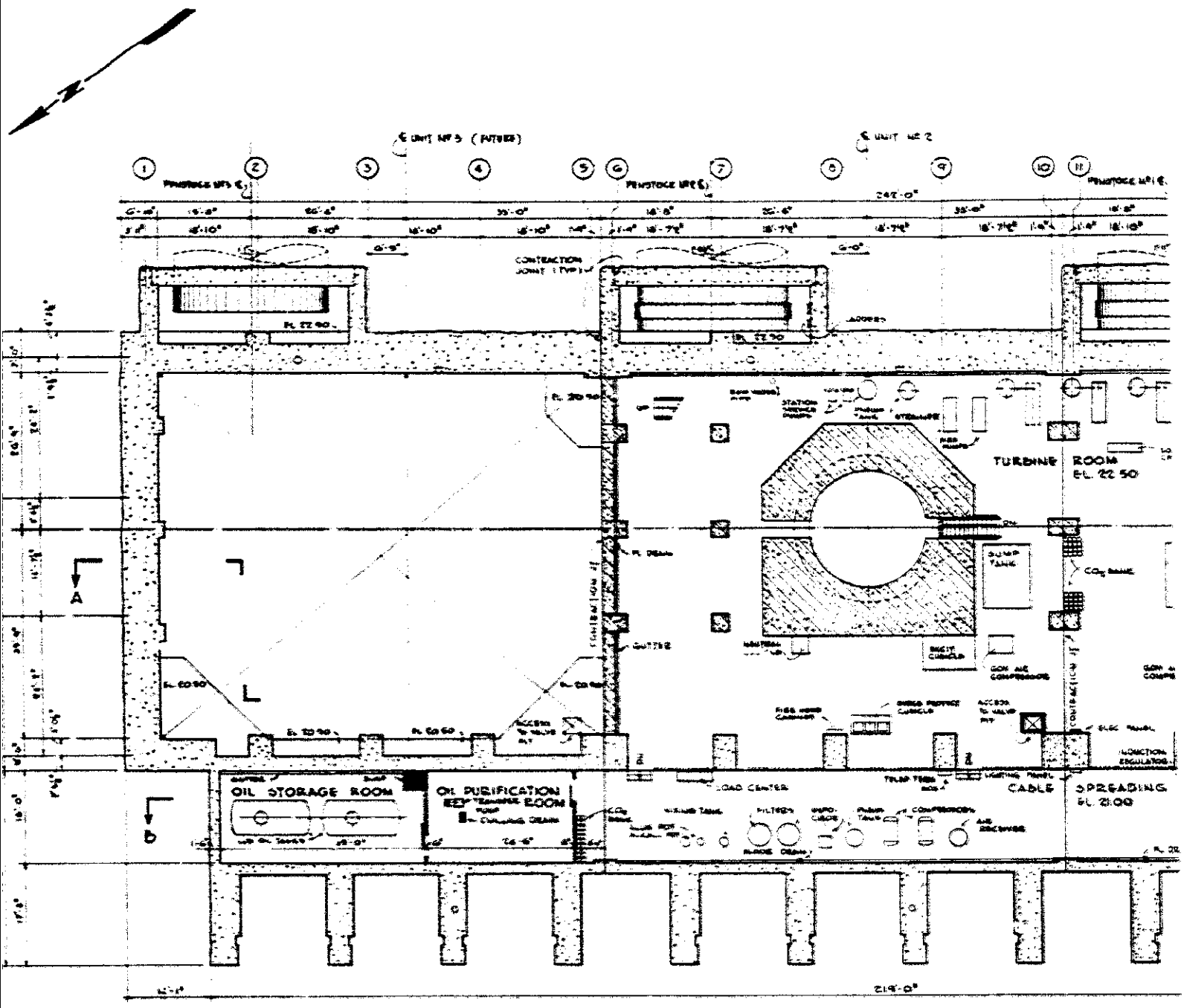
KARNAFULI PROJECT

POWERHOUSE
FLOOR PLAN EL. 3600

INTERNATIONAL ENGINEERING CO. INC. - ENGINEERS

KA.-X-1	SAN FRANCISCO, CAL. DATE OCT 24, 1956	FL-42-222

REVISIONS


UNIT Nº 5 (FUTURE)
UNIT Nº 2
PENSTOCK Nº 5
PENSTOCK Nº 6
PENSTOCK Nº 6
242'-0"
CONTRACTION JOINT (TYP.)
EL. 22.50
EL. 22.50
TURBINE ROOM
EL. 22.50
CONTRACTION JT.
CONTRACTION JT.
A
b
OIL STORAGE ROOM
OIL PURIFICATION ROOM
LOAD CENTER
CABLE SPREADING
EL. 21.00
219'-0"

NOTE:
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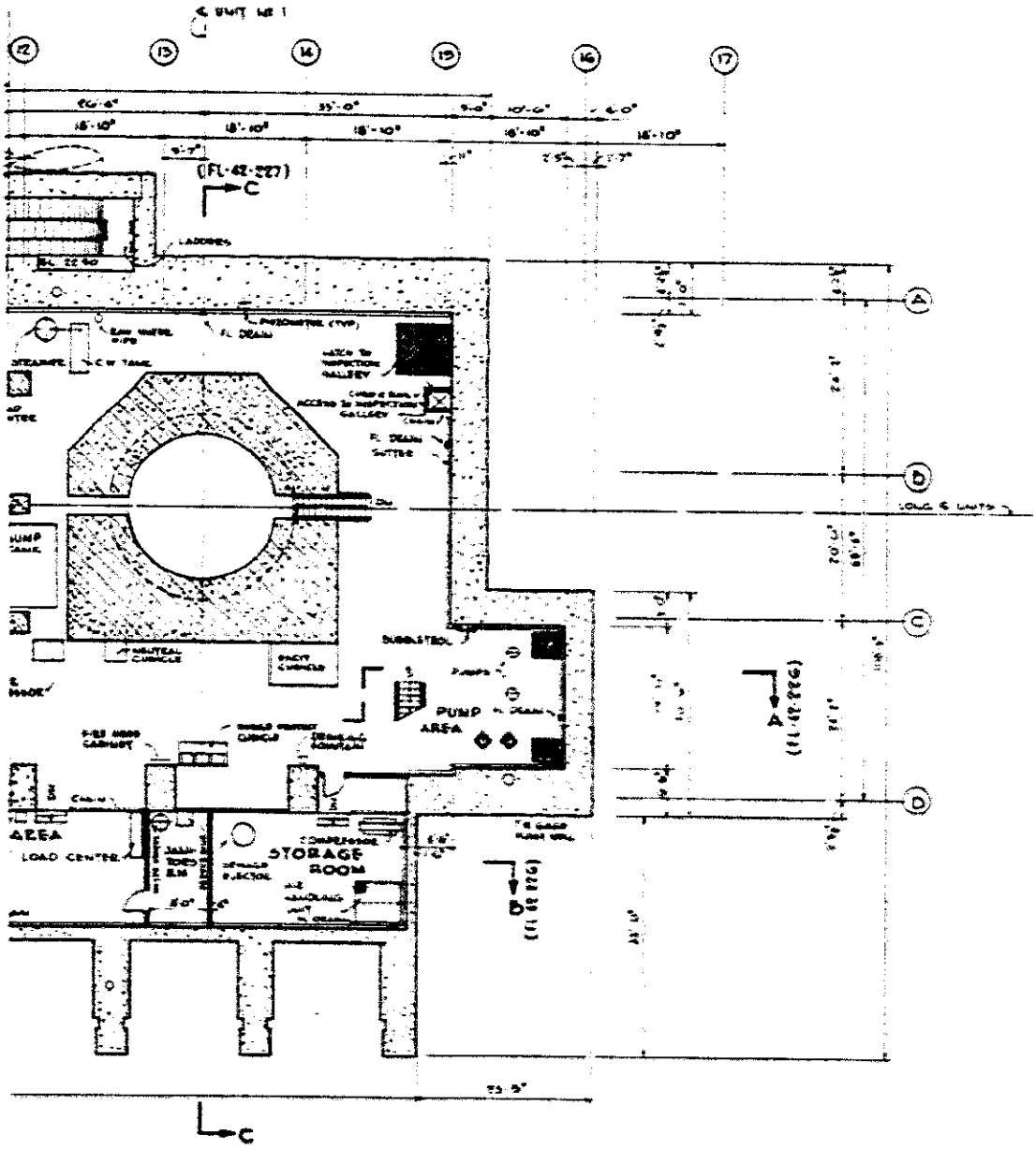
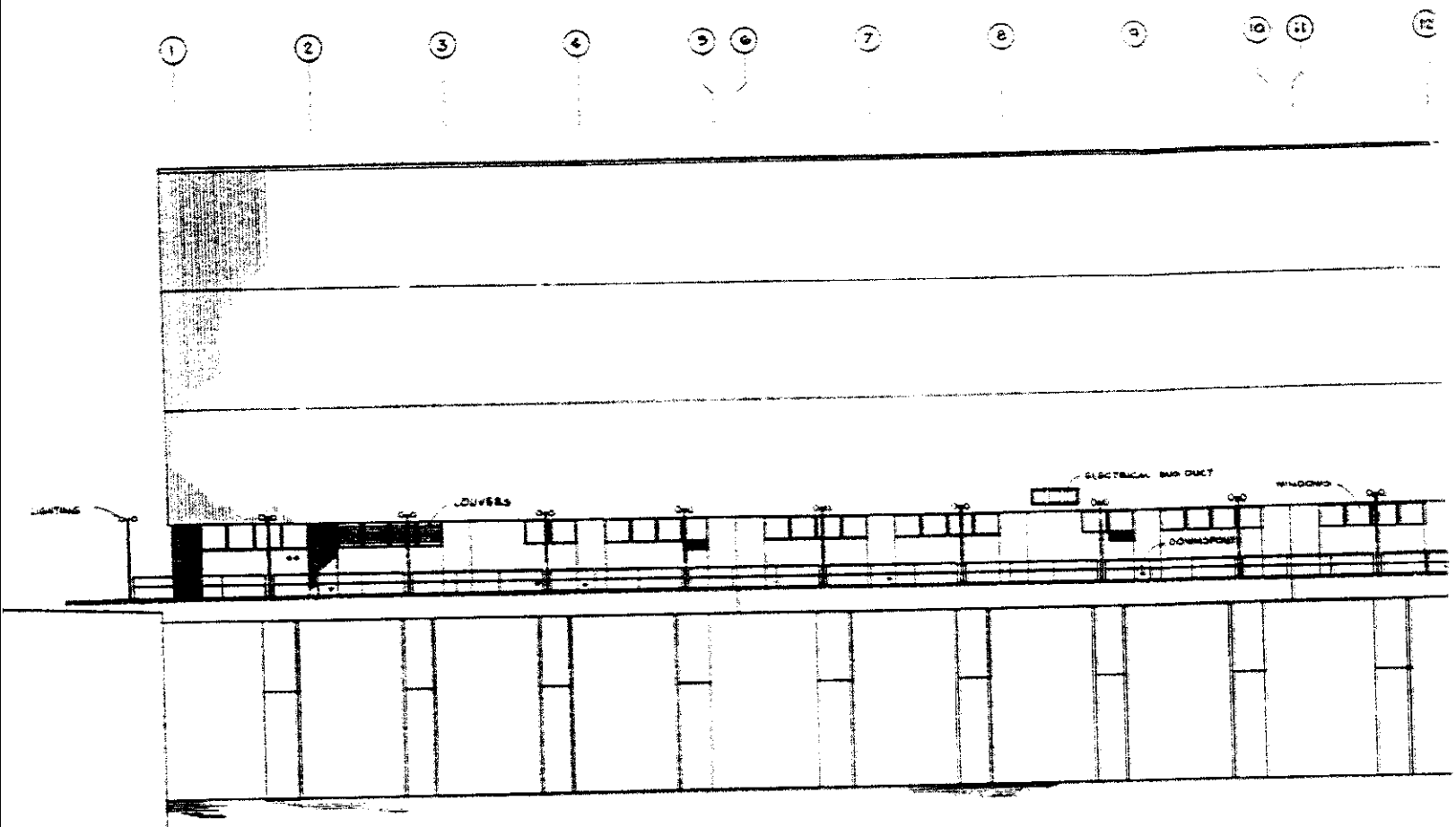
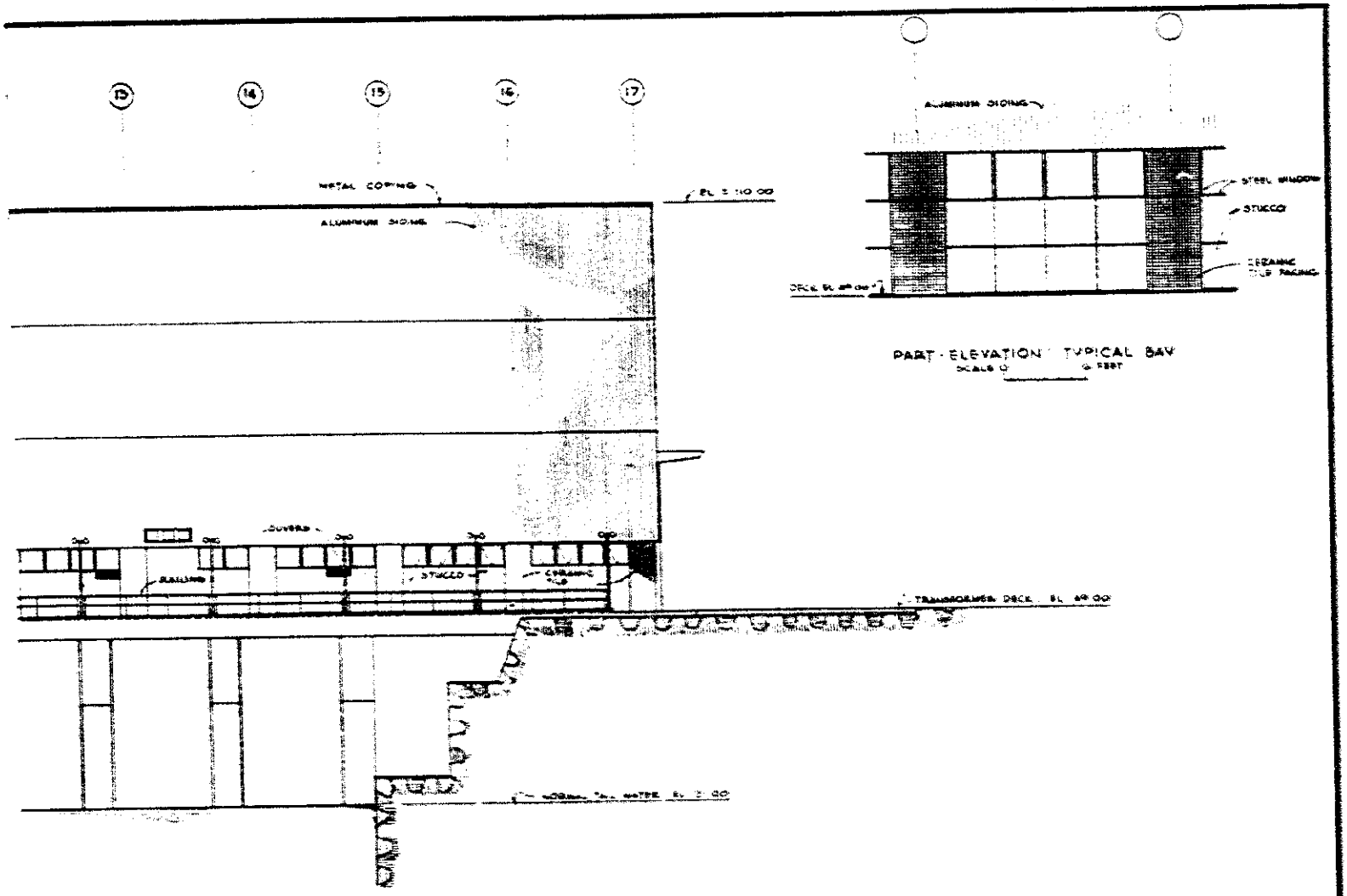


GOVERNMENT OF PAKISTAN MINISTRY OF INDUSTRIES	
KARNAFULI PROJECT POWERHOUSE FLOOR PLAN EL. 22.50	
INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS	
KA-X-1	SAN FRANCISCO, CAL. DATE OCT. 24, 1958 — FL-42-223



DOWNSTREAM ELEVATION



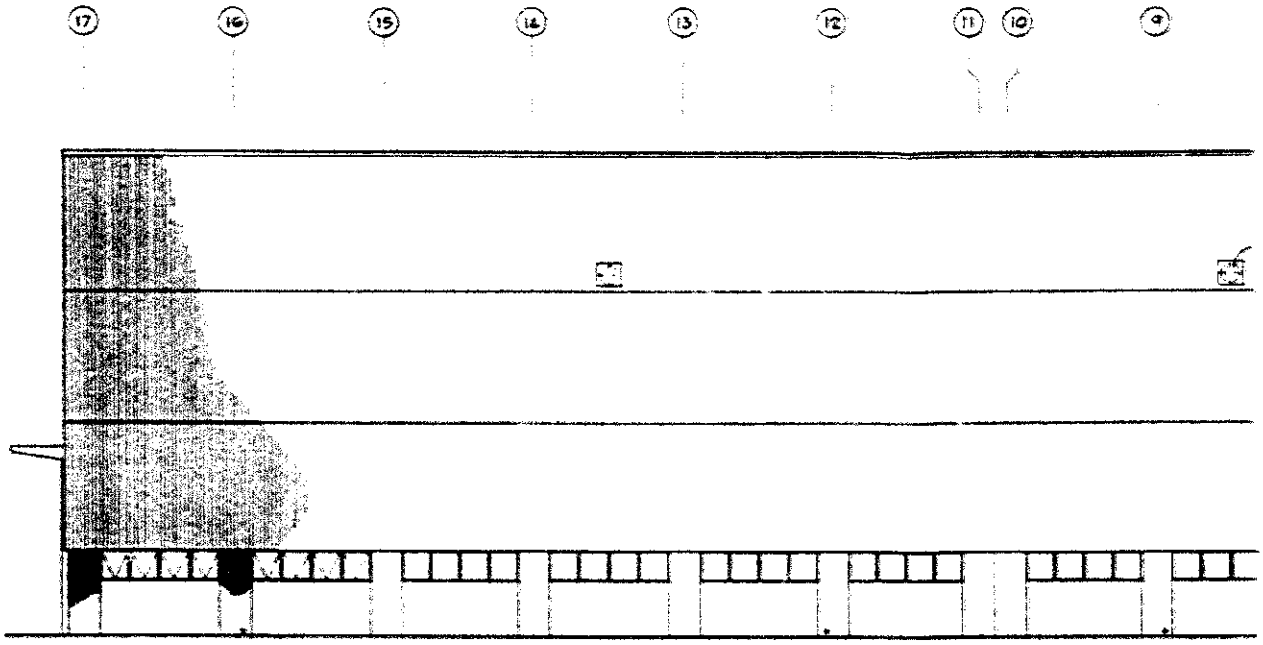
GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

KARNAFULI PROJECT
POWERHOUSE
DOWNSTREAM ELEVATION

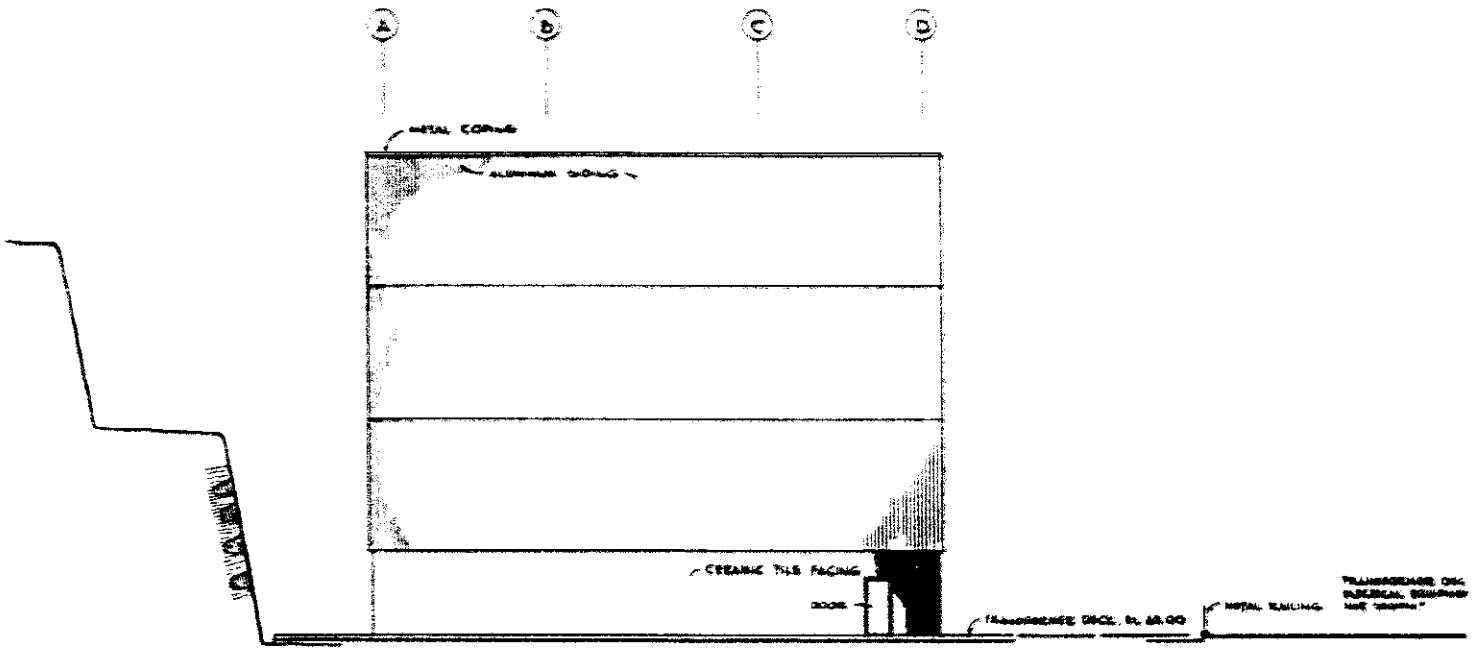
INTERNATIONAL ENGINEERING CO. INC — ENGINEERS

DESIGNED J.E.	CHKD.	SUBMITTED	
DRAWN J.T.C.		RECOMMENDED	
		APPROVED	

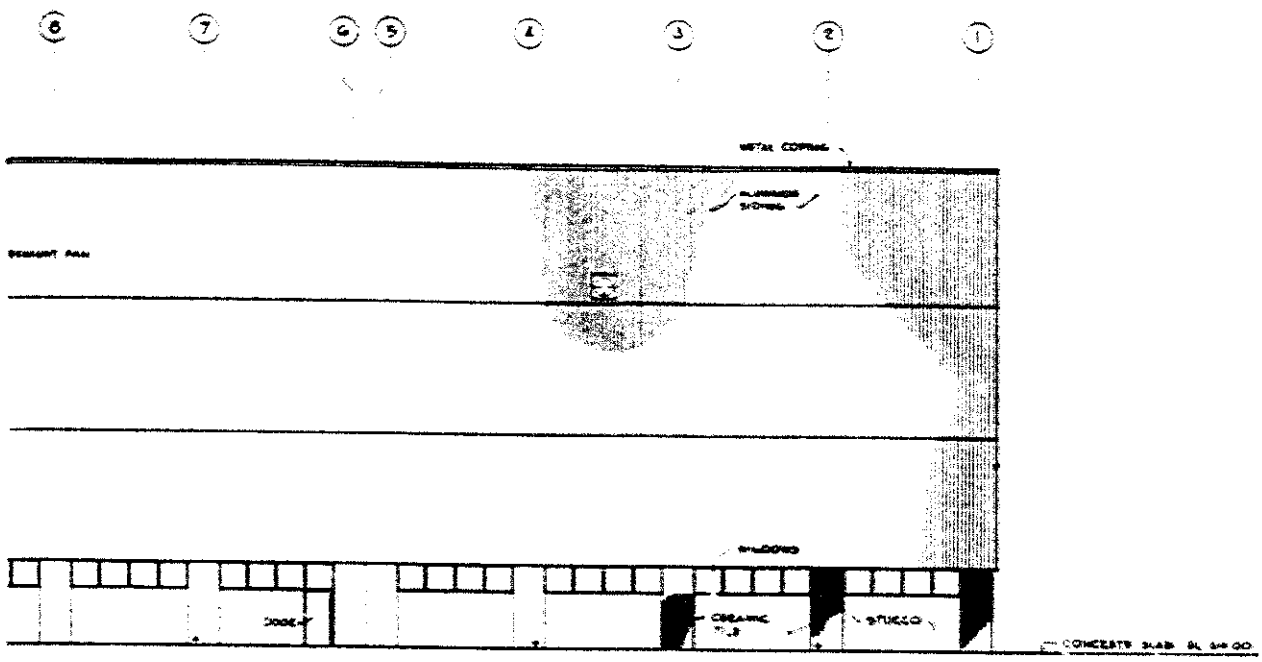
ICA-X-1	SAN FRANCISCO, CAL. DATE OCT. 24, 1962	FL-42-224	

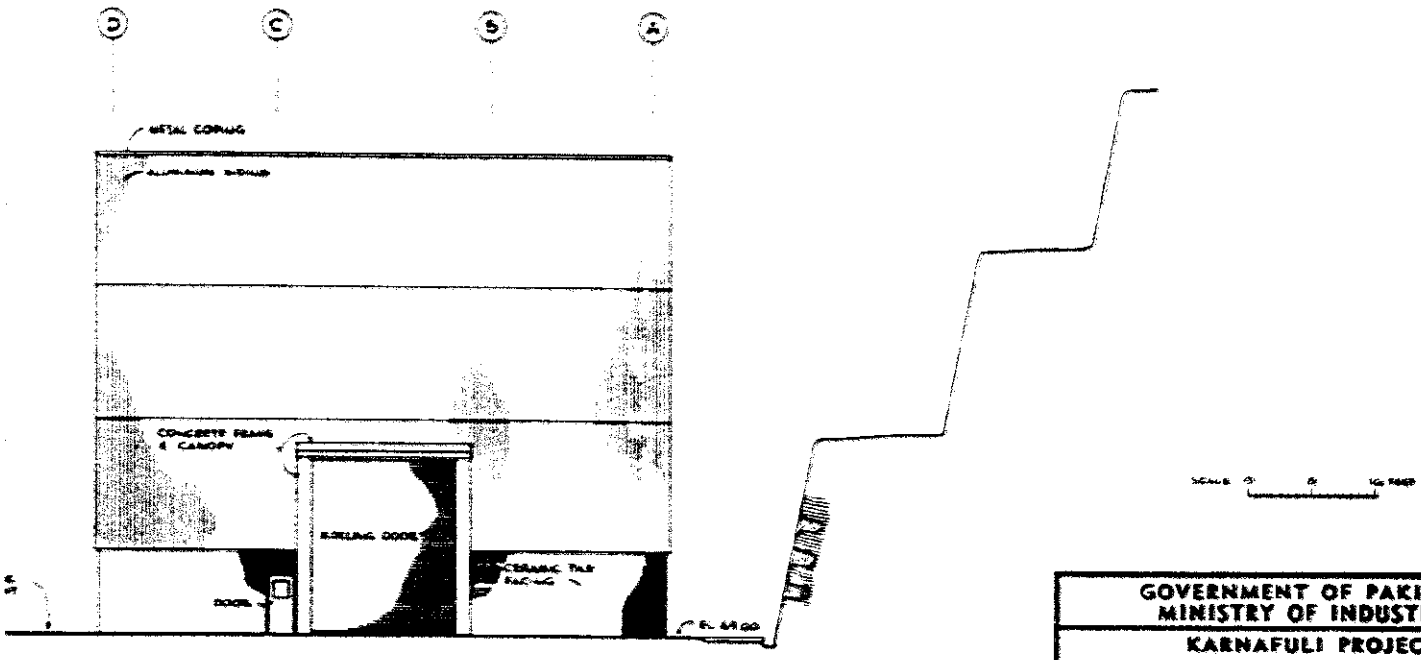


UPSTREAM ELEVATION



NORTH END ELEVATION





SOUTH END ELEVATION

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

KARNAFULI PROJECT

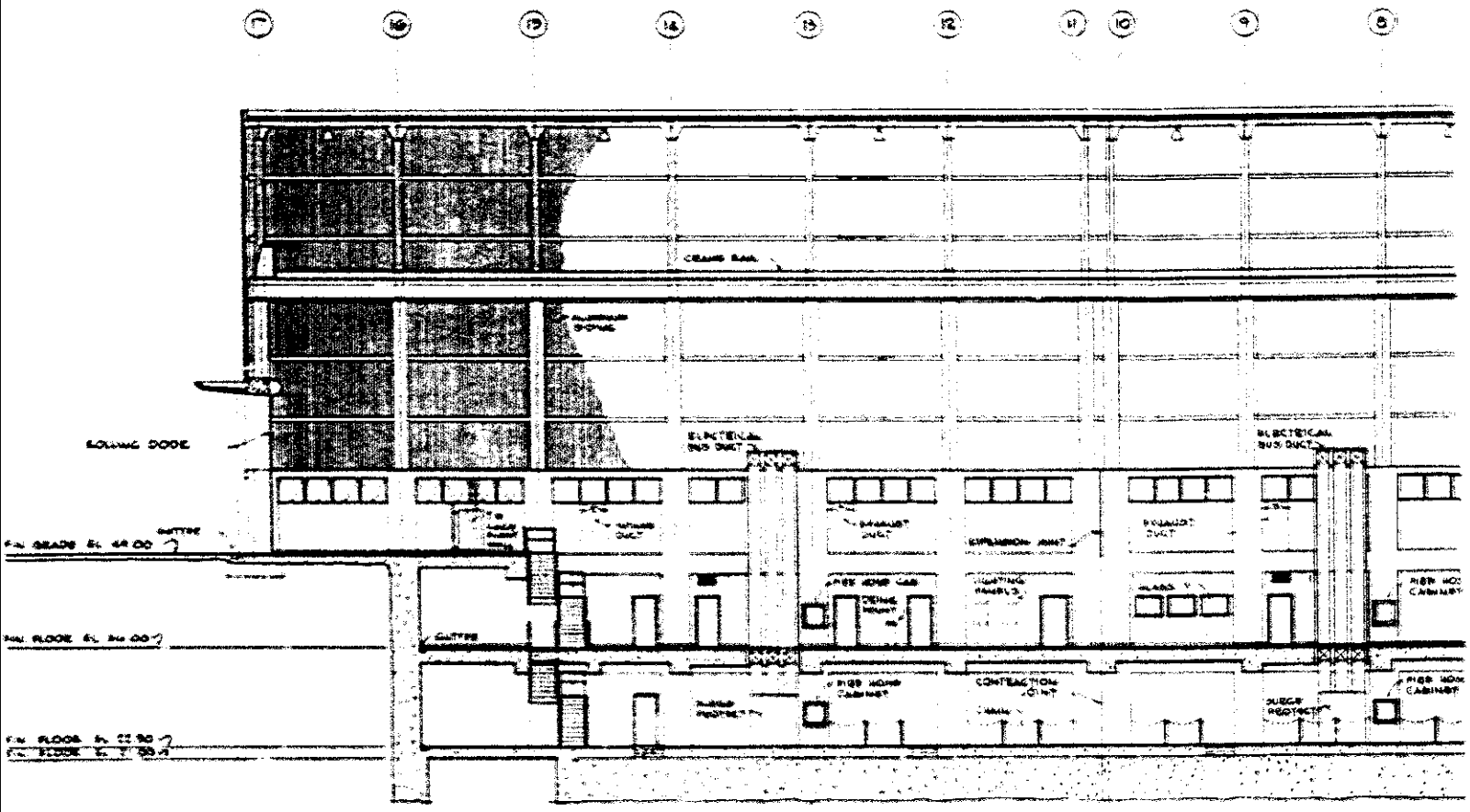
POWERHOUSE
ELEVATIONS

INTERNATIONAL ENGINEERING CO., INC. — ENGINEERS

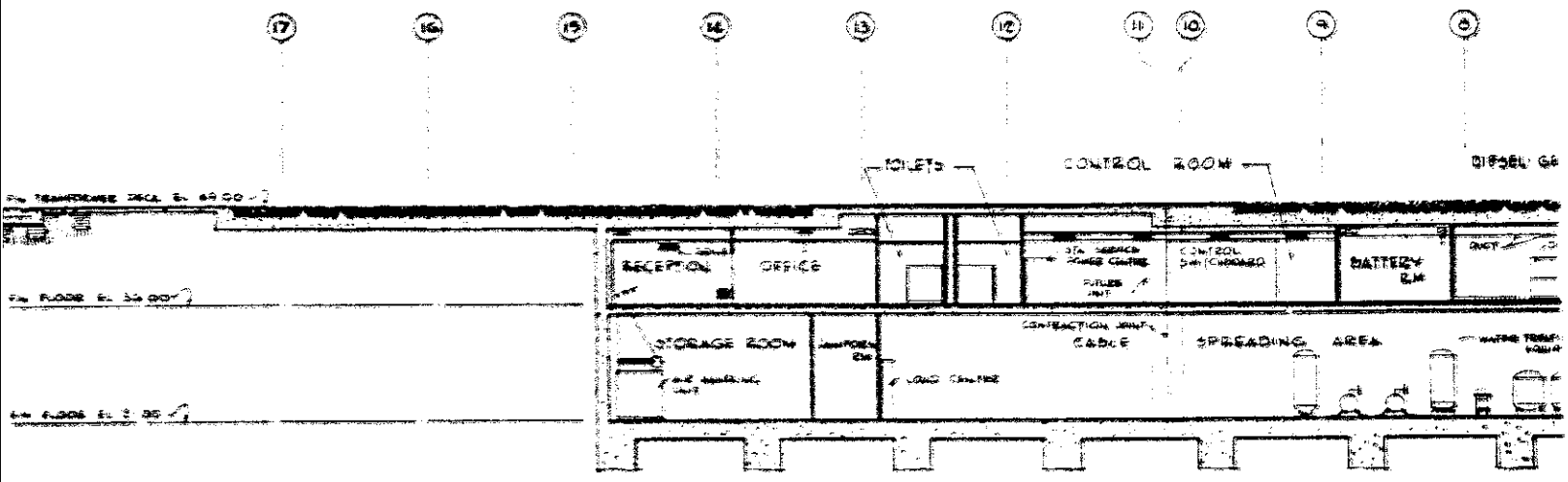
KA-X-1

DATE OCT. 26, 1956

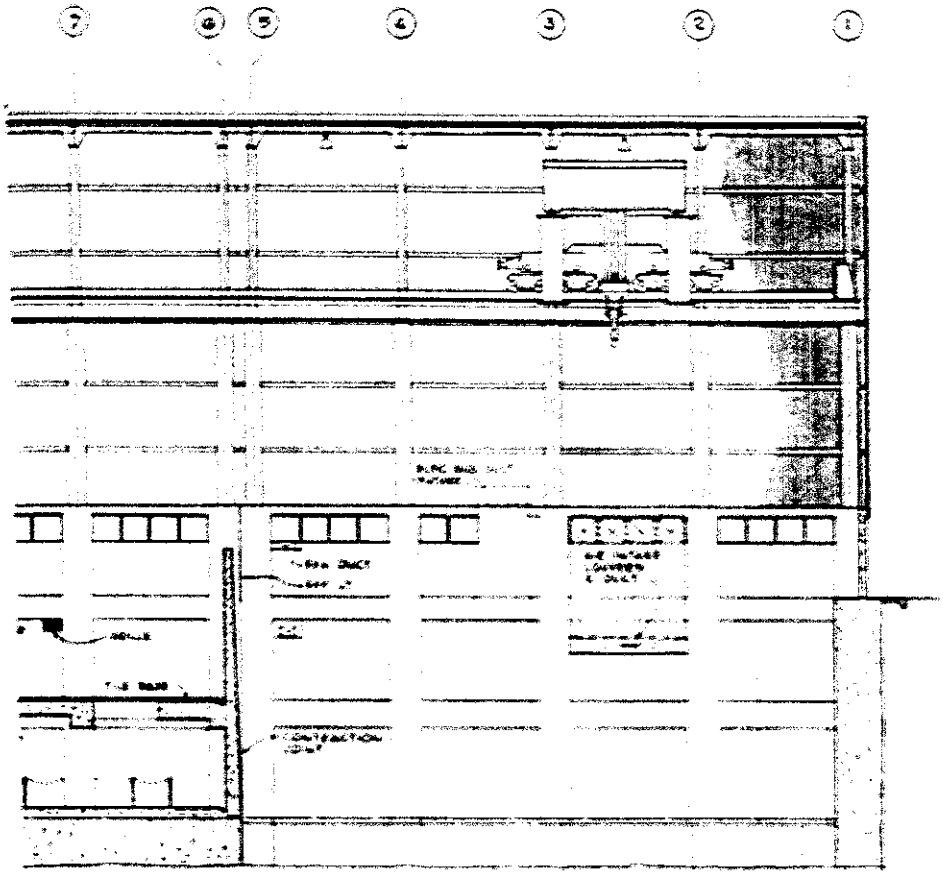
FL-42-225

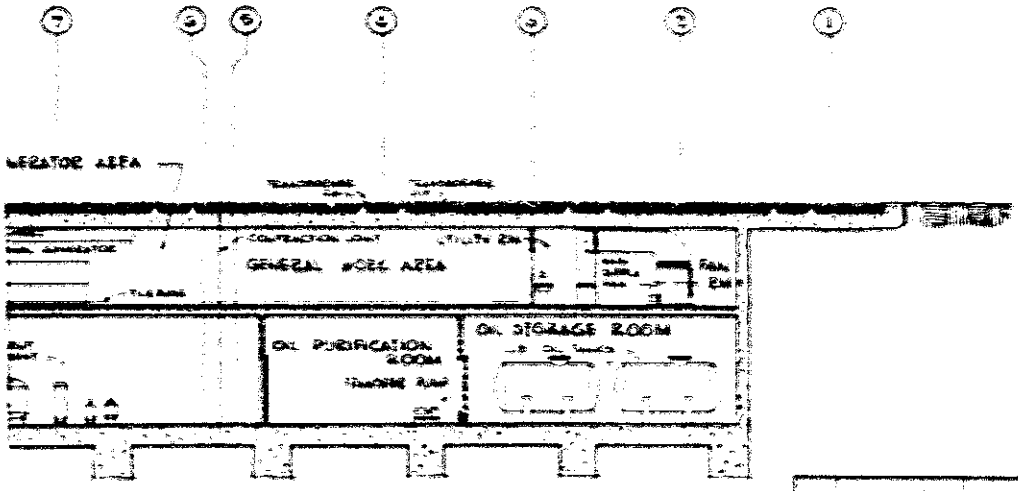


SECTION A-A



SECTION b - b





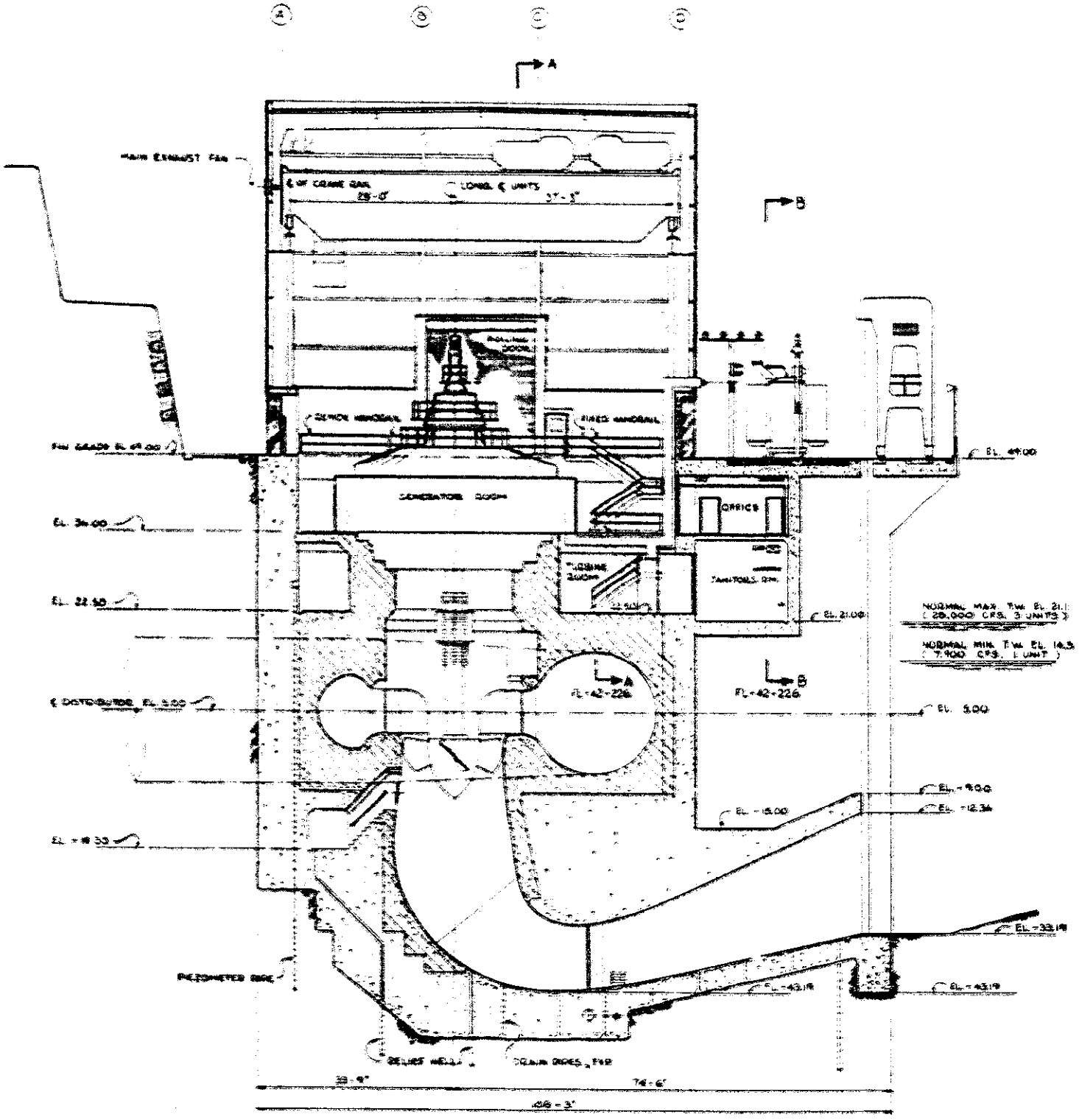
GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

KARNAFULI PROJECT
POWERHOUSE
LONGITUDINAL SECTIONS

INTERNATIONAL ENGINEERING CO., INC. — ENGINEERS

ICA-2-1

FL-42-226



TRANSVERSE SECTION C-C

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

KARNAFULI PROJECT
POWERHOUSE
TRANSVERSE SECTION

INTERNATIONAL ENGINEERING CO., INC. — ENGINEERS

ICA-X-1

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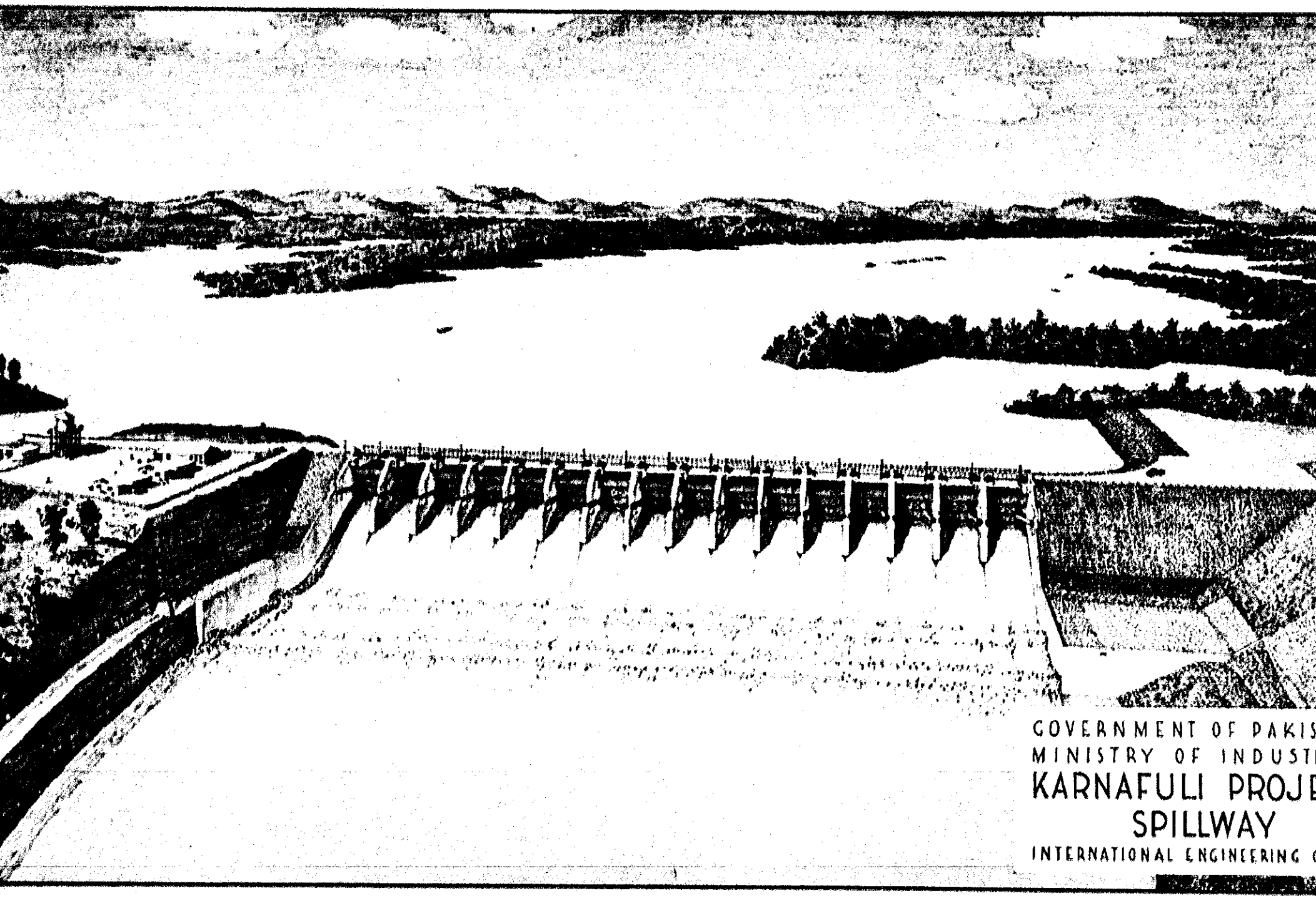
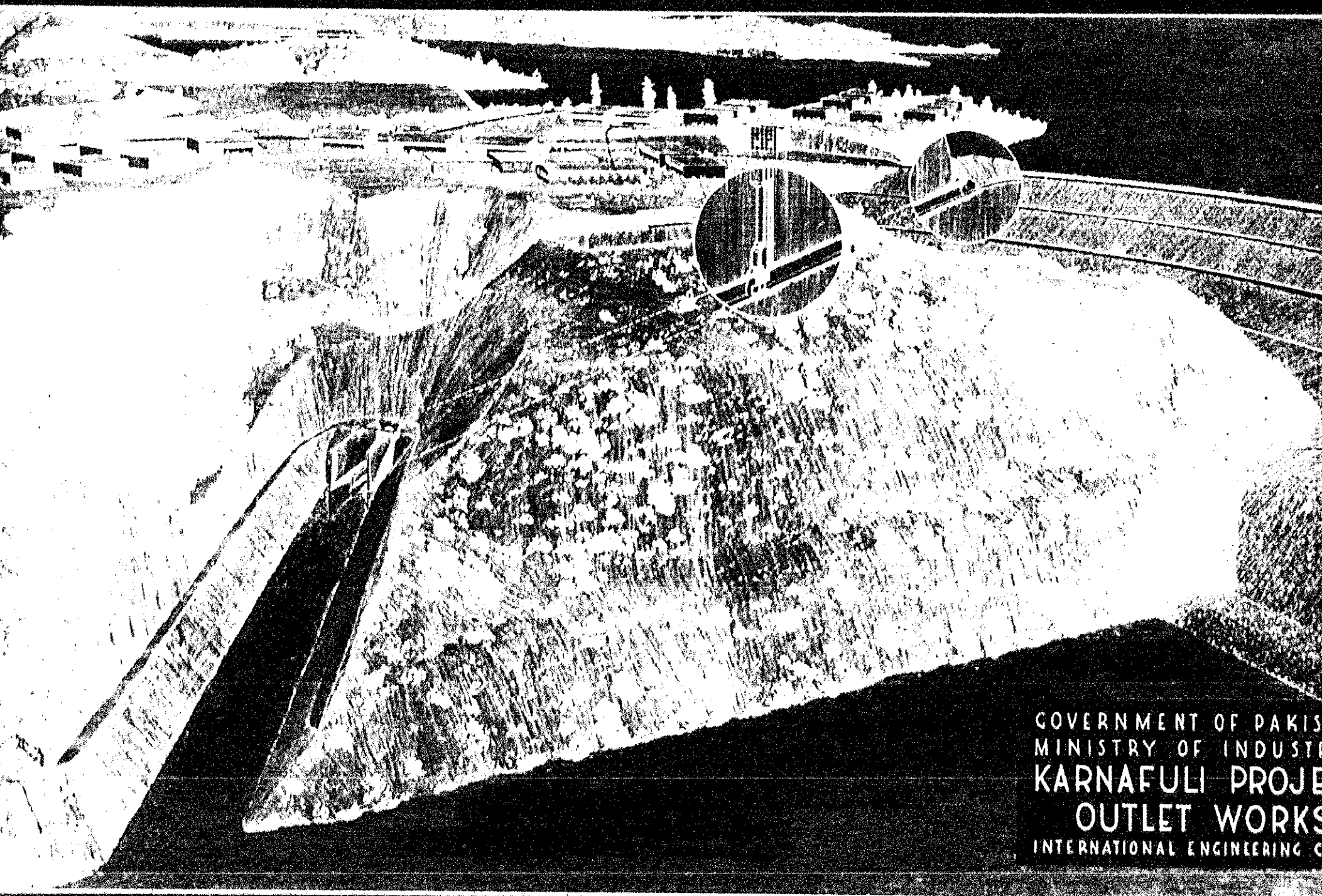

GOVERNMENT OF PAKIS
MINISTRY OF INDUSTI
KARNAFULI PROJE
SPILLWAY
INTERNATIONAL ENGINEERING


GOVERNMENT OF PAKIS
MINISTRY OF INDUST
KARNAFULI PROJE
OUTLET WORKS
INTERNATIONAL ENGINEERING C

PART IV

CHAPTER III

OTHER STRUCTURES

Although the powerhouse is the major structure in the Karnafuli Project, the other structures have been given careful study. Simplicity and repetition were emphasized in both design and finish, to be consistent with the scale of the project. The equipment, and railings, particularly on the spillway and intake structures, were arranged and painted to give a neat appearance. Horizontal ledges, where dust may collect and wash down the walls causing streaks and water marks, were held to a minimum. Painted surfaces were given treatments, coatings and colors consistent with those on the powerhouse to improve appearance and to simplify maintenance problems.

Perspectives of the spillway and the outlet works precede this page. (Figures A and B)

KARNAFULI PROJECT

DESIGN REPORT

PART V

EQUIPMENT AND SYSTEMS

CHAPTER I GENERAL

CHAPTER II SPILLWAY

2.01 General
2.02 Tainter Gates
2.03 Tainter Gate Hoists
2.04 Stoplogs
2.05 Electrical Power and Lighting

CHAPTER III DIVERSION AND OUTLET TUNNEL

3.01 General
3.02 Bulkhead
3.03 Fixed-Wheel Gate
3.04 Slide Gate
3.05 Hoists
3.06 Trashrack
3.07 Ventilation
3.08 Electrical Power and Lighting

CHAPTER IV POWERHOUSE

4.01 General
4.02 Electrical Equipment and Systems
4.03 Mechanical Equipment and Systems
4.04 Other Powerhouse Equipment

CHAPTER V POWER INTAKE

5.01 General
5.02 Trashrack
5.03 Trashrack Rake
5.04 Intake Gates
5.05 Stoplogs
5.06 Bulkheads
5.07 Hoists
5.08 Other Equipment
5.09 Electrical Power and Lighting

CHAPTER VI SWITCHYARD

6.01 General
6.02 Design Characteristics
6.03 Ratings
6.04 Interconnections
6.05 Controls
6.06 Interlock Schemes
6.07 Signaling Devices
6.08 Electrical Power and Lighting
6.09 Communications
6.10 Safety Provisions

CHAPTER VII CARGO TRANSFER SYSTEM TROLLEYS

7.01 General
7.02 Design

PART V

CHAPTER I

GENERAL

This part of the Design Report pertains to the design of the structural, mechanical and electrical equipment permanently installed in the various project features. Structural equipment includes gates, trashracks, bulkheads and stoplogs. Design of structural steel frames including the powerhouse superstructure, switchyard, etc. is covered in Part III, Structures.

The design criteria for each item of equipment and system is presented. A description and other pertinent information such as capacity, protection features and layout exhibits are given. The design analysis of each item of structural equipment is also given.

PART V

CHAPTER II

SPILLWAY

2.01 GENERAL

The spillway crest structure has 16 gate openings each 40 feet wide, separated by 7-foot thick intermediate piers. The spillway crest is at Elevation 80.25 feet, and the top of gates is at Elevation 118.00 feet. The bridge and the hoist deck are at Elevation 127.00 feet.

The spillway is equipped with tainter gates, operated by individual drum-and-cable hoists. Electric power for the hoists is supplied by a transmission line from the powerhouse. However, an emergency source of electric power is provided by a diesel-electric generator installation at the spillway.

One set of interchangeable stoplogs is provided to enable unwatering for inspection, maintenance or repair of a gate when the reservoir level is between Elevation 80 feet and Elevation 102 feet.

2.02 TAINTER GATES

A. General

1. Selection - The selection of tainter gates with individual hoists was determined during preliminary design to be the most satisfactory choice for combining reliability and economy. An alternate plan involving vertical lift gates with two traveling cranes was given preliminary consideration, but was eliminated because of lesser reliability, with little, if any, reduction in cost.

The adopted arrangement is in keeping with current United States practices where tainter gates are being used for spillway control. Such gates have been made as large as 50 feet wide by 50 feet high, and present planning calls for even larger units.

2. Description - Design of the gate, as shown on Exhibit II-1 (PART V), follows conventional lines, using horizontally spanning skin plates supported by vertical ribs. The ribs, in turn, are supported by a system of horizontal girders, from which the load is delivered to the trunnions through box girders and struts.

The center of the trunnion has been placed low, but clear of the upper nappe of the maximum flood flow, to reduce to a minimum the vertical component of the maximum gate load on the pier.

Each trunnion pedestal bolts onto a nearly vertical anchorage girder which is embedded in the concrete. The anchorage girders are held in place by a system of post-tensioned, high-tensile rods.

An angular hoisting movement of 44.5 degrees raises the bottom gate edge to Elevation 107.00 feet, which level is approximately one foot above the estimated design flood nappe.

Standard J-type molded rubber seals are provided for the sides and sills of each gate.

B. Stresses and Loads

Under all conditions of normal loading, the maximum allowable design stresses were limited to 90 percent of those given in the American Institute of Steel Construction (AISC) Specification for Structural Steel for Buildings. Normal loads are considered to consist of all combinations of hydrostatic loads up to full reservoir level, together with dead loads and normal operating loads, with the gate in any position from fully open to closed.

Maximum reservoir elevation is set at Elevation 119 feet, resulting in a surcharge of only one foot above the top of the gate. Increased stresses resulting from flood surcharge are normally considered permissible if they do not exceed the normal allowable stresses by more than one-third, (33.3%). In this case, the one-foot surcharge represents a load increase of five percent.

C. Provision for Corrosion

Provision for corrosion was made by providing 1/16 inch of extra thickness to the skin plate of the gates and by using a minimum thickness of 3/8 inch for all steel work.

D. Structural Framing

Exhibit II-2 (PART V) presents in graphical form the loading conditions considered and the assumptions made for design of the various gate components.

The structural framing of the face of the gate and its supporting horizontal beams conforms to the best conventional method of framing tainter gates. The skin plate is supported on vertical stiffeners or ribs which are curved to the radius of the gate. Horizontal elements of the skin plates are designed to span continuously over the ribs. The ribs are designed as straight beams with varying hydrostatic loading, continuous over two spans with cantilevers at each end. Portions of the skin plate are assumed to act integrally with the stiffening ribs. Span lengths of the ribs are such as to equalize rib moments as much as practicable. Combined stresses at critical points, resulting from skin plate action in conjunction with rib bending, were investigated.

The skin plate ribs are carried on three horizontal girders which, in turn, are supported near the fifth points to approximately equalize the positive and negative bending moments. The middle girder is designed to resist the entire thrust of 270,000 pounds in addition to bending forces.

The design of these gates differs from the conventional, in that the means for carrying the hydrostatic load from the horizontal girders to the trunnion bearing consists of but one strut at each end of the gate, instead of multiple strut arms, one from each horizontal girder to the bearing. The single strut design was adopted primarily to provide clearance in the pier recess between the gate and the embedded anchorages, with the anchorages placed almost horizontally and parallel to the maximum load resultant. Other advantages of this compact layout of gate supports are that the component parts may be more easily heat-treated after shop welding, they may be shipped as units, and they require no field welding to reassemble. The maintenance problem is also reduced. The disadvantage of this type of box girder and strut construction is that the weight of steel required is slightly in excess of that of the conventional multiple-arm system. However, there are savings in fabrication, shipping, and erection costs that offset any additional cost of material.

As shown on Exhibit II-2, the centerline of the strut is located slightly below the line of the full gate load resultant, so that strut moments (of opposite signs) are balanced when the gate is at its critical opening of 13.5 feet.

E. Trunnion Bearing

The trunnion bearing is the hinge about which the gate pivots. It consists of a hub on the downstream end of the strut, with a pin connecting the hub to the supporting pedestal. One side of the pedestal is designed to resist the lateral thrust of 270,000 pounds.

The bronze bushing and thrust ring are of the self-lubricating "Lubrite" type to ensure positive lubrication at all times. This is a standard type of installation for connections with high bearing pressures which make it impossible to maintain a film of ordinary grease on the sliding surfaces. Also, this type installation eliminates the necessity for a high-pressure lubrication system. The forged steel pin is equipped with a stainless-steel sleeve to prevent corrosion on the contact surface.

F. Anchorage

Anchorage design provides for the use of individual anchorage girders for each trunnion, with the upper portion of each girder projecting out of the concrete.

Modern design and construction practices in the field of prestressing permit the use of a lighter anchorage arrangement for securing the girder to the concrete pier. A group of post-tensioned high-strength steel rods within the pier is used for this purpose. The weight of these rods is only a fraction of that of a structural steel anchorage, and the

post-tensioning force which clamps the anchorage girder against the concrete is considerably in excess of the maximum water pressure on the gate. This type of anchorage has two distinct advantages, in that (1) it eliminates any movement of the trunnion when the gate is loaded, and (2) it places the pier concrete in compression, thus preventing the formation of cracks.

2.03 TAINTER GATE HOISTS

A. General

1. Purpose - Sixteen spillway tainter gate hoists are provided for operation of individual gates as will be required for proper spillway releases.

2. Location - The hoists are located at Elevation 127.00 feet on the cantilever portions of the spillway piers adjacent to and above the spillway tainter gates.

3. Description - Each hoist is a 75-ton (2,000 pounds per ton) nominal capacity, wire rope, double-drum, double-pickup, electrically driven hoist with a raising or lowering speed, under power, of two feet per minute. The hoists are built for outdoor operation as shown on Figure A.

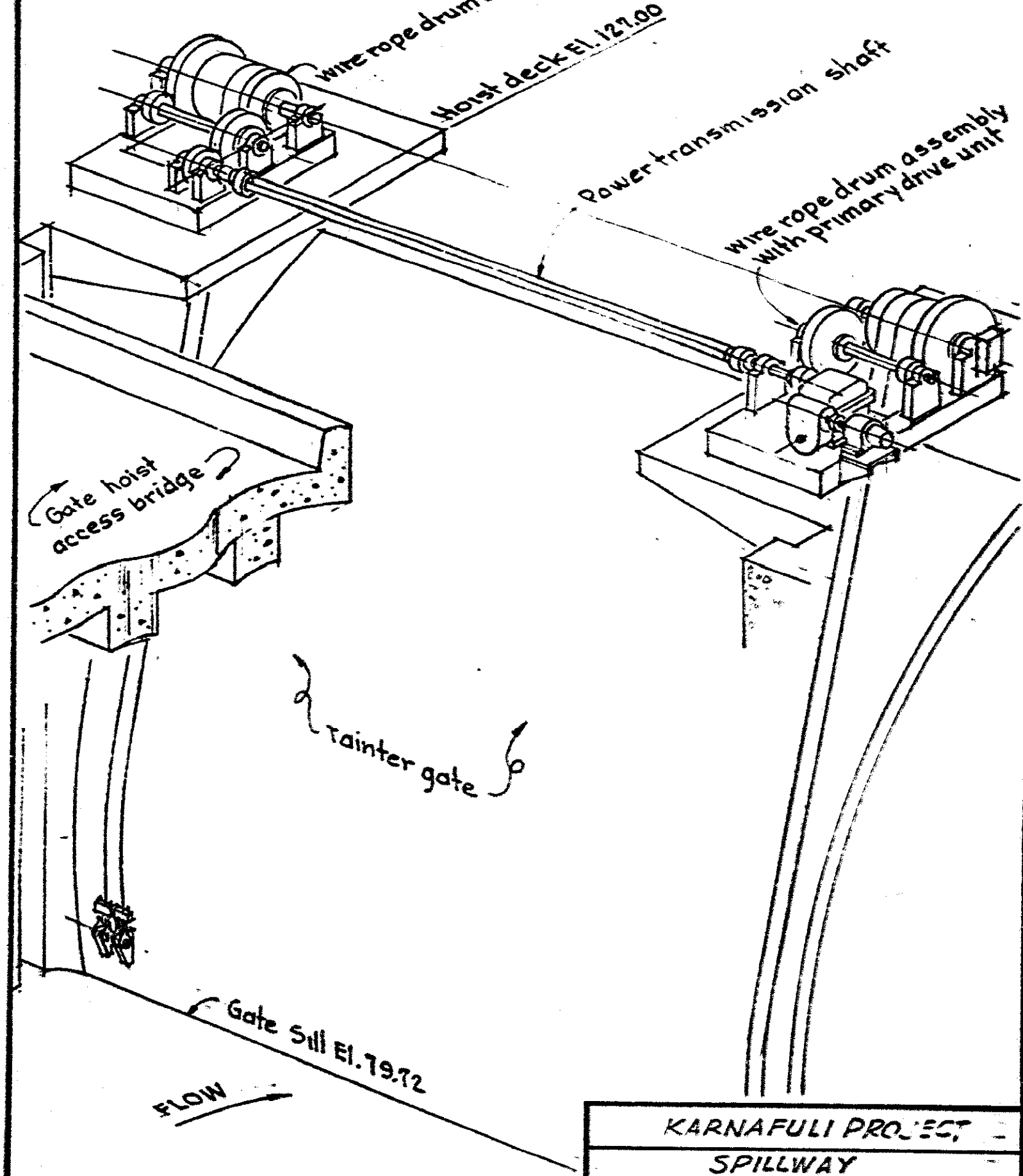
B. Selection of Type

The cantilever supported hoist arrangement was selected to eliminate the cost of a supporting bridge.

C. Design

1. Loads and Stresses - The hoist capacity is based on the gate weight, seal and trunnion frictions, and the hydraulic loads and is rated at the gate connections. Hoist loads are shown on Exhibit II-2. The hoist parts are designed with a safety factor of five based on the ultimate strength of the materials used. The wire rope is selected with a safety factor of five based on the breaking strength of the rope. The stresses induced under the maximum torque of the hoist drive motor do not exceed eighty percent of the yield point of the materials.

2. Mechanical Features - Mechanical features include two separate drum assemblies, mounted on the cantilever portion of the piers. The two drum assemblies are connected by a power transmission shaft which spans the opening between piers. One drum assembly contains the primary drive unit. The drums can receive all the wire ropes in single layers. The ropes are secured to the gate through open bridge rope sockets to provide a means for adjusting the rope tension. The power transmission shaft is hollow to provide rigidity at comparatively small weight and is equipped with flexible couplings at each end to compensate for misalignment and deflection. A self-locking worm gear is incorporated in the primary drive machinery to function as a load brake in controlling the lowering speed. A gate-position indicator is mounted in the vicinity of the hoist control station.



Hoist Capacity = 75 tons
Raising Speed = 2 fpm
Lowering Speed = 2 fpm

KARNAFULI PROJECT

SPILLWAY
TAINTER GATE HOIST
DESIGN REPORT

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR.	RECOMMENDED	APPROVED

3. Structural Features - Each wire rope drum assembly is mounted on a rigid structural steel base constructed as a unit. These bases are equipped with lifting eyes to facilitate installation of the hoists. The bases are secured and leveled by means of anchor bolts embedded in the concrete.

4. Electrical Features - All electrical equipment is suitable for outdoor operations and for the power source of 230/400 volts grounded wye, 3-phase, 50-cycle, alternating current. Electrical wiring runs in rigid metal conduits, tagged and marked for permanent identification. Hoists operations are push-button controlled. To raise or lower the gate, the operator must press the required push-button to start the hoist. The hoist will continue in operation until the operator presses the stop push-button or until the limit switch stops the hoist. An alarm bell is provided to ring for each foot of gate travel. Each push-button has its control function designated by a nameplate.

An electrical holding brake is provided to stop and hold the gate. This brake sets automatically on interruption of electrical power in the motor circuit; it releases when the motor circuit is energized. Release can be effected manually. The hoists are equipped with limit switches to control upper and lower gate positions. Instructions for operating the hoist, wiring control diagrams and maintenance instructions are permanently placed in the vicinity of the control cabinet.

2.04 STOPLOGS

A. General

A set of seven interchangeable, structural steel stoplogs is provided to permit inspection, maintenance, or repairs to the tainter gates. This set of stoplogs permits unwatering of any one tainter gate when the reservoir water surface is at Elevation 102.00 feet or below, a condition which will prevail for several months each year. The stoplogs are stored in a convenient place from where they can be trucked or barged to the spillway for use.

B. Loads

The hydrostatic pressure on the upstream face of the stoplogs is the primary load. Secondary loads are induced by friction between the individual stoplogs, since the water pressure against them causes unequal lateral deflection.

The stoplogs were designed to safely resist the pressures resulting from a reservoir water surface at Elevation 113.00 feet. In the event that it should become desirable to unwater a gate while the reservoir level is above Elevation 102.00 feet, additional stoplogs can be designed for placing above the original set.

C. Bearings and Seals

Bolted-on timbers form both bearing and seal against the downstream faces of the stoplog slots. Timber bumpers are provided at the ends of each stoplog to prevent damage to the slots.

Sealing at the sill and between the individual stoplogs is accomplished by means of molded J-type rubber seals.

D. Handling

A lifting beam is provided for use in positioning the stoplogs. Lowering and raising is accomplished with the use of a mobile crane. When a stoplog has been lowered to its final position, a pull on the tag lines disengages the two hooks for recovery of the lifting beam. When removing stoplogs, the lifting beam is lowered until the hooks engage the lifting eyes by gravity.

2.05 ELECTRICAL POWER AND LIGHTING

A. General

The remote location of the spillway tainter gates and hoists from the powerhouse requires that electrical services be provided by the 11-kv aerial distribution line.

Standard distribution practice for this class of service requires a pole-mounted bank of three single-phase distribution transformers with fuse cutout and distribution lightning arrestors for step-down of the 11-kv distribution voltage to the 230/400 volt, grounded wye utilization voltage.

Tainter gate hoist motors and power outlets for maintenance equipment are provided with 400-volt, 3-phase, 50-cycle service. Lighting and convenience outlets are provided with 230-volt, single-phase, 50-cycle service.

B. Secondary Protection

Modern practice provides secondary protection for the transformer bank and branch circuits to motors, power outlets, and lighting and convenience outlets. To provide this protection, an outdoor weatherproof convertible, 3-phase, 4-wire distribution type circuit breaker panelboard is installed at the spillway transformer pole. This type arrangement is preferable from the standpoint of simplicity, safety, and economy.

C. Motors

All motors are provided with magnetic starters and other control equipment required for local tainter gate control. Electrical automatic or remote control equipment will not be provided, as emergency conditions requiring such controls will not exist.

D. Lighting

Lighting is provided by standard incandescent industrial type fixtures, following the criteria established by general project lighting. The tainter gate hoist control locations have illumination levels of 3-foot candles. The gate-hoist access bridge has 1-foot candle. Convenience outlets for extension lights and other temporary operating and maintenance service, and power outlets for use of power hand tools are located at 100-foot intervals across the spillway.

E. Grounding

A copper grounding system is provided to protect personnel from danger of high voltage due to faulty equipment or short circuit conditions. All exposed metalwork is connected to the main ground cable to keep all such metal to equal potential, and the main ground cable is grounded by connections to driven ground rods in the approach channel.

F. Telephone

To provide communications with other project features during routine operations and maintenance activities, a telephone is installed at the spillway transformer pole.

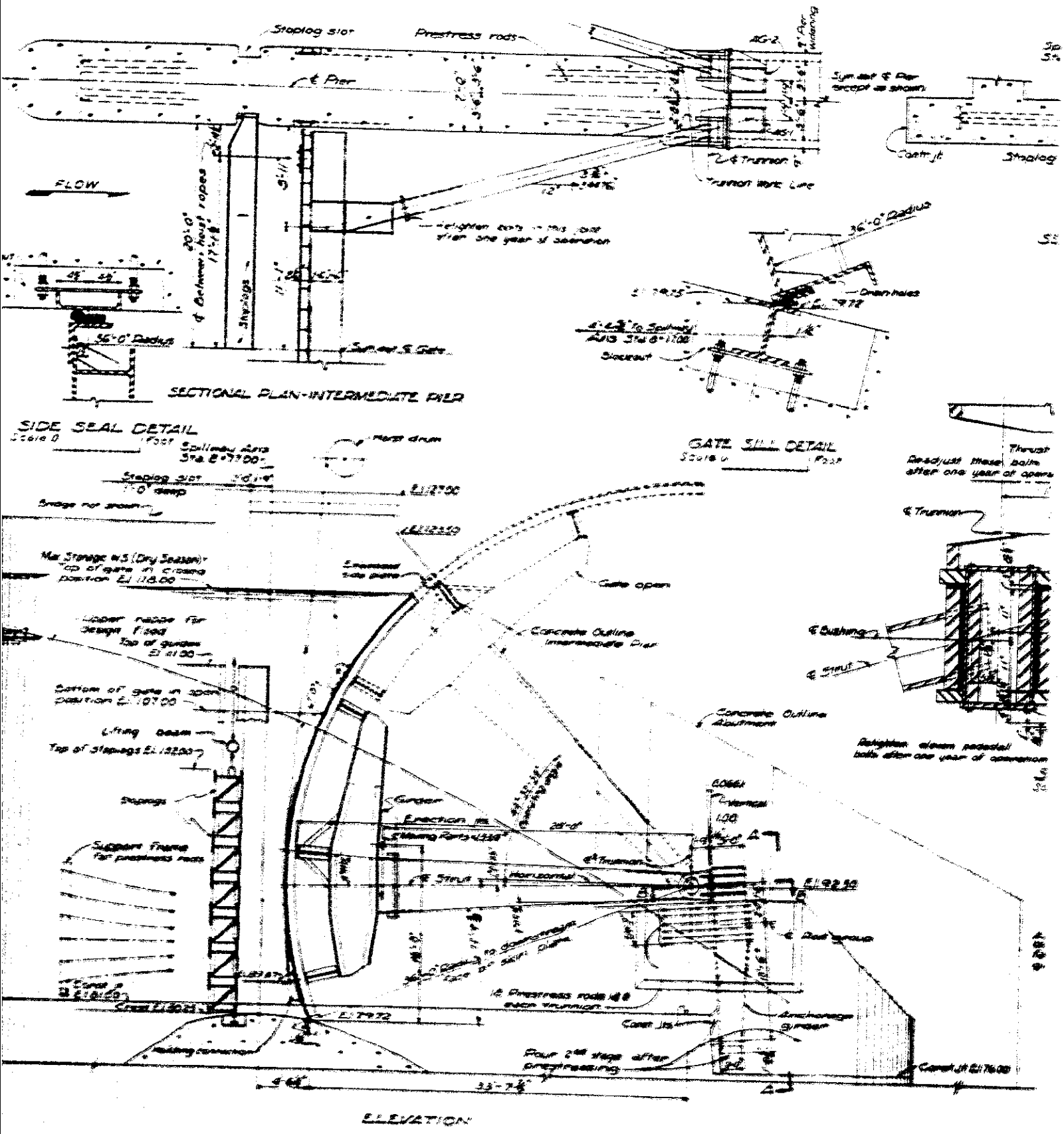
G. Emergency Power and Lighting

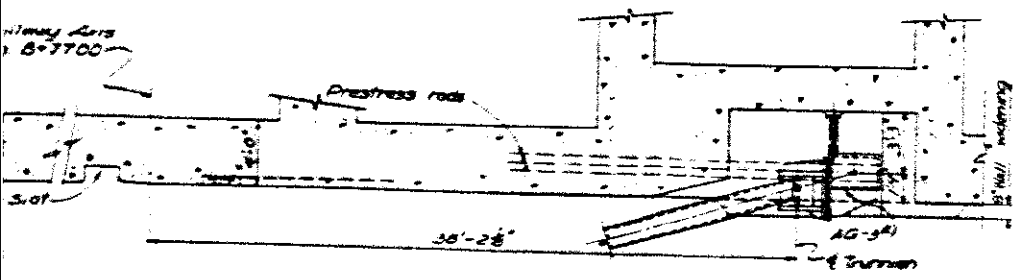
A diesel electrical generating plant to provide emergency power and lighting for the spillway tainter gates is installed in the spillway area.

PART V - CHAPTER II

LIST OF EXHIBITS

Exhibit	Title
II-1	Spillway Tainter Gates and Stoplogs - General Arrangement
II-2	Spillway Tainter Gates - Design Analysis Data


Stoplog slot
Prestress rods
℄ Pier
FLOW
Stoplogs
56'-0" Radius
SECTIONAL PLAN-INTERMEDIATE PIER
SIDE SEAL DETAIL
GATE SILL DETAIL
36'-0" Radius
Drain holes
Stoplog slot
Gate open
Concrete Outline Intermediate Pier
Concrete Outline Abutment
Lifting beam
Stoplogs
Girder
Support Frame for prestress rods
℄ Trunnion
℄ Strut
℄ Bushing
Anchorage girder
Pour 2nd stage after prestressing
ELEVATION



CTIONAL PLAN-ABUTMENT

ESTIMATED GATE WEIGHTS:

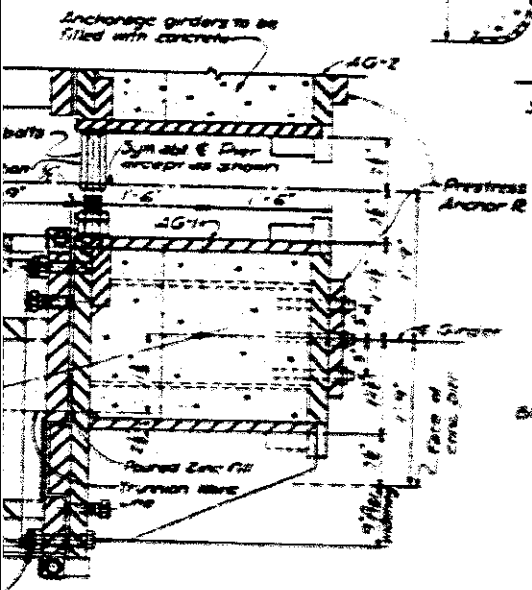
Gate Leaf	77,200 lbs
Box Girders	23,600 "
Struts	31,800 "
Trunnion Pedestals	9,400 "
Anchorage Girders	31,600 "
Sills and Side Plates	2,800 "
Total each Gate Assembly	77,400 "
Total required for 16 Gate Assemblies	2,838,400 lbs

(See dwg FL-15-229 for estimated weight of prestress material)

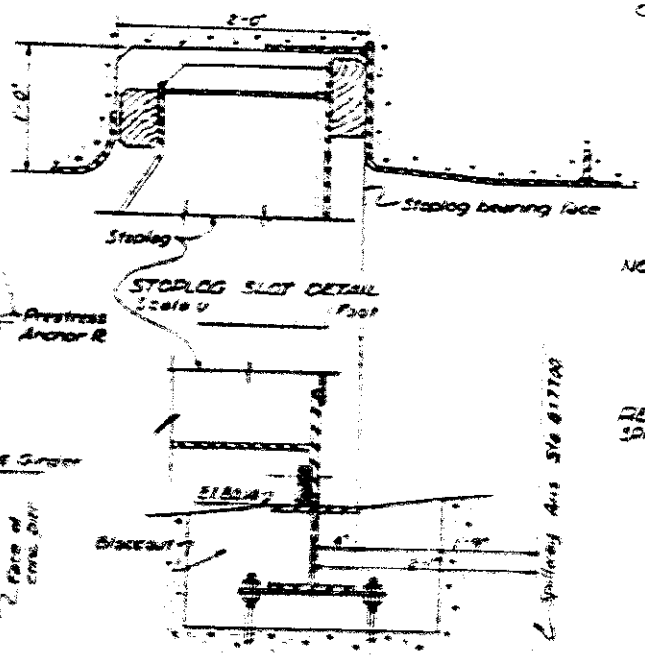
ESTIMATED STOPLOG WEIGHTS:

Total seven Stoplogs	70,400 lbs
Stoplog Lifting Beam	1,500 "
Sills and Guides	118,000 "
Total Stoplogs	197,900 lbs

GRAND TOTAL 3,036,300 lbs



CTION B-B
Scale 0 Feet



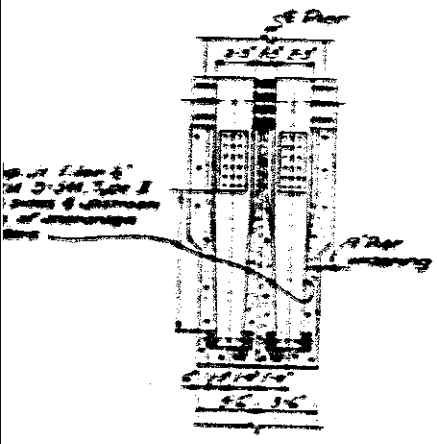
STOPLOG SILL DETAIL
Scale 0 Feet

NOTES:
Refer to individual drawings for notes.

REFERENCE DRAWINGS:
SPILLWAY-Tainter Gates-Leaves
Framing & Details FL-15-220
-Tainter Gates-Leaves
Details & Seals FL-15-223
-Tainter Gates-Girders & Struts
Sheet 1 of 2 FL-15-224
-Tainter Gates-Girders & Struts
Sheet 2 of 2 FL-15-225
-Tainter Gates-Trunnions & Anchorage
Girders-Sheet 1 of 2 FL-15-226
-Tainter Gates-Trunnions & Anchorage
Girders-Sheet 2 of 2 FL-15-227
-Tainter Gates
Sills & Side Plates FL-15-228
-Tainter Gates-Anchorages
Prestress Rods FL-15-229
-Tainter Gates 40'-0" x 37'-9"
Design-Analysis Data FL-15-230
-Stoplogs-Panels FL-15-231
-Stoplogs-Sills & Guides FL-15-232
-Stoplogs-Lifting Beam FL-15-233

-Crest Structure-Pier Block-
Concrete Outline-Stage II FL-15-3[illegible]
Crest Structure-Abutment
Concrete Outline-Stage II-Sheet 1 FL-15-3[illegible]

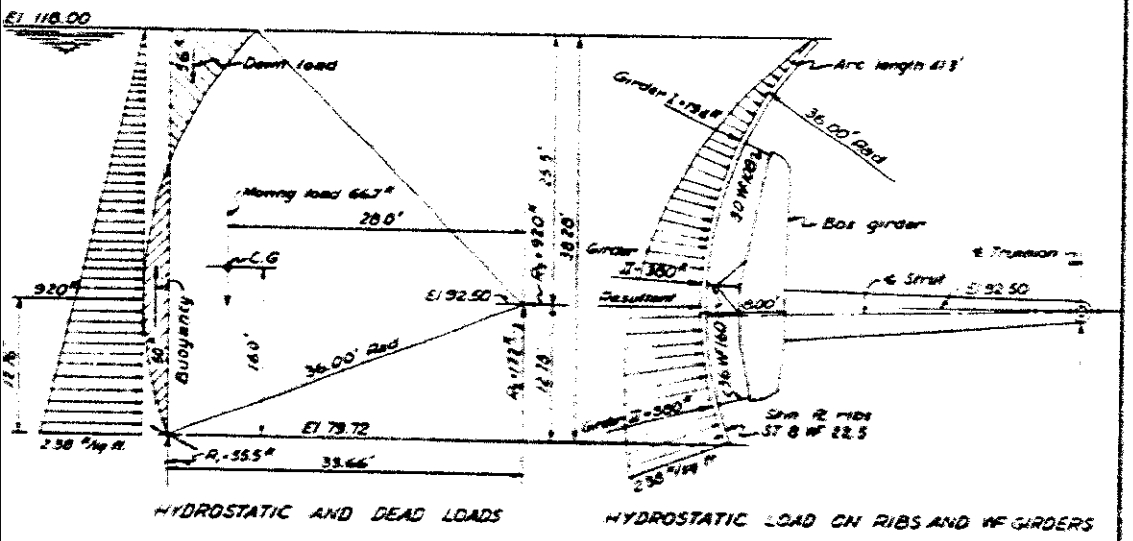
Scale: 0 5 10 Feet
ENGLISH AS PLATED



SECTION A-A

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
SPILLWAY
TAINTER GATES & STOPLOGS
GENERAL ARRANGEMENT
INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS

ICA-II-1	San Francisco, Cal. AUGUST 25, 1959	FL-15-221

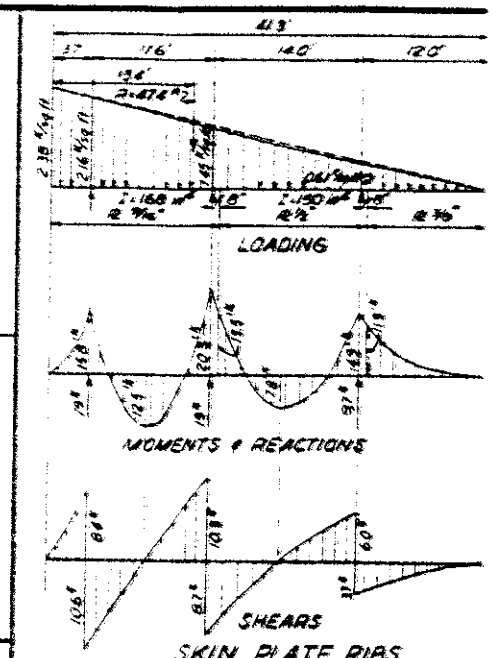


HYDROSTATIC AND DEAD LOADS

HYDROSTATIC LOAD ON RIBS AND WF GIRDERS

FULL LOAD AND REACTIONS ON HALF GATE

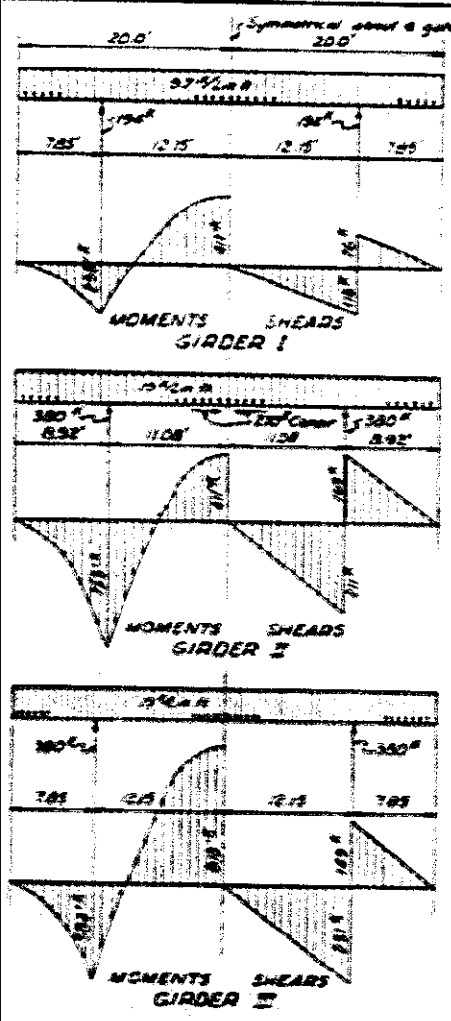
Scales: 1/8" = 1'-0", 1" = 2 #/sq.ft.



SKIN PLATE RIBS

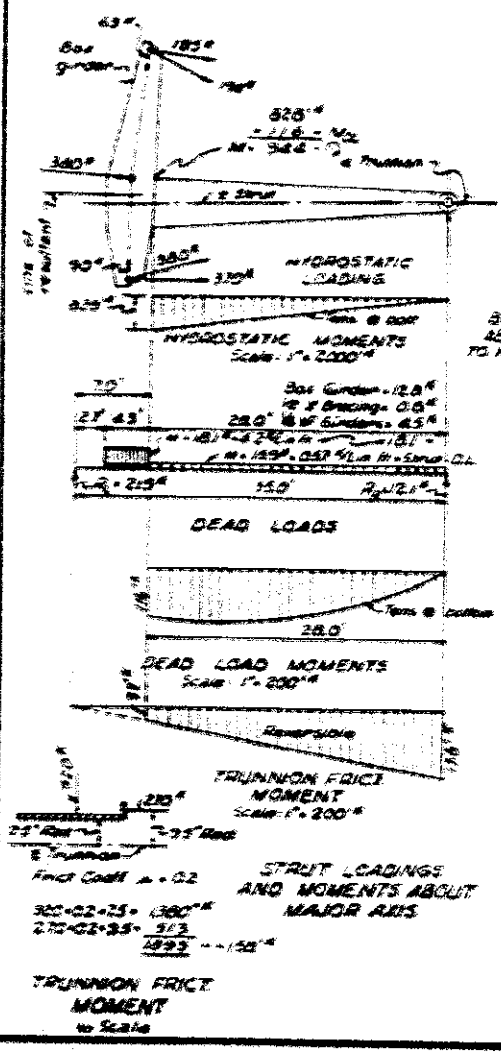
Computed on a 1'-0" wide strip

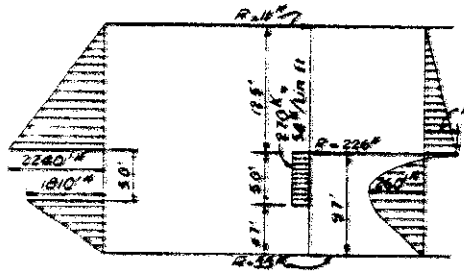
Scales: 1/8" = 1'-0", 1" = 2 #/sq.ft., 1" = 20#, 1" = 10#



HORIZONTAL WF GIRDERS

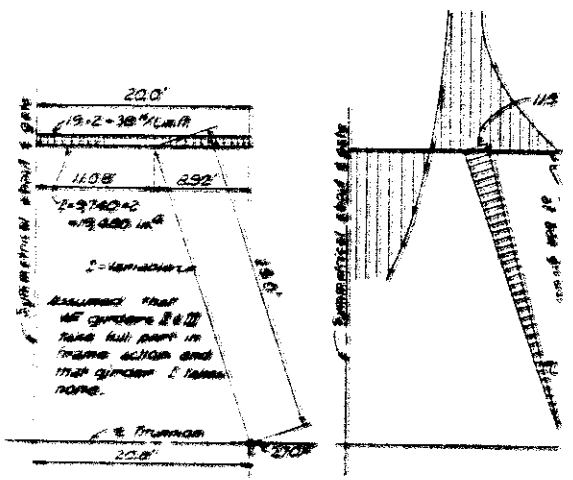
Scales: 1/8" = 1'-0", 1" = 300#, 1" = 200#





BOX GIRDER MOMENTS ABOUT MAJOR AXIS DUE TO HYDROSTATIC LOADING 1" = 2000#

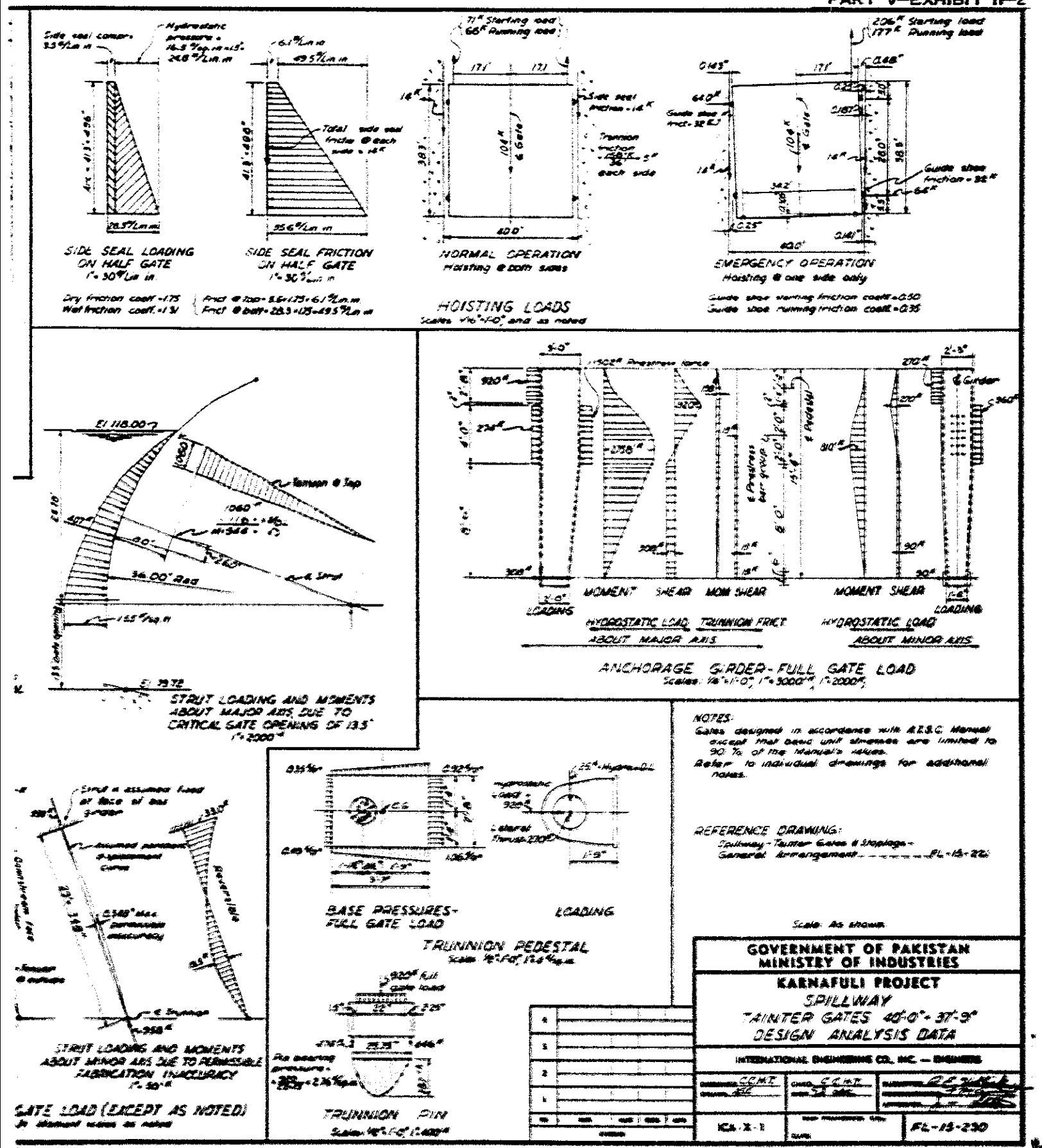
BOX GIRDER LOADING AND MOMENTS ABOUT MINOR AXIS, DUE TO LATERAL THRUST OF STRUT 1" = 400#



STRUT LOADING AND MOMENTS ABOUT MINOR AXIS DUE TO HYDROSTATIC LOAD ON WF GIRDERS 1" = 400#

BOX GIRDER AND STRUT - FULL

Scales: 1/8" = 1'-0", 1" = 2 #/sq.ft.


SIDE SEAL LOADING ON HALF GATE
SIDE SEAL FRICTION ON HALF GATE
HOISTING LOADS
NORMAL OPERATION
EMERGENCY OPERATION
STRUT LOADING AND MOMENTS ABOUT MAJOR AXIS DUE TO CRITICAL GATE OPENING OF 13.5'
ANCHORAGE GIRDER - FULL GATE LOAD
MOMENT SHEAR MOM SHEAR
MOMENT SHEAR
HYDROSTATIC LOAD TRUNNION FRICT ABOUT MAJOR AXIS
HYDROSTATIC LOAD ABOUT MINOR AXIS
STRUT LOADING AND MOMENTS ABOUT MINOR AXIS DUE TO PERMISSIBLE FABRICATION INACCURACY
GATE LOAD (EXCEPT AS NOTED)
BASE PRESSURES - FULL GATE LOAD
LOADING
TRUNNION PEDESTAL
TRUNNION PIN
NOTES:
Gates designed in accordance with A.I.S.C. Manual except that basic unit stresses are limited to 90% of the Manual's values.
Refer to individual drawings for additional notes.
REFERENCE DRAWING:
Spillway - Tainter Gates & Stoplogs - General Arrangement
Scale: As shown
GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
SPILLWAY
TAINTER GATES 40'-0" x 37'-9"
DESIGN ANALYSIS DATA
INTERNATIONAL ENGINEERING CO., INC. — ENGINEERS
FL-15-230

CHAPTER III

DIVERSION AND OUTLET TUNNEL

3.01 GENERAL

Design of the diversion and outlet tunnel provided for use of the tunnel first as a temporary diversion structure, and second, as a permanent outlet works. Design requirements for the outlet works included the following:

1. Discharge capacity shall be 2000 cubic feet per second at the minimum reservoir operating level of Elevation 60.0 feet.

2. Means for closure shall be provided to allow; (1) installation of a control gate, and (2) normal and emergency inspection and maintenance of the control gate.

The minimum tunnel cross-sectional area required for passage of diversion flows was somewhat larger than that necessary for the outlet works function. Consequently, a concrete bulkhead with an orifice sufficiently large to pass the required flow is provided, as shown on Exhibit III-1. A fixed-wheel gate is located on the upstream face of the bulkhead. This bulkhead and gate combination provides the means for closure for the purposes set forth in Item 2 above. A second stage concrete plug with a rectangular opening is located downstream of the bulkhead. This plug restricts the area in the control section of the tunnel and provides anchorage for the high pressure slide gate which is used for control of flows after diversion.

Hoisting equipment includes a monorail service hoist and a drum-and-cable fixed-wheel gate hoist, both located in the gate shaft house. The hydraulic hoist for the control gate is an integral part of the gate.

3.02 BULKHEAD

A. General

1. Selection - Selection of the type of bulkhead was based on the results of comparative economic studies of a 20-foot high by 14-foot wide fixed-wheel gate, and a fixed bulkhead of the same dimensions, with a small opening which could be closed by a fixed-wheel gate on the upstream face of the bulkhead. The bulkhead and gate combination was adopted.

2. Description - As shown on Exhibit III-2, the 20-foot high by 14-foot 4-inch wide concrete bulkhead is 3 feet 2 inches thick and has a total weight of approximately 112,600 pounds. A rectangular-shaped bell-mouth opening, 6 feet wide and 7 feet 6 inches high, has the same minimum dimensions as the opening through the tunnel plug immediately downstream of the bulkhead.

B. Design

1. Criteria and Assumptions - The bulkhead was designed to withstand the hydrostatic forces due to a reservoir level at Elevation 80.25 feet, the elevation of the spillway crest. Allowance for greater head was unnecessary, since the second-stage concrete behind the bulkhead provides the support necessary to resist the forces resulting from higher heads. The design provided for this concrete to be placed before the reservoir surface rose above Elevation 80.25 feet.

Maximum allowable stresses in the bulkhead were limited as follows:

a. Compressive strength of concrete, 3750 pounds per square inch at 28 days.

b. Tensile unit stress of steel, 20,000 pounds per square inch.

c. All other stresses, in accordance with the American Concrete Institute Building Code, ACI 318-56.

2. Analysis - Details of the bulkhead design analysis are shown on Exhibit III-3. The component parts of the bulkhead were analyzed as beams with hydrostatic and gate-wheel loads. Since the horizontal beams designated as C, B, and E are short, and have unyielding supports at their ends, it was assumed for analysis that the vertical D beams were continuous two-span beams, supported by beams C, B, and E. Beam supports and loads are as shown on Exhibit III-3.

3.03 FIXED-WHEEL GATE

A. General

1. Selection - As previously mentioned, a fixed-wheel gate was selected for use in conjunction with the bulkhead. This type of gate is considered to be the most economical kind for the intended use.

2. Description - The fixed-wheel gate, framing and details of which are shown on Exhibit III-4, is 10 feet 2-1/2 inches wide, 10 feet 5 inches high and 20 inches thick. Total weight of the gate is approximately 26,400 pounds, of which 15,200 pounds is steel, and 11,200 pounds is concrete ballast.

The gate operates on the upstream face of the bulkhead. Slots in the upstream portion of the gate shaft are equipped with gate guides, to facilitate the raising and lowering of the gate. Dogging devices are provided at Elevation 135 feet for holding the gate in its storage position.

An equalizing line introduces water from the upstream side of the fixed-wheel gate to the chamber between it and the slide gate to permit operation under balanced head conditions. However, in the event that the slide gate should be jammed in the open position, permanent concrete ballast inside the fixed-wheel gate permits closure against the hydrostatic head due to a reservoir level at Elevation 97 feet.

B. Design

1. Criteria and Assumptions - The fixed-wheel gate was designed for normal steel stresses (basic tensile stress being 18,000 pounds per square inch) under full hydrostatic head due to a normal pool water surface at Elevation 118 feet. Also considered, but not governing, was the condition of maximum pool water surface at Elevation 119 feet, with an allowable overstress of 33-1/3 percent (24,000 pounds per square inch).

The skin plate was designed for bending at a basic tensile stress of 24,000 pounds per square inch under normal load, both alone and as part of the girder stresses. An allowance of 1/16 inch was made for skin plate and side plate corrosion.

2. Analysis - In the design, the girder flanges were considered to be part of the skin plate, thus forming a continuous series of haunched beams. Skin plate and girder stresses, which are at right angles to each other, were combined by adding three-tenths of the smaller stress to the larger stress, in accordance with Poisson's ratio for carbon steel. Complete details of the fixed-wheel gate design analysis are given in Exhibit III-5.

3.04 SLIDE GATE

A. General

1. Operational Requirement - Operation of the control gate will be required when the reservoir water surface is at or below the spillway crest, Elevation 80.25 feet, and the powerhouse units are shut down.

2. Structural Requirement - The gate must resist hydrostatic pressures from full head at reservoir water surface Elevation 119.0 feet. This amounts to about 105 feet of static head.

3. Selection - A slide gate was selected to fulfill the above requirements. Selection of this type of gate was based on its economy, relative simplicity of design, and dependability of operation.

4. Description - The gate and hoist, as shown on Exhibit III-6, is a standard type of slide gate which has been developed over the past fifty years and is in wide general use. The gate is constructed of cast iron, with bronze and steel used for special parts. Principal parts are as follows:

a. Gate leaf, measuring 6 feet by 7 feet-6 inches
b. Upstream and downstream frames
c. Bonnet, with recess for gate leaf
d. Bonnet cover
e. Connecting stem, gate leaf to hoist
f. Hydraulic hoist, with cylinder about 18 inches in diameter by 8 feet-6 inches high.

B. Design

The one-piece, cast-iron gate leaf, with internal webs which span horizontally across the opening, was designed to resist hydrostatic pressure from the full reservoir. The maximum horizontal beam stress at this head is about 1,500 pounds per square inch.

The gate frames and bonnet were designed to resist the reaction of the hydraulic hoist and to resist external pressures of 30 pounds per square inch. The frames and bonnet are strongly cross-ribbed to distribute the hoist reaction. Internal hydrostatic pressure on the frames and bonnet is carried into the concrete which encases the gate.

The bonnet cover was designed to resist internal hydrostatic pressures from the full reservoir head and to resist the reaction from the hoist.

The operating load on the hoist to raise the gate, when the reservoir surface is at Elevation 80.25 feet, is approximately 152 kips. This load is made up of dead weight of gate leaf and stem, downpull, and friction of the gate leaf on its seats. A conservative starting coefficient of sliding friction of 0.6 for bronze on bronze was assumed. The load of 152 kips requires a differential oil pressure on the hoist piston of approximately 660 pounds per square inch. This compares to the required test pressure on the cylinder of 1,200 pounds per square inch, which gives a hoop tension stress of 9,500 pounds per square inch.

Composition of the bronze sealing seats on the gate leaf and frame was developed by the U. S. Bureau of Reclamation to carry heavy loads with sliding contact and to resist abrasion. Bearing stress on the seats during operation is under 500 pounds per square inch, as compared to an allowable stress of 3,000 pounds per square inch for this type of bronze.

The pumping equipment and controls for the hydraulic hoist operating system are located in the shaft house. A manual pump and by-pass valves are located in the gate chamber.

3.05 HOISTS

A. General

1. Purpose - A fixed-wheel gate hoist is provided for operation of the fixed-wheel gate. A monorail hoist is provided to lift the gate from its "dogged" position into the shaft house.

2. Location - These hoists are located in the shaft house of the gate shaft structure above Elevation 135.0 feet. The fixed-wheel gate hoist is placed directly above the fixed-wheel gate and held in place on the hoist deck by anchor bolts. The monorail hoist is suspended from a monorail just below the roof of the shaft house and over the centerline of the tunnel.

3. Description - The fixed-wheel gate hoist is a 15-ton (2000 pounds per ton) nominal capacity, reeved-wire rope, single-drum, single-pick-up, electrically driven hoist built for indoor operation as shown on Figure A.

The monorail hoist is a 10-ton (2000 pounds per ton) rated capacity, hand-operated, chain hoist, with traveling trolley, and monorail as shown on Figure A.

B. Fixed-Wheel Gate Hoist

1. Selection of Type - Comparative studies were made of an electric-powered monorail hoist versus a stationary fixed-wheel gate hoist. The stationary hoist was selected because of its economy and reliability. The capacity of 15 tons was required for operation of the gate against flowing water. The hoist is capable of raising the gate above maximum high water elevation for storage or maintenance. It is equipped with safety devices for personnel protection.

2. Design

a. Loads and Stresses - The hoist parts were designed with a safety factor of five based on the ultimate strength of the material used. The wire rope was selected with a safety factor of six as to size with due allowance for reeving efficiency. The stresses induced under the maximum torque of the hoist drive motor did not exceed eighty percent of the yield point of the materials. Under maximum loading, the hoist base bearing loads on the concrete did not exceed 600 pounds per square inch.

b. Mechanical Features - The mechanical features of the hoist include the speed reducer connected by shafting to the drum, and wire rope with sheaves and load block. The drum is large enough to receive all the wire rope in a single layer. The fleet angle is a maximum of 3-1/2 degrees. The load block design is such that the ropes will not leave the sheaves at any operating position. All sliding and meshing surfaces are adequately lubricated. A gate-position indicator is mounted near the hoist control panel.

c. Structural Features - Because the hoist must be moved from its position to remove the fixed-wheel gate, it is mounted on a rigid structural steel base constructed as a single unit. The base is equipped with lifting eyes, and a lifting sling is provided with the hoist for this function.

d. Electrical Features - All electrical equipment is suitable for indoor operation and for the power source of 230/400 volts, grounded wye, 3-phase, 50-cycle, alternating current.

Electrical wiring runs in rigid metal conduits, tagged and marked for permanent identification. Hoist operations are push-button controlled; each push-button has a name plate designating its function.

The hoist speed, raising or lowering, is 2 feet per minute. The lowering speed is controlled by regenerative braking of the drive motor.

An automatic electric holding brake is installed on the drive motor shaft extension. This brake sets, holding the gate, in the event of electrical power failure. The hoist is equipped with upper and lower limit switches to control the respective position of load block travel. In addition, a separate upper limit switch, directly activated, controls gate maximum raised position.

Instructions for operating and maintaining the hoist, together with electrical wiring diagrams, are mounted in the vicinity of the control cabinet.

C. Monorail Hoist

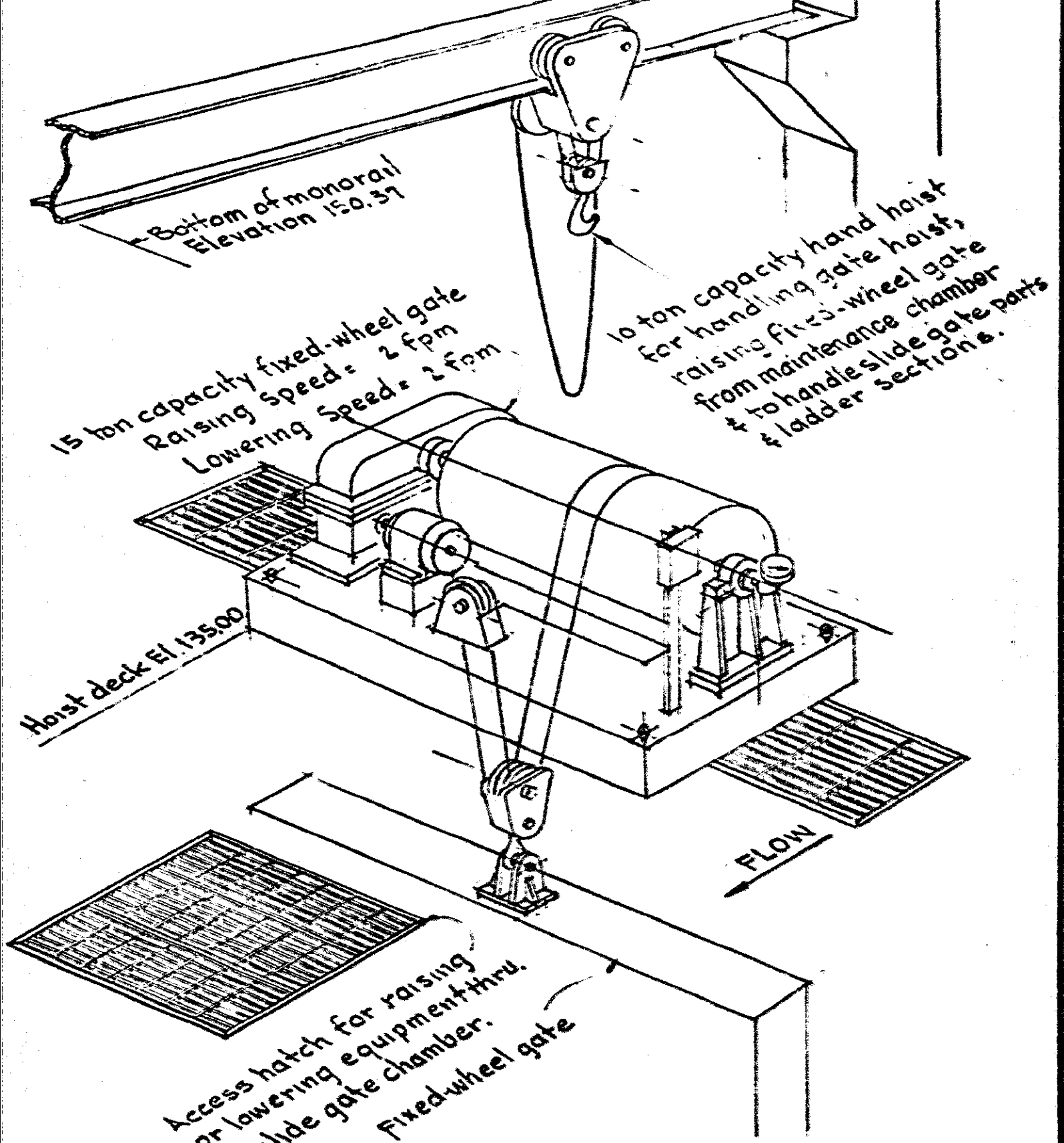
1. Description and Design - The monorail hoist is a commercial hand-operated hoist with a 10-ton rated capacity and a lift of 18 feet. The hoist and lifting beam were designed with a safety factor of four based on the ultimate strength of the material used. Its load hook was designed to accommodate the lifting pin at the fixed-wheel gate, and is swivel mounted on a thrust bearing. The hoist trolley has an easy-rolling action requiring a minimum effort to operate along the monorail beam.

A standard structural steel member, in one piece and of adequate size and strength, is used as a monorail. The monorail is held in place by embedded anchor bolts, and is equipped with trolley stops at each end.

3.06 TRASHRACK

A. General

To protect the emergency and control gates from damage or disruption of operation, a trashrack with openings one foot wide by three feet-three inches high, is provided at the entrance to the tunnel. Framing and details of the trashrack, which fully covers the tunnel entrance, are shown on Exhibit III-7.



KARNAFULI PROJECT

DIVERSION & OUTLET TUNNEL
FIXED-WHEEL GATE & MONORAIL HOIST
DESIGN REPORT

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR.	RECOMMENDED	APPROVED

B. Design

The trashrack was designed to ensure failure of the metalwork before overstressing the concrete. Design differentiates between the supports, or horizontal girders, and the bars. The supports were designed as simple beams for a stress of 33,000 pounds per square inch under a differential head of 40 feet. The resultant supports have a varying cross section, with the maximum thickness in the middle to resist bending moment.

The bars were designed as continuous beams with a stress of 33,000 pounds per square inch under a differential head of 35 feet.

Buckling of both the supports and bars was investigated.

3.07 VENTILATION

A ventilation system is provided for the gate chamber at the bottom of the shaft.

Exhaust is provided by an exhaust grille and a 16-inch diameter metal duct extending from the gate chamber up through the shaft and shaft house to a ventilator on the roof. The ventilator provides suction by natural and wind-induced drafts or by a self-contained electric motor and fan. Natural draft operation provides an estimated two air changes per hour in the gate chamber. Fan operation provides ten air changes per hour.

Fresh air is drawn down the shaft into the gate chamber from two 4-foot square louvers in the back wall of the shaft house.

All components of the vent are galvanized for protection against corrosion.

3.08 ELECTRICAL POWER AND LIGHTING

A. General

The remote location of the Diversion and Outlet Tunnel from the powerhouse required that electrical service be provided by the aerial distribution line. Service is furnished only to the shaft house, as no permanent power-operated equipment is located at the intake or outlet portals.

Standard distribution practice for this class of service required a pole-mounted bank of three single-phase distribution transformers, with fuse cutout and distribution lightning arrestors, for step-down of 12-kilovolt distribution voltage to 230/400 volt grounded wye utilization voltage.

Gate hoist motors and power outlets for maintenance equipment are provided with 400-volt, 3-phase, 50-cycle service. Lighting, ventilation, and convenience outlets are provided with 230-volt, single-phase, 50-cycle service.

B. Secondary Protection

Modern practice required secondary protection for the transformer bank and branch circuits to motors, power outlets, and lighting and convenience outlets. To provide this protection, a standard convertible three-phase, four-wire distribution type circuit breaker panelboard is installed in the shaft house. This type of arrangement is preferable to other arrangements from the standpoint of simplicity, safety, and economy.

C. Motors

All motors are provided with magnetic starters and other control equipment required for local control. No automatic or remote control equipment is provided, since there are no emergency conditions which would require such equipment.

D. Lighting and Convenience Outlets

Lighting provided by standard incandescent industrial-type fixtures, following the criteria established, is 10-foot candles over Elevation 135 feet, and 5-foot candles at all other locations. Vapor-tight fixtures are installed below Elevation 55 feet to reduce the possibility of damage from flooding of these areas during high tailwater conditions.

Convenience outlets are located at each working elevation for use of power hand tools and extension lights.

E. Grounding

A copper grounding system is provided to protect personnel from danger of high voltage due to faulty equipment or short circuit conditions. All exposed metalwork is connected to the main ground cable to keep all metalwork at equal potential. The main ground cable is grounded by connection to rods driven into the ground and to metal constantly in contact with water in the diversion tunnel.

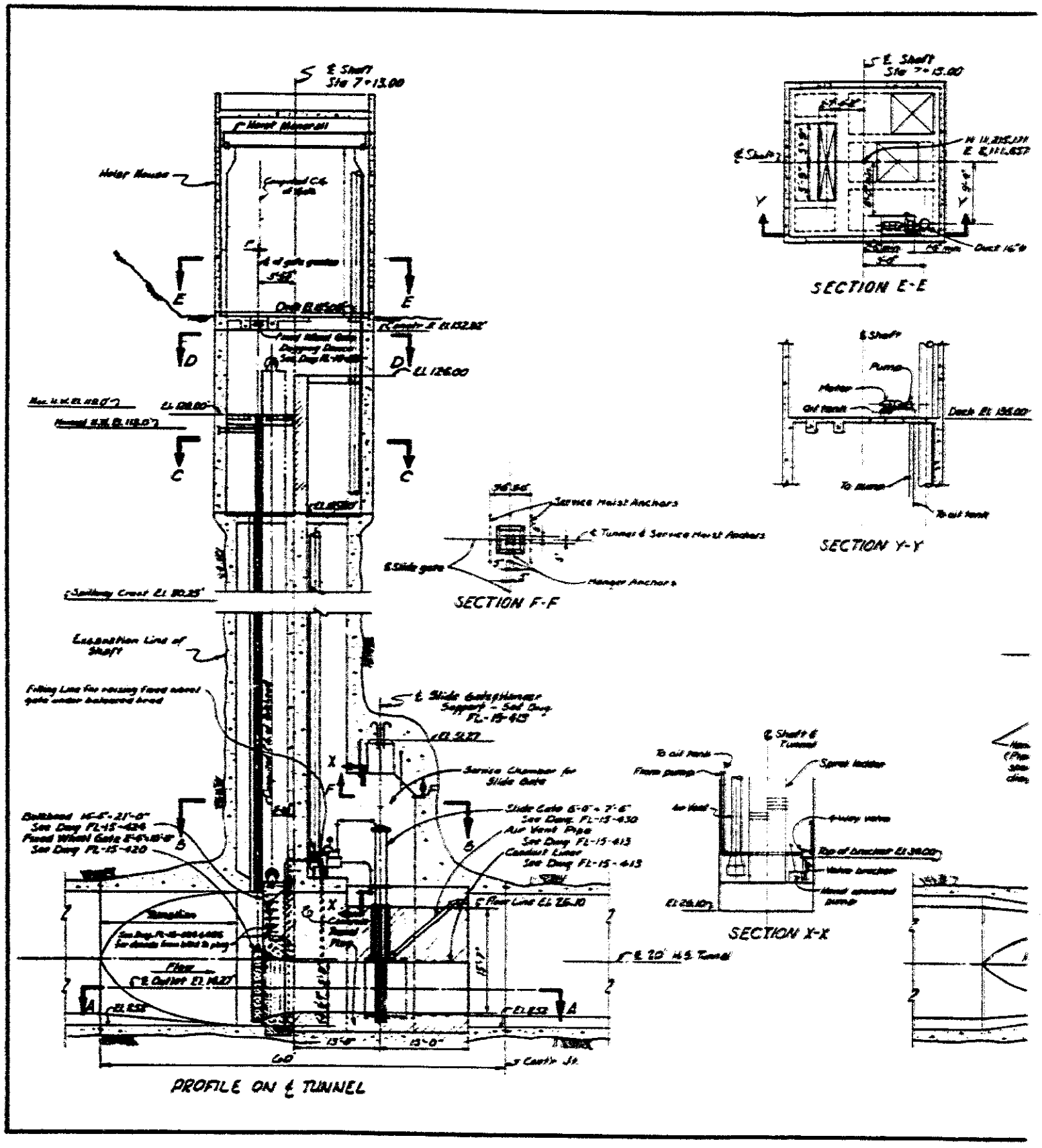
F. Telephone

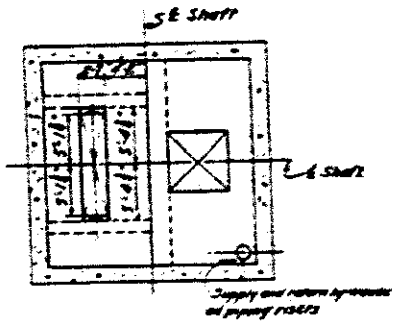
To provide communications with other project features during routine operations and maintenance activities, a telephone is installed in the shaft house.

PART V - CHAPTER III

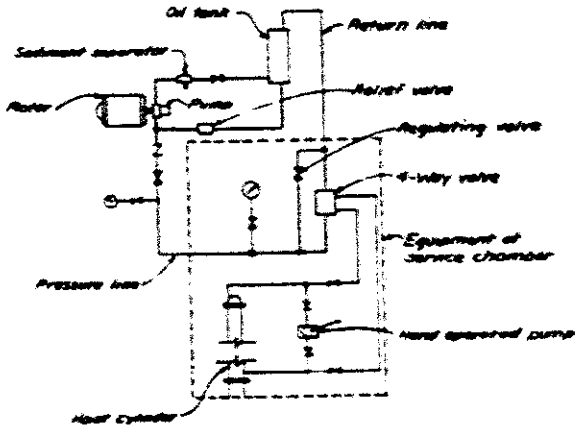
LIST OF EXHIBITS

Exhibit	Title
III-1	General Arrangement
III-2	Bulkhead Framing and Details
III-3	Bulkhead Design Analysis
III-4	Fixed-Wheel Gate Framing and Details
III-5	Fixed-Wheel Gate Design Analysis
III-6	Slide Gate Assembly
III-7	Trashrack Framing and Details


℄ Shaft
Sta 7+13.00
Hoist House
Hoist Monorail
Fixed Wheel Gate
Dropping Chamber
See Dwg FL-15-418
EL 126.00
Spillway Crest EL 110.25'
Location Line of Shaft
Service Hoist Anchors
℄ Tunnel & Service Hoist Anchors
℄ Slide gate
Hanger Anchors
SECTION F-F
Deck EL 136.00'
To Sump
To oil tank
SECTION Y-Y
℄ Slide Gate & Hanger Support - See Dwg. FL-15-413
Service Chamber for Slide Gate
Slide Gate 6'-6" x 7'-6"
See Dwg. FL-15-430
Air Vent Pipe
See Dwg. FL-15-413
Conduit Liner
See Dwg. FL-15-413
Bulkhead 16'-6" x 21'-0"
See Dwg FL-15-424
Fixed Wheel Gate 5'-6" x 18'-0"
See Dwg FL-15-420
Flow Line EL 26.10
℄ 20' H.S. Tunnel
Flow
℄ Outlet EL 19.27
EL 8.58
EL 8.58
13'-0"
13'-0"
60'
℄ Centrly Jt.
PROFILE ON ℄ TUNNEL
℄ Shaft & Tunnel
To oil tank
Floor pump
Spiral ladder
Air Vent
4-way valve
Top of bracket EL 36.00'
Valve bracket
Hand operated pump
EL 26.10'
SECTION X-X



SECTION D-D



SLIDE GATE HYDRAULIC SYSTEM PIPING DIAGRAM

NOTE TO FIELD:
For "as Fabricated" metalwork see manufacturer's drawings.

REFERENCE DRAWINGS:

Diversion & Outlet Tunnel
Bulkhead & Fixed Wheel Gate
Embedded Metal, Sheet 1 — FL-15-411
Embedded Metal, Sheet 2 — FL-15-412

Slide Gate 6'-0"x7'-6"
Misc. Embedded Metal — FL-15-413

Bulkhead & Fixed Wheel Gate
Fixed Wheel Gate 6'-6"x19'-0" - Framing & Details - FL-15-420
Fixed Wheel Gate 6'-6"x17'-0" - Misc. Details — FL-15-421
Fixed Wheel Gate 6'-6"x17'-0" Design Analysis — FL-15-422

Bulkhead 16'-6"x21'-0" Framing & Details — FL-15-424
Bulkhead 16'-6"x21'-0" - Reinforcing — FL-15-425
Bulkhead 16'-6"x21'-0" Design Analysis — FL-15-426

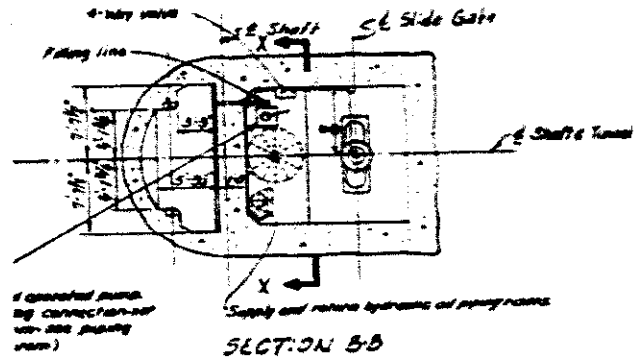
Slide Gate 6'-0"x7'-6"
Assembly — FL-15-430

Bulkhead 16'-6"x21'-0"
Proposed Construction & Installation Scheme — FL-15-[illegible]

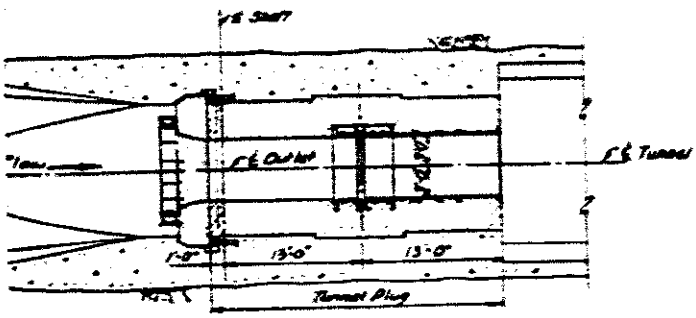
Transition & Chamber - Concrete Outline Stage I - FL-32-[illegible]
Concrete Outline - Stage II — FL-32-[illegible]

Gate Shaft - Concrete Outline Stage I — FL-32-[illegible]
Gate Shaft Building - Concrete Outline — FL-32-[illegible]

15 Ton Outlet Fixed-Wheel Gate Hoist — FL-15-[illegible]



SECTION B-B



SECTION A-A

Scale 1/8" = 1'-0"

THIS DRAWING REDUCED ONE-HALF

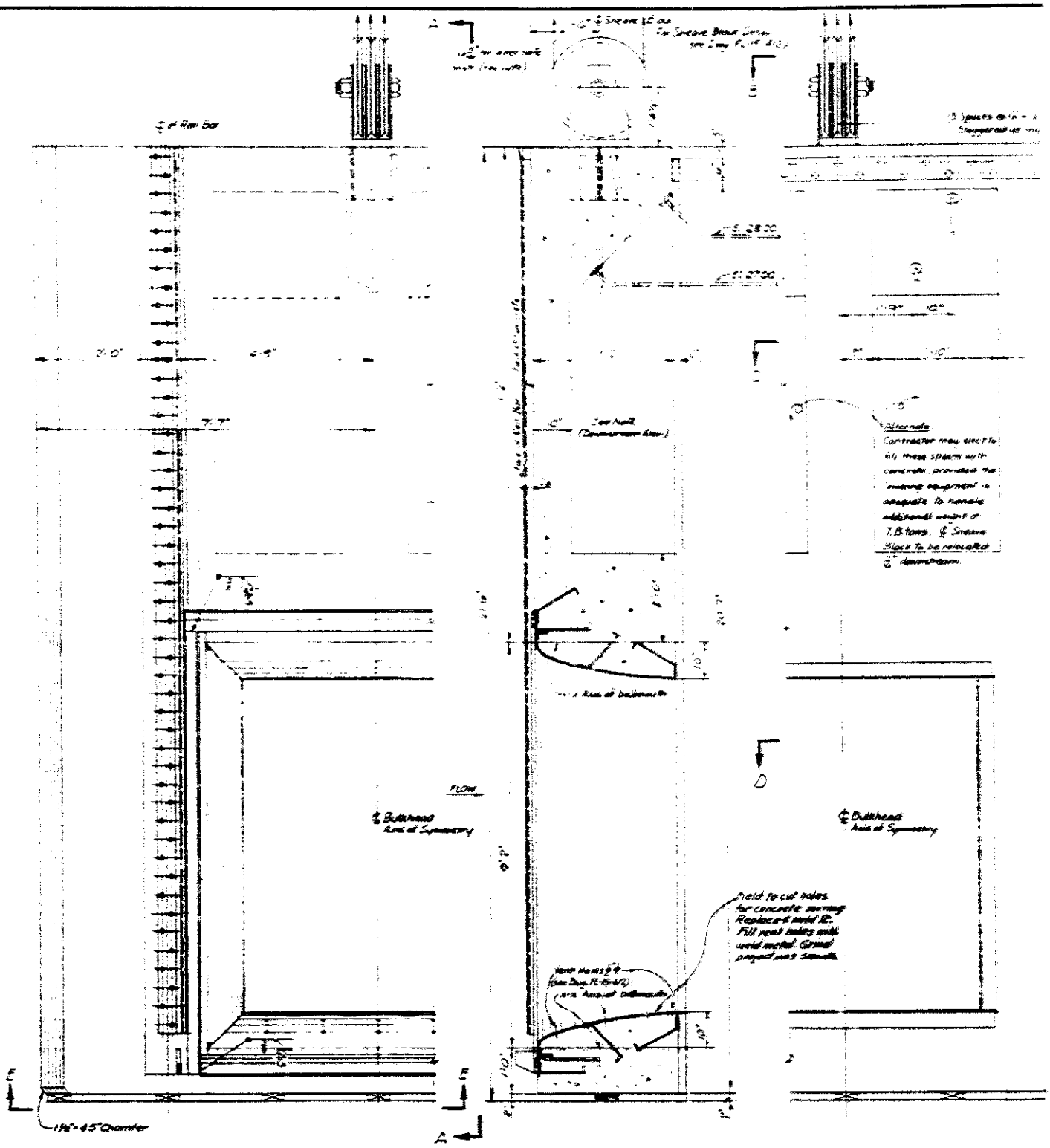
REVISIONS				
4				
3				
2				
1	Nov. 13 1958 RFC			
	Added note to Field			

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
DIVERSION & OUTLET TUNNEL
BULKHEAD, FIXED WHEEL & SLIDE GATES
GENERAL ARRANGEMENT

INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS

KA-X-1	SAN FRANCISCO, CAL. April 1, 1958	FL-15-410R1

17



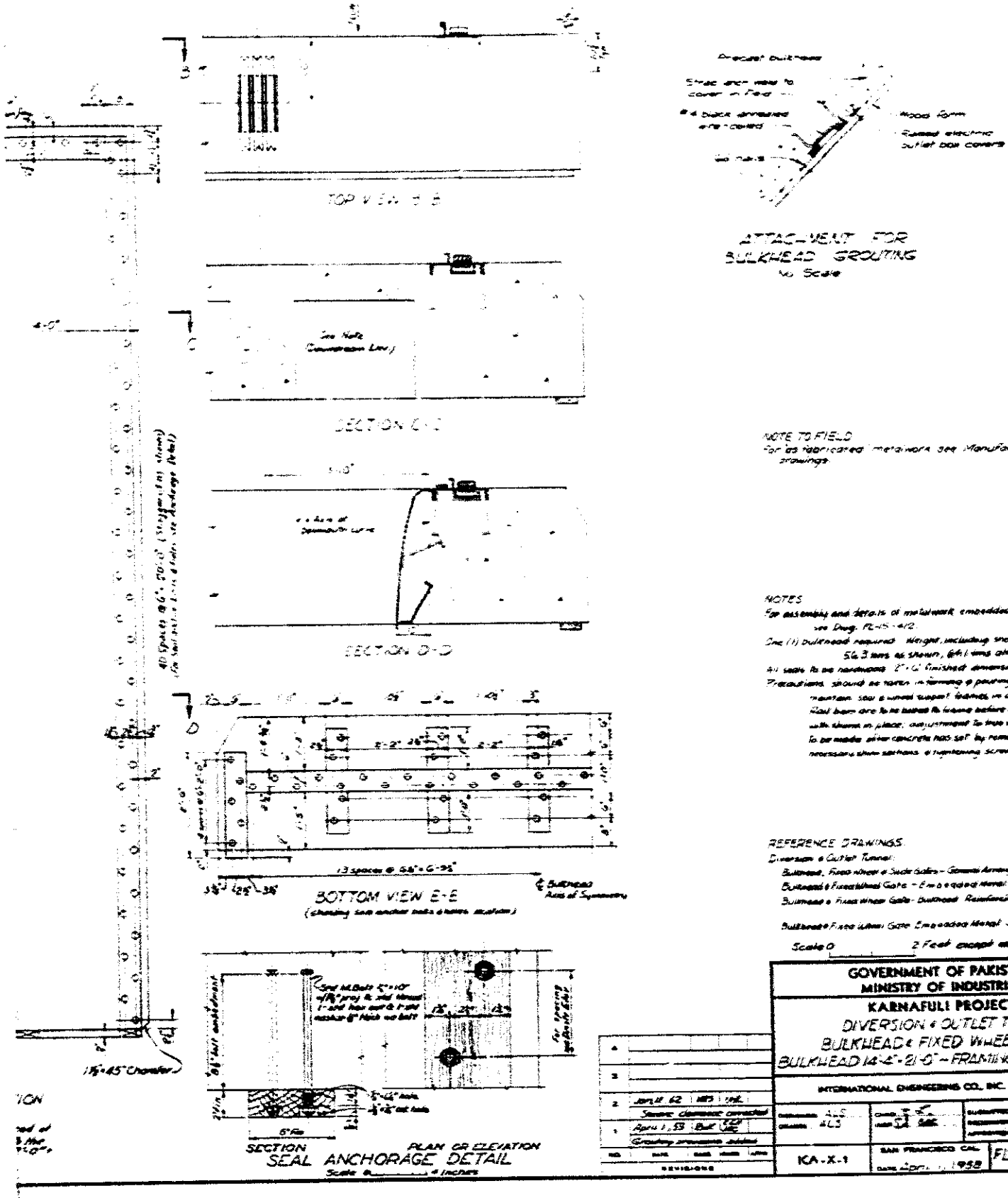
UPSTREAM ELEVATION

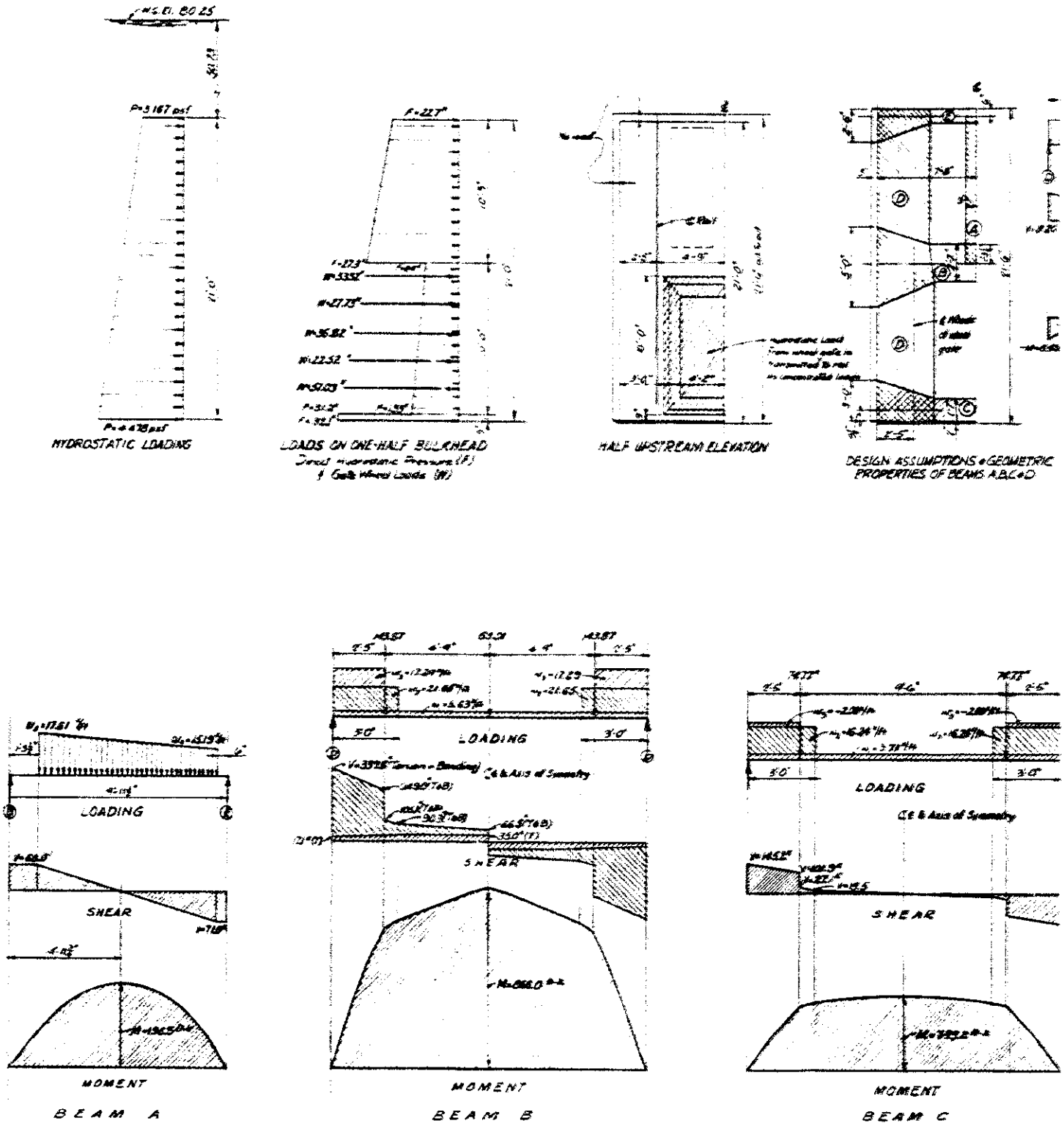
SECTION A-A

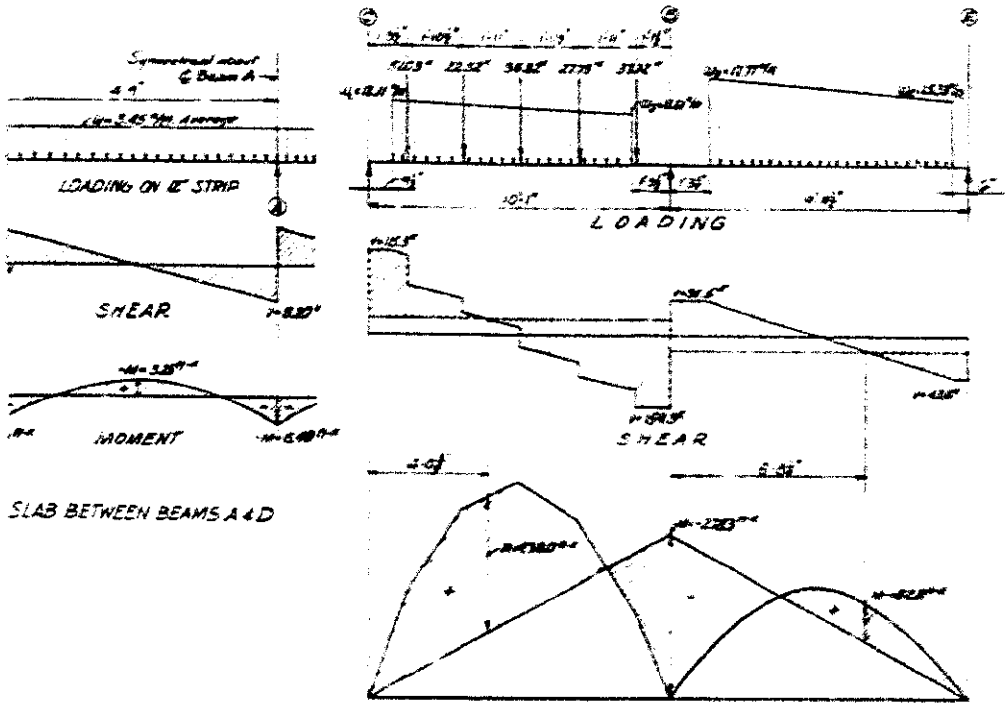
DOWNSTREAM ELEVAT

Construction Note: Concrete contractor to insert ⅝" ϕ coil ties or shear bolts, or equivalent, spac 1'-6" c.c. way in the downstream face of the bulkhead. Remove coil bolts or equivalent aft forms and replace after bulkhead has been lowered, with coil rods or similar shear, long, to serve as dowels for the tunnel plug concrete. Not required at 10" slab.

178


TOP VIEW B-B
SECTION C-C
See Note (Construction Law)
SECTION D-D
BOTTOM VIEW E-E
(showing sole anchor bolts & holes, alternate)
13 spaces @ 5'-6" = 6'-2"
℄ Bulkhead Axis of Symmetry
SECTION
PLAN OR ELEVATION
SEAL ANCHORAGE DETAIL
Scale 0 4 Inches
1½"x45° Chamfer
Precast bulkhead
Wood form
Cutout electric outlet box covers
ATTACHMENT FOR BULKHEAD GROUTING
No Scale
NOTE TO FIELD
For as fabricated metalwork see Manufacturer's drawings.
NOTES
For assembly and details of metalwork embedded in Bulkhead see Dwg. FL-15-412.
One (1) bulkhead required. Weight, including shoe block: 56.3 tons as shown, 64.1 tons alternate as noted.
All seats to be machined. 2'-6" finished dimensions.
Precautions should be taken in forming & pouring concrete, to maintain seal & wheel support faces in a true plane. Rail bars are to be tested to insure before concrete pour with shims in place. Adjustment to true vertical plane is to be made after concrete has set by removing or adding necessary shim sections & tightening screws.
REFERENCE DRAWINGS
Dwg. No.
Diversion & Outlet Tunnel:
Bulkhead, Fixed Wheel & Slide Gates – General Arrangement ... FL-15-410
Bulkhead & Fixed Wheel Gate – Embedded Metal Sh. 1 ... FL-15-411
Bulkhead & Fixed Wheel Gate – Bulkhead Reinforcing ... FL-15-412
Bulkhead & Fixed Wheel Gate – Embedded Metal Sh. 2 ... FL-15-412
Scale 0 2 Feet except as noted
GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
DIVERSION & OUTLET TUNNEL
BULKHEAD & FIXED WHEEL GATE
BULKHEAD 14'-4" - 21'-0" – FRAMING & DETAILS
INTERNATIONAL ENGINEERING CO., INC. – ENGINEERS
KA-X-1
SAN FRANCISCO, CAL.
1958
FL-15-424 R2
REVISIONS


HYDROSTATIC LOADING
LOADS ON ONE-HALF BULKHEAD
HALF UPSTREAM ELEVATION
DESIGN ASSUMPTIONS & GEOMETRIC PROPERTIES OF BEAMS A,B,C & D
LOADING
SHEAR
MOMENT
BEAM A
BEAM B
BEAM C
LOADING, SHEAR & MOMENT DIAGRAMS



LOADING, SHEAR & MOMENT DIAGRAMS

DESIGN DATA:

Bulkhead is to allow river diversion [illegible] passes spillway crest elev. 8025. [illegible] gate is lowered and the hinge plug [illegible] the latter becoming part of main concrete.

Maximum allowable stresses:

Concrete compressive strength at 28 days [illegible]
Steel tensile unit stress: 20,000 p.s.i.
All other appurtenant stresses in accordance with ACI Building Code ACI 318-56

DESIGN ASSUMPTIONS:

Two design assumptions have been considered:

(a) Beams B & C are supported by beams D which in turn are continuously supported along the cantilevered [illegible]

(b) Beams D are continuous two-span beams [illegible] beams C, B & E.

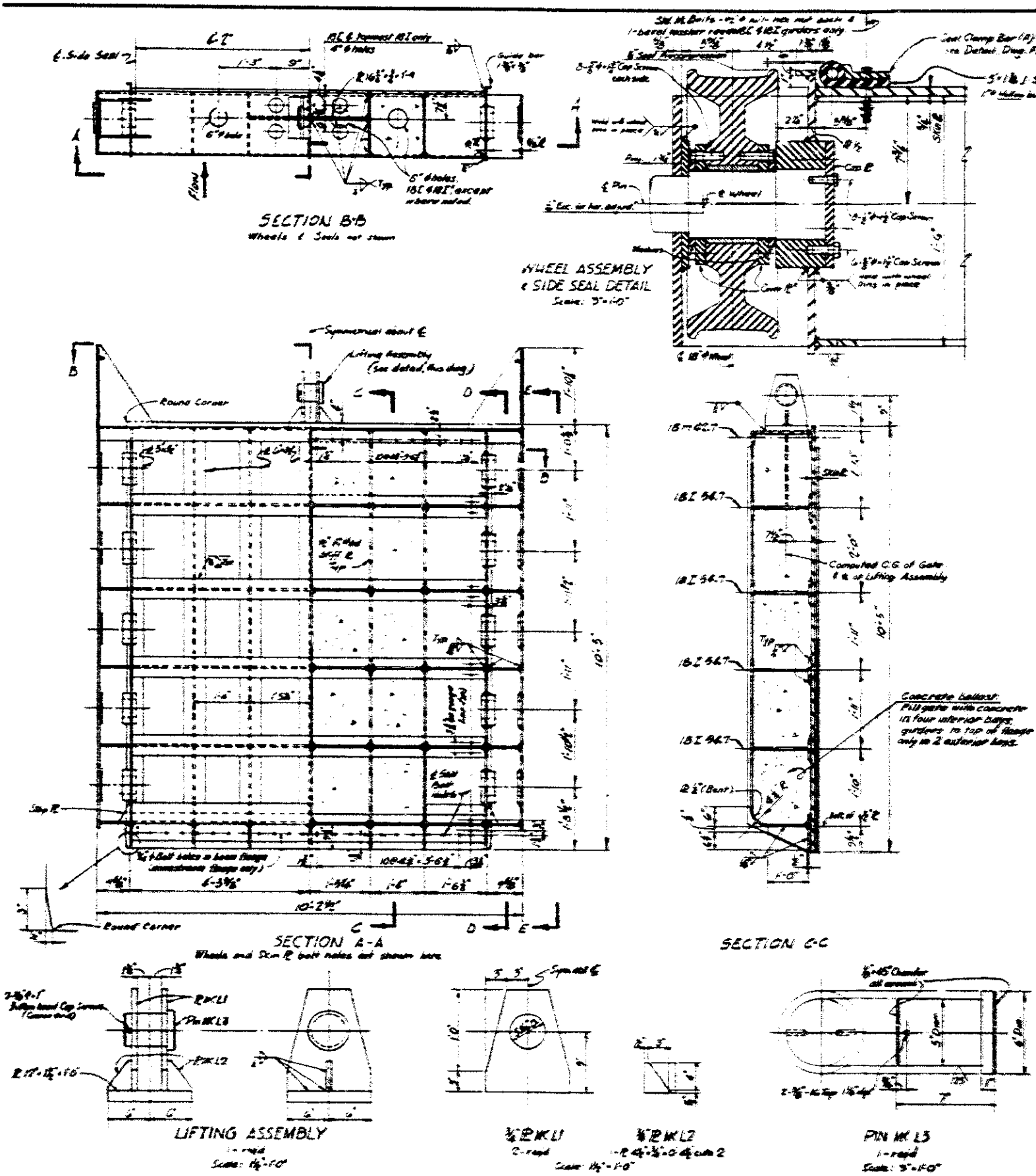
The assumption (b) appears to be more realistic [illegible] C, B & E are short and possess unyielding supports at their ends. The presented design is therefore based on the method (b).

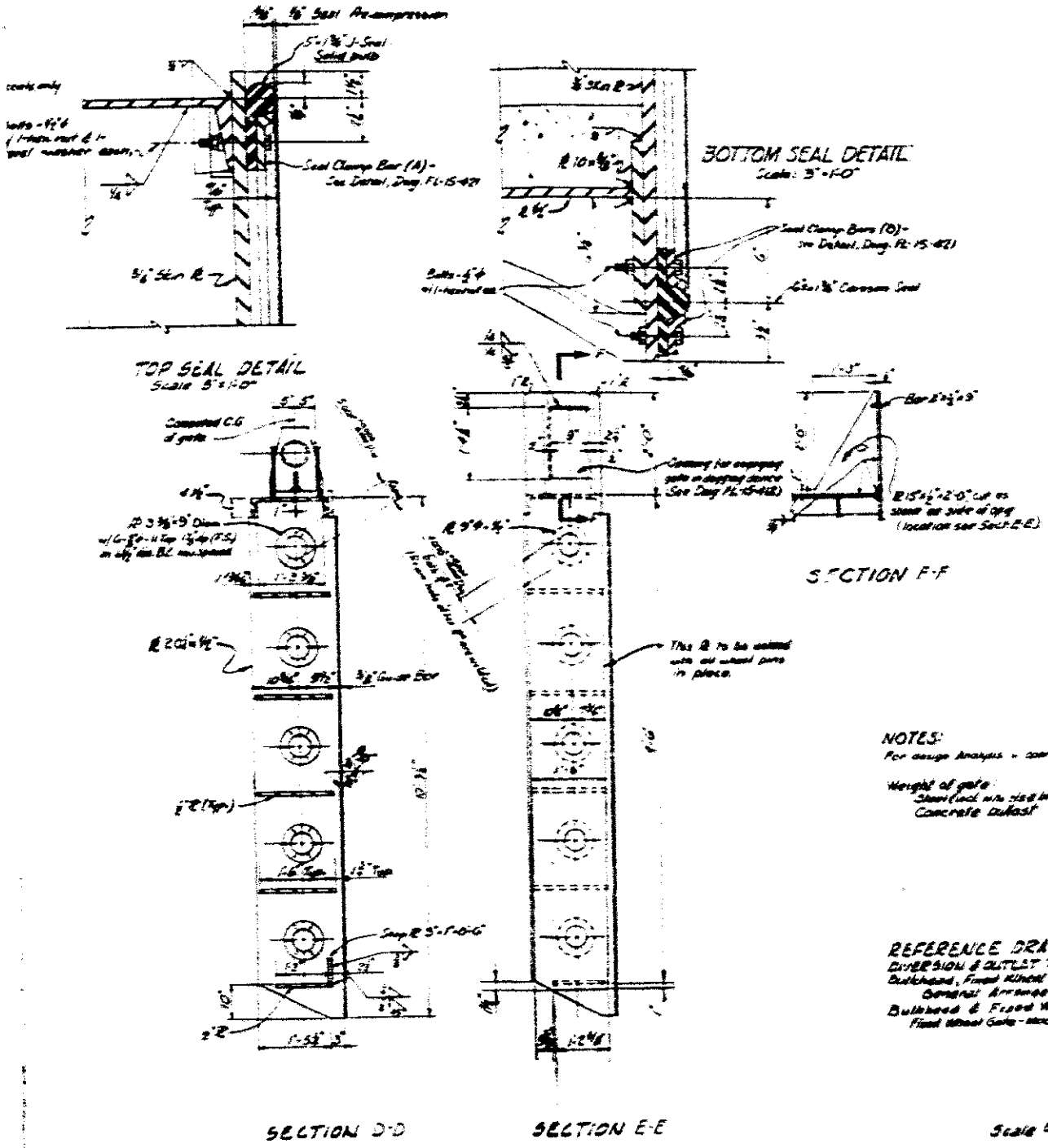
REFERENCE DRAWINGS:

Diversion & Outlet Tunnel
Bulkhead Fixed Wheel & Slide Gates
General Arrangement — — — — — — F2-5-8.3

Bulkhead & Fixed Wheel Gate
Bulkhead 14'-4"×21'-0" Framing & Details — — F2-15-434
Bulkhead 14'-4"×21'-0" Reinforcing — — — F2-5-435

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
DIVERSION AND OUTLET TUNNEL
BULKHEAD AND FIXED WHEEL GATE
BULKHEAD 14'-4"×21'-0" – DESIGN ANALYSIS
INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS
KA-X-1
SAN FRANCISCO, CAL.
DATE April [illegible] 1958
F2-15-436


SECTION B-B
Wheels & Seals not shown
WHEEL ASSEMBLY & SIDE SEAL DETAIL
Scale: 3"=1'-0"
Round Corner
Lifting Assembly (See detail this dwg.)
Computed C.G. of Gate & of Lifting Assembly
Concrete ballast: Fill gate with concrete in four interior bays; girders to top of flange only in 2 exterior bays.
SECTION A-A
Wheels and Skin PL bolt holes not shown here
SECTION C-C
LIFTING ASSEMBLY
1-reqd
Scale: 1½"=1'-0"
PIN WK L3
1-reqd
Scale: 3"=1'-0"



NOTES:

For design Analysis & operating conditions see Dwg FL-15-422

Weight of gate	Dry	Submerged
Steel (incl. w/o side & lifting assembly)	15,200#	13,800#
Concrete Ballast	11,200#	6,300#
Total	26,400#	19,500#

REFERENCE DRAWINGS:

DIVERSION & OUTLET TUNNEL
Bulkhead, Fixed Wheel & Slide Gate
General Arrangement FL-15-410
Bulkhead & Fixed Wheel Gate
Fixed Wheel Gate - Miscellaneous Details FL-15-421

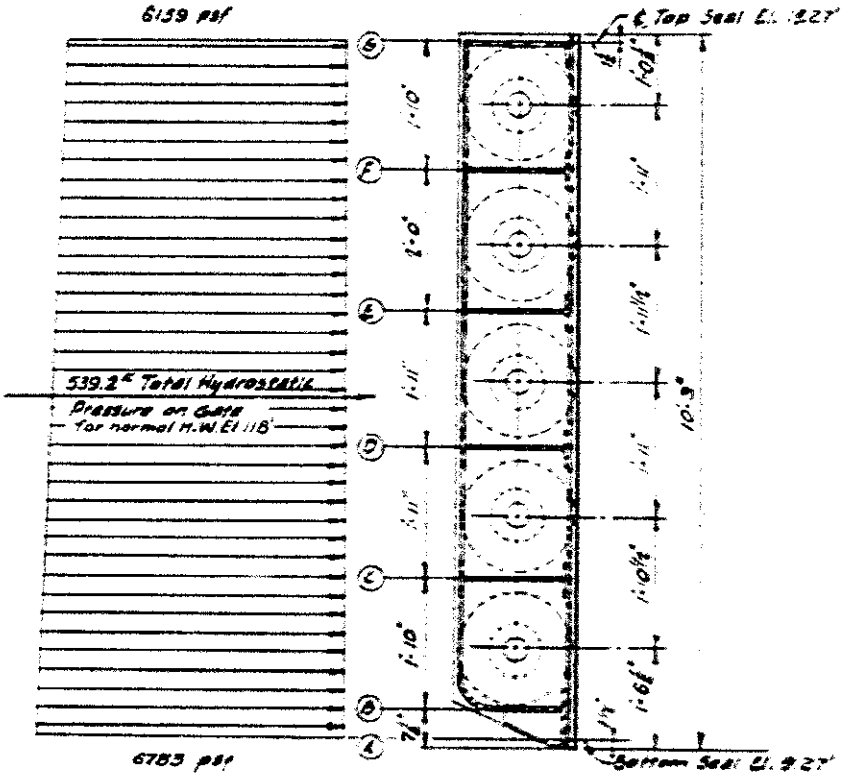
Scale 3/4" = 1'-0" Unless Shown

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
DIVERSION & OUTLET TUNNEL
BULKHEAD & FIXED WHEEL GATE
FIXED WHEEL GATE 5'-4" x 10'-0" FRAMING & DETAILS

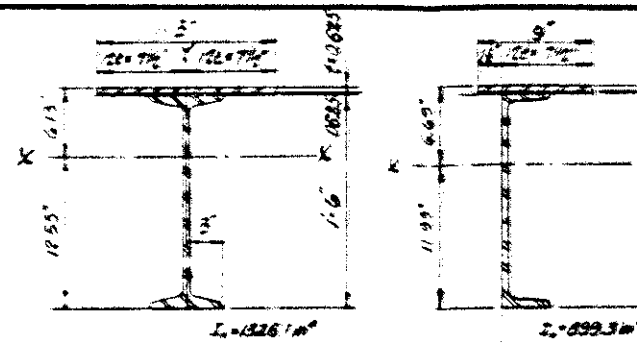
INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS

ALS RLD

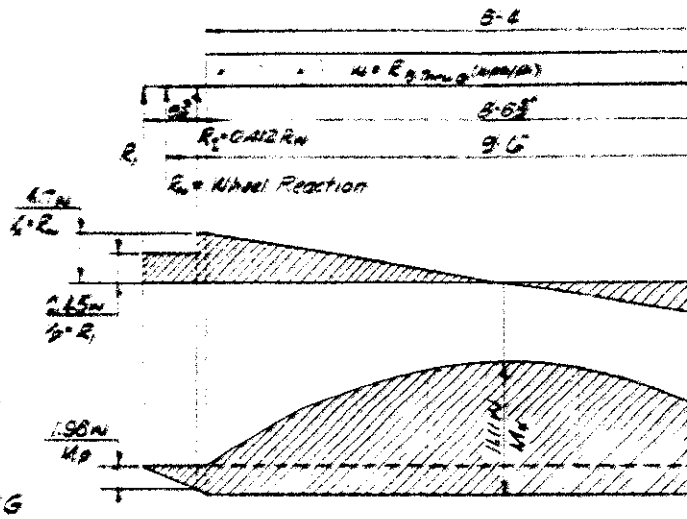
KA-X-1 | SAN FRANCISCO, CAL. | FL-15-420



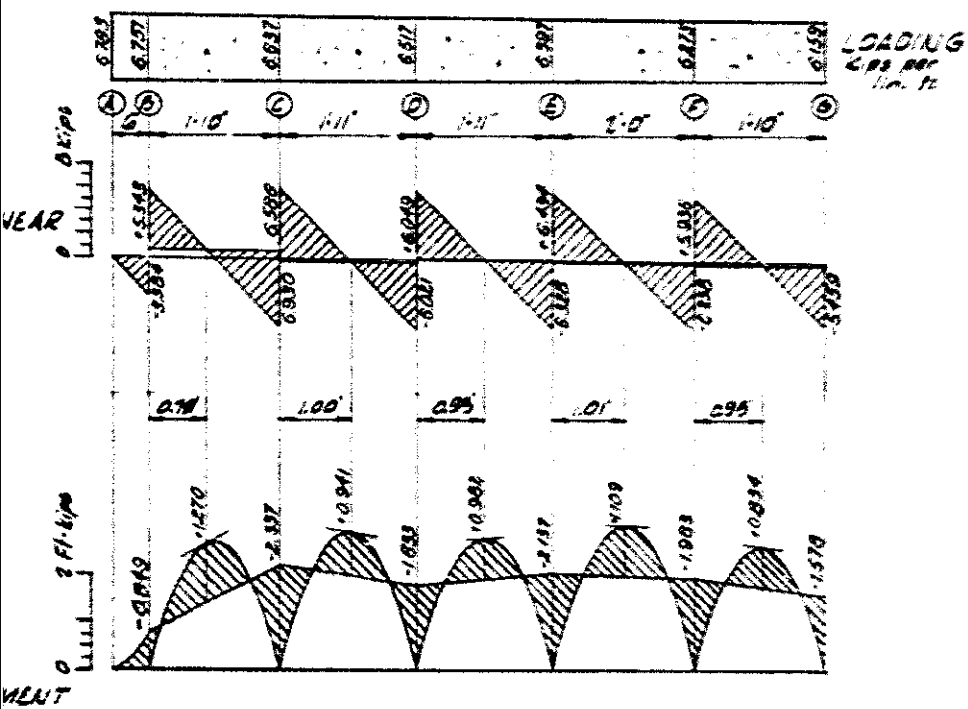
HYDROSTATIC PRESSURE DIAGRAM



PROPERTIES OF COMBINED SECTIONS OF HORIZONTAL BEAMS



NORMAL TO X-X AXIS



SHEAR & MOMENT DIAGRAMS
Scale 0 Feet

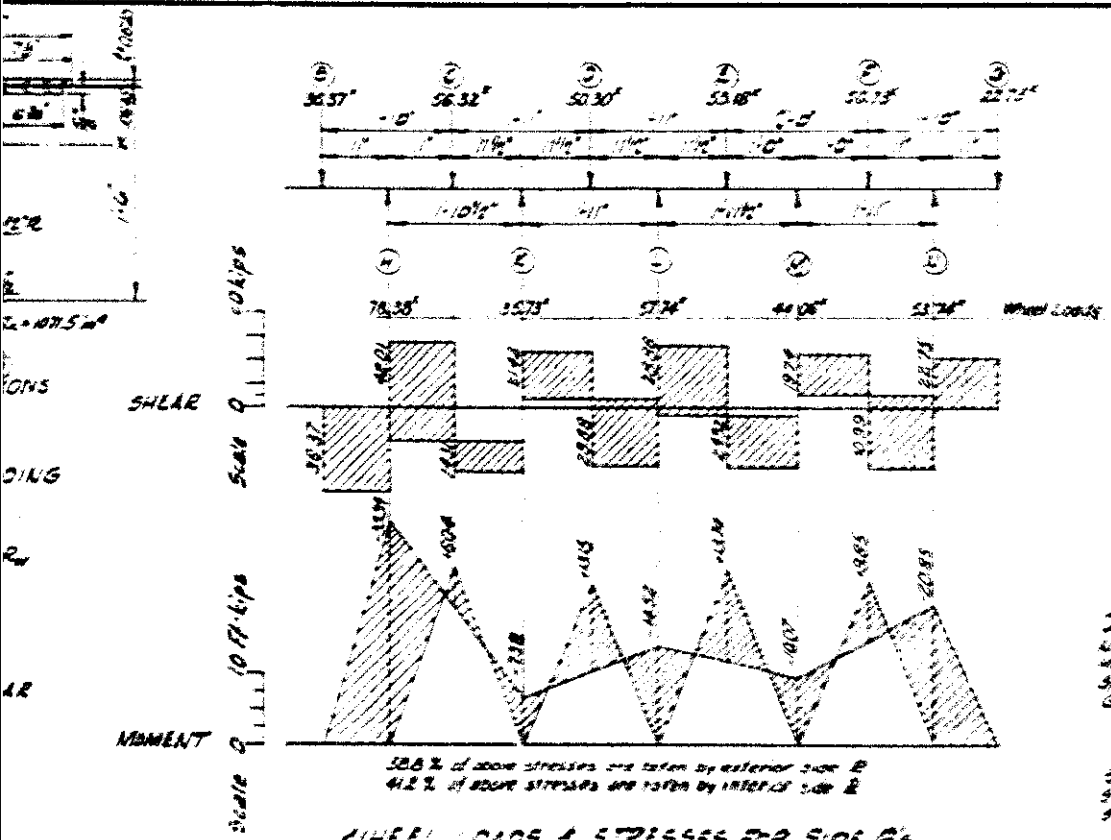
Girder	W	$R_W = V_A$ 4.17 W	$V_W = R_L$ 2.45 W	M
Girder C (Most Interior Girder)	13.52	56.52	33.12	
Girder G (Top Girder)	5.46	22.75	13.38	
Girder B (Bottom Girder)	8.79	36.57	21.39	
Girder D	12.07	50.30	Reactions	
Girder E	12.76	53.18	Moments	
Girder F	12.17	50.73	Loads	

GIRDER STRESSES
Scale 0 Feet

SKIN & STRESSES

BY MOMENT DISTRIBUTION BASED ON VERT. STRIP 1'-0" WIDE

Skin & is considered as continuous haunched beam. For simplification of calculations all haunches have been made of same depth & width. The resulting error for the exterior supports is small & on the safe side. The top support, S, is considered fully fixed rather than partially so, on the basis of the relative stiffness imparted to it by the series of 8" stiffener &'s spanning vertically between girders which tend to prevent rotation of the top girder. Calculation of distribution & carryover factor was made by method presented in Engineering Monograph #10 of the U.S. Bureau of Reclamation (October 1942). Skin & thickness = 3/8" for design (see note on corrosion allowance).



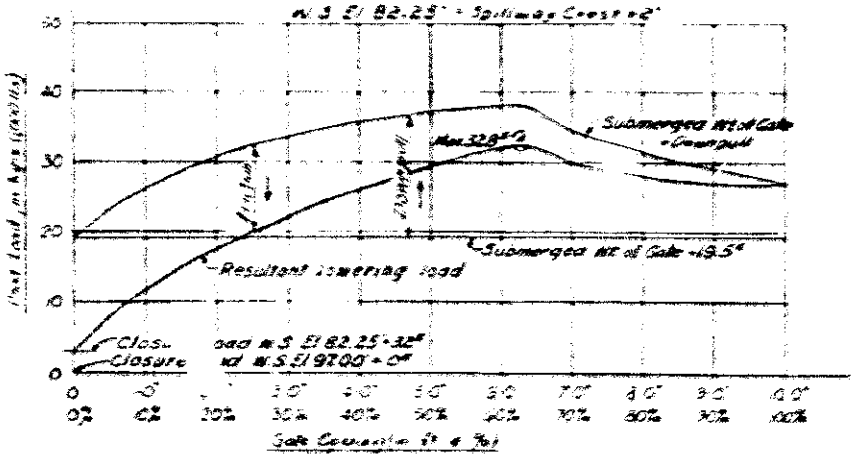
WHEEL LOADS & STRESSES FOR SIDE B's
Scale 0 ____ feet

DESIGN DATA:
Normal High Water Elevation 118'
Max High Water Elevation 119'
Spillway Crest Elevation 80.25'
Gate designed for following maximum stresses for structural steel (ASTM designation A-7):

	Normal W.S. El. 118' (90% Allowable)	Max W.S. El. 119' (33⅓% Over normal)
Bending	18,000 psi	24,000 psi
Shearing	11,700 psi	14,000 psi

Since the stresses for normal W.S. El. 118' have been found critical in all cases by inspection, no details are given on this dwg. for the loads from max. W.S. El. 119'.

A corrosion allowance of 1/16" for faces exposed alternately to water and air has been made for skin plate and side plates.



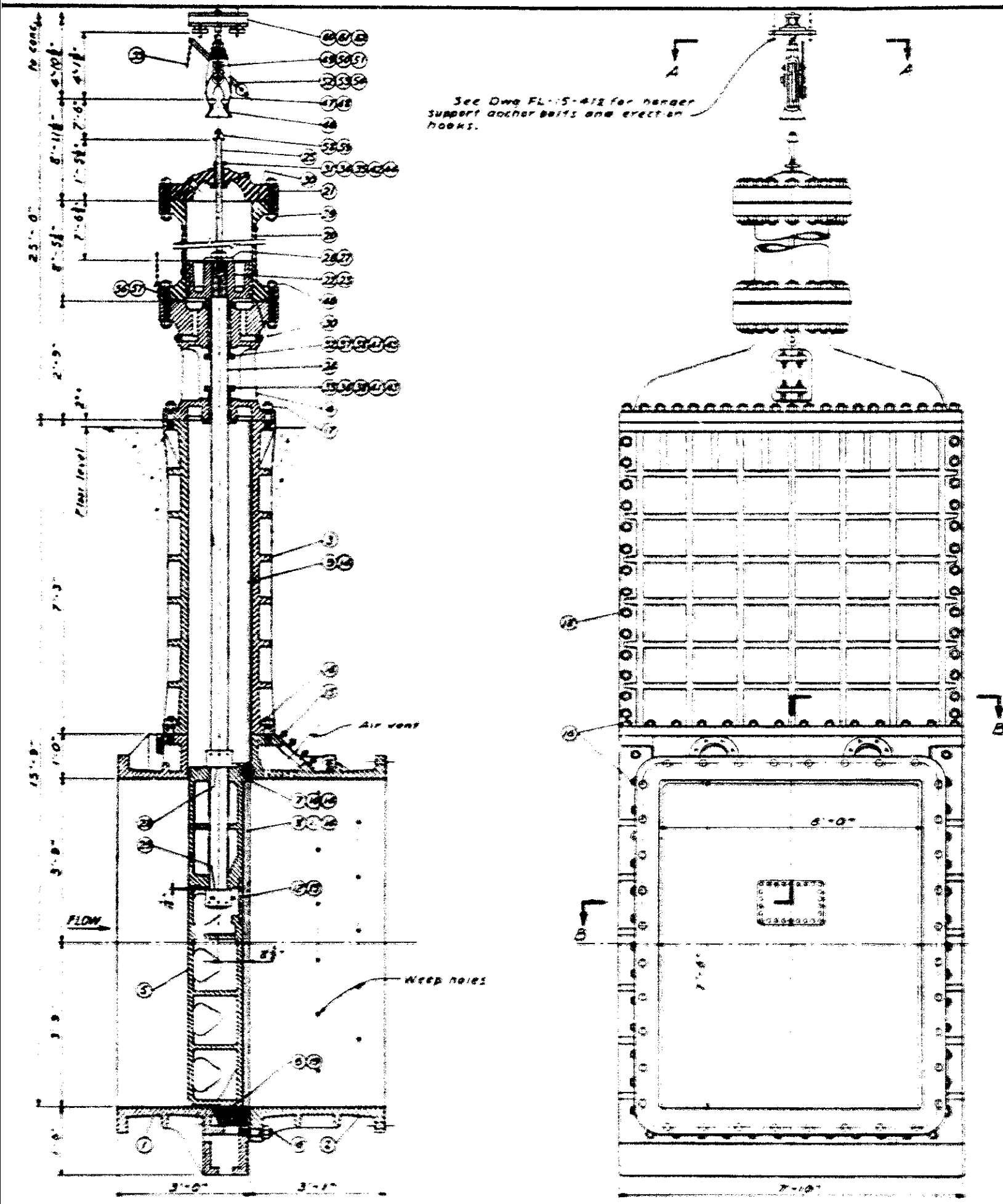
CLOSURE & INITIAL LOWERING LOADS

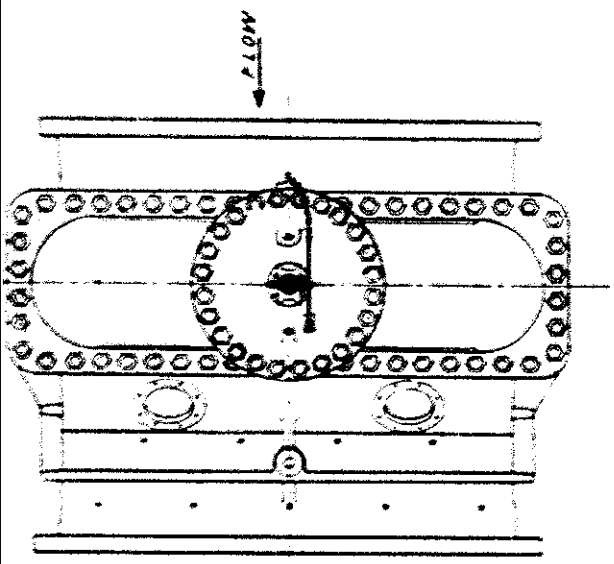
Initial lowering of gate by construction equipment. Subsequent raising or lowering under permanent hoist (side gate closed). In case slide gate is jammed open the gate can be lowered against a pool elev. of 97.00' max. The permanent hoist has a capacity of 15 tons adequate to handle the max. pull under that emergency.

Initial lowering of gate to occur after pool level reaches spillway crest elev. 80.25'. It has been assumed that pool will have risen an additional 2'-0" by the time gate is lowered. Hence loads in diagram above given for W.S. El. 82.25 except where noted.

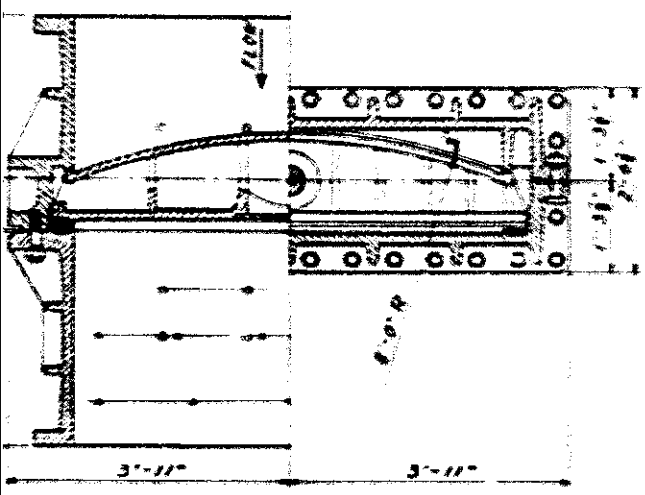
REFERENCE DRAWINGS:
Diversion & Outlet Tunnel
Bulkhead, Fixed Wheel & Slide Gates
General Arrangement — — — — — — — — FL-15-410
Bulkhead & Fixed Wheel Gate
Fixed Wheel Gate 6'-4"x10'-0" - Framing & Details — FL-15-420
15 Ton Outlet Fixed-Wheel Gate Hoist — — — FL-15-401

GOVERNMENT OF PAKISTAN	
MINISTRY OF INDUSTRIES	
KARNAFULI PROJECT	
DIVERSION & OUTLET TUNNEL BULKHEAD & FIXED WHEEL GATE FIXED WHEEL GATE-DESIGN ANALYSIS	
INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS	
MLS / ELD	


See Dwg FL-5-412 for hanger support anchor bolts and erect on hooks.
Air vent
Weep holes
FLOW
Floor level
8'-0"



PLAN A-A



SECTION B-B

LIST OF PARTS FOR ONE GATE ASSEMBLY

	PART No.	No. REQD	DESCRIPTION	MATERIAL	ASTM DESIGNATION except as noted
GATE	1	1	Upstream frame	Cast iron	A48, Class 30
	2	1	Downstream frame	do.	do.
	3	1	Bonnet	do.	do.
	4	1	Bonnet cover	do.	A48, Class 40
	5	1	Gate leaf	do.	A48, Class 30
	6	1	Gate slit	Cast iron & babbitt	A48, Cl 30 & B23, Gr 3
	7	1	Horizontal leaf seat	Cast bronze	B145, No. 4A
	8	2	Vertical leaf seat	do.	do.
	9	2	Bonnet seat	do.	B143, No. 2B
	10	1	Horizontal frame seat	do.	do.
	11	2	Vertical frame seat	do.	do.
	12	1	Leaf recess cover	Structural stee.	A7
	13	26	⅝"-2" Flat hd capscrew	Bronze	B138, Type A, half hd, cold
	14	89	1"-4½" do	do.	do.
	15	16	⅝"-3¾" Stud	Bolt steel	A107, Grade 1020
	16	72	1⅛"-7¾" Stud	do.	do.
	17	50	1⅛"-7⅞" Stud	do.	do.
	18	33	1⅛"-7⅞" Bolt	do.	do.
	19	3	1"-4¾" Bolt	do.	do.
HYDRAULIC HOIST	20	1	Cylinder	Forged steel	See dwg. FL-15-43
	21	1	Cylinder head	Cast steel	A27, Grade 65-35
	22	1	Piston	Cast iron	A126, Class B
	23	4	Piston ring	Cast mang. bronze	B147, No. 8B
	24	1	Stem	High tensile brz.	B138, Type B, half hard
	25	1	Stem extension	Rolled bronze	B103, Gr A, half hard
	26	1	Locking key	Steel	S.A.E. 1120
	27	1	Key retainer	Cold rolled steel	A108, Grade 1025
	28	1	Top leaf nut	Cast mang. bronze	B147, No. 8B
	29	1	Bottom leaf nut	do.	do.
	30	2	Cylinder gasket	Copper	B152, Type ETP Lt. cold-rolled
	31	1 Set	1¼" Oil packing	Fabric	See notes, dwg FL-15-43
	32	1 Set	4⅝" Oil packing	do.	do.
	33	1 Set	4⅝" Water packing	do.	do.
	34	1	1½" Packing seat	Cast bronze	B144, No. 3B
	35	1	1½" Packing gland	do.	do.
	36	1	4⅝" Packing seat	do.	do.
	37	1	4⅝" Packing seat	do.	do.
	38	2	4⅝" Packing gland	do.	do.
	39	24	1½" Stud	Bolt steel	A107, Grade 1045
	40	24	1½" Stud	do.	do.
	41	8	⅝" Stud	Rolled bronze	B103, Gr A, Half hard
	42	2	⅜" Stud	do.	do.
	43	8	Fillister hd capscrew	do.	B138, Type A, half hard cold
	44	1	do.	do.	do.
	45	1	Spanner wrench	Forged steel	A235, Class C
HANGER	46	1	Hanger body	Cast iron	A48, Class 40
	47	2	Hanger hook	Cast steel	A27, Grade 65-35
	48	1	Hook pin	Cold rolled steel	A108, Grade 1025
	49	1	Cam	Cast steel	A27, Grade 65-35
	50	1	Cam shaft	Rolled bronze	B103, Gr A, half hard
	51	1	15/16" I.D. Washer	Cold rolled steel	A108, Grade 1025
	52	1	Lever	Structural steel	A7
	53	4	Counter weight	Lead	B29
	54	1	⅝"-4" Machine bolt	Steel	Commercial
	55	1	Chain & spring	Steel spring wire	A68 (Galvanized)
	56	1	Chain hook	Structural steel	A7
	57	2	½"-1" Capscrew	Steel	Commercial
	58	1	Safety stud cap	Cold rolled steel	A108, Grade 1025
	59	4	Safety stud	Rolled bronze	B103, Gr A, half hard
	60	1	1⅛"-3'-9" Bar	Bolt steel	A107, Gr 1020
	61	1	Hanger support beam	Structural steel	A7
	62	2	Hanger support base	do.	A7

DESIGN DATA

Iron gate leaf class 30 – Maximum designed head 110 feet.
Crete surrounding frames & bonnet castings designed to carry all the load.
Iron bonnet cover class 40 – Maximum designed head 110 feet
to 5 – 2 seats – Coefficient of friction 0.6 (starting)
Hydraulic hoist cylinder – Working oil pressure 750 p.s.i.
– Designed oil pressure 1000 p.s.i.

SHOP NOTE

screws 14 and bronze seats 7, 8, 9, 10, and 11 in place
ole screws with 4 center punch scores in head. After
is assembly, make final machining on sliding faces of
bronze seats which must be in a true plane.

FIELD NOTE

assembling for installation, the finished faces of
flanged joints are to be smoothly coated with a
ixture of white lead and graphite and bolted
together while coating is plastic. The shanks and
threads of all bolts and studs are to be similarly
coated.

REFERENCE DRAWINGS:

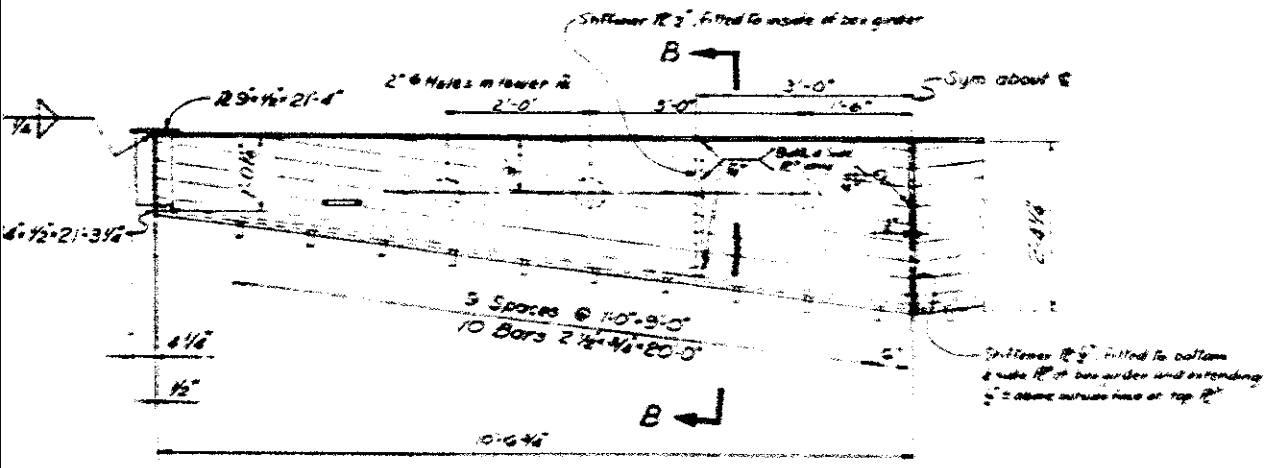
Diversion and Outlet Tunnel –
Bulkhead, Fixed wheel gate, & Slide gate – General Arrgt – FL-15-4
Slide Gate 6'-0" x 7'-6" – Misc. Embedded Metal – FL-15-4
do. – Upstream Frame – FL-15-4
do. – Downstream Frame – FL-15-4
do. – Leaf, Recess Cover, & Bolts – FL-15-4
do. – Bonnet, Gate Slit & Seats – FL-15-4
do. – Bonnet Cover – FL-15-4
do. – Hoist Cylinder, Pr. & Stems – FL-15-4
do. – Hoist Packing Glands & Studs – FL-15-4
do. – Jaw-Automatic Gate Hanger – FL-15-4

Scale: ¾" = 1'-0"

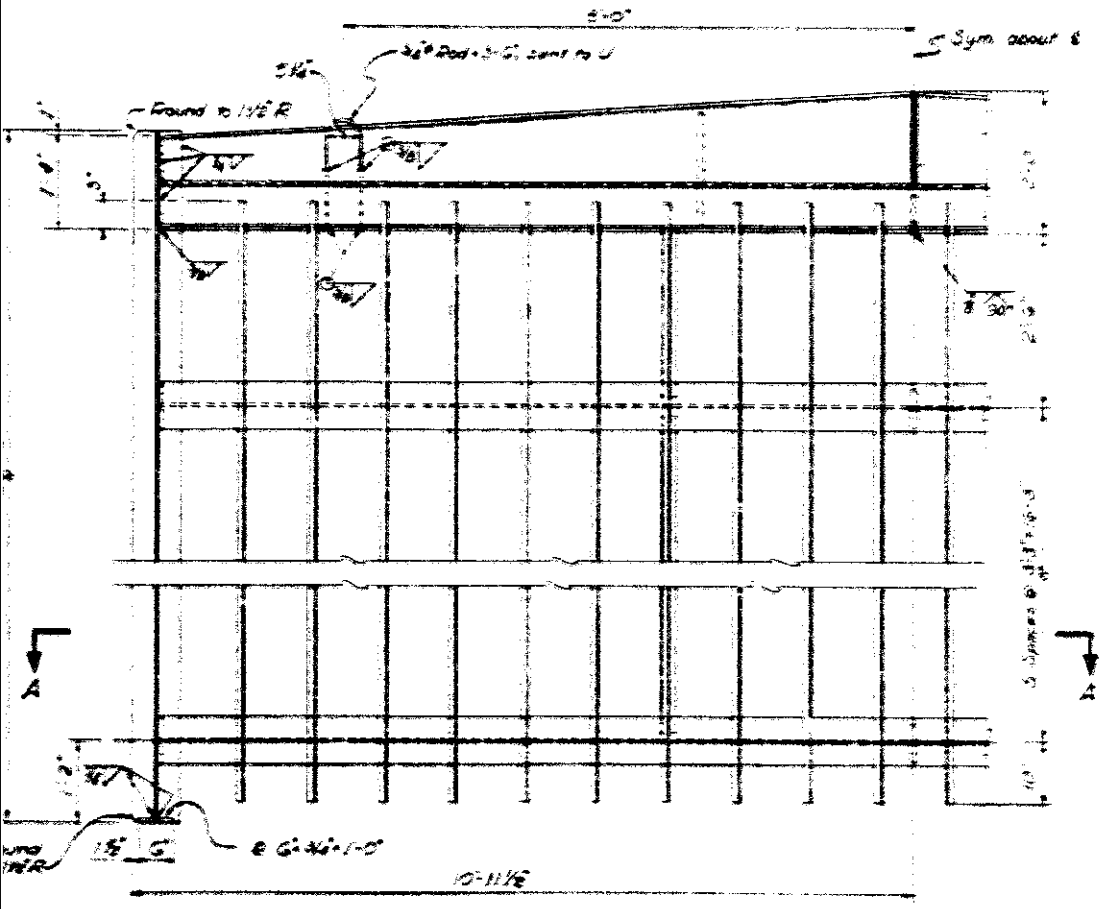
GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
DIVERSION AND OUTLET TUN
SLIDE GATE 6'-0" x 7'-6"
ASSEMBLY

INTERNATIONAL ENGINEERING CO., INC. - ENGINEER

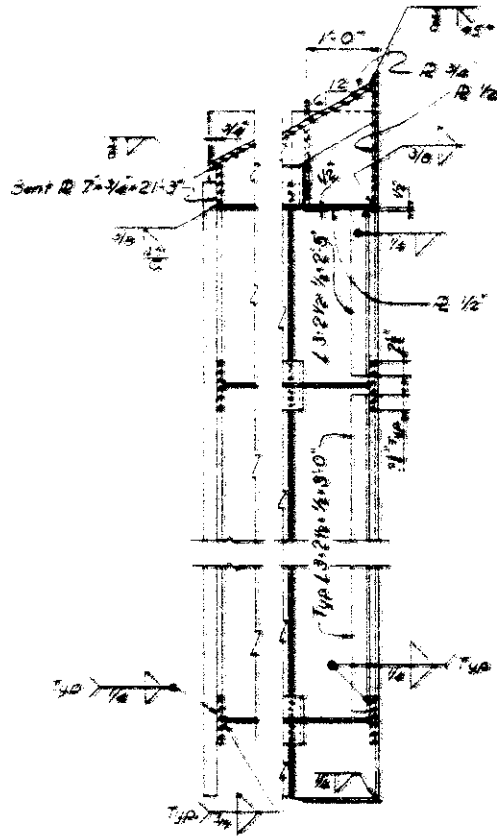
L.E.Co.	AC	
AC		



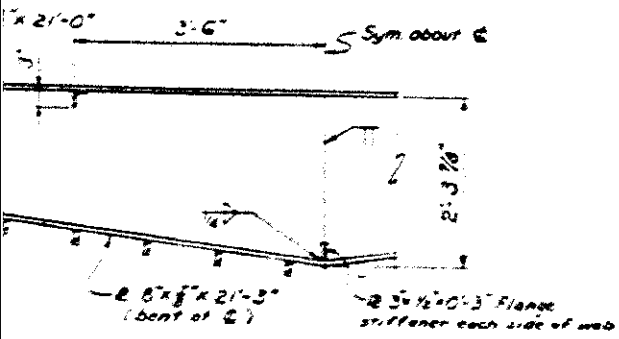
PLAN



UPSTREAM ELEVATION



SECTION B-B



A-A

ms
der at top

NOTES:
Trashracks frame designed for 40' differential head at a stress of 33,000 psi.
Trashrack bars designed for 35' differential head at a stress of 33,000 psi.
No corrosion allowance is made.
Estimated weight of trashrack - 19,000 lbs.

REFERENCE DRAWINGS:
Miscellaneous Metalwork - Sheet 2 ... FL-32-902

Scale: 3/4"=1'-0"

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
DIVERSION & OUTLET TUNNEL
TRASHRACK
FRAMING & DETAILS

INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS

ALS
R.D.
C.A.K.

SAN FRANCISCO, CAL.

NO.	DATE	REVISED	CHECKED	APPROVED
4				
3				
2				
1				

CHAPTER IV

POWERHOUSE

4.01 GENERAL

This chapter covers all of the electrical and mechanical equipment and systems installed in the powerhouse. Information given includes equipment description, design capacities and functional description.

4.02 ELECTRICAL EQUIPMENT AND SYSTEMS

The electrical installation for the Karnafuli Project powerhouse has been designed on the basis of the unit system, with each generator and main power transformer bank operating as an electrical unit. Each unit is connected through a high voltage oil circuit breaker to a common high voltage bus. Generation is at 11 kilovolts and high voltage transmission is at 132 kilovolts.

Generation voltage was chosen to meet local standards, as there will be some power distribution to associated project features at this voltage. Transmission voltage was determined by necessity to match the high voltage transmission system from Chittagong to Dacca presently under construction.

Installation follows the modern principles and practices being used in the United States for equivalent projects and is intended to be as economical as practicable. The manufacture, rating, installation, testing, and operation of all electrical materials, machinery and equipment is in accordance with United States standards applicable to this type of equipment.

The paragraphs which follow in this section present pertinent information pertaining to the machinery, equipment and materials to be used at the Project. Exhibits IV-1, IV-2, and IV-3 are single line diagrams for main, auxiliary, and miscellaneous power.

A. Generators and Auxiliary Equipment

1. Generators - The initial two hydraulic turbine-driven vertical shaft alternating-current generators are each rated 50,000 kilovolt-amperes at 0.8 power factor, 11 kilovolts, 3-phase, 50 cycle per second, 107 revolutions per minute units with direct-connected main and pilot exciters, neutral grounding transformers, excitation and voltage regulation cubicles, gage boards, and other necessary appurtenances as further described in this chapter. The generators are self-ventilating with a closed system of aircooling, utilizing water air coolers. The generators are provided with carbon dioxide fire protection equipment in accordance with modern practice for units of this size.

The generators are equipped with the usual protective and indicating devices, such as detectors and alarms for winding temperature, bearing temperature, bearing oil temperature and level, air cooler and bearing oil cooler water flow, and exciter air temperature. Also relay protection and instrumentation are provided as described under C-1, Main Control Switchboard.

2. Excitation and Voltage Regulation Equipment - The main and pilot exciters are integral parts of each main generator, mounted above the main generator rotors. Each main exciter is rated 250-volts direct current, will be shunt wound, with interpoles, and will be excited from pilot exciters. Each main exciter is provided with a field circuit breaker and a field discharge resistor. The pilot exciters are 250-volt direct current, flat-compound wound, self-exciting. The exciters are operated in conjunction with automatic voltage regulating equipment.

The voltage regulation equipment is the automatic, rheostatic type of latest design, capable of properly regulating the generator voltage for all generator operating requirements and conditions. The equipment is installed in totally enclosed switchgear type cubicles with control equipment furnished for mounting on the main control switchboard.

3. Generator Neutral Grounding - Each generator has neutral grounding transformers with secondary resistors as required, to limit the severity of ground faults in the generator winding. The neutral grounding equipment is mounted in totally enclosed sheet steel cubicles. The transformers are single-phase, oil-filled distribution type, rated 11-kilovolt - 120/240 volts, 50 cycles alternating current.

4. Generator Main Connections - In order to conserve space and provide adequate current carrying capacity and safety, isolated phase bus duct is used for the 11-kilovolt connection from the generator via the main terminal equipment to the outdoor type 11-kilovolt generator disconnecting switches adjacent to the powerhouse wall.

The generator terminal equipment, consisting of surge protecting capacitors and lightning arresters, potential transformers, and current transformers, is provided in isolated type steel enclosures matching the main bus enclosures.

Standard outdoor type construction is used for the main bus from the 11-kilovolt generator disconnecting switches to the main power transformers, station service transformers and 11-kilovolt distribution line oil circuit breakers.

B. Transformers and High Voltage Equipment

1. Main Power Transformers - The main power transformers are single-phase, two-winding, oil-immersed, forced air-cooled, outdoor, 14,400/19,200 kilovolt-amperes, 0.8 power factor, 50 cycle, 11-kilovolt delta to 76/132 kilovolts grounded wye. The transformers are connected in three-phase banks with one spare unit. Single-phase transformers are used in preference to three-phase units because of their reliability and to

overcome transportation problems which would be inherent in the larger units. The transformers are installed on a deck at the downstream wall of the powerhouse, in order to provide a symmetrical arrangement for the unit system, and to keep the generator-transformer connections as short as possible.

2. Powerhouse High Voltage Connections - The high voltage equipment at the powerhouse consists of the 132-kilovolt lightning arresters and the 132-kilovolt line between the transformers and the switchyard.

The connections are outdoor-type pipe and cable bus arranged as required for the unit system to provide as compact an installation as possible in order to reduce space requirements on the transformer deck.

C. Station Service Equipment and Systems

1. Main Control Switchboard - A main control switchboard of the duplex completely metal-enclosed cubicle type, with front and rear sections, top enclosure, and end sections is provided to house instruments, control devices, relays and their auxiliary devices for the proper operation and protection of the powerhouse electrical equipment.

All equipment is modern large central station type. The principal protective equipment consists of generator differential overcurrent, over voltage, and loss of excitation relays, generator-main transformer differential relays, main bus differential relays and transmission line distance relays. Indicating instruments are provided to record all major equipment electrical values such as voltage-current, power, KVA, KVAR, and temperature. Audible and visual annunciation of all protective devices also are provided.

2. Station Battery and Battery Charger - The station battery is a lead-acid glass cell type rated 250 volts direct current, to furnish direct current for operation, control and protection of electrical equipment where limited dependable power is required. The battery is provided with a fully automatic charger with auxiliary equipment, meters, switches, rheostats, fuses, and ground detector lamps, all mounted in a suitable cabinet.

3. Station Service Equipment - A connection is taken off the 11-kilovolt bus of each unit to a 500 kilovolt-amperes, 3-phase, 50-cycle, 11 kilovolt - 400/230 volt, oil-filled, station service transformer. These transformers furnish power to the station service switchgear, load centers and power distribution panels for auxiliary usage in the powerhouse.

4. Diesel Engine-Driven Emergency Generator - A 150 kilovolt-ampere, 3-phase, four-wire, 230/400 volts, 50-cycle diesel engine-driven generator provides a limited amount of emergency station service power in the event of trouble or failure of the station service transformers.

5. Carrier-Current Communication Equipment - Carrier-current communication equipment is provided for voice communication over the transmission lines to Chandraghona, Chittagong and Siddhirganj. The installation consists of the necessary transmitter-receivers, coupling capacitors, line traps, line tuning equipment, and interconnecting cable.

6. Telephone Equipment - Private manual telephone equipment, including switchboard relays, battery, rectifying and ringing equipment with transformer, distribution frame, protective equipment, carrier-current matching equipment, code-call box, telephone sets at each project feature, and cable and wiring are provided.

7. Power Intake Structure - Power is furnished at the intake structure by a 400-volt feeder from the powerhouse station service. A distribution panel, motor control equipment, lighting and grounding systems and other equipment are provided.

8. Lighting System - The powerhouse is provided with a complete alternating-current lighting system and with limited emergency direct-current lighting where required for operation. The transfer to direct-current emergency lighting is automatic low-voltage actuated. The equipment is either indoor, weatherproof, explosion-proof or other appropriate type as required for each location.

9. Grounding System -The powerhouse is provided with an electrical grounding system with main ground connections to the main generator and turbine frames, generator grounding transformers, generator surge protection, main transformer tanks and neutral bus, main transformer lightning arresters, 132-kilovolt circuit breaker tanks, and 132-kilovolt steel structures. Secondary ground connections are made to all other electrical equipment, conduit, pipe, handrails, ladders and miscellaneous exposed metalwork.

D. Service To Other Project Features

An 11-kilovolt aerial line distributes power from the powerhouse to the spillway and outlet tunnel control equipment, area lighting and other project loads. Descriptions of these systems will appear in the appropriate chapters for these features.

4.03 MECHANICAL EQUIPMENT AND SYSTEMS

Mechanical equipment and systems are provided in the Karnafuli Project powerhouse for the purposes of installation, operation, maintenance and repair of all powerhouse facilities.

This section of the Design Report presents descriptions and pertinent information concerning design of the following:

A. Turbines and Governors
B. Auxiliary Systems
C. Draft Tube Bulkheads
D. Cranes and Hoists

A. Turbines and Governors

1. Turbines

a. General - The general setting of the turbines is shown on Figure E.

Each turbine is guaranteed to develop 55,000 horsepower at an effective net head of 68 feet when operating at 107.1 revolutions per minute. The maximum guaranteed efficiency is 90 percent at an effective net head of 80 feet. The maximum output is limited to 63,000 horsepower which is equivalent to 115 percent of the generator rated capacity.

b. Working Stresses - The maximum unit stresses for the rotating parts of the turbine were specified not to exceed one-half of the yield point at runaway speed. When the turbine is subject to maximum hydrostatic pressure (including pressure rise due to full load rejection) and is under maximum normal operating conditions, the stresses in the different metals will not exceed those shown in the following table:

	Maximum Unit Stress in Pounds per Square Inch	
	Tension	Compression
Cast iron	2,000	10,000
Cast steel	10,000	10,000
Alloy cast steel	20 percent of ultimate strength or 33 percent of yield point	20 percent of ultimate strength or 33 percent of yield point
Plate steel for spiral case or other principal parts	12,000	12,000
Bronze bearings	--	3,500

Exception was made on the spiral case where a maximum stress of 13,200 pounds per square inch was allowed during maximum pressure rise. Also, two-thirds of the yield point was allowed in the wicket gates, wicket gate stems and levers at the breaking point of the breaking element of the gate operating mechanism.

c. Spiral Case - The spiral case is of the riveted single buttstrap design. The horizontal centerline is located at Elevation 5.00 feet (as recommended by the manufacturer) to prevent excessive cavitation or pitting of the runner.

d. Runner - The runner is composed of six cast-steel blades set in a cast-steel hub. The hub contains the blade operating mechanism which consists of a cast-steel cross head connected to

levers on each of the blades. The space inside the hub is completely filled with lubricating oil from the Kaplan oil head. The lower end of the runner is covered with a cast-iron cone. Critical areas of the blade have a 1/8 inch thick layer of stainless steel placed where tests and experience have indicated that pitting is most likely to occur.

e. Main and Intermediate Shaft - The main turbine shaft is 34 inches in diameter and 245-3/8 inches long. A 15-inch diameter opening for the blade operating rod extends throughout its entire length. The runner blade servomotor cylinder is integrally cast with the main shaft and bolted to an intermediate shaft. The other end of the intermediate shaft has a flange which is bolted to the generator shaft coupling flange.

f. Blade Operating-Mechanism - The blade operating rod is connected at the lower end to the cross head in the runner hub and at the upper end to the operating piston in the blade servomotor cylinder. Oil pressure for operating and controlling the position of the runner blades is admitted to the servomotor cylinder through pipes inside the generator shaft.

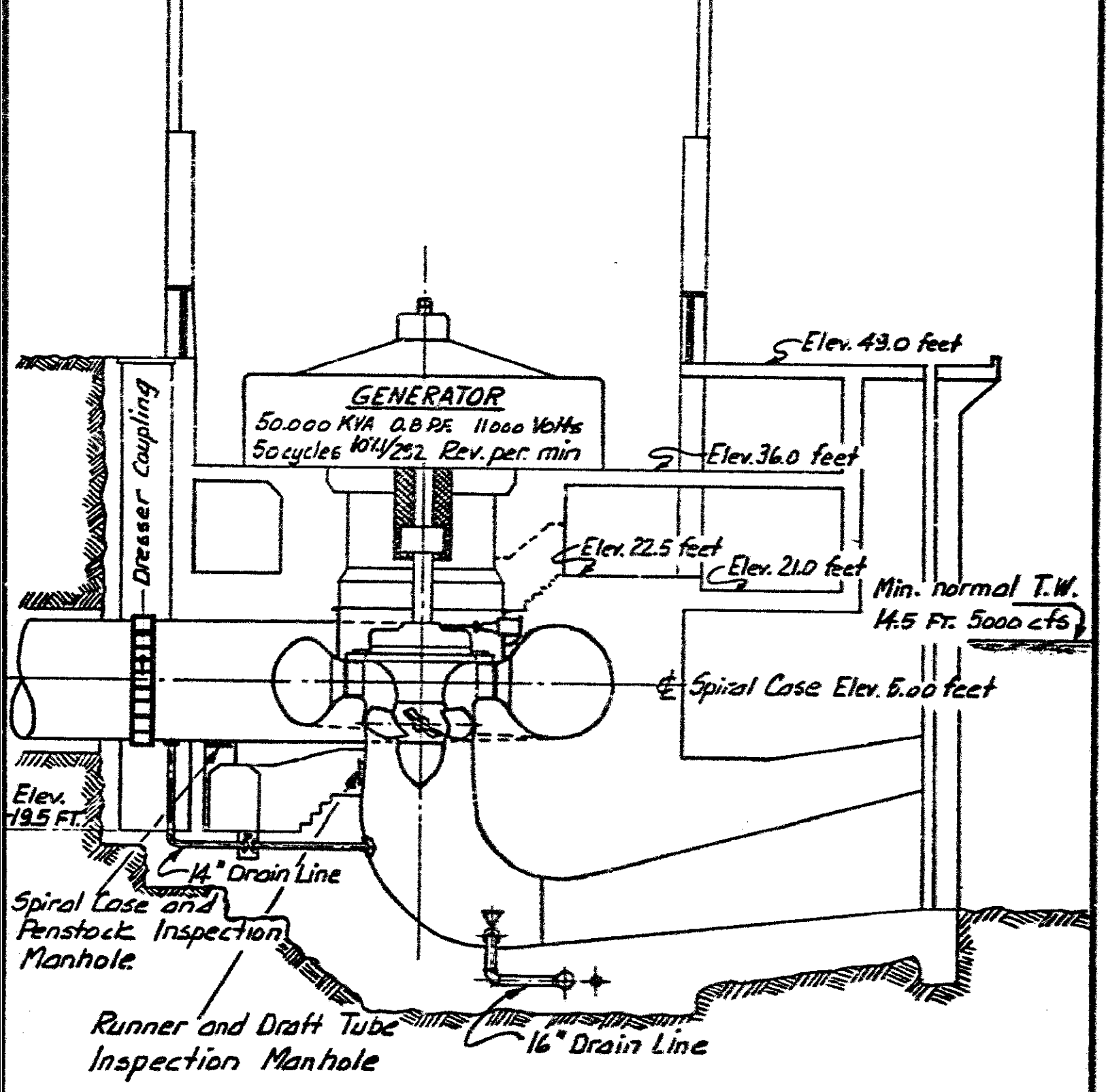
g. Guide Bearing - The turbine bearing is of the oil pressure lubricating type. One continuously running A. C. pump supplies the pressure to the bearing. If this pump should fail, a flow switch located in the pressure line will start the emergency D. C. oil pump. If the oil flow to the bearing is insufficient, an additional flow switch located in the pressure line will initiate the full shut-down of the unit.

h. Outer Head Cover - The outer head cover contains the upper and intermediate bronze bushed bearings and the stuffing boxes for the wicket gate stems. Plate strips of a material resistant to erosion are bolted to the head cover in the outer passage area.

i. Inner Head Cover - The gate operating ring which moves on bronze strips, the turbine guide bearing and the stuffing box are located on the inner head cover. The stuffing box can be supplied with filtered water by a hand-operated valve in the turbine pit. This water is used on the stuffing box during motoring of the units and possibly during long operating periods at speed-no-load.

A valve located on the inner head cover and operated by linkage from the operating ring admits air between the head cover and the runner to reduce vibration when the turbine is operated at light loads. On a sudden load rejection the valve is designed to open by the vacuum developed in the draft tube.

j. Centralized Lubrication System - A semi-automatic lubricating system is furnished for each turbine. The system is of the dual line type where No. 1 circuit around the pit liner lubricates exclusively all wicket gate stem bearings subject to water pressures and No. 2 circuit lubricates all remaining points on the turbine. The lubricating cycle for No. 1 circuit, lubricating the stem bearings is fully automatic and controlled by an adjustable interval timer. The No. 2 circuit, lubricating all other points on the turbines, is interconnected to No. 1 circuit by two hand



TURBINE

55000 HP at 68 feet net effective head and 107.1 Rev. per. min. Max. runaway speed 292 Rev. per. min.

KARNAFULI PROJECT	
POWER HOUSE CROSS SECTION DESIGN REPORT	
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.	
DR. RLH / CK.	RECOMMENDED / APPROVED
DATE: October 5, 1960	Fig. E

control valves. No. 2 cycle will catch lubricant from No. 1 cycle during one of its cycles. The air-operated lubricant pump will continue to run until the booster valve on the return lines reaches full pressure on the complete system.

k. Tailwater Depressing System - Provisions have been made on the turbine for future installation of a tailwater depressing system which may be used to force the tailwater level below the bottom of the runner blades. This will be required if in the future it would be desired to motor the units with the wicket gates closed, and with the generators connected to the line; for voltage and power factor correction and as a spinning reserve.

l. Spare Parts - The following tools and spare parts are furnished with the turbine:

One (1)	Wrench Board with a complete set of wrenches and special tools for assembling and dismantling the turbine equipment.
One (1)	Lifting Device for the main shaft, runner and inner head cover assembly.
One (1)	Device for runner and shaft assembly during field erection.
Two (2)	Wicket Gates and stems completely machined with bushings ready for installation.
One (1)	Complete set of breaking elements for the gate operating mechanism on the turbine.
One (1)	Babbitted liner for the turbine guide bearing.
One (1)	Sleeve for the main shaft where it passes through the packing box, including keys, screws, etc., required for attaching the sleeve to the main shaft.
One (1)	A. C. motor-driven turbine bearing oil pump complete with motor.
One (1)	D. C. motor-driven turbine bearing oil pump complete with motor.
One (1)	A. C. motor-driven head cover drainage pump complete with motor.
One (1)	D. C. motor-driven head cover drainage pump complete with motor.
One (1)	Complete set of all packings for one turbine.

2. Governors

a. General - Each governor has been designed to produce 570,000 foot-pounds for automatic operation of the gate servomotor and 362,000 foot-pounds for similar operation of the blade servomotor. The minimum oil pressure is rated at 300 pounds per square inch. Each governor consists of an actuator cabinet, a pressure tank and a separate sump tank equipped with pumps and motors. The actuator cabinet and the pressure tank are located on floor Elevation 36 feet and the sump tank on floor Elevation 22.5 feet.

b. Actuator Cabinet - Each actuator cabinet is provided with a shut-down solenoid and a speed-no-load solenoid.

The shut-down solenoid will be de-energized by protective relays which will be actuated for the following conditions:

(1) Overspeed of 180 percent.

(2) Governor low oil pressure or low oil level in tank.

(3) Turbine bearing with either low oil flow or high temperature.

(4) Kaplan head packing boxes with high temperature.

(5) Generator bearings with either low oil level or high temperature.

(6) Release of CO_2 system.

(7) Generator differential or generator main transformer differential.

(8) Generator overcurrent.

(9) Generator neutral ground overvoltage.

(10) Manual operation of control switch on governor panel to "Stop position".

The speed-no-load solenoid will be de-energized for the following conditions:

(1) Generator overvoltage or loss of excitation.

(2) Generator oil circuit breaker trip, manual or by line and/or bus, differential relays.

(3) Generator and transformer winding overtemperature.

(4) Overspeed of 160 percent.

(5) Manual operation of master control switch on switchboard panel to "Stop position".

c. Pressure Tanks - The pressure tanks each contain a volume of 444 cubic feet which is approximately 20 times larger than the gate and blade servomotor cylinders. Each tank is provided with one air compressor which automatically starts and stops in response to a float switch and a pressure switch located on the pressure tank. The two air compressors are interconnected and if one compressor is out of operation the other, by manual operation, can be used for both tanks.

d. Sump Tanks - The sump tanks contain not less than 110 percent of the total volume of oil in the entire system. On each tank are mounted two 75 horsepower motor driven oil pumps of the positive displacement screw type. The pumps have a combined capacity per minute of three times the total oil volume of the turbine servomotors.

e. Spare Parts - The governors are furnished with the following spare parts and tools:

One (1)	Complete set of tools and special wrenches required to completely assemble or dismantle the governors.
One (1)	Oil pump complete with motor and pilot, unloader, safety and shut-off valves.
One (1)	Head drive generator complete with rotor, stator and bearing.
One (1)	Head motor complete with stator and rotor.
One (1)	Complete head governor.
One (1)	Complete dashpot assembly.
One (1)	Blade relay valve plunger and bushings.
One (1)	Gate relay valve plunger and bushing.
One (1)	Pilot valve plunger and bushing.
One (1)	Complete set of packing.

B. Auxiliary Systems

Design of the powerhouse auxiliary systems was accomplished in accordance with appropriate codes, giving due consideration to practical and economic aspects of the design.

Criteria for sizing and selection of piping materials was derived from the United States Bureau of Reclamation publication, "Turbines and Pumps", Design Supplement No. 6, dated November 7, 1950.

Design or selection of other materials and equipment was generally based on recommendations, specifications or standards of the following authorities:

United States Bureau of Reclamation (USBR)
American Standards Association (ASA)
National Board of Fire Underwriters (NBFU)
National Association of Fan Manufacturers (NAFM)
American Society of Mechanical Engineers (ASME)
American Water Works Association (AWWA)
American Society of Heating and Air-conditioning Engineers (ASHAE)
Interstate Commerce Commission (ICC)

1. Raw Water System - The raw water system as shown on Exhibit IV-4 was designed to utilize individual water take-offs from each penstock to a header on the upstream side of the turbines. Water pressures in the penstocks at points of take-off vary from 10 to 30 pounds per square inch, depending upon the headwater elevation. The capacity of each raw water take-off will meet the cooling water requirements for 1-1/2 turbines, plus fire protection requirements and station service requirements including domestic water. The raw water take-offs and distribution header have sufficient capacity to supply all three turbines from any two penstock take-offs. A shutoff valve is provided at each take-off from the penstock. A 1-1/2-inch compressed air blowout connection is provided on the penstock side of the shutoff valve to permit blowing-out of the pipe should fouling occur.

An exposed 16-inch raw water header along the upstream wall of the turbine room floor serves as a distribution header to all services with a minimum pressure drop in the system. Since the pressure available at the penstock is not sufficient to overcome friction losses to equipment and other services, it is boosted as required at each of the services involved. Only minimum provisions for thermal expansion of the raw water header are required, since the raw water temperatures vary only from 75 to 85 degrees Fahrenheit. The 16-inch diameter header is supported on hangers from overhead, and all connections from the raw water intake to the header will be sufficiently long to provide the necessary flexibility.

An 8-inch and a 2-inch branch are taken off the raw water header and cross-connected to the normal cooling water supply to the emergency diesel generator. This ensures that in case of a power failure rendering pumps inoperable, a supply of water is available to the generator at a pressure of 10 to 30 pounds per square inch.

All piping for raw water, cooling water, station service water, treated water and fire protection systems conforms to the following requirements:

a. Piping over 8 inches in diameter was selected for the pressure involved, and is black, seamless pipe with beveled ends for welding. Wall thicknesses are not less than 0.25 inches. Portions of piping which are embedded are generally heavier than indicated by pressure requirements only, but in no case heavier than Schedule 40.

b. Piping from 2-1/2 inches to 8 inches in diameter, inclusive, is Schedule 40 black, seamless pipe with beveled ends for welding.

c. Piping 2 inches in diameter and smaller is standard weight galvanized, wrought steel with threaded ends and galvanized, malleable iron fittings and couplings.

d. All piping 2-1/2 inches in diameter and larger is welded with slip-on and welding neck flanges as applicable.

Gate-type valves are used for general purposes. Two and one-half inch and larger size valves are flanged, valves 2 inches and smaller are screwed.

2. Cooling Water Services - Approximately 1650 gallons per minute of cooling water is required for each main generator. About 1550 gallons per minute at less than 10 degrees Fahrenheit temperature rise are required for the generator air cooling coils; 50 gallons per minute for the thrust bearings; and approximately 35 gallons per minute for the turbine bearings. This water is supplied, as shown on Exhibit IV-4, by a valved branch from the raw water header to a duplex strainer and centrifugal pump which meets the capacity and head requirements for delivery of water at approximately 40 pounds per square inch at the generator connections. Since the raw water header pressure varies to the extent previously mentioned, the delivery pressure from the pump varies accordingly. Consequently, a pressure reducing station at each generator provides a constant pressure supply of water to the generator components involved.

Each individual supply to the generator has, following the pressure reducing valve, a flow meter and plug valve for air cooler water supply and sight glasses and balancing cocks for other services. This arrangement permits balancing of the individual requirements of the different systems. In addition, the cooling water supply to the generator cooling coils is provided with a valved by-pass and three-way valve, thermostatically controlled to allow the passage of cooling water at periods of low load for protection of the pumps.

Each generator coil has a valved vent line from the top of the coil which combines into a vent header. These vent lines run outside the generator casing to floor drains. The drains from the cooling coils for thrust and turbine bearings combine and discharge into the tailrace.

Three cooling water pumps, one of which is a stand-by, are provided with the initial installation of two turbines. The design provided for a fourth cooling water pump to be installed at the time of the installation of the third turbine.

The cooling water pumps are of the horizontal, split-case type with bronze impeller, bronze sleeves and wearing rings, and outboard ball bearings.

Duplex strainers, which can be cleaned without interrupting operations, are provided on all pump suction lines. The baskets are bronze, with 3/16-inch perforations, and are capable of withstanding 125 pounds per square inch pressure if fully clogged. The ratio of the straining area of each compartment to the strainer inlet area is approximately four to one. The body of the strainers is provided with vent cocks and and drains.

Pressure regulating valves are of the full-flow needle type. The high capacity and low pressure drop permits sensitive regulation, free from binding. The valves are diaphragm and external pilot operated, utilizing water from the main cooling water line. Valves have externally guided stems to minimize binding from water deposits. Valves are installed with bypass, "Y" strainer and isolation gate valves.

Balancing valves are of the lubricated plug-cock type. Two and one-half-inch and larger size valves are flanged; those 2 inches and smaller are screwed. The semi-steel valves are wrench-operated.

Thermostatic regulating valves are of the three-port, balanced piston type, air-actuated, with large capacity thermal bulb.

3. Station Service Water System - Two 60-gallon per minute pumps at 163 feet total dynamic head delivery are provided for station service water requirements. These pumps take suction from the 8-inch raw water branch and strainer serving the fire protection system, as shown on Exhibit IV-4. In addition to supplying the treated water system, the pumps discharge into a station service pneumatic tank which provides storage and minimizes pump cycling.

A 3-inch header and a 2-1/2-inch header run the length of the powerhouse with branch take-offs for the air conditioning condensing unit cooling coils, and for the numerous hose bibs and service connections throughout the plant. These headers also supply the normal cooling water requirements of the emergency generator when in operation.

The station service water pumps are of the close-coupled, end-suction centrifugal type, with bronze impellers, shaft sleeves and wearing rings, and are fitted with screwed connections. Pumps, rated as shown on Exhibit IV-4, are complete with ball bearing mountings, motor, mechanical seals, and all necessary accessories. Each pump is fitted with a gate valve on the suction side, and check and gate valves on the discharge line. The pumps are pressure-controlled from pressure switches mounted on the pneumatic tank.

The 125-pound per square inch pneumatic pressure tank is of the vertical type, and is of welded construction, completely galvanized, and mounted on structural base. A dual float-and-pressure control of the mercury bulb, electric-operated type starts the pumps at a predetermined low water level, admitting water to the tank. If the water is raised to a higher level than normal for the pressure involved, a solenoid valve on the station compressed air line opens, restoring the normal

operating level. Should the water level be lower than normal at the pressure setting involved, a level control opens a solenoid valve and allows excess air to escape until the lower range of acceptable pressure is reached.

4. Water Fire Protection System - The water fire protection system shown on Exhibit IV-4 consists of two 500-gallon per minute, 100-pound per square inch pumps which discharge into an exposed 6-inch header running the length of the powerhouse close to the ceiling of the mechanical gallery. Hose racks are located in accordance with recommendations of the National Bureau of Fire Underwriters. In addition, fire hydrants are installed at both ends of the powerhouse at Elevation 48.5 feet, and also in the switchyard.

The branch take-off from the raw water header to the fire pumps and other services is equipped with a duplex strainer which serves fire pumps, station service pumps, and the branch take-off to the emergency generator.

The fire main will normally be supplied and maintained at a pressure of 60 to 80 pounds per square inch by means of a cross-connection from the station service pumps. In case of a heavy demand, such as represented by a hose or fire hydrant discharge, the pressure drops below the 60 pounds per square inch normally supplied. If the pressure drops to 56 pounds, the number one fire pump will automatically cut in on pressure control. If the demand is sufficient to cause the pressure to continue to drop to 52 pounds per square inch, the second fire pump automatically cuts in on pressure control. The combined pumps discharge is provided with a relief valve set above 100 pounds per square inch, but lower than the shut-off head of the pumps. On cessation of demand, the discharge pressure rises, and a portion of the water is recirculated through the pump or pumps to prevent overheating. Once started, the fire pumps continue to operate until manually shut off and reset.

The fire pumps are of the horizontal split-case type with bronze impellers, bronze sleeves and wearing rings, and outboard ball bearings.

Interior fire hose racks are of the Fire Underwriters-labelled swinging type, each containing 75 feet of 1-1/2-inch standard linen fire hose. The racks are either pipe-clamp or wall-bracket mounted, with 1-1/2-inch fixed type, non-adjustable fog nozzles.

Outside fire hydrants are a post type with a 6-inch riser barrel and with two independently valved 2-1/2-inch hose thread outlets. Each hydrant is installed with a 5-inch gate valve and a 6-inch branch to the main. Gate valves are wrench-operated and installed below grade, with an adjustable valve box (slip type) connector.

5. Treated Water System - The treated water system, as shown on Exhibit IV-5, obtains its water from the station service pumps through a 2-1/2 inch header.

The header is provided with an orifice plate which directs a portion of the water through an alum and alkali pot to act as a coagulant for the suspended matter in the water to be treated. Following the alum and alkali pot a tank, with a capacity of approximately 100 gallons, is provided for mixing of the alum and alkali with the turbid water. The water is then run through two rapid sand filters which have a design capacity based on 2.5 gallons per minute per square foot of surface. Each filter is equipped with an orifice plate to prevent the flow from exceeding the capacity of the filter. Also provided are regulated backwash control valves in the multiport system.

The filtered water, suitable for service to the turbine gland seals, is run from the treated water header to each turbine gland seal through a one-inch branch line. Remaining treated water passes through a flow meter which determines the quantity flow and controls the rate of chlorine application by the chlorinator. An automatic hypochlorinator injects the prescribed amount of chlorine into the system. The treated water then flows into a pneumatic tank which provides storage and decreases fluctuations prior to going into the normal domestic water supply, either through a hot water heater for hot water, or normal cold water runs to the fixtures.

The estimated maximum cold water demand of 60 gallons per minute was based on the fixture list.

The alum and alkali feeders are of the proportioning pot type, complete with necessary valves, control valves, sediment traps, orifice plate, and piping. The alum and alkali feeders utilize ammonium alum and lump sal soda, respectively.

The mixing tank is of welded steel construction with dished heads suitable for pressures to 125 pounds per square inch and is completely galvanized. The tank has supports for vertical floor mounting with necessary tappings.

Rapid sand filters are of the vertical, floor-mounted, pressure type, with manually operated multiport valves for accomplishing the various phases of the filtering operation, such as backwashing, filtering to waste, filtering and closed. Also provided are access plugs, distributors, manual air vents, supporting legs, and a filtering media of graded fine and coarse silica sand and gravel. The tanks are of welded steel construction with dished heads designed for pressures up to 125 pounds per square inch. Orifices are installed in the filtered water discharges for regulation of the flow rate, to prevent undue agitation and carry-over of filtering media during backwashing.

The automatic hypochlorinator is electric-water operated, meter-actuated type, so designed that the rate of chlorine application is adjustable and is in proportion to flow of water. The unit has a capacity of at least 60 gallons per 24 hours, or five pounds of chlorine per 24 hours when running continuously using a one percent hypochlorite solution. Not more than 0.5 gallons of water per minute is required for operation and control of the apparatus.

The pneumatic water tank is a 125-pound per square inch, welded, completely galvanized vertical type, mounted on a structural base. A dual float and pressure control of the mercury bulb, electric-operated type opens a solenoid valve on the water inlet at a predetermined low water level, admitting water from the supply main. If the air pressure is below normal after restoration of the normal water level, the pressure control opens a solenoid valve on the station compressed air line, restoring the normal operating pressure. If the air pressure is above normal after restoration of the normal water level, the pressure control opens a solenoid valve, allowing excess air to escape until normal operating pressure is restored.

Solenoid valves for water inlet and for the pneumatic tank air inlet and outlet are of the line voltage, magnetic type, for operating on 125-pound per square inch air or water pressure lines.

6. CO_2 Fire Protection Systems - The CO_2 fire protection system for generators, shown on Exhibit IV-6, consists of carbon-dioxide cylinders on the turbine room floor, with pipe runs for initial and delayed discharge to each generator. The number of cylinders and the arrangement and size of pipe and discharge nozzles were determined by the manufacturer, based on the static air volume of the machine and the deceleration period without the application of the generator field.

Operation of the system is automatic, by use of electric thermostats and tie-in with the tripping circuit of the generator differential protection relays. In addition, provision is made for manual operation from a remote-control electric switch box.

The cylinder bank consists of two group of cylinders, one group furnishes an initial discharge to rapidly build up the CO_2 concentration. The other group furnishes a delayed discharge to compensate for normal leakage throughout the deceleration period of the machine. Because of the remoteness and inaccessibility of the project, a 100 percent supply of spare bottles is neatly mounted adjacent to the connected system. The entire cylinder battery is assembled in a secure manner and arranged so that each cylinder can be weighed in place without disturbing the connections, and without putting the system out of service. The cylinders stand in their normal upright position and are guarded with removable wire mesh screening. Pressure-operated switches for red indicating lights in the manual switch boxes, and a remote alarm indicate when gas is being released.

In addition to providing fire protection for the generators, CO_2 protection is also provided for the oil storage and purification rooms as shown on Exhibit IV-6. This system is fully automatic, providing about a 34 percent concentration in a volume of 13,500 cubic feet. This is accomplished with fixed nozzles which utilize rate-of-rise type thermostats for actuation. The system controls use CO_2 pressure in lines to individual pieces of equipment to simultaneously trip duct dampers, exhaust fans, and fire doors when thermostats open cylinder valves. Automatic electric releases, manual release, cylinder banks, and other appurtenances are similar to those for the generator fire protection system previously described.

All of the cylinders are constructed of seamless, drawn steel and charged with carbon dioxide. Each cylinder conforms in all respects with the shipping container specifications of the Interstate Commerce Commission.

All pipe and fittings are hot-dipped galvanized and scale-free. Pipe up to and including 3/4 inch in diameter is standard weight. For larger sizes, extra-heavy pipe is utilized. All fittings are galvanized and have a minimum bursting pressure of 6000 pounds per square inch.

Pressure-operated switches are of the mercury type, mounted on a pivoted framework in such a manner that a small piston advance by the pressure of the gas will operate the switch. The thermostats are of the fixed temperature, bi-metallic type.

7. Compressed Air Systems - The station service compressed air system, shown on Exhibit IV-6, consists of two tank-mounted reciprocating compressors and an additional vertical receiver amply sized to satisfy the normal powerhouse compressed air demands. A system operating pressure of 125 pounds per square inch was selected.

Compressed air from this system is distributed throughout the powerhouse through mains running at or near ceilings, with branch supply lines to the sewage ejector, pneumatic tanks, generator air brakes, grease pumps, raw water line blowouts and to service hose bibs located throughout the powerhouse.

The governor compressed air system supplies air to the governor accumulator tanks to maintain the optimum hydraulic oil level for turbine servomotor operation. Normally, a pressure of 300 pounds per square inch is required to overcome line losses and to charge an accumulator tank which varies in operating pressure from 225 to 275 pounds per square inch.

Precautions for keeping the hydraulic oil as free from water and other contaminants as possible include (1) taking all branch run-outs off the tops of the mains to prevent inclusion or carry-over of water, (2) providing drains in the compressed air receivers, and (3) providing a valved drip leg and an air dehydrator cartridge in each air line, before connecting to a hydraulic oil accumulator tank.

In general, all branch take-offs from station service compressed air mains and governor compressed air mains are taken off the top of the subject main to prevent inclusion of water. Drip legs with valved drains or water traps are installed at the ends of runs of mains to collect condensed moisture and permit its removal.

System piping is generally black wrought steel pipe with malleable iron-screwed fittings for pipe 2 inches in diameter and smaller. Flanged and welded fittings are used for pipe 2-1/2 inches in diameter and larger.

Valves in general for sizes 2 inches and smaller are globe valves or lubricated plug cocks. Valves 2-1/2 inches and larger are flanged lubricated plug cocks.

Station service air compressors are a "V"-belt driven, reciprocating two-stage type, complete with motors, drives, mounting bases (welded to horizontal steel air receivers), support legs, muffler-filter units on intakes, automatic controls, belt guards, pressure gages, relief valves, intercoolers, and all necessary accessories. Compressors are multicylinder, with isolated, finned cylinder heads bolted to frame, suitable for operation at 125 pounds per square inch. Receivers are of American Society of Mechanical Engineers Code design for 200-pound per square inch working pressures. Control is of the automatic "dual" type, actuated by the air receiver pressure.

The station air receiver is a vertical, floor-mounted type, in accordance with the requirements of the American Society of Mechanical Engineers Code for Unfired Pressure Vessels for 125-pound per square inch working pressures, complete with all necessary inlets, outlets, inspection plugs, drains and control tappings.

Air traps for removing moisture from drain pockets in the compressed air lines are of the regular float type, designed for operation at a minimum pressure of 125 pounds per square inch.

The governor air compressors are a "V"-belt driven, reciprocating, multistage type complete with motor, drives, mounting bases, support legs, muffler-filter units on intakes, automatic controls, pressure gages, relief valves and all necessary accessories. Receivers are of American Society of Mechanical Engineers Code design for 300-pound per square inch working pressures. Control is of the automatic "dual" type, actuated by the air receiver pressure.

8. Unwatering System - The unwatering system is provided to drain the penstocks, scroll cases, and draft tubes for access, inspection, and maintenance. The system, as shown on Exhibit IV-7, provides the means to accomplish the following:

a. Draining water from the scroll case when the wicket gates are closed, and/or draining all water from the penstock to the draft tube. A 14-inch valved line for each unit takes suction from the low point of the penstock and scroll case. A valve station is located in the inspection gallery. The discharge connects to the steel-lined portion of the draft tube.

b. Draining the draft tubes to the unwatering sump. A 16-inch valved line for each unit takes suction from a point near the bottom of the draft tube through an inlet grating. A valve station is located in an adjacent access pit. Discharge is into an unwatering sump at one end of the powerhouse.

c. Removing collected water from the unwatering sump. Two vertical turbine unwatering pumps with characteristics as shown on Figure A are located in the unwatering sump for this purpose.

d. Refilling draft tube and scroll case to normal tailwater elevation from the tailrace. A 10-inch valved line for each unit takes suction from the tailrace through a beehive strainer, located below normal minimum tailwater on the downstream wall of the powerhouse. A valve station is located in the inspection gallery. Discharge is into the lower portion of the draft tube.

Normal unwatering is accomplished at periods of normal tailwater by closing the intake gates and draining through the penstocks, scroll case and draft tube, to normal tailwater elevation. At this point, the draft tube gates are closed, leaving approximately 1,540,000 gallons remaining in the unit. The 14-inch scroll case drain line to the draft tube and the 16-inch draft tube drain to the unwatering sump are opened and the unwatering pumps are started. With the sump full, the combined discharge is in the order of 7000 gallons per minute, gradually dropping to approximately 5500 gallons per minute as the sump level recedes. This operation permits unwatering of a unit in about four hours. Designed operation and pump selection is as shown on Figure A.

To reactivate a unit, the drain valves are closed and the 10-inch draft tube fill line is opened, permitting the unit to fill to normal tailwater elevation, equalizing pressure on the draft tube gates prior to hoisting.

Unwatering sump pump control is accomplished by means of an automatic bubbler type level measuring and pump controller utilizing compressed air and measuring pressures involved to determine the sump levels. Automatic pump stop-start and high-level alarm positions are incorporated in the system.

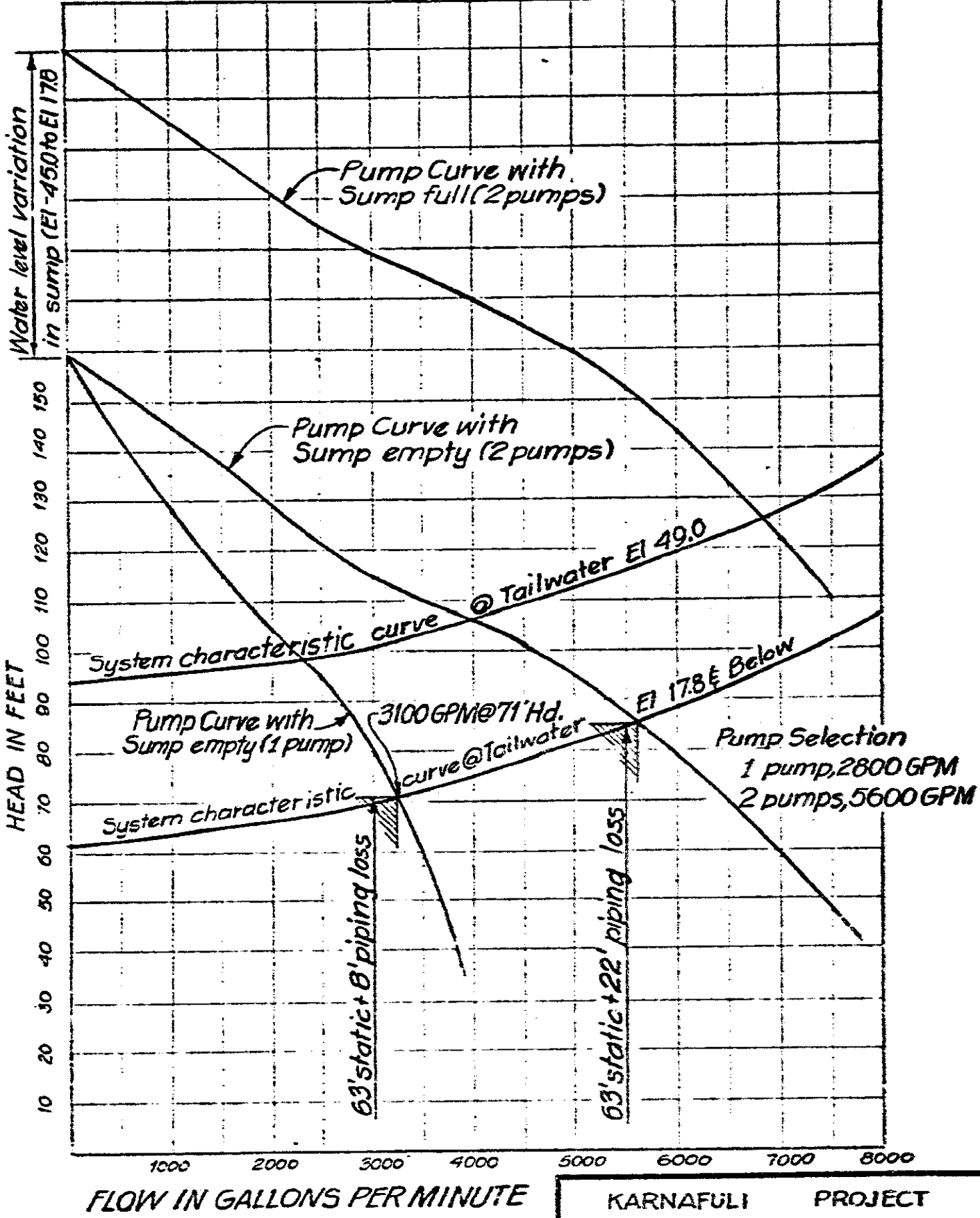
Pipe and fittings for the penstock drain line to the draft tubes are black welded steel, Schedule 30 where embedded, and Schedule 20 where exposed, with flanged connections at the valves.

Pipe and fittings for the draft tube unwatering lines to the unwatering sump is American Water Works Association Class 150 bell and spigot, cast iron pipe with Class "B" fittings.

Pipe and fittings for the draft tube filling line from the tailrace to draft tubes is black welded steel, Schedule 30 where embedded, and Schedule 20 where exposed, with flanged connections at the valves.

Valves, for general purposes, are of the gate type. Sizes of 2-1/2 inches and larger are flanged, and those 2 inches and smaller are screwed.

Unwatering pumps are a vertical turbine type, with bronze enclosed impellers, cast iron flanged and bolted bowls, basket strainer, and oil-lubricated steel shafts. Units have non-overloading characteristics and operate at not over 1760 revolutions per minute. Pump discharges are fitted with checks and gates. Motors are a squirrel cage induction type, with vertical hollow shafts and non-reversing ratchets.



Note: Pump Curves are based on ratings for 75HP Vertical Turbine, 50-cycle, 1460 RPM, 3-stage pumps.

KARNAFULI PROJECT		
DESIGN REPORT UNWATERING PERFORMANCE CURVES		
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. BLM CK.	RECOMMENDED	APPROVED
DATE: JULY 10, 1958	FIGURE A	

9. Drainage System - The powerhouse drainage system includes all floor, gutter, equipment and area drains discharging into a main 8-inch header installed below the lowest drain point and discharging into a drainage sump at one end of the powerhouse. As shown on Exhibit IV-7, two vertical turbine pumps in the unwatering sump discharge the water to the tailrace. A valved cross-connection between the unwatering and drainage sumps permits complete emptying of the unwatering sump. An additional valved cross-connection between sumps is provided at an elevation above normal pump starting level. In the event drainage pumps are inoperable, water overflows from the drainage sump to the unwatering sump and is removed by the unwatering pumps.

Pump capacities and pipe sizes are based on practical and economic considerations. Water to be drained includes seepage through walls and foundations, turbine and equipment drains, floor washdown, filter backwash and other overflow and leakage. Pipe sizes selected are considered conservative, consistent with economic feasibility.

Roof drains are provided on the roof of the powerhouse for removal of rain water. The downspouts discharge into trenches around the outside of the building and into the transformer drain pits.

Transformer pit drains remove collected water through three 6-inch drain lines which discharge into the tailrace.

Pipe and fittings for the drainage system are standard weight, cast iron, bell and spigot materials.

Roof drain downspouts are Schedule 40, black welded steel with Class 150 malleable iron, black, banded fittings.

Drainage pumps are a vertical turbine type, with bronze semi-open impellers, cast iron flanged and bolted bowls, basket strainer, and oil-lubricated steel shafts. Units have non-overloading characteristics and operate at not over 1760 revolutions per minute. Pump discharges are fitted with checks and gates. Motors are a squirrel cage induction type with vertical hollow shafts with non-reversing ratchets.

10. Plumbing System - The powerhouse plumbing, shown on Exhibit IV-7, consists of a conventional building plumbing system, with a pneumatic sewage ejector, a septic tank, and effluent discharge to tailrace. A hot water heater and hot and cold water distribution lines to all fixtures are included.

The pneumatic ejector lifts sewage from the lower levels of the powerhouse to a septic tank buried adjacent to the powerhouse. The effluent flows by gravity to the tailrace.

All sanitary waste and soil mains are standard weight, cast iron bell and spigot pipe, with cast iron bell and spigot fittings. Waste and vent lines smaller than 2 inches in diameter are galvanized steel and are equipped with drainage fittings.

Plumbing fixtures, complete with all accessories, brass, trim and supports, are as follows:

a. Water closets are a siphon-jet type with flush valves, vacuum breakers and screw-driver stops, hard rubber open-front seats and check hinges.

b. Eastern type water closets with all necessary appurtenances are set level with the floor.

c. Lavatories are 19-inch by 17-inch, with direct-lift waste fitting trim, "P" traps, cleanouts, and all necessary accessories.

d. Urinals are a wall-mounted, flushing rim type, with flush valve connections, strainers and traps, cleanouts, and all necessary accessories.

e. The lead-lined sink provided for the battery room is equipped with lead strainer, lead plug and lead tail piece, for connection to the acid waste line.

f. Floor and area drains in general are of cast iron, round pattern, with square tops and hinged grates.

g. Service sinks are 24 inches by 20 inches, with double faucets, 3-inch cast iron "P" trap standards, metal strainers, rough brass cleanout plugs and all necessary accessories.

The sewage ejector is a single-unit pneumatic type, operated by the station compressed air system. The unit includes a steel receiver, gate and check valves, electrode control and air-piping strainer, reducing valve, 3-way solenoid valve, and all necessary accessories.

The septic tank is constructed of heavy-gage welded steel with a thick asphaltic coating inside and out. The tank has two compartments, with a flanged top cover.

11. Ventilating System - All portions of the powerhouse, other than the control room, are provided, as shown on Exhibit IV-8, with forced, filtered air ventilating systems. No provision was made for tempering, cooling, dehumidifying or recirculation.

Temperatures at the site range from a maximum of 110 degrees Fahrenheit during summer months, to a minimum of 50 degrees Fahrenheit during the winter. For design purposes, the summer and winter outdoor temperatures tabulated below were assumed, based on American Society of Heating and Air-conditioning Engineers design temperature definitions:

Summer dry bulb	97 degrees Fahrenheit
Summer wet bulb	83 degrees Fahrenheit
Summer relative humidity	56 percent
Winter dry bulb	68 degrees Fahrenheit

The amounts of fresh outside air distributed to the various portions of the powerhouse are as follows:

	Air Changes per hour
Generator Room	1
Turbine Floor	5
Battery Room	5
Oil Rooms	8
Inspection Gallery	5
Toilets	10.

Taking into account all equipment, light and solar heat gains, the system will limit temperature rise to less than 10 degrees Fahrenheit.

Limiting air velocities used throughout the system are as follows:

	Feet per minute
Main fan outlet velocity	1850
Intake air louvers	800
Air volocity through filters	300
Maximum duct velocity	1500
Discharge air registers	750 - 1000.

The source of supply air is through a louvered opening on the downstream wall of the generator room floor, above maximum high water level. Air is drawn through the louvers, down a duct shaft, into a plenum and through a filter bank, by a centrifugal supply fan. The air is distributed throughout the building by a network of supply ducts. Air is circulated through the inspection gallery at Elevation -19.00 feet by the negative pressure produced by circulating fans. The fans, mounted in the penstocks coupling chambers draw air from the inspection gallery into embedded ducts, discharging the air into the generator room through registers mounted on the upstream wall. Initially, one fan is mounted at the Unit No. 1 penstock and one at the Unit No. 2 penstock. A third fan will be attached to the embedded duct in the Unit No. 3 penstock when that unit is installed. The air discharged into the turbine area at Elevation 22.5 feet exfiltrates up to the generator room floor through stairway and access openings.

Three exhaust fans mounted high on the upstream wall of the generator room floor provide a positive exit route for the supply air distributed into the building. These fans were designed to maintain a slight positive pressure in the building. Gravity dampers are attached to the fans to close the fan openings should the fan be turned off or should power failure occur. All pieces of equipment are independently controlled by on-off switches.

The toilet, battery, and oil rooms are provided with exhaust fans. These fans are connected to ducts which discharge air out the downstream wall through louvers located above the maximum high

water level. The toilet room and battery room fans are manually operated by on-off switches. The oil room fan, in addition to having the manual off-on switch, is interconnected with the CO_2 fire extinguishing controls. These controls, through a network of CO_2 piping act simultaneously to trip fire doors and fire dampers shut, and to shut off the exhaust fan.

Fans and blowers conform to the standard requirements of the National Association of Fan Manufacturers. Motors for fan service are ball-bearing, general purpose, squirrel cage induction motors, of sufficient capacity to perform the work required. Drives are generally of the "V"-belt type with belt guards.

The main supply fan is a centrifugal type, double width, double inlet, with backward inclined blades, Class I.

Circulating fans and main exhaust fans are of the direct drive propeller type with constant speed drive.

The battery room, toilet room, and oil room exhaust fans are of a direct drive, utility set type, single width, single inlet, with forward curved blades mounted or suspended on vibration-free supports.

Filters are a permanent, impingement, washable, all-metal panel type, 4 inches thick, encased in steel metal frames. Air velocity through the filter is limited to 300 feet per minute.

Ductwork is fabricated of hot-dipped, galvanized sheet steel. Bracing for ducts is of galvanized sheet angle-iron members, with supporting rods as recommended by the American Society of Heat and Air-conditioning Engineers. Sheet metal gages for the various duct sizes are as follows:

Maximum Side	U. S. Standard Gage
Up to 24 inches	24
24 to 40 inches	22
40 to 72 inches	20
72 inches and larger	18.

Splitter dampers of the locking quadrant type are provided at all supply branches. All right angle turns are provided with duct turns. Round bends have a radius of one-and-one-half the duct width, where practicable. All inlet and discharge connections to fans are made with heavy canvas-asbestos cloth made tight to the fans and ducts.

12. Air-Conditioning System - The air-conditioning system for the powerhouse control room is, as shown on Exhibit IV-8.

Temperatures at the site range from a maximum of 110 degrees Fahrenheit during summer months to a minimum of 50 degrees Fahrenheit during the winter. For design purposes, the summer and winter outdoor temperatures tabulated below were assumed, based on the American

Society of Heating and Air-conditioning Engineers design temperature definitions:

Summer dry bulb	97 degrees Fahrenheit
Summer wet bulb	83 degrees Fahrenheit
Summer relative humidity	56 percent
Winter dry bulb	68 degrees Fahrenheit.

The air conditioning system was designed to maintain interior conditions in the control room, office, and reception room as follows:

Summer cooling	82 degrees Fahrenheit
Summer relative humidity	50 percent
Winter temperature	70 degrees Fahrenheit.

The conditioning of the air includes filtering, heating and cooling. A constant minimum amount of outside air is brought through a duct connected to a louver mounted on the downstream wall of the generator room. This duct connects to an air handling unit which distributes the treated air into the conditioned areas. In addition, an amount of air equal to the outside air intake is exhausted from the toilet rooms. The exhaust fans for these rooms are thus utilized for two purposes. A return air duct system returns air to the air handling unit. Manual dampers on the air handling unit for outside and return air can be adjusted to maintain a slight positive pressure in the conditioned areas. An electric heating coil in the supply duct is actuated by a room summer-winter type thermostat to maintain the area above a minimum temperature during winter months. The same thermostat actuates a freon compressor to maintain the temperature below a maximum limit during the summer. The freon compressor-condensing unit is connected to a direct expansion coil in the air handling unit to dehumidify, as well as cool, the air.

The air handling unit for the control room and office area air conditioning system is a self-contained type, including fan, casing, filter section, mixing box, direct-expansion cooling coils, and electric heating coils.

All other components are similar to those previously described for the ventilating system.

13. Insulating Oil System - The insulating oil system, as shown on Exhibit IV-9, permits treatment, transfer and storage of transformer insulating oil. The system consists of one clean oil storage tank and one dirty oil storage tank, each capable of storing the oil charge of one transformer; supply and return headers, with valve stations at each bank of three transformers and in the switchyard for the oil circuit breakers; a portable insulating oil transfer pump, capable of filling or emptying a transformer in less than two hours; a portable filter press that can be utilized for both insulating oil and lube oil; and a portable centrifuge that also can be utilized for both insulating oil and lube oil.

When a transformer is required to be taken out of service and untanked, the oil is drained to the dirty oil tank, using the portable transfer pump, to enable moving the transformer without additional weight. The

oil is then purified by the filter press (or centrifuge, depending on the condition of the oil) and transferred to the clean oil tank. The transfer pump is used to move the clean oil back to the transformer prior to restoring the unit to service. Purifying is normally performed with the filter (or centrifuge) directly connected to the transformer. The oil is circulated to purify it and increase its di-electric quality. The insulating oil system in general permits the following operations:

a. Fill clean and dirty oil storage tanks by gravity.

b. Fill transformers or oil circuit breakers from clean oil storage tanks, using the transfer pump.

c. Empty transformers or oil circuit breakers into the dirty oil storage tank, using the transfer pump.

d. Move dirty oil from the dirty oil tank through the filter press to the clean oil tank.

e. Circulate oil from transformers or oil circuit breakers through filter press back to transformer or oil circuit breaker, to effect purification.

f. Flush supply lines with oil from the clean oil storage tank, and return the oil to the dirty oil storage tank, using the transfer pump.

g. Discharge waste oil to the truck filling and unloading station.

All insulating oil piping is Schedule 40, black wrought steel pipe, pickled and oiled with 150 pound per square inch malleable iron screwed fittings for fittings 2 inches and smaller in size. Welded fittings are used for 2-1/2-inch sizes and larger. Lengths of flexible metal-lined, synthetic, oil-resistant hose with braided cover are provided, each with male and female standard type metallic hose couplings and adapters with American Standard screw thread. Valves in general are of the gate type.

The insulating oil storage tanks are a gravity type, with welded construction. Each tank is fitted with a bolted manhole, a vent, and suction, fill, drain and stick gage tappings. The vent is further provided with a dehydrating filter to prevent inclusions of water. The suction and fill connections extend to within 6 inches of the bottom of the tanks, and are fitted with foot valves and antisplash deflectors, respectively. Insulating oil tanks are finished with a heavy bitumastic coating on the outside, and a synthetic oil-resistant enamel on the inside.

The portable filter is a press type, complete with motor-driven pump, cast iron frame and plates with movable and fixed heads and hand-wheel closing device, steel drip pan, basket type strainer, air bell, pressure relief valve, and pressure gages, all mounted on a rigid steel frame with heavy casters. The rotary-gear type pump operates at a 15-foot suction lift and a 70-pound per square inch discharge pressure.

A filter paper drying oven, with sufficient capacity to handle several charges of filter paper is provided. The oven is of sheet steel with adequate heat insulation and has manual electrical controls for low, medium and high heat. A thermometer and thermostat is also provided. Filter paper is hung vertically from rods, and the sheets are separated slightly to facilitate rapid and thorough drying. The interior is painted with oil-resistant paint. Wiring has heat-resistant insulation and is installed concealed within the outer casing.

The portable insulating oil transfer pump is a positive-displacement, direct-connected, electric motor-driven type. The pump has close-grained cast iron castings, hard alloy iron herring-bone gears, forced-feed lubrication, built-in relief valves, and an enclosed type motor. This pump may also act as a standby unit for lube oil service. The unit is mounted on a rigid steel base, supported on heavy duty casters, and is provided with a tow bar, 100 feet of electrical cable, a cable rack and a motor starter.

14. Lubricating Oil System - The powerhouse lubricating oil system permits treatment, transfer, and storage of the lubricating and governor oil. The system, as shown on Exhibit IV-9, consists of supply and return headers, with connections to generator and turbine bearings, governor systems, clean and dirty oil storage tanks, and a transfer pump.

Each of the oil tanks, which are located in the lube oil storage room, has sufficient capacity to store all the lubricating and governor oil from one turbine, generator, and governor. The portable filter press and portable centrifuge are as described under Insulating Oil System. The supply and return headers are located at the ceiling of the floor at Elevation 21.50 feet and the turbine room floor at Elevation 22.50 feet. All equipment branches (except bearing overflows) are valved.

The lubrication and governor oil system provides facilities for the following operations:

a. Filling storage tanks by gravity.

b. Moving oil from a dirty oil tank (or any other reservoir in the system) through the filter press to a clean oil tank.

c. Filling a governor sump, generator bearing, or turbine bearing from clean oil storage, using the transfer pump.

d. Draining generator bearings, by gravity.

e. Draining a governor sump, using the transfer pump.

f. Flushing supply lines with oil from a clean oil storage tank to a dirty oil tank, using the transfer pump.

g. Discharging waste oils to the truck filling and unloading station.

At the entrance of the oil storage room a curb of sufficient height is provided to permit containment of all oil within the room, in case of oil spill or tank break of both tanks. A sump and pumping connection is provided in the oil room to permit removal of (1) waste oil by pumping, or (2) water by gravity.

All lubricating and governor oil piping is Schedule 40, black wrought steel pipe, pickled and oiled, with 150-pound per square inch malleable iron screwed fittings for sizes 2 inches and smaller, welded fittings for 2-1/2-inch sizes and larger. Lengths of flexible, metal-lined, synthetic, oil-resistant hose, with braided cover are provided; each with male and female standard type, metallic hose couplings and adaptors with American Standard screw thread. Valves in general are of the gate type.

The lubricating oil storage tanks comply with the Fire Underwriters requirements for welded construction. Each tank is fitted with a bolted manhole, a vent, and suction, fill, drain and gage glass tappings. The suction and fill connections extend to within 6 inches of the bottom of the tanks, and are fitted with foot valves and antisplash deflectors, respectively. Lubricating oil tanks are furnished with a heavy bitumastic coating on the outside, and a synthetic oil-resistant enamel on the inside.

The lubricating oil transfer pump is a positive displacement, direct-connected, electric motor-driven type. The pump has close-grained cast iron castings, hard alloy iron herringbone gears, forced-fed lubrication, a built-in relief valve and an enclosed type motor.

15. Tailwater Gage System - The tailwater gaging system provided includes (1) a stilling or gage well from Elevation 55.00 feet to below low tailwater elevation; (2) a connecting pipe to the tailrace with screened inlet; (3) a valved compressed air connection to the inlet pipe to permit cleaning of screen with compressed air; and (4) a valved drain from the bottom of the stilling well.

A water level measuring device incorporating a float, counterweight and pulley arrangement actuates a transmitter which transmits the water level to a recorder in the control room.

C. Draft Tube Bulkheads

1. General

a. Requirements - For unwatering any powerhouse unit for inspection or maintenance, a set of three identical bulkheads is required to close off the draft tube from the tailwater.

b. Selection - Selection of the type of bulkhead was based on the combined cost of bulkheads and gantry crane. Reinforced concrete bulkheads were estimated to weigh more than 50 tons each, as compared to 11.5 tons each (includes a lifting beam weighing 1,100 pounds) for structural steel bulkheads. The obvious economy of a gantry crane for the smaller weight dictated the choice of the structural steel bulkheads.

c. Description - Each bulkhead is of all welded construction, 21 feet-2 inches high by 15 feet-1 inch wide, with ten 21WF62 beams which span the draft tube openings horizontally. The 21WF62 beams support a 9/16-inch thick skin plate on the upstream side.

Each draft tube bulkhead opening is provided with an 8WF24 sill, stainless steel side and top seal plates, and dogging devices for holding bulkheads in normal storage position just below the deck at Elevation 49.00 feet.

The bulkheads are equipped with standard rubber seals. To seat the bulkhead and to facilitate unwatering a unit, four springs on side brackets are provided to force the bulkhead against the stainless steel seal plates before a differential pressure is established by the unwatering pumps. If a differential pressure is built up on the upstream face of the bulkhead by inadvertently filling the draft tube from the reservoir instead of from tailwater, the bulkhead will simply "blow out" without damage, except for shearing off the spring brackets

Two 4-inch flap valves are located near the top of each bulkhead to prevent trapping air when the draft tubes are filled.

2. Design

a. Criteria and Assumptions - The draft tube bulkheads were designed for normal steel stresses (basic tensile stress being 18,000 pounds per square inch) under an unbalanced hydrostatic head from tailwater at Elevation 24.00 feet.

The skin plate was designed for bending at a basic tensile stress of 24,000 pounds per square inch under normal load, both alone and as part of the girder stresses. A corrosion allowance of 1/32 inch for all exposed surfaces was made.

An increase in allowable stresses of about 1/3 was permitted for tailwater at Elevation 44.0 feet (5 feet below the deck).

b. Analysis - In the design the beam flanges were considered to be part of the skin plate, thus forming a continuous series of haunched beams. Skin plate and beam stresses, which are at right angles to each other, were combined by adding three-tenths of the smaller stress to the larger stress, in accordance with Poisson's ratio for carbon steel.

Complete details of the draft tube bulkhead analysis are given in Figure B.

D. Cranes

1. General

a. Purpose - A powerhouse bridge crane is provided for installation and maintenance of the turbines, generators, and other equipment within the powerhouse.

A draft tube bulkhead gantry crane is provided for handling the bulkheads for the draft tubes.

b. Location - The powerhouse bridge crane is located inside the building. It travels the full length of the powerhouse on a runway supported by wall columns.

The draft tube bulkhead gantry crane is located and travels on rails extending along the full length of the transformer deck.

c. Description - The powerhouse crane is a 350-ton (2,000 pounds per ton) nominal capacity, two-trolley, cab-operated, electrically driven, double-girder, indoor traveling bridge type with lifting beam, as shown in Figure C. The bridge girders have a span of 65 feet-3 inches between rails. The lifting beam connects the main hooks of the two trolleys to provide the 350-ton nominal capacity at the eye of the lifting beam. Each trolley has a main hoist of 175-ton nominal capacity, with allowance for one half of the lifting beam weight, and an auxiliary hoist of 25-ton nominal capacity.

The draft tube crane is a 12-ton (2,000 pounds per ton) nominal capacity, fixed hoist, two-point pick-up, platform-operated, electrically driven, outdoor cantilever gantry type, as shown on Figure D.

2. Powerhouse Bridge Crane

a. Selection of Type - A bridge crane with two trolleys, each with a main and auxiliary hoist arrangement with the lifting beam, was selected to provide versatility to accommodate all the required lifts.

b. Design

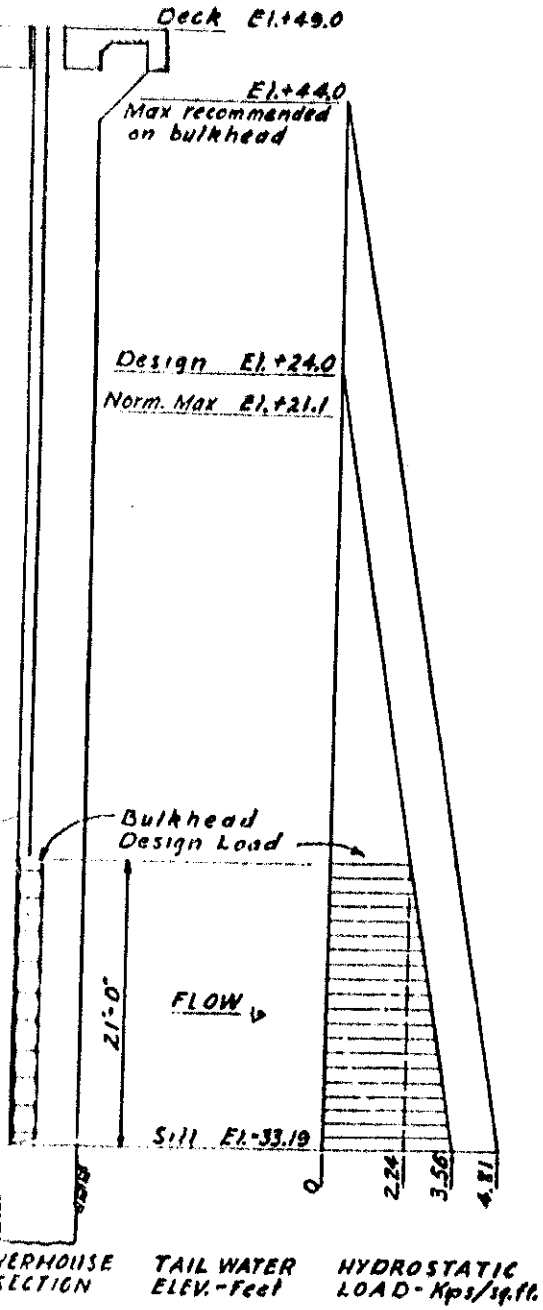
(1) Lifts and Speeds - The crane was designed for the following lifts and speeds:

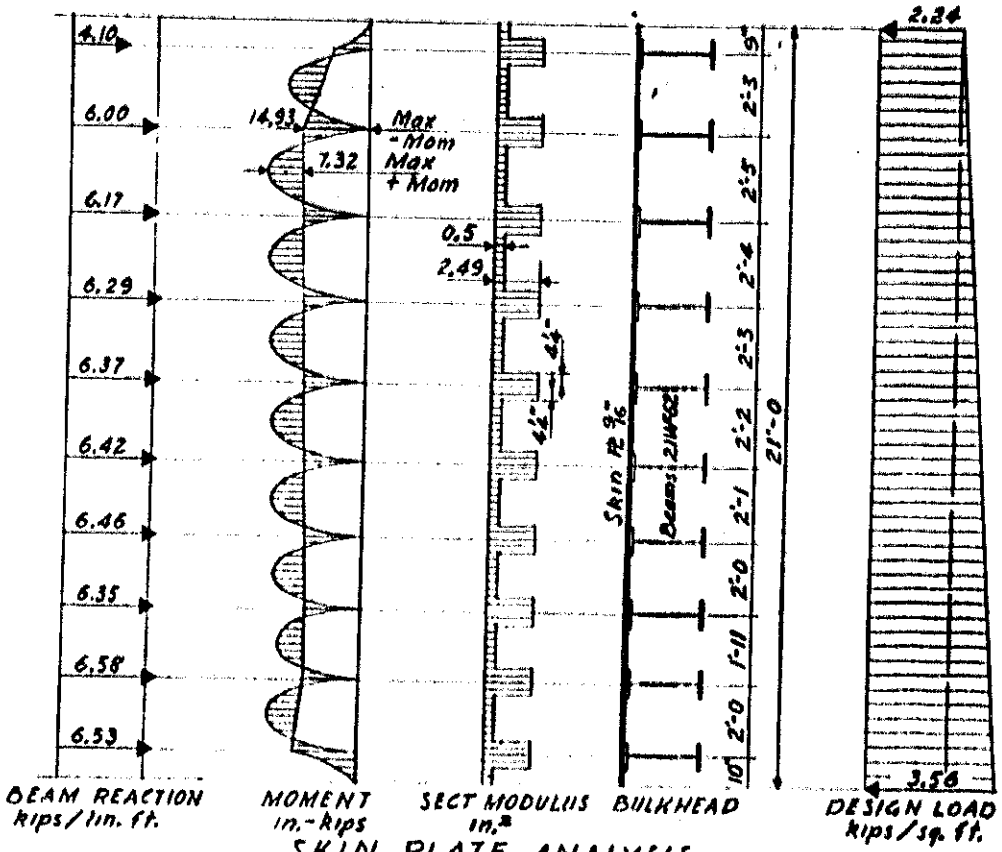
	Lift (feet)	Speed (feet per minute)
Main hooks	68	4
Auxiliary hooks	107	25
Trolley travel		25
Bridge travel		50

(2) Loads and Stresses - The crane was designed for the following loads:

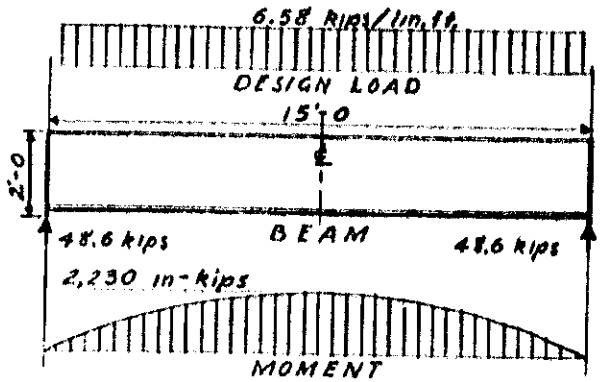
(a) Dead Load - Total weight of crane bridge, cab, drive machinery, walkways, and bridge trucks and wheels.

(b) Live Load - Weight of trolleys, blocks, hooks, ropes, lifting attachments, and lifted loads.

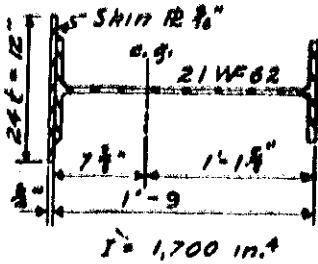




SKIN PLATE ANALYSIS



BEAM ANALYSIS



EFFECTIVE BEAM SEC
Corrosion reduction 1/8
from all exposed surface

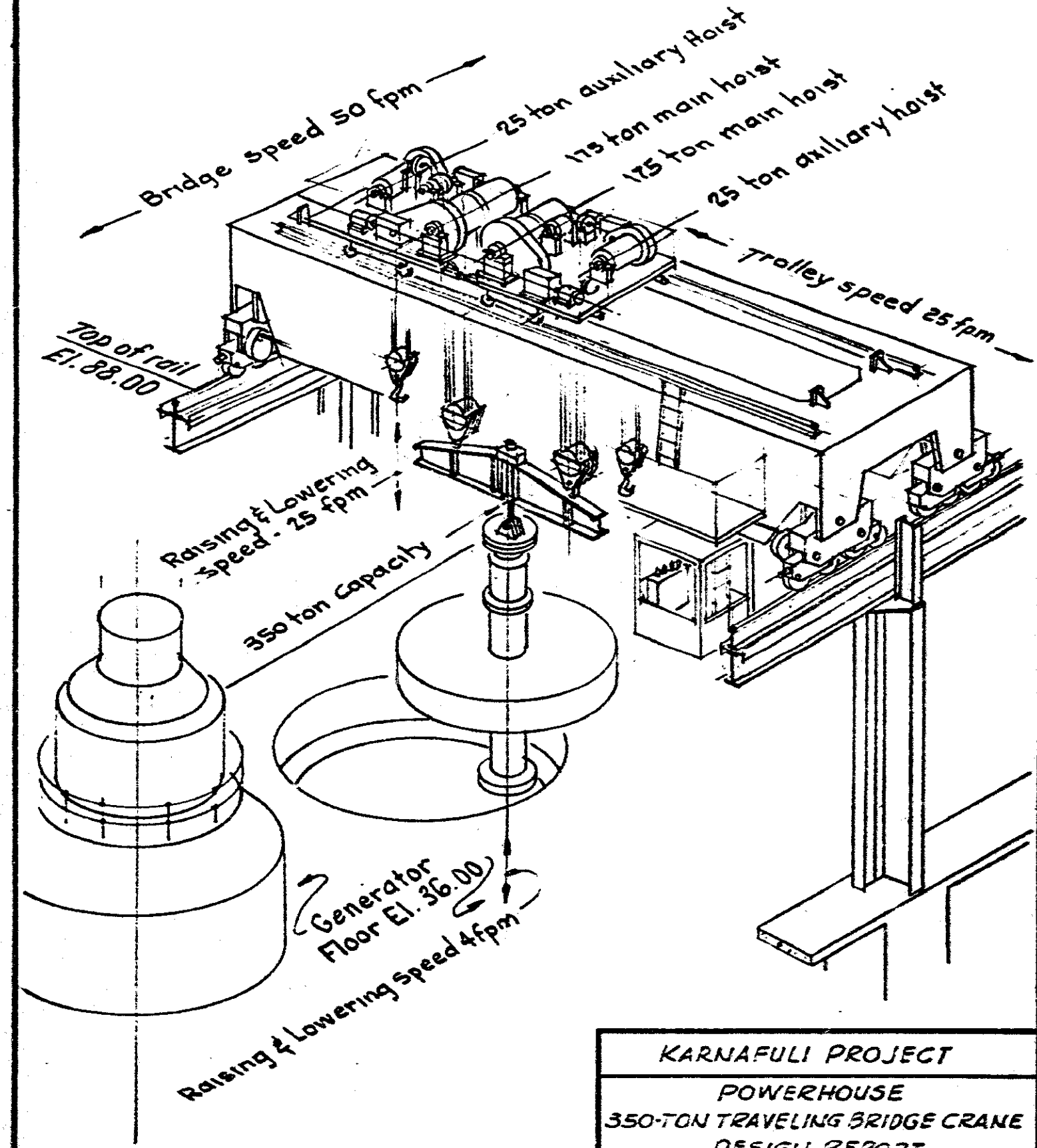
NOTES:
Design stresses are 90
AISC Specifications allowab
(Basic stress 18,000 psi wit
T.W. El. +24.00)

No Scale

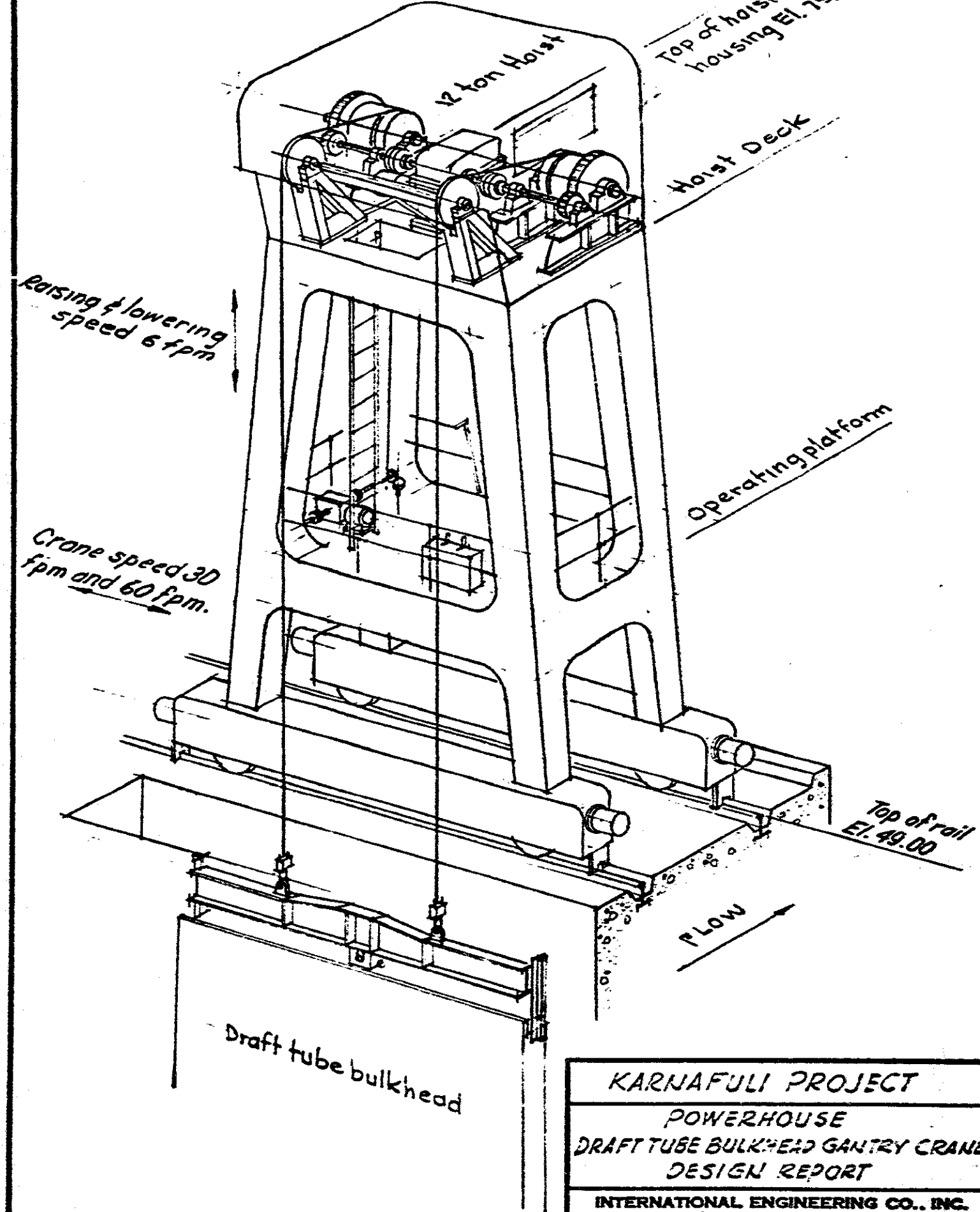
GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNIFULI PROJECT
POWERHOUSE
DRAFT TUBE BULKHEAD
DESIGN ANALYSIS

INTERNATIONAL ENGINEERING CO., I
SAN FRANCISCO, CALIF.

	RECOMMENDED	APPROV
DR. A.R.		
CK.		
DATE: May 25, 1959	FIG. B	



KARNAFULI PROJECT		
POWERHOUSE 350-TON TRAVELING BRIDGE CRANE DESIGN REPORT		
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. IK CK.	RECOMMENDED	APPROVED
DATE: May 25, 1959	FIG. C	



KARNAFULI PROJECT

POWERHOUSE
DRAFT TUBE BULKHEAD GANTRY CRANE
DESIGN REPORT

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. JK	RECOMMENDED	APPROVED
CK.		

DATE: May 25 1959 FIG. D

(c) Earthquake - Ten percent of live load and dead load applied horizontally or vertically with 33-1/3 percent overstress allowed.

(d) Crane Impact Loads - Ten percent of the wheel loads.

(e) Trolley Traction - Ten percent of total live load applied to bridge rail equally on each girder.

(f) Bridge Traction - Ten percent of live load plus ten percent of dead load; seventy-five percent of this total load applied to one side of crane.

The structural members were designed with allowable stresses of 18,000 pounds per square inch in tension and 10,000 pounds per square inch in shear. Allowable compressive stresses were in accord with the American Institute of Steel Construction Specification for the Design, Fabrication and Erection of Structural Steel for Buildings. With rated load, the bridge girder deflection will not exceed 0.001 inch per inch of span.

Mechanical components were designed with a safety factor of five, based on the ultimate strength of the material used. The wire rope was selected with a safety factor of six, as to size, with due allowance for reeving efficiency. Stresses induced at stall torque of the hoist drive motors will not exceed eighty percent of the yield point of the material of any crane component.

(3) Mechanical Features - Mechanical features of the crane include equalizing bridge trucks with sixteen wheels, eight per side, to provide wide distribution of loads. The hoists are drum and reeved wire rope type, with drum capacity to receive all the rope in a single layer. The fleet angle is a maximum of 2-1/2 degrees. The load block design is such that the ropes will not leave the sheaves at any operating position. The main hooks are of the sister type. All bearings are of the anti-friction type, except for the sheaves. All sliding and meshing surfaces are adequately lubricated.

(4) Structural Features - The crane is welded construction. The bridge girders are box type with standard crane rails for the trolley runway mounted on the vertical neutral axes. The girders are secured with rigid-end connections. The bridge trucks are equalizing box-type construction equipped with safety lugs, rail sweeps, and spring bumpers. The trolley frames are box type, single weldments without field joints, and are equipped with safety lugs, rail sweeps, and spring bumpers. A structural steel lifting beam has been provided to connect the two main hooks, for lifts greater than the single hook capacity. An indoor structural steel operator's cab is suspended from bridge to provide the operator with an optimum view. Operating and lubricating instructions are mounted within the cab. Walkways, safety hand rails and ladders have been provided.

(5) Electrical Features - The electrical equipment is suitable for indoor operation and for the power source of 230/400 volts, grounded wye, 3-phase 50-cycle alternating current. The power collector systems for the crane and the trolleys utilize non-corrosive, formed conductors and sliding shoes. Independent full-magnetic controls have been provided for each hoist and travel operation. All electrical components of the controls will "Fail Safe". An alternating current reversing, direct current dynamic lowering brake system has been provided for each hoist. Each hoist and travel operation has been equipped with a holding brake that is released when the respective motor is energized and sets when the motor circuit is interrupted. The bridge is also equipped with a hydraulic-electric brake for slowing and stopping the travel. Each hoist is equipped with upper and lower limit switches and a separate block-activated upper limit switch. The crane is equipped with lights to provide proper illumination under the crane and in the cab. The electrical wiring is enclosed in rigid metal conduits, tagged and marked for permanent identification.

(6) Spare Parts - The following spare parts for the crane are provided:

Brake Lining	100 %
Motor Carbon Brushes	100 %
Contactor Tips	100 %
Fuses	100 %
Wire Rope	50 %

3. Draft Tube Bulkhead Gantry Crane

a. Selection of Type - The cantilever type gantry crane was selected to provide working clearance between the transformers and the gantry legs while hoisting or lowering the draft tube bulkheads. The two-point pickup, with a shallow-depth lifting beam, was used to reduce the crane height to provide adequate clearance below the power transmission lines. The operator's platform was elevated to keep it and the electrical components above high flood levels.

b. Design

(1) Lifts and Speeds - The crane was designed for the following lifts and speeds:

	Lift (feet)	Speed (feet per minute)
Hoist	85	6
Crane Travel		30 and 60

(2) Loads and Stresses - The crane was designed for the following loads:

(a) Dead Load - Total weight of crane structure, hoist machinery, operator's platform, hoist housing, and gantry drive trucks and wheels.

(b) Live Load - Weight of blocks, hook, ropes, lifting beam, and lifted load.

(c) Earthquake - Ten percent of live load and dead load applied horizontally or vertically with 33-1/3 percent overstress allowed.

(d) Wind Load - Ten pounds per square foot for operating condition and 30 pounds per square foot for static condition, both calculated on crane and bulkhead together. Wind load and earthquake loads were not considered to occur simultaneously.

(e) Crane Impact Loads - Ten percent of the wheel loads.

The structural members were designed with allowable stresses of 18,000 pounds per square inch in tension and 10,000 pounds per square inch in shear. Allowable compressive stresses were in accord with the American Institute of Steel Construction Specification for the Design, Fabrication and Erection of Structural Steel for Buildings. With rated load, crane deflections will not exceed 0.001 inch per inch of span.

Mechanical components were designed with a safety factor of five, based on the ultimate strength of the materials used. The wire rope was selected with a safety factor of six, as to size, with due allowance for reeving efficiency. Stresses induced at stall torque of the hoist drive motors will not exceed eighty percent of the yield point of the material of any crane component.

The crane righting moment is a minimum of 110 percent of the combined overturning moment caused by maximum hoist motor torque and wind loads when the crane is operating.

(3) Mechanical Features - Mechanical features include a motor-driven, self-locking worm gear reducer with two output shafts connected by flexible couplings to the two hoist drum assemblies. The drums can receive all the wire rope for the direct lift in single layers. The ropes are secured to the bulkhead lifting beam through open bridge rope sockets to provide a means for adjusting the rope tension. (Note: The lifting beam was supplied with the bulkheads.) The self-locking feature of the gear reducer functions as a load brake during the lowering operation. All bearings are the anti-friction type, except for the sheaves. All sliding and meshing surfaces are adequately lubricated.

(4) Structural Features - The crane is box-type construction with a hoist housing on the top. A counterweight is provided to maintain the righting moment. An operator's platform is located within the legs of the crane to afford an optimum view. The operating and lubricating instructions are mounted adjacent to the controls.

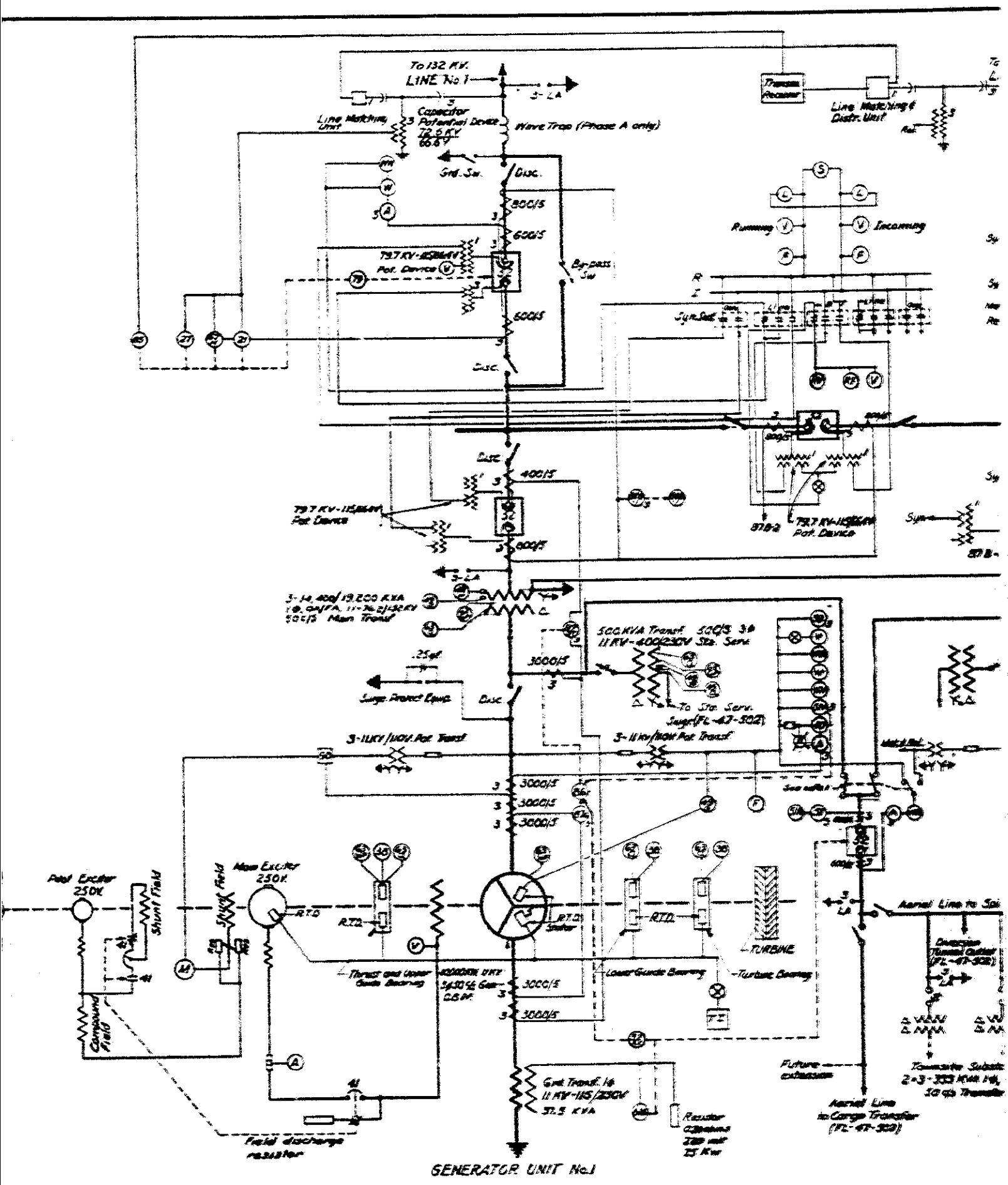
Safety hand rails and ladders are provided. The trucks are equipped with safety lugs, rail sweeps, and spring bumpers.

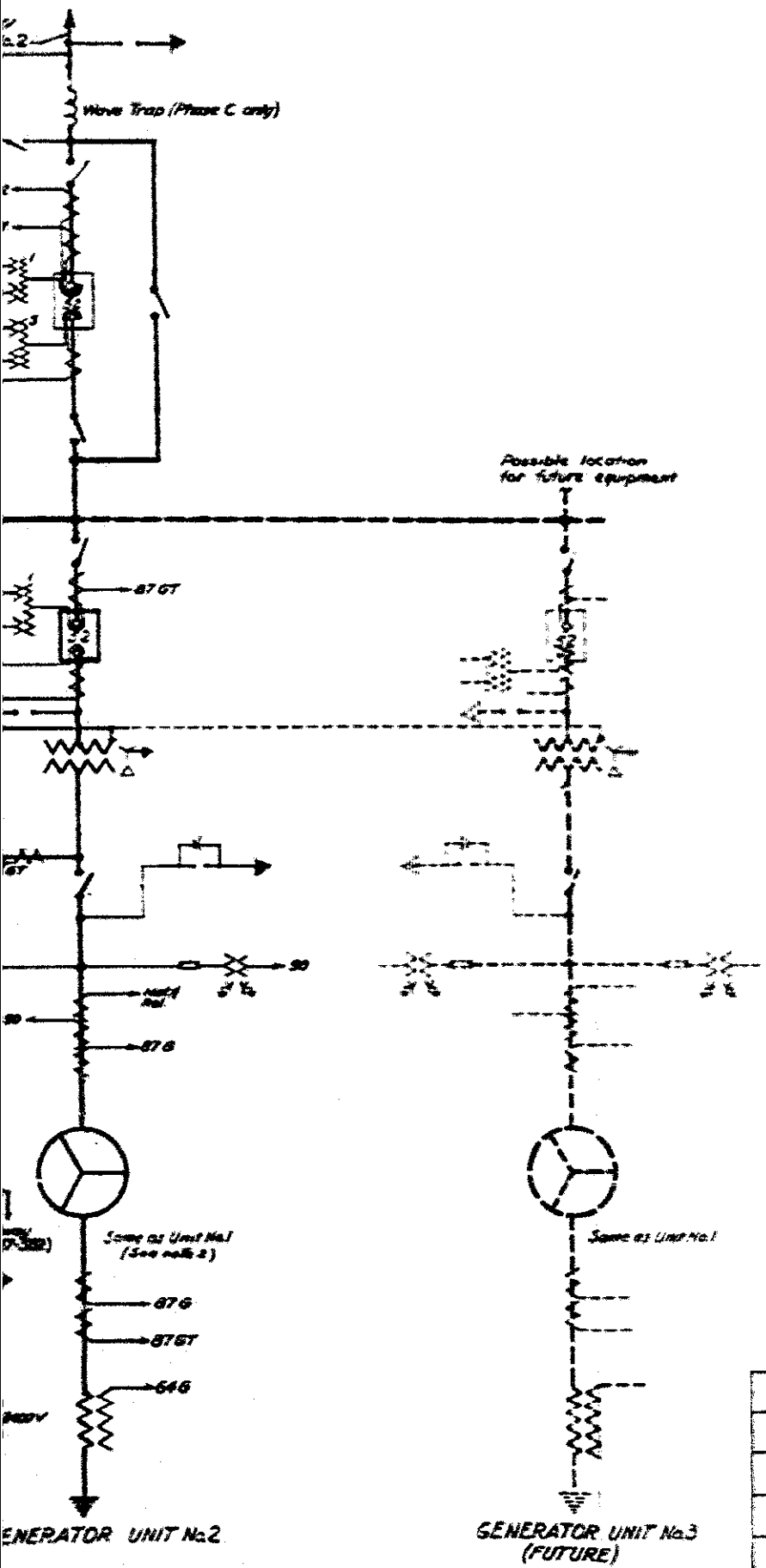
(5) Electrical Features - The electrical equipment is suitable for outdoor operation and for the power source of 230/400 volts grounded wye, 3-phase 50-cycle alternating current. The power is supplied to the crane by a portable power cable and automatic cable reel. All the electrical components of the controls will "Fail Safe". The crane is equipped with manual controls. The hoist is equipped with a holding brake that is released when the hoist motor is energized and sets when the motor circuit is interrupted. The hoist is equipped with upper and lower limit switches. Adequate lighting has been supplied on the crane to facilitate night operation. The electrical wiring is enclosed in rigid metal conduits, tagged and marked for permanent identification.

PART V - CHAPTER IV

List of Exhibits

Exhibit	Title
IV-1	Main Single Line Diagram
IV-2	Auxiliary Power Single Line Diagram
IV-3	Misc. Small Alternating-Current and Direct-Current Power Single Line Diagram
IV-4	Raw, Cooling, Service and Fire Water Piping Diagram
IV-5	Treated Water System Piping Diagram
IV-6	Governor and Service Air and CO_2 Fire System Piping Diagram
IV-7	Unwatering, Drainage, Soil and Waste Piping Diagram
IV-8	Heating, Ventilating and Air Conditioning Piping Diagram
IV-9	Insulating and Lubricating Oil Systems Piping Diagram


To 132 KV. LINE No 1
Line Matching Unit
Capacitor Potential Device
Wave Trap (Phase A only)
Line Matching & Distr. Unit
Grd. Sw.
Disc.
800/5
600/5
Pot. Device
By-pass Sw
500/5
Running
Incoming
Sys. Det.
400/5
Pot. Device
600/5
3-14,400/19,200 KVA
Main Transf.
500 KVA Transf. 500/5 3φ
11 KV-400/230V Sta. Serv.
3000/5
Surge Protect Equip.
To Sta. Serv.
3-11KV/110V Pot. Transf.
3-11KV/110V Pot. Transf.
Pilot Exciter 250V.
Shunt Field
Main Exciter 250V.
Compound Field
Thrust and Upper Guide Bearing
Stator
Lower Guide Bearing
TURBINE
Turbine Bearing
Kanal Line to Sal.
Future extension
Kanal Line to Cargo Transfer
Field discharge resistor
Resistor
SENERATOR UNIT No.1



LEGEND

ASA standard device numbers and symbols are used for relays and instruments.
Relay suffix letters indicate location or condition to which device responds as follows:

- B — Bus
- G — Generator
- GP — Gas Pressure
- GT — Generator-Transformer Unit
- N — Neutral
- O — Oil
- V — Voltage Restraint
- W — Winding
- WF — Water Flow

Instrument letters are defined as follows:

- A — Ammeter
- F — Frequency Meter
- L — Lamp
- M — Motor
- PMG — Permanent Magnet Generator
- RF — Recording Frequency-Meter
- RV — Recording Voltmeter
- S — Synchroscope
- SI — Speed Indicator
- TI — Temperature Indicator
- V — Voltmeter
- VAR — VARmeter
- W — Wattmeter
- Wh — Watthour-meter
- ⊗ — Instrument Switch
- ⊗ — Indicating Lamp, blue cap

NOTES

1.- Air break transfer switches connecting the 11KV OCB to the generator buses shall be interlocked to prevent simultaneous closing of both switches.
2.- Details of protection, metering and control circuits are not shown for unit No.2 unless different from unit No.1.

REFERENCE DRAWINGS

Auxiliary Power Single Line Diagram	FL-47-[illegible]
Misc. Small A.C. & D.C. Single Line Diagram	FL-47-[illegible]

No Scale

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT

MAIN SINGLE LINE DIAGRA[illegible]

INTERNATIONAL ENGINEERING CO., INC. - ENGINEER[illegible]

No.	Date	Revision	Drawn	Checked
5	March 21, 1956	Built	[illegible]	[illegible]
		As built		
4	May 18, 1955	RC	JA	[illegible]
		General Changes		
3	March 5, 1953	JA	CC	[illegible]
		Chg PT & EPO V Relays		
2	Sept 24, 1938	AL	JA	[illegible]
		Add Voltage Line & Meters		
1	Sept 2, 1938	[illegible]	JA	[illegible]
		Added bus interrupting OCB		

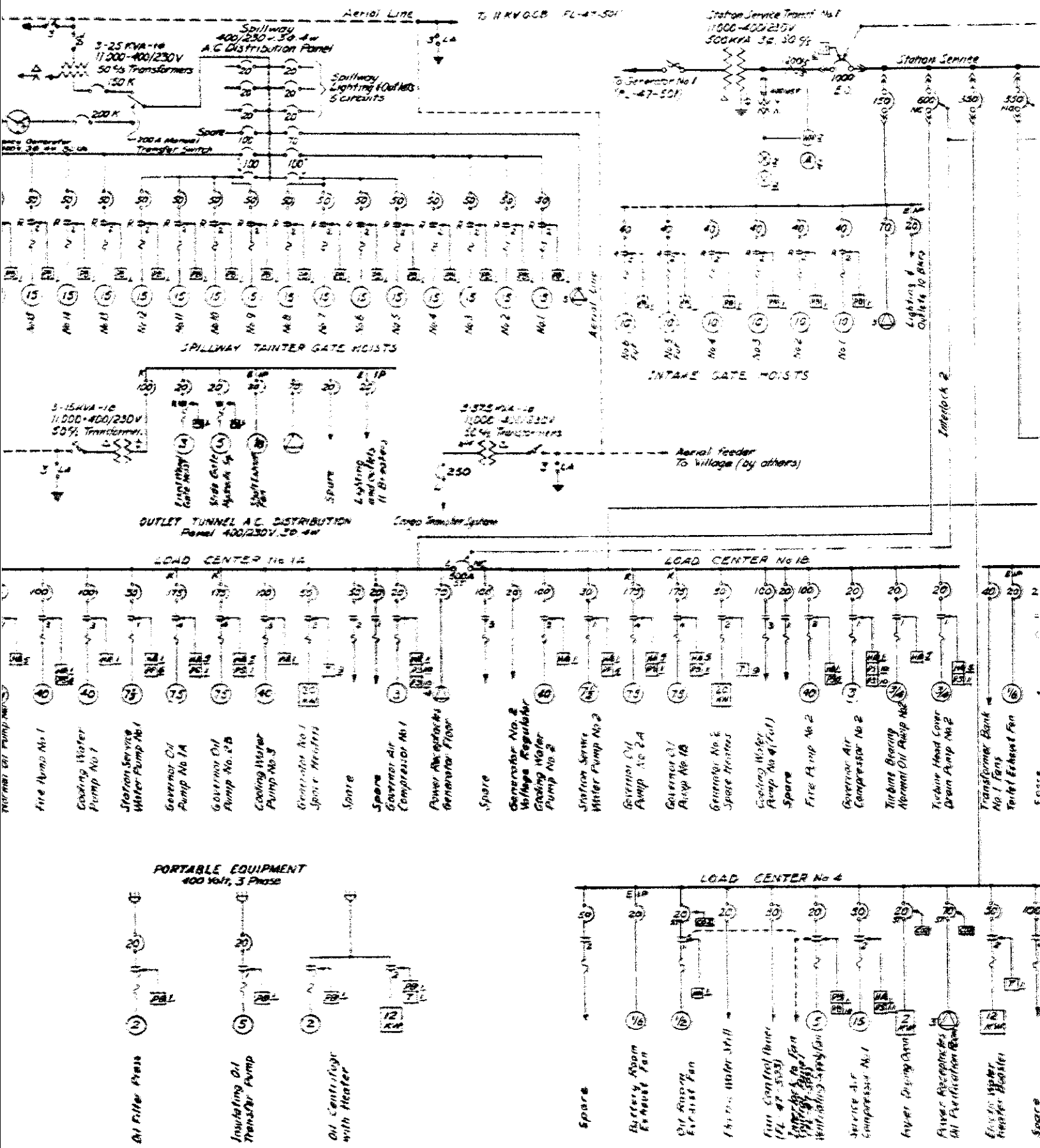
REVISIONS

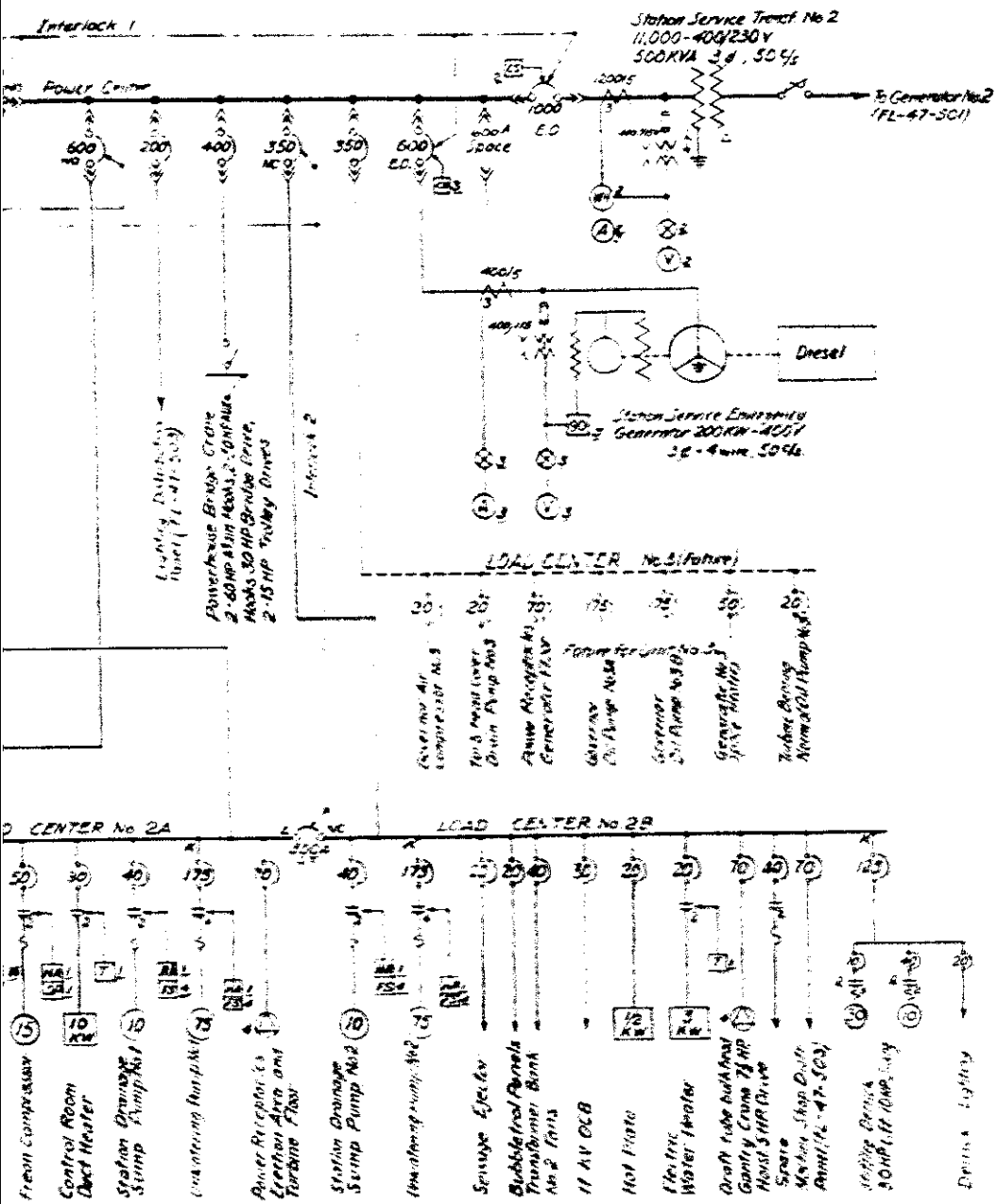
R.K. — Drawn; I.R. — Checked; JA, DGG — Approved; FL [illegible] — Scale; [illegible] — Date

KA.-X.-1

SAN FRANCISCO, CAL. March 28, 1938

FL-47-5[illegible]


Aerial Line
Spillway 400/230 V, 3φ 4w A.C. Distribution Panel
3-25 KVA-1φ 11000-400/230V 50% Transformers
Spillway Lighting (Outlets) 5 circuits
Spare
Transfer Switch
Station Service Transf. No 1 11000-400/230V 500 KVA 3φ 50 ~
Station Service
To Generator No 1
SPILLWAY TAINTER GATE HOISTS
INTAKE GATE HOISTS
Lighting & Outlets to Bldg
Interlock 2
3-15 KVA-1φ 11000-400/230V 50% Transformers
Aerial feeder To Village (by others)
OUTLET TUNNEL A.C. DISTRIBUTION Panel 400/230V, 3φ 4w
Cargo Transfer System
LOAD CENTER No 1A
LOAD CENTER No 1B
Fire Pump No 1
Cooling Water Pump No 1
Station Service Water Pump No 1
Governor Oil Pump No 1A
Governor Oil Pump No 1B
Cooling Water Pump No 3
Spare
Governor Air Compressor No 1
Power Receptacles Generator Floor
Generator No 2 Voltage Regulator
Cooling Water Pump No 2
Station Service Water Pump No 2
Governor Oil Pump No 2A
Governor Oil Pump No 2B
Fire Pump No 2
Governor Air Compressor No 2
Turbine Bearing Normal Oil Pump No 2
Turbine Head Cover Drain Pump No 2
Transformer Bank No 1 Fans
Toilet Exhaust Fan
PORTABLE EQUIPMENT 400 Volt, 3 Phase
Oil Filter Press
Insulating Oil Transfer Pump
Oil Centrifuge with Heater
LOAD CENTER No 4
Battery Room Exhaust Fan
Oil Room Exhaust Fan
Service Air Compressor No 1
Power Receptacles Oil Purification Room
Electric Water Heater Booster



LEGEND

- A — Ammeter
- CS — Control Switch
- DP — Dual Pressure Control
- E.O. — Electrically Operated Breaker
- FS — Float Switch
- HA — Hand-off-Automatic Switch
- NC — Normally Closed
- NO — Normally Open
- PB — Push Button
- PF — Combined Pressure & Float Control
- PS — Pressure Switch
- ST — Breaker equipped with Shunt Trip Coil
- T — Thermostat
- V — Voltmeter
- W.H. — Watt hour meter
- Instrument Switch
- Electrical Interlock
- Combination Motor Starter number indicates NEMA size, letter R reversing starter
- Power Receptacle, number indicates quantity
- Motor, Number indicates H.P.
- Overcurrent Relay
- Oil Circuit Breaker
- Voltage Regulator
- CO2 — Breaker trips when CO_2 released

LOCATION NUMBERS

- Control station or equipment
- Duplex control board
- Station service emergency generator
- Sump
- Actuator Panel
- Pneumatic service water tank
- Load Center No 1
- Load Center No 2
- Main generator housing
- Governor oil pressure tank
- Compressed air receiver
- Governor air receiver
- Turbine pit alcove
- Fan Control panel
- Air Conditioning control panel

NOTES

All ratings shown are subject to change upon receiving final information from equipment manufacturers.

Interlock 1 shall prevent paralleling of normal sources or paralleling of normal and emergency sources.

Interlock 2 shall prevent paralleling of feeders to Load Centers.

All load center and distribution panel circuit breakers are E frame, 3 pole unless otherwise indicated.

REFERENCE DRAWINGS

Main Single Line Diagram	FL-47-501
Misc. Small A.C. & D.C. Single Line Diagram	FL-47-503
Interlocking and Control Diagrams - Sheet 1	FL-47-504
Interlocking and Control Diagrams - Sheet 2	FL-47-505

Not to Scale

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT

AUXILIARY POWER SINGLE LINE DIAGRAM

INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS

REV.	DATE	DRAWN	CHECKED	APPROVED
4	May 13, 1959	RC	JA	[illegible]
	General Changes			
3	4-13-59	RC	[illegible]	[illegible]
	General Changes			
2	3-5-59	JA	[illegible]	[illegible]
	General Changes			
1	5-28-58	Br M	JA	[illegible]
	General Changes			

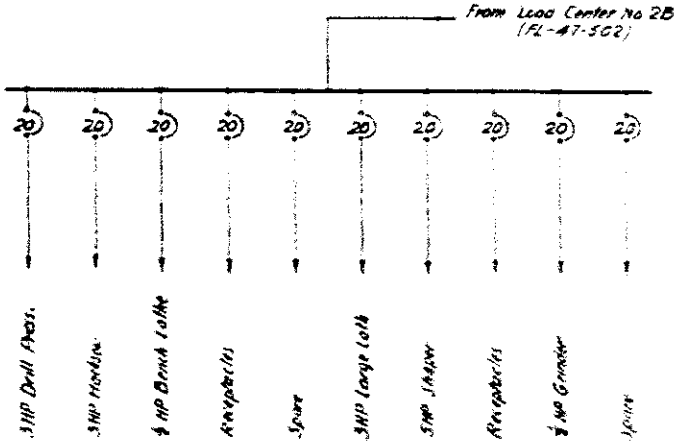
REVISIONS

Designed JA
Drawn ZR
Chkd. DGG
Appd. [illegible]

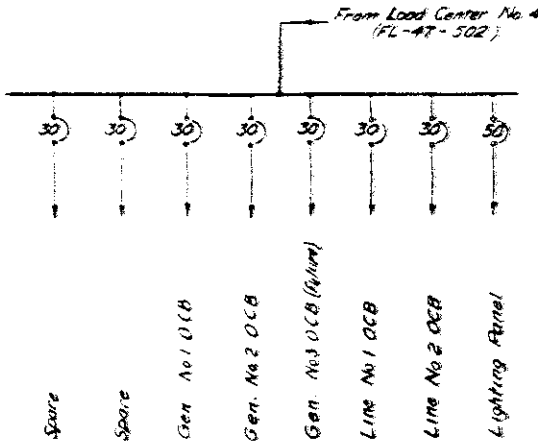
ICA-X-1

San Francisco, Cal.
Date March 28, 1958

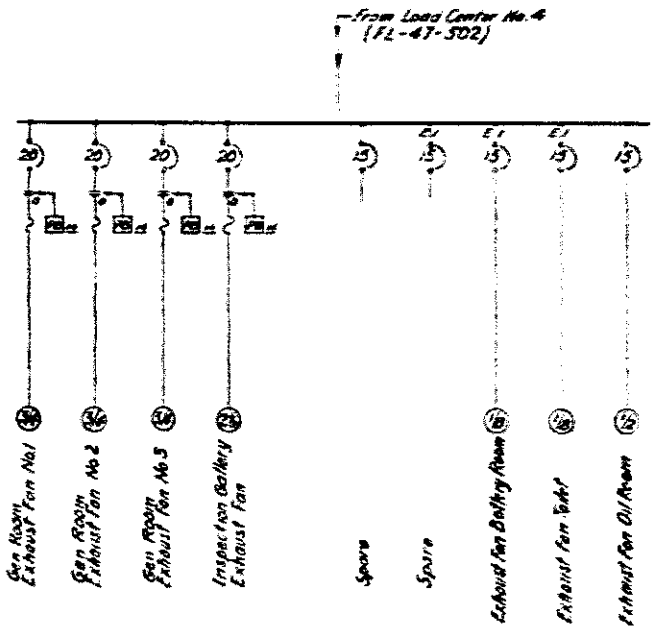
FL-47-502 R4



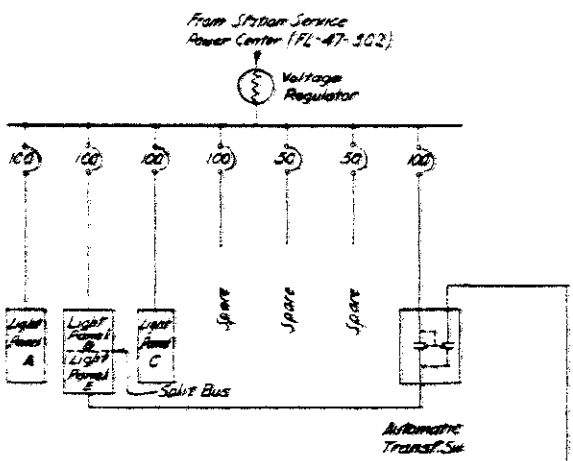
MACHINE SHOP A.C. Distribution Panel
400/230V-3φ.4w



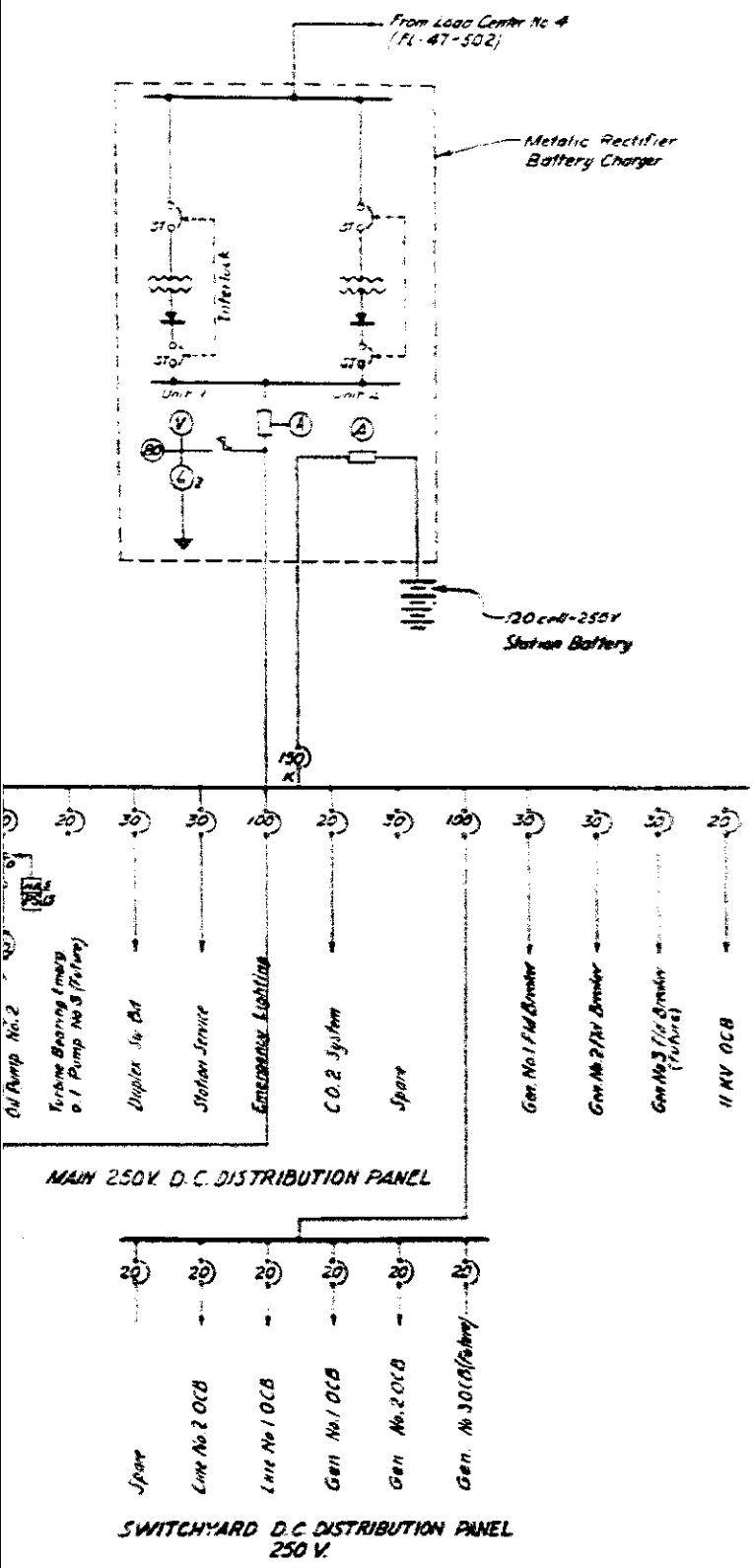
SWITCHYARD A.C. Distribution Panel
400/230V. 3φ-4W



EXHAUST FAN PANEL
400/230V - 3φ.4w



LIGHTING DISTRIBUTION PANEL
400/230V. 3φ-4w



LEGEND

CS Control Switch
HA Hand-off-Automatic Switch
PS Pressure Switch
Contactor Motor Starter, number indicate NEMA size
A Ammeter
V Voltmeter
Motor, Number indicates HP
Ground Detector Light
80 D-C Undervoltage Relay
E/1 Single Pole, 230 volt E frame circuit breaker
Transformer
Metallic Rectifier
ST Shunt Trip attachment

LOCATION NUMBERS

1 Actuator Panel
2 Turbine bearing
4 Fan Control Panel

NOTES

All ratings shown are subject to change upon receiving final information from equipment manufacturer.

All AC circuit breakers are F frame, 3 pole unless otherwise indicated.

All DC circuit breakers are F frame, 2 pole unless otherwise noted.

REFERENCE DRAWINGS

Main Single Line Diagram	FL-47-501
Auxiliary Power – Single Line Diagram	FL-47-502

Not to Scale

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT

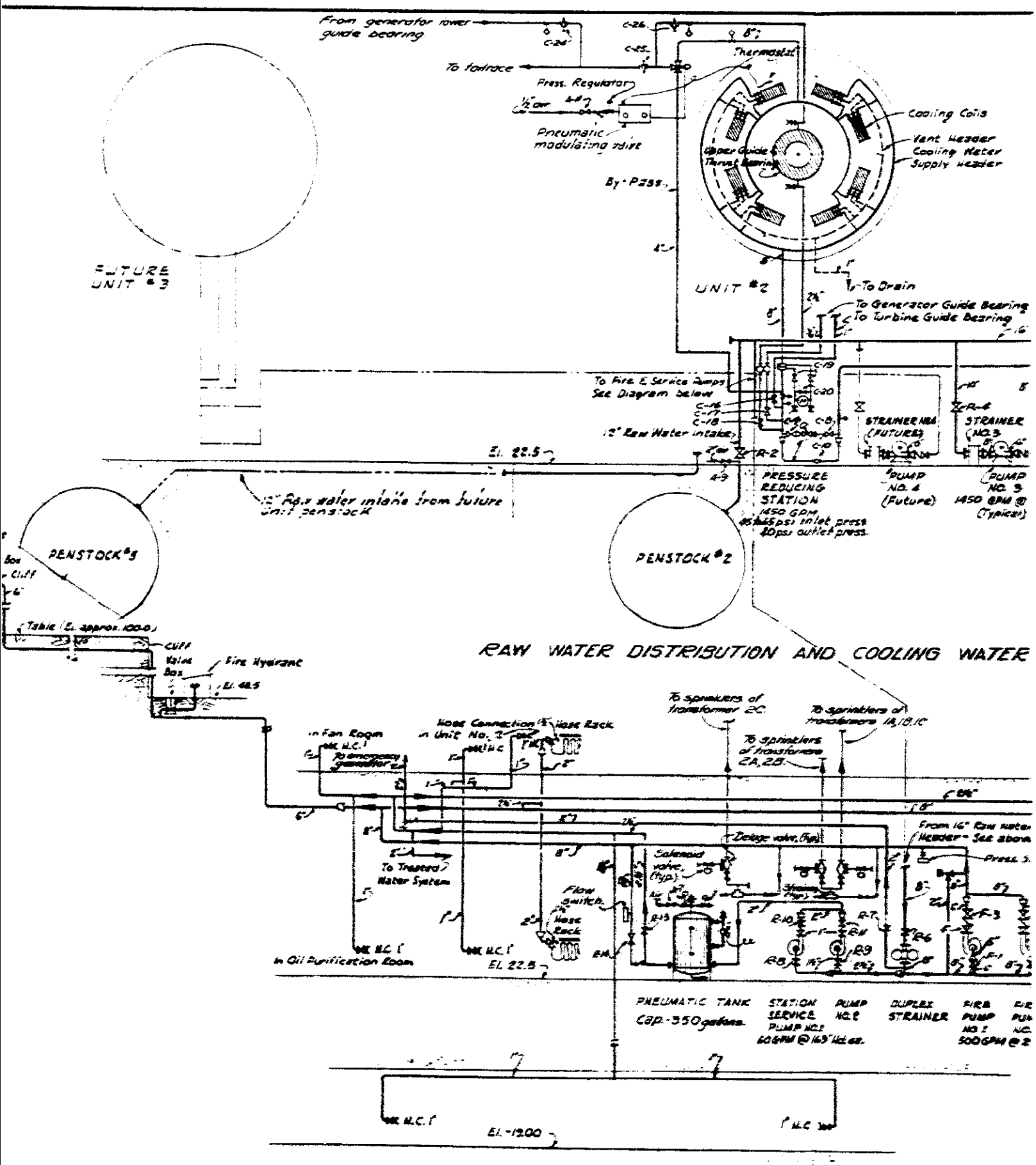
MISC. SMALL A.C. & D.C. POWER SINGLE LINE DIAGRAM

INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS

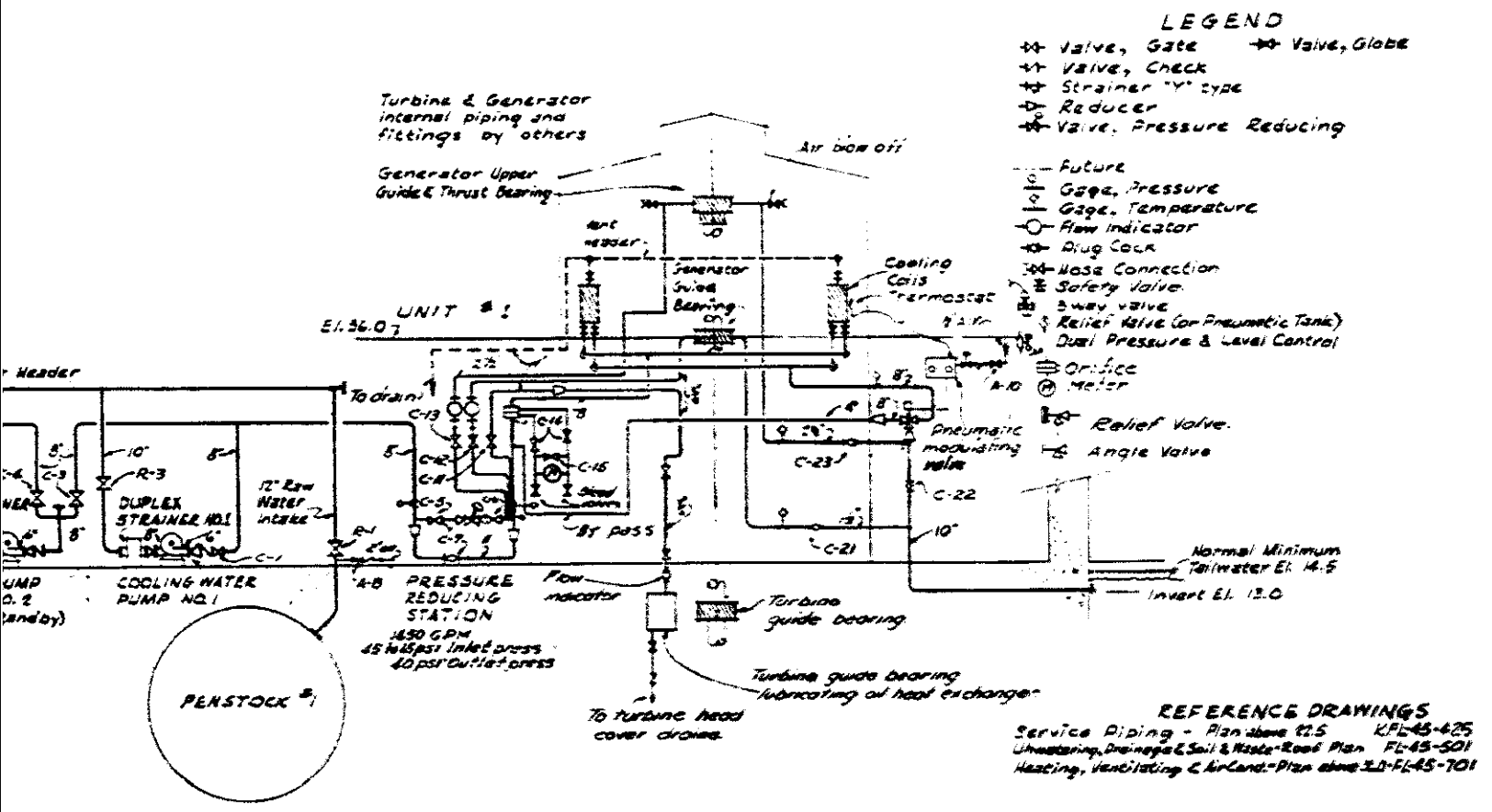
4			
3			
2			
1			
No.	Date	Revision	Approved

REVISIONS

ICA.-X-1	SAN FRANCISCO, CAL. March 28, 1958	FL-47-503

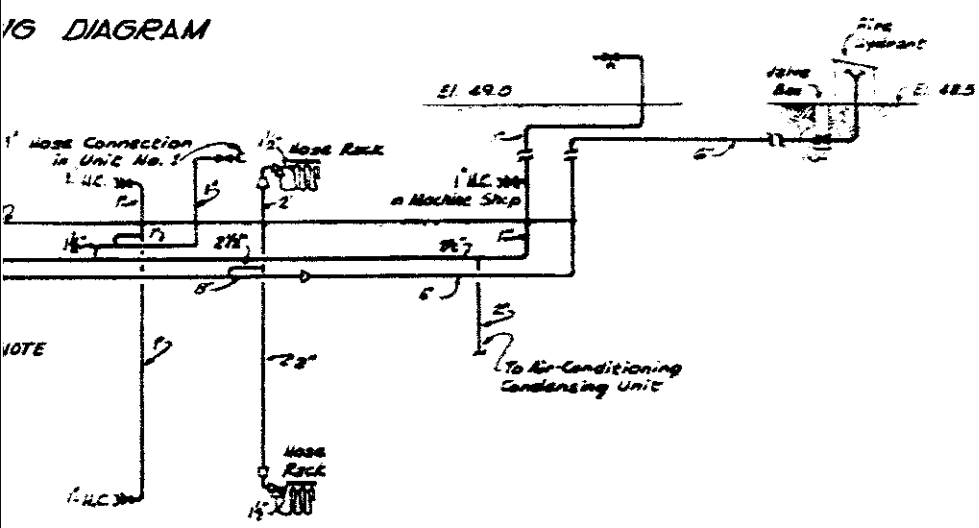


FIRE PROTECTION AND STATION SERVICE WATER PIPING DIAGRAM



REFERENCE DRAWINGS

Service Piping - Plan above 12.5 KFL-45-625
Plumbing, Drainage & Soil & Waste-Roof Plan FL-45-501
Heating, Ventilating & Air Cond.-Plan above 3.0 FL-45-701



LEGEND

R - Raw water
C - Cooling water
F - Fire water
D - Domestic water (after chlorination)
L - Lube oil
A - Air
T - Insulating oil
GS - Gland seal water

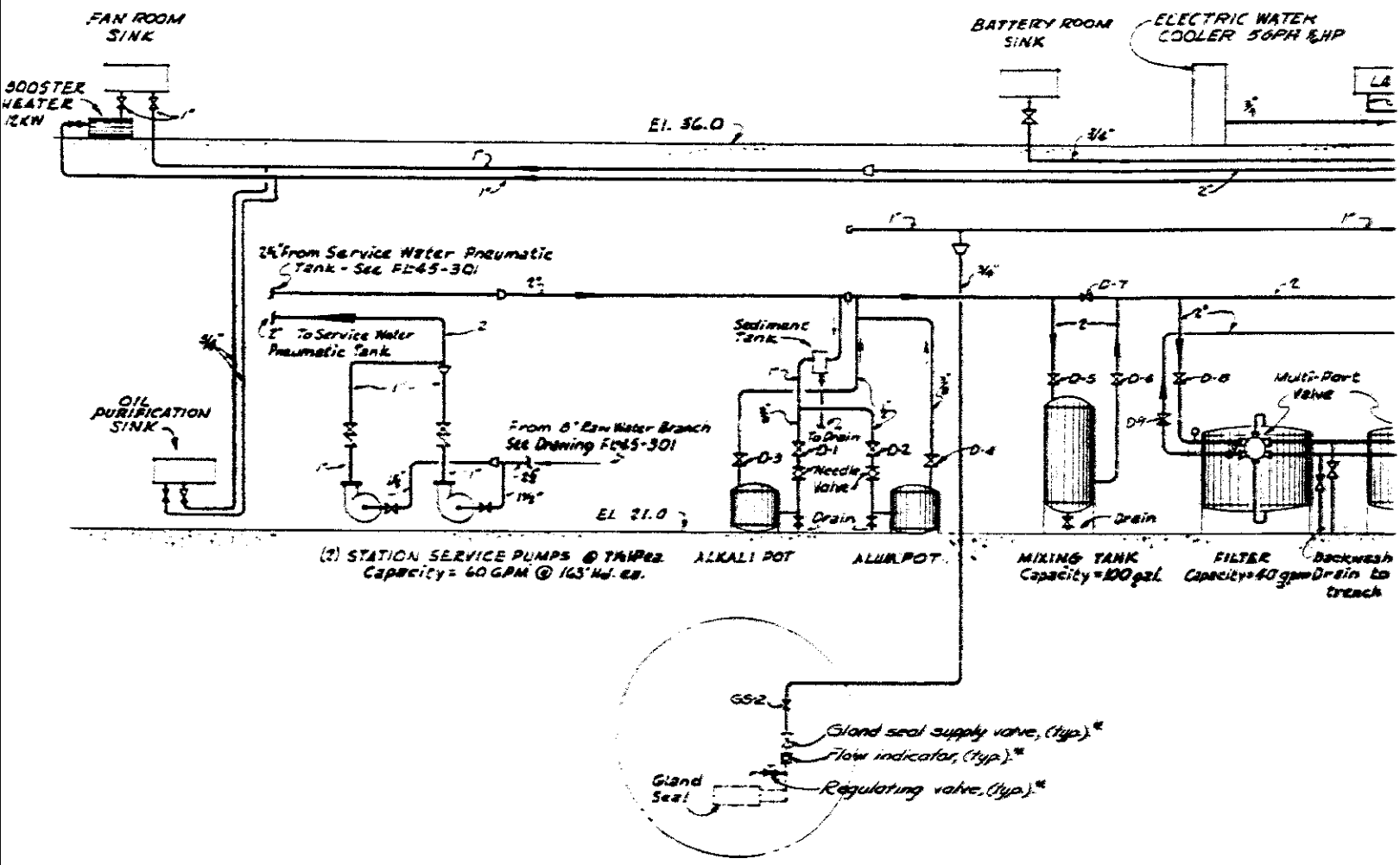
NOTE:
Equipment valves that are not numbered are supplied by the manufacturer of the equipment.
Fire pumps pressure switch setting:
#1 Pump ON at 56 psig
#2 Pump ON at 52 psig

REV.	DATE	DRAWN	CHKD	APPD	REVISIONS
6	Jan 19, 1962	J.T.			Selected valve added to pressure test
5	Jan 4, 1962	J.T.			Minor revisions
4	Oct. 20-61	R.L.	BMM		Minor Revisions
3	Sep 8, 61	J.T.			General revision
2	June 16-59	CMD			
1	Feb 2-1959	H.T.	HW		Minor Revisions

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
POWERHOUSE
RAW, COOLING, SERVICE AND FIRE WATER PIPING DIAGRAM
INTERNATIONAL ENGINEERING CO. INC. - ENGINEERS

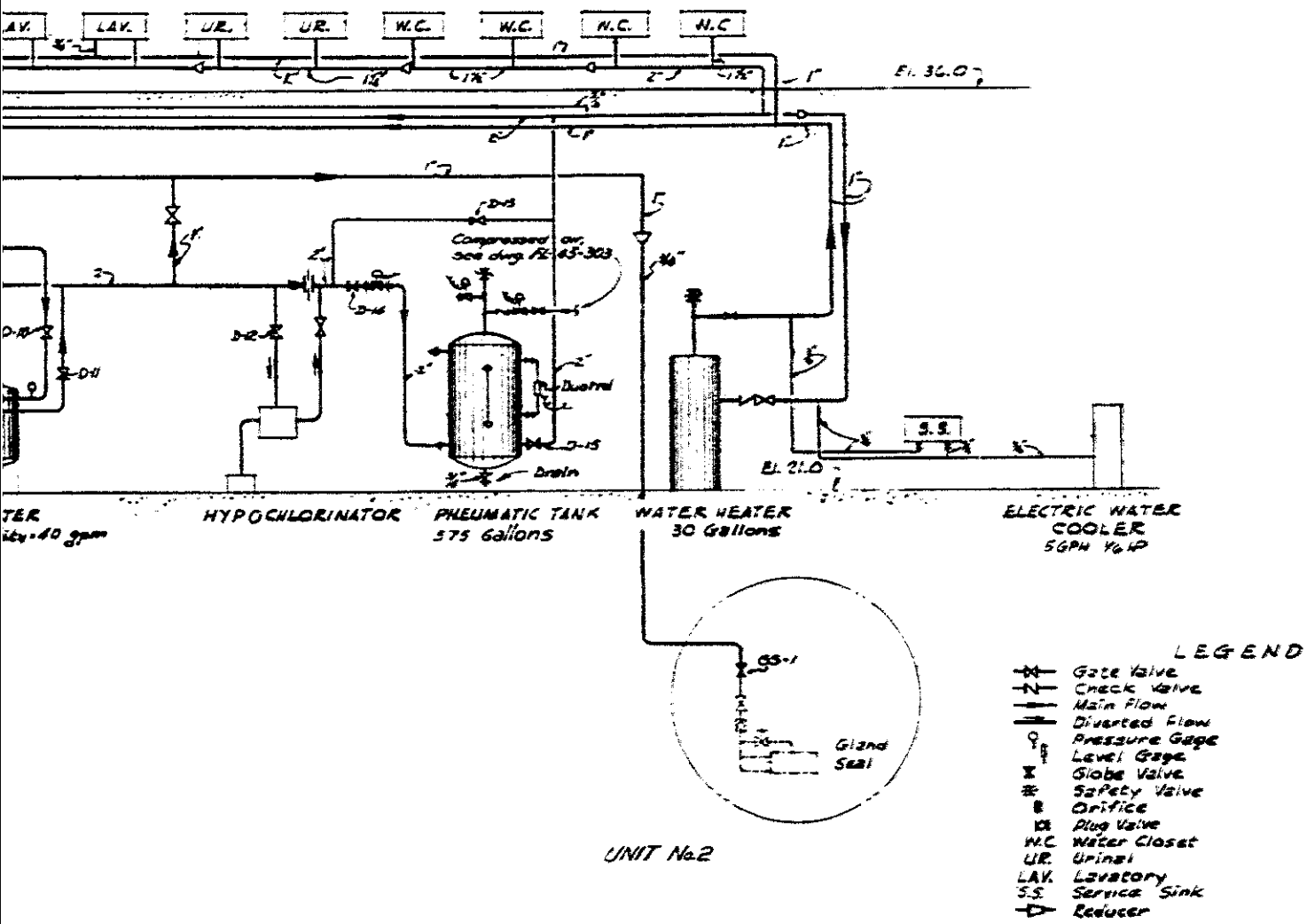
DESIGNED RAS
DRAWN RAS
CHECKED HW
APPROVED E.S.

KA-X-1
SAN FRANCISCO, CAL.
DATE 7/10/58
FL-45-301



UNIT No. 1

* Furnished by turbine manufacturer.



UNIT No.2

LEGEND

- Gate Valve
- Check Valve
- Main Flow
- Diverted Flow
- Pressure Gage
- Level Gage
- Globe Valve
- Safety Valve
- Orifice
- Plug Valve
- W.C. Water Closet
- U.R. Urinal
- LAV. Lavatory
- S.S. Service Sink
- Reducer

REFERENCE DRAWINGS

For Reference Drawings and General Notes see drawing Fz45-301

4	Jan. 19, 1962	JT	
	Minor revisions		
3	Jan. 4, 1962	JT	
	Minor revisions		
2	Oct. 20-1961	PC	BGM
	Minor Revisions		
1	Feb. 2-1959		
	Minor Revisions		

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

KARNAFULI PROJECT

POWER HOUSE

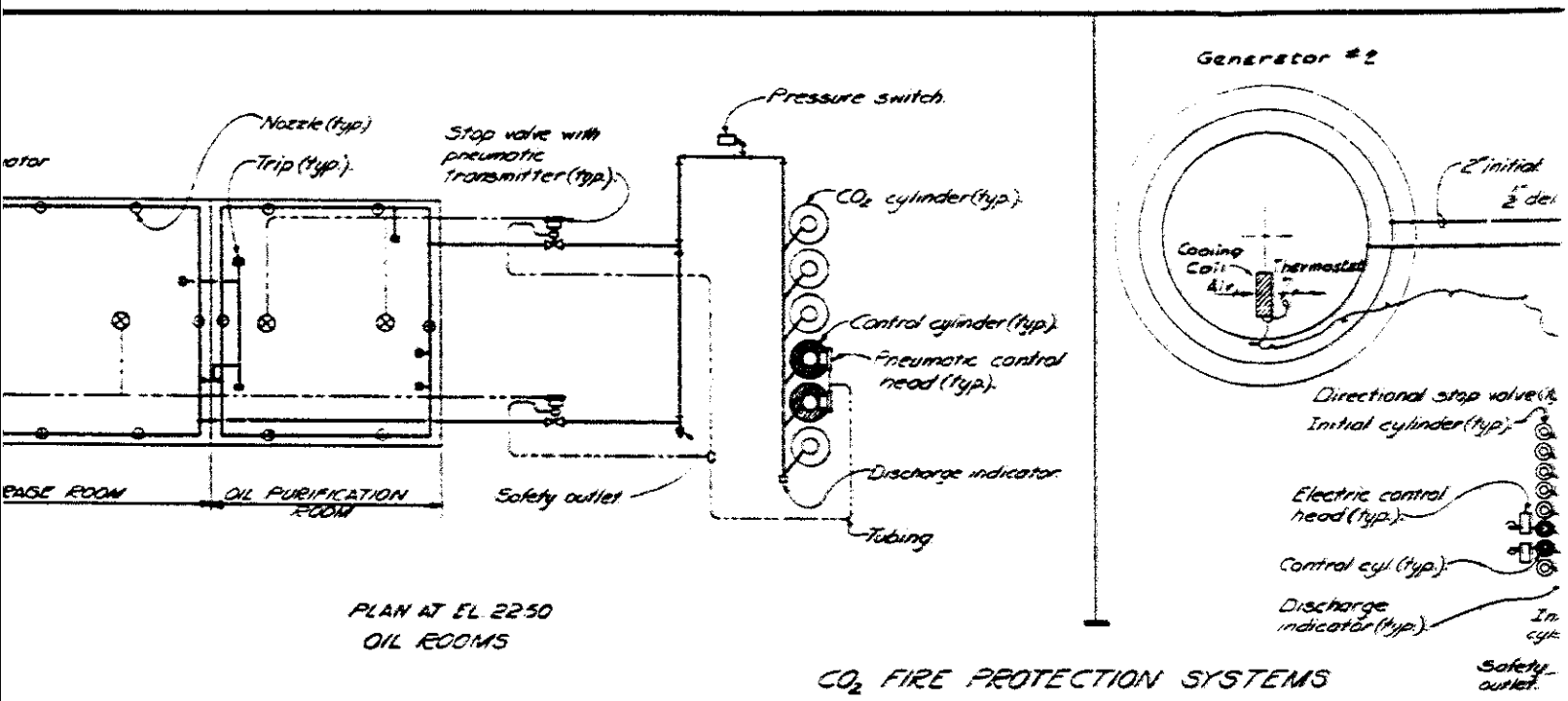
TREATED WATER SYSTE

PIPING DIAGRAM

INTERNATIONAL ENGINEERING CO. INC. - ENGINEER

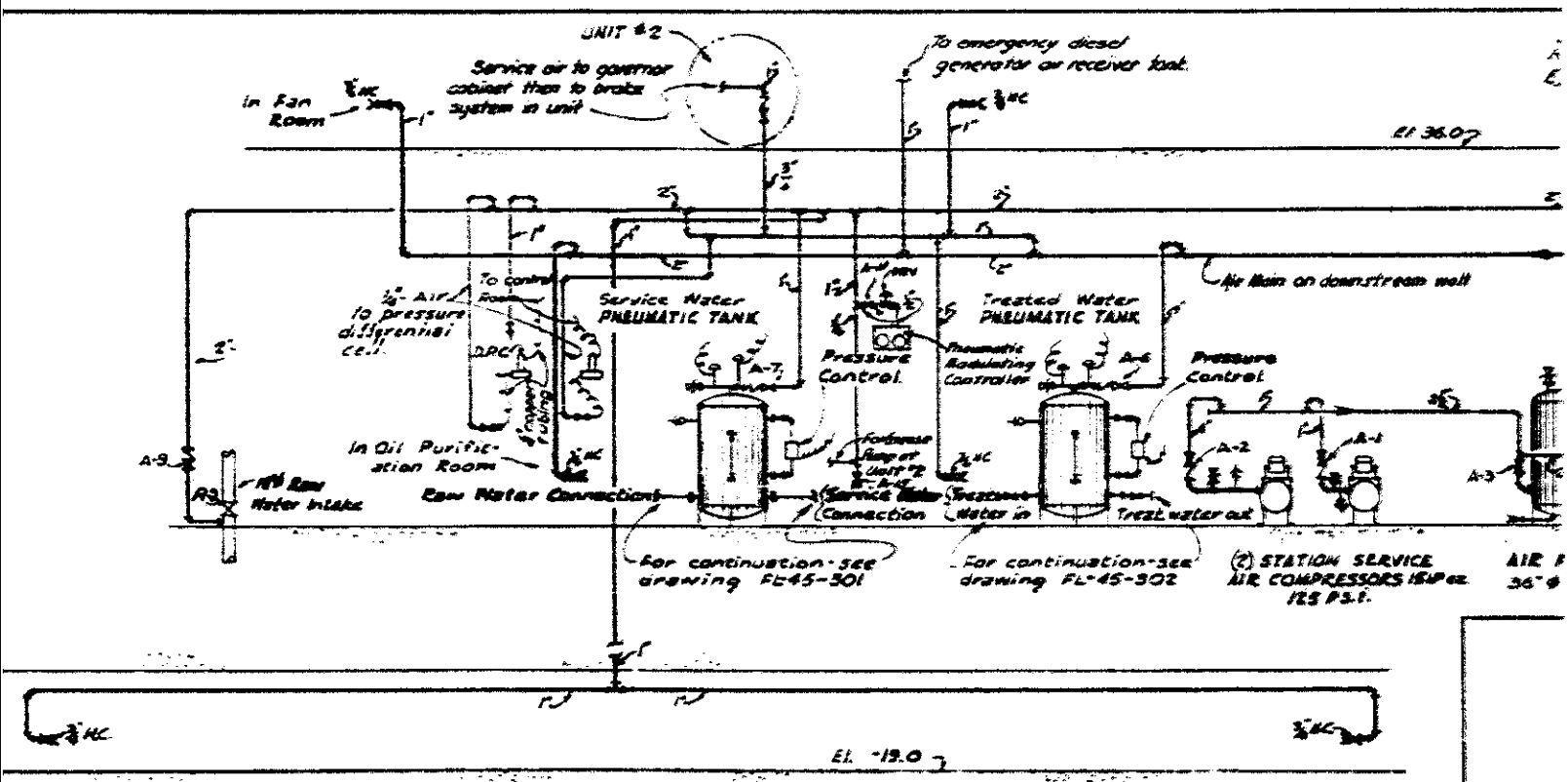
RAS RAS

SAN FRANCISCO, CAL.

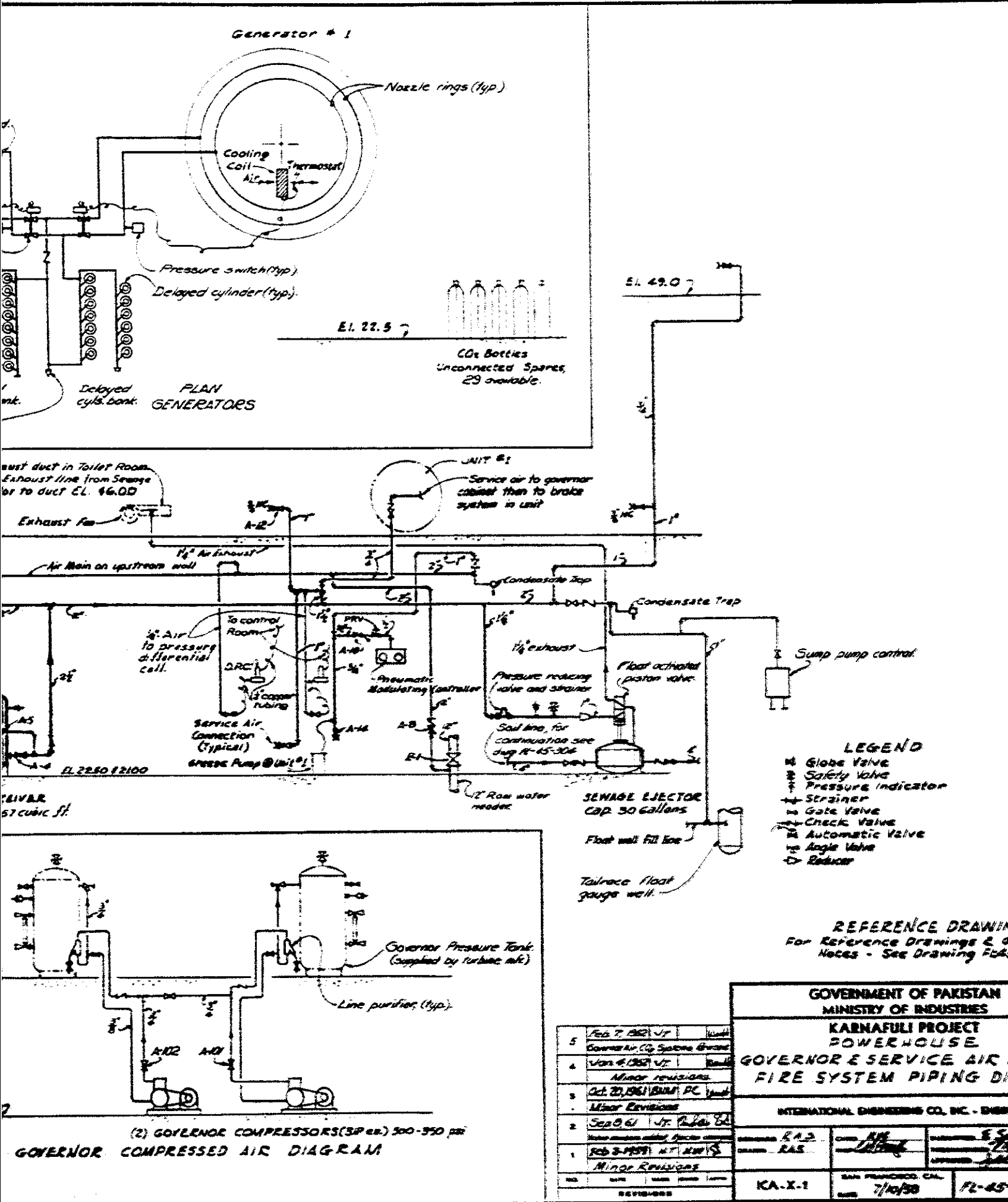


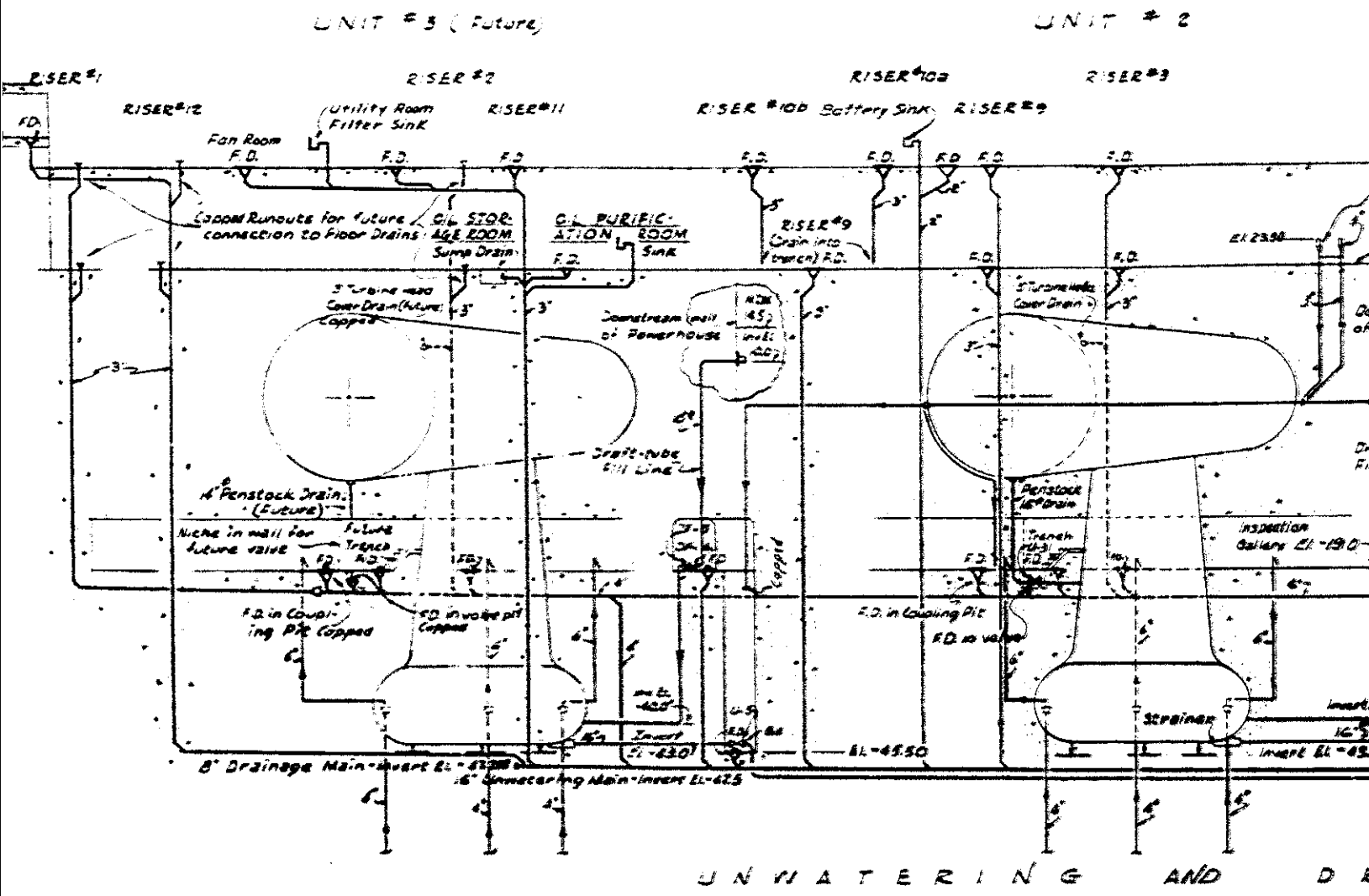
PLAN AT EL 22'50
OIL ROOMS

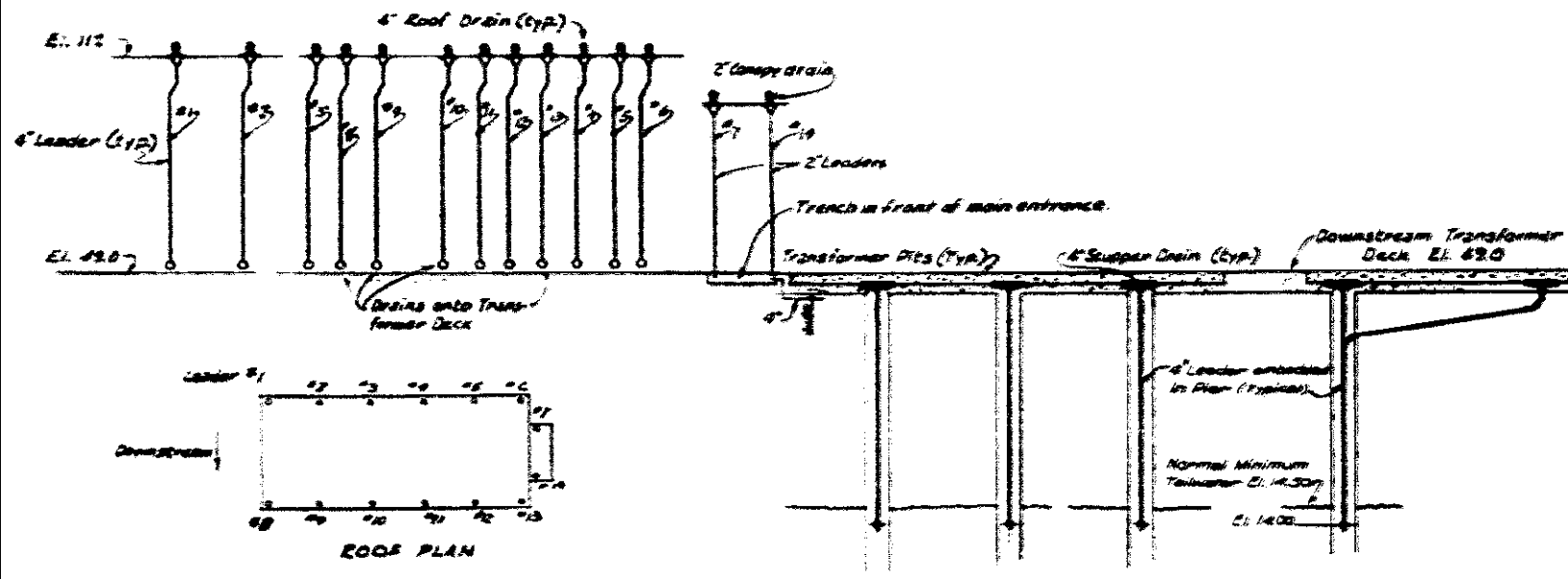
CO_2 FIRE PROTECTION SYSTEMS



STATION SERVICE COMPRESSED AIR DIAGRAM

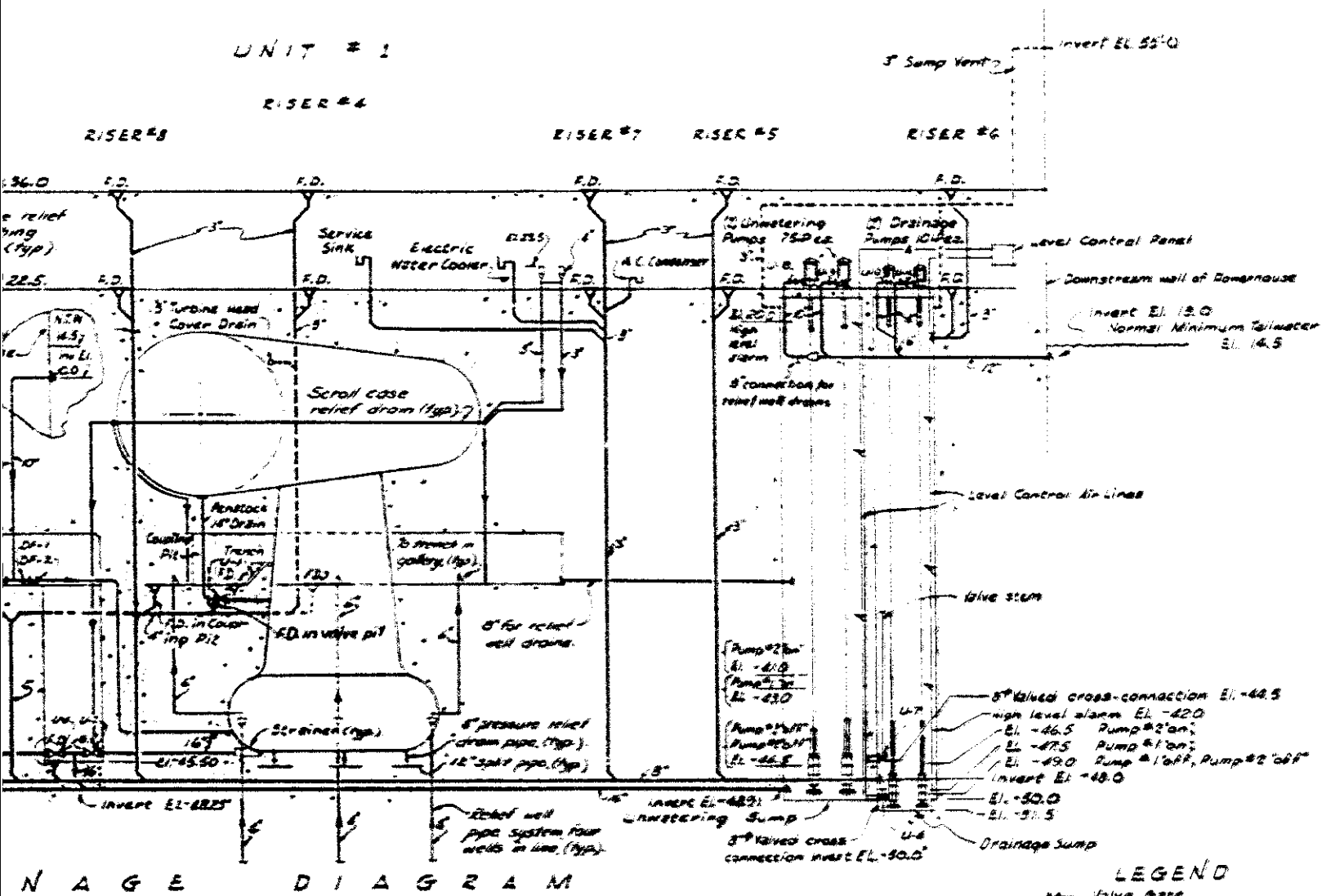

Generator # 1
Nozzle rings (typ.)
Cooling Coil
Thermostat
Air
Pressure switch (typ.)
Delayed cylinder (typ.)
Delayed cyls. bank
PLAN GENERATORS
El. 22.5
CO2 Bottles Unconnected Spares, 29 available.
El. 49.0
UNIT #1
Service air to governor cabinet then to brake system in unit
Exhaust Fan
1¼" Air Exhaust
Air Main on upstream wall
Condensate Trap
Condensate Trap
To control Room
¼" Air to pressure differential cell.
PRV
Pneumatic Modulating Controller
1¼" exhaust
Pressure reducing valve and strainer
Float activated piston valve
Sump pump control
Service Air Connection (Typical)
Grease Pump @ Unit #1
Seal line, for continuation see dwg. PL-45-304
EL 2250 & 2100
2" Raw water header
SEWAGE EJECTOR cap. 30 Gallons
Float well Fill line
Tailrace Float gauge well
LEGEND
Globe Valve
Safety Valve
Pressure Indicator
Strainer
Gate Valve
Check Valve
Automatic Valve
Angle Valve
Reducer
Governor Pressure Tank (Supplied by turbine mfr.)
Line purifier (typ.)
A-102
A-101
(2) GOVERNOR COMPRESSORS (3P ea.) 300-350 psi
GOVERNOR COMPRESSED AIR DIAGRAM
REFERENCE DRAWIN
For Reference Drawings & o
Notes - See Drawing PL-4
GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
POWERHOUSE
GOVERNOR & SERVICE AIR
FIRE SYSTEM PIPING DI
INTERNATIONAL ENGINEERING CO. INC.
KA-X-1
7/10/58
PL-45
REVISIONS



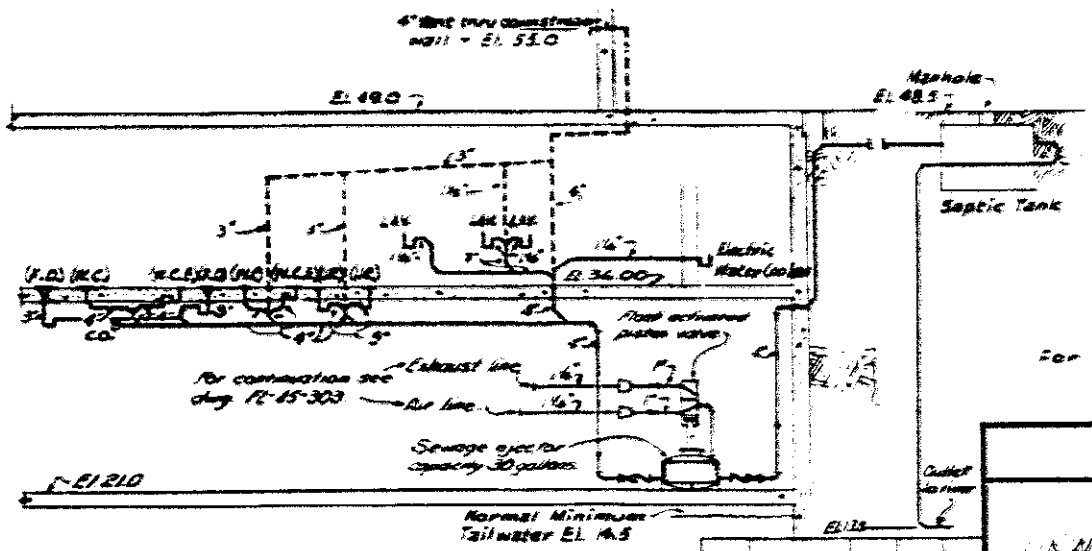


ROOF DRAINAGE DIAGRAM

TRANSFORMER DECK DRAINAGE DIAGRAM



N A G E D I A G R A M



SOIL AND WASTE DRAINAGE DIAGRAM

LEGEND

- Valve, Gate
- Valve, Check
- Drain, Floor F.D.
- Drain, Scupper
- Drain, Roof R.D. w/basket strainer
- W.C. Water Closet (Western)
- W.C.E. Water Closet (Eastern)
- LAV. Lavatory
- U.R. Urinal
- Vent
- C.O. Cleanout
- Reducer

REFERENCE DRAWINGS

For Reference Drawings & General Notes See Drawing FL-45-301

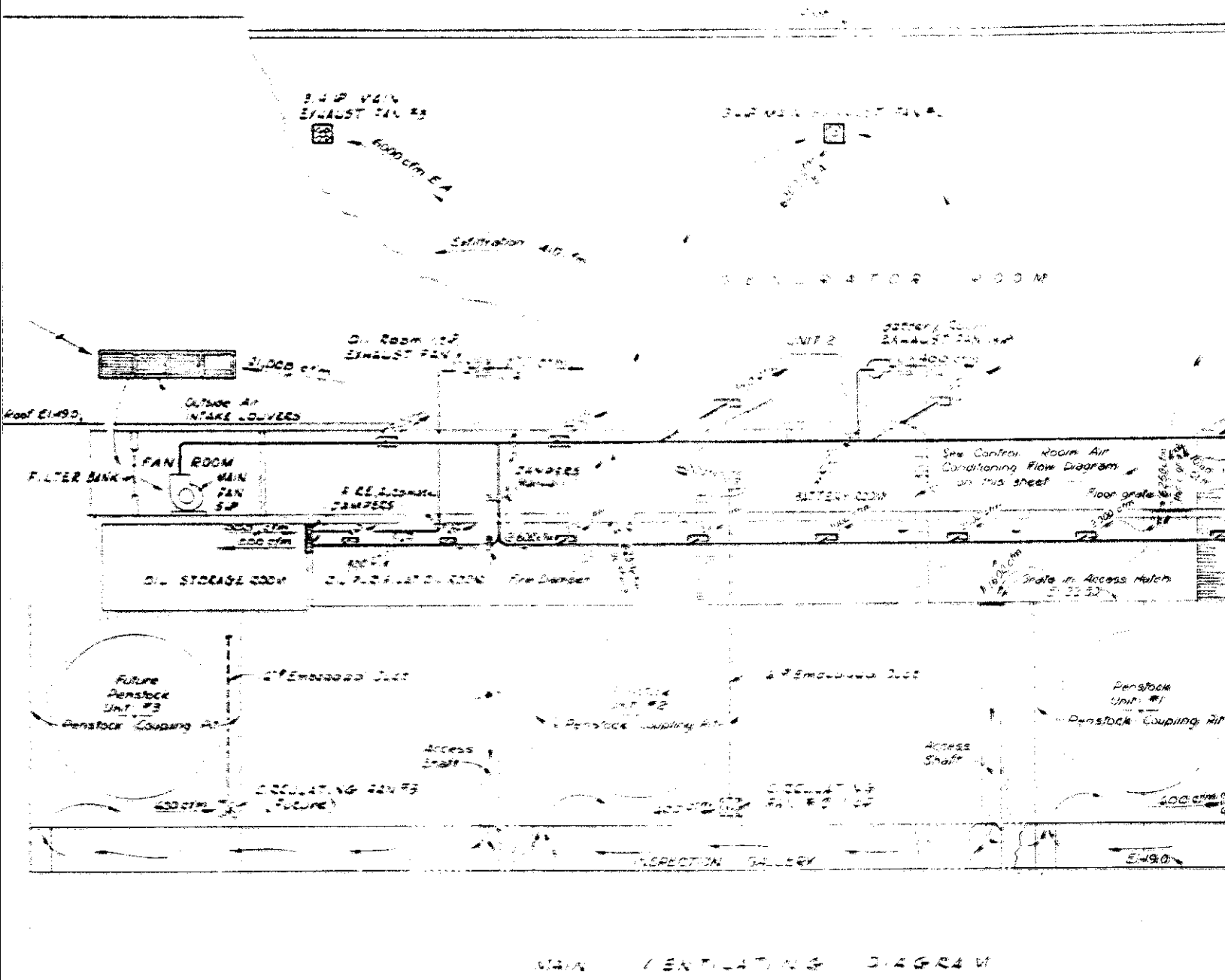
4			
3	Oct 30, 61	Minor Revisions	
2	Sep 13, 61	General revision	
1	Feb 8, 61	Minor Revisions	
REVISIONS			

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

KARNAFULI PROJECT
POWERHOUSE
UNWATERING, DRAINAGE, SOIL & WASTE PIPING DIAGRAM

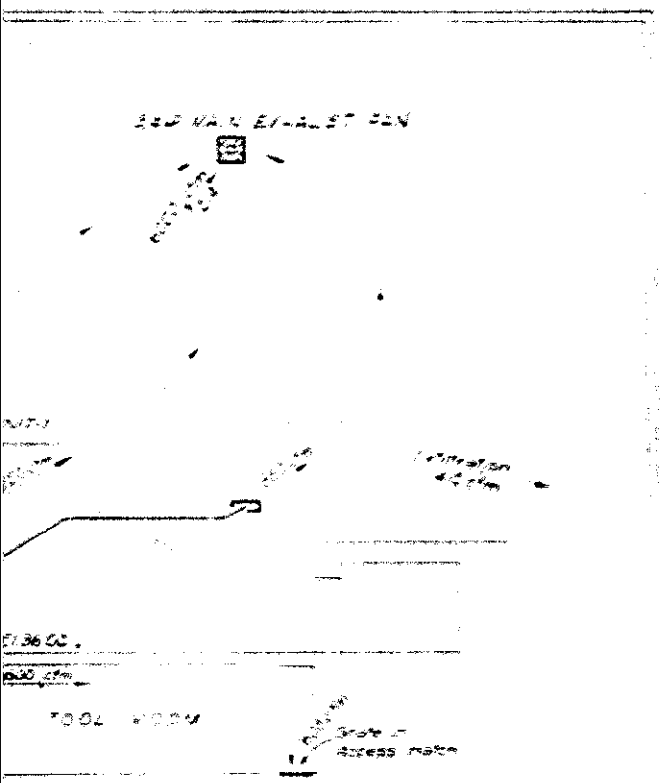
INTERNATIONAL ENGINEERING CO. INC. - ENGINEERS

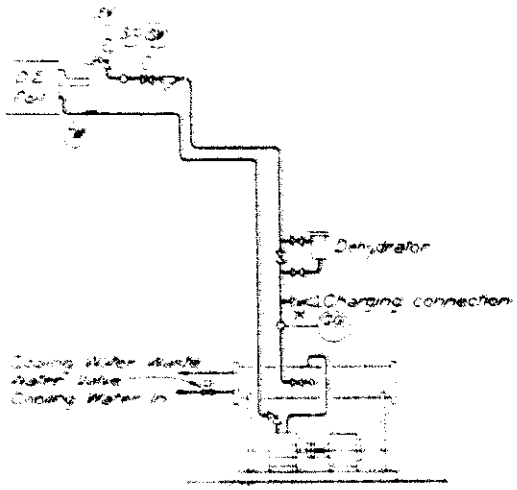
DESIGNED RAS / DRAWN RAS	CHKD. / APPD.	SUBMITTED E.S.
KA.-X-1	SAN FRANCISCO, CAL. 7/26/58	FL-45-304



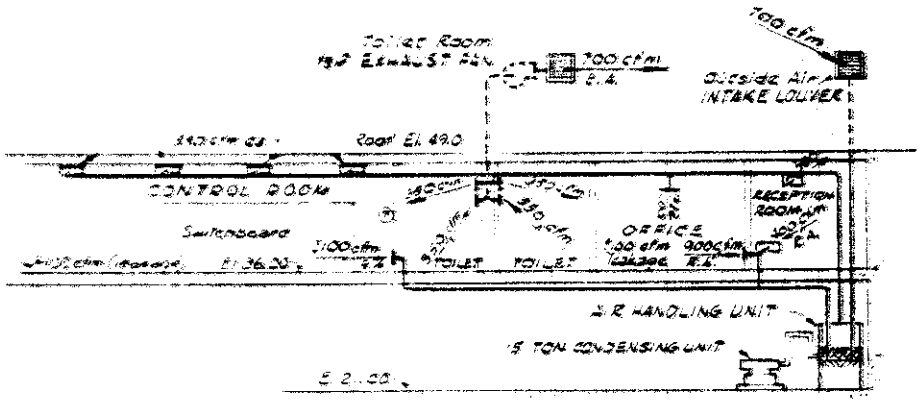
MAIN VENTILATING DIAGRAM

Best Available Copy





.5 TON FREON COMPRESSOR



CONTROL ROOM DIAGRAM

LEGEND:

- EV Expansion Valve
- SG Sight Glass
- SV Solenoid Valve
- TW Thermal Well
- T Thermostat
- Refrigerant Stop Valve
- Charging Valve
- Y Strainer
- Direct Expansion Coil
- Electric Heater
- Damper
- Air Blower (Fan)
- Propeller Fan
- Grille
- E.A. Exhaust Air
- O.A. Outside Air
- R.A. Return Air
- Air Flow
- Fire Damper

REFERENCE DRAWINGS
For Reference Drawings & General Notes see Drawing F2-45-301

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

KARNAFULI PROJECT

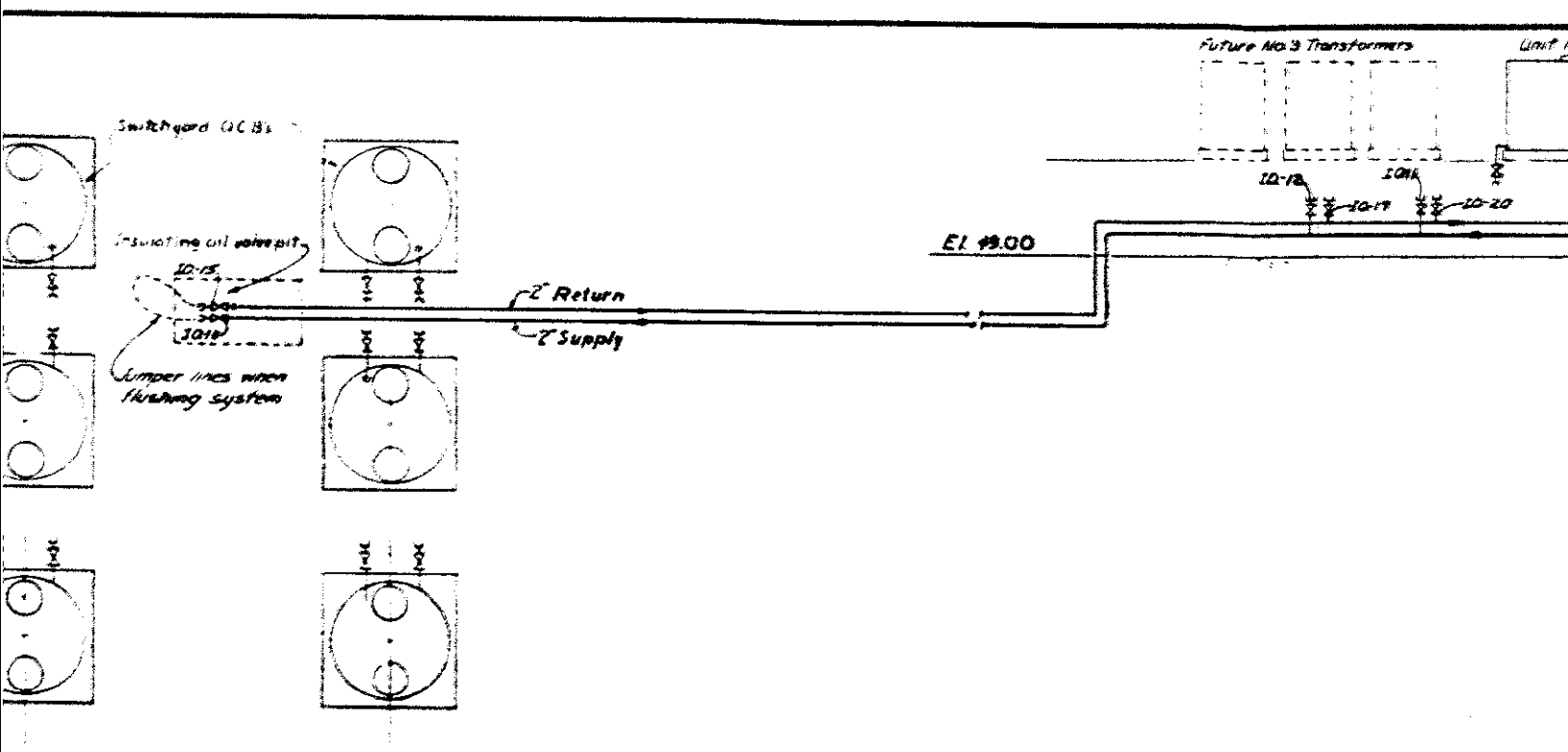
POWERHOUSE

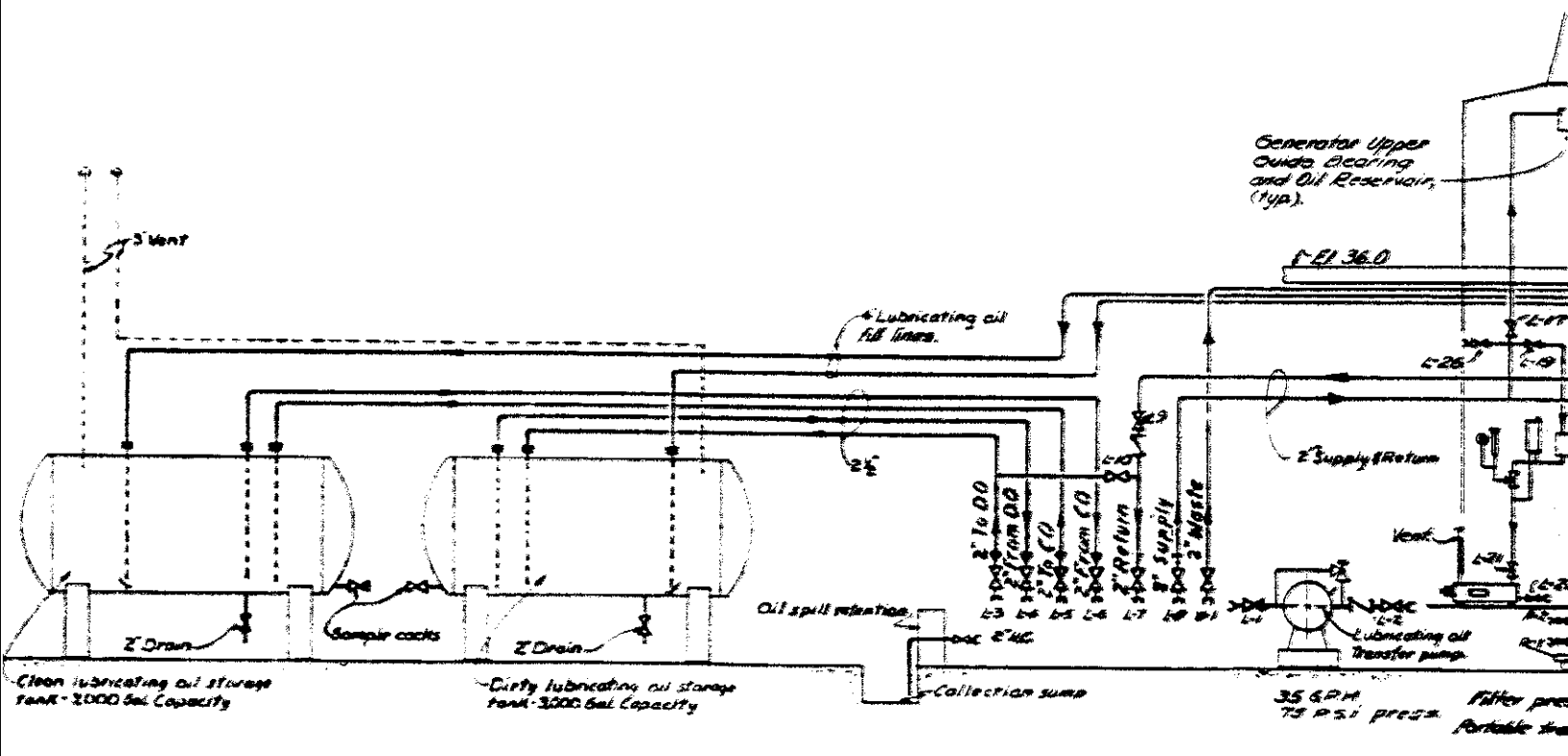
HEATING, VENTILATING & AIR CONDITIONING

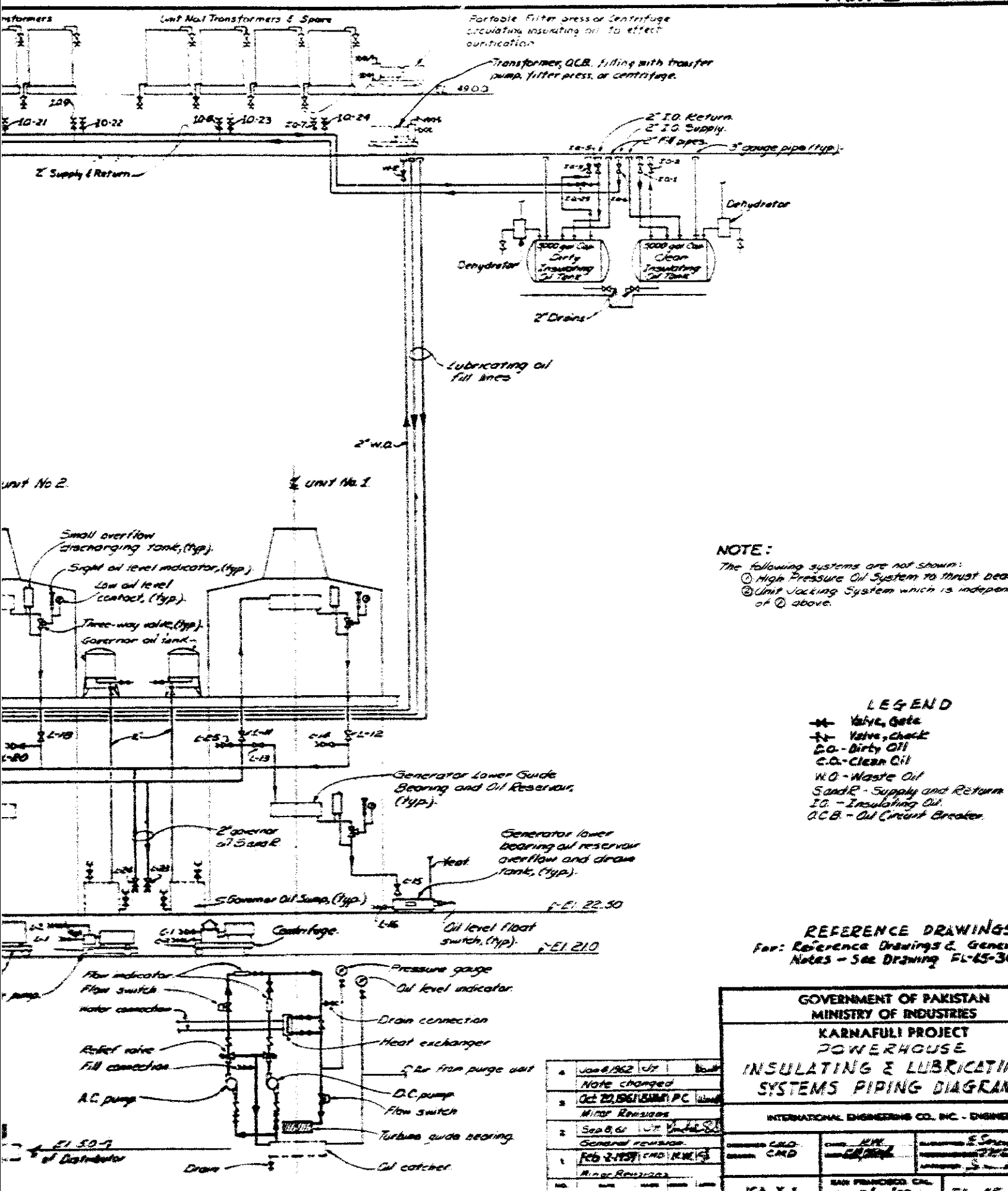
PIPING DIAGRAM

INTERNATIONAL ENGINEERING CO., INC. - ENGINEERS

Best Available Copy


Future No. 3 Transformers
Switchyard OCB's
Insulating oil valve pit
10-15
10-16
Jumper lines when flushing system
2" Return
2" Supply
El. 49.00
10-18
10-19
10-20


Generator Upper Guide Bearing and Oil Reservoir (typ.)
El. 36.0
3" Vent
Lubricating oil fill lines.
2" Supply & Return
2" To OO
2" From OO
2" To CO
2" From CO
2" Return
2" Supply
2" Waste
Vent
Oil spill retention
2" Drain
Sampler cocks
2" Drain
Lubricating oil transfer pump
Clean lubricating oil storage tank - 2000 Gal Capacity
Dirty lubricating oil storage tank - 3000 Gal Capacity
Collection sump
35 G.P.M. 75 P.S.I. press.
Turbine guide bearing



NOTE:
The following systems are not shown:
① High Pressure Oil System to thrust bea
② Unit Jacking System which is indepen of ① above.

LEGEND
Valve, Gate
Valve, Check
D.O. - Dirty Oil
C.O. - Clean Oil
W.O. - Waste Oil
S and R - Supply and Return
I.O. - Insulating Oil
O.C.B. - Oil Circuit Breaker

REFERENCE DRAWINGS
For: Reference Drawings & Gener Notes - See Drawing FL-65-3

4	Jan 8, 1962	J.T.	
	Note changed		
3	Oct 27, 1961	P.C.	
	Minor Revisions		
2	Sep 8, 61	J.T.	
	General revision		
1	Feb 2, 1959	C.M.D.	
	Minor Revisions		

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
POWERHOUSE
INSULATING & LUBRICATI SYSTEMS PIPING DIAGRA
INTERNATIONAL ENGINEERING CO., INC. - ENGINE
C.M.D.
SAN FRANCISCO, CAL.

PART V

CHAPTER V

POWER INTAKE

5.01 GENERAL

This chapter discusses the design of the structural, mechanical and electrical equipment of the power intake structure. The general arrangement of this equipment is shown on Exhibit V-1. The power intake structure is described in Part III, Chapter IV.

5.02 TRASHRACK

A. Description

To protect the powerhouse units from damage by trash, sixteen identical sets of trashracks are provided at the upstream face of the intake structure. Each set consists of six panels. Each panel is 10 feet 6 inches wide by 16 feet high, and is fabricated from 2-inch by 3/8 inch vertical bars which are supported by 12-inch deep horizontal I-beams and channels. Adjacent panels in a set are bolted together at a combination splice plate and lifting connection, so that all installation or removal operations can be performed from the intake structure deck. A dogging device is provided to hold the assembled trashracks in position while attaching or removing a panel. The trashracks are installed in slots formed in the intake structure columns and bear on embedded angles.

B. Loads and Stresses

The trashrack bars were designed to yield under a uniform load of 625 pounds per square foot (10 feet of head). The trashrack framing was designed to withstand a uniform load of 940 pounds per square foot (15 feet of head) at stresses not to exceed those recommended by the American Institute of Steel Construction.

C. Design

The trashrack bar spacing is 6 inches on centers as specified by the turbine manufacturer. The bar thickness and depth was determined from a United States Bureau of Reclamation formula for the failure stress in flat trashrack bars. This formula equates the failure stress to the yield stress times $(1.23 - 0.0153\ L/t)$, where L is the unsupported span and t is the bar thickness. The depth and spacing of the support members was determined by relating the section modulus of available I-beams to the required section modulus for the contributing loaded area. I-beams having the least total weight were chosen.

5.03 TRASHRACK RAKE

A. General

1. Purpose - The trashrack rake is provided to facilitate the removal of debris from the trashracks.

2. Location - The trashrack rake is installed on rails which are mounted on the intake structure deck near the upstream edge.

3. Description - The trashrack rake, as shown on Figure A, is a 2-ton capacity, self-dumping, log-grappling type. It is equipped with a fixed electric powered hoist, which is mounted on an electrically-driven supporting frame and is suitable for outdoor operation.

4. Selection of Type - The selection of this type of trashrack rake was based on the experience of its successful use in other installations with similar requirements.

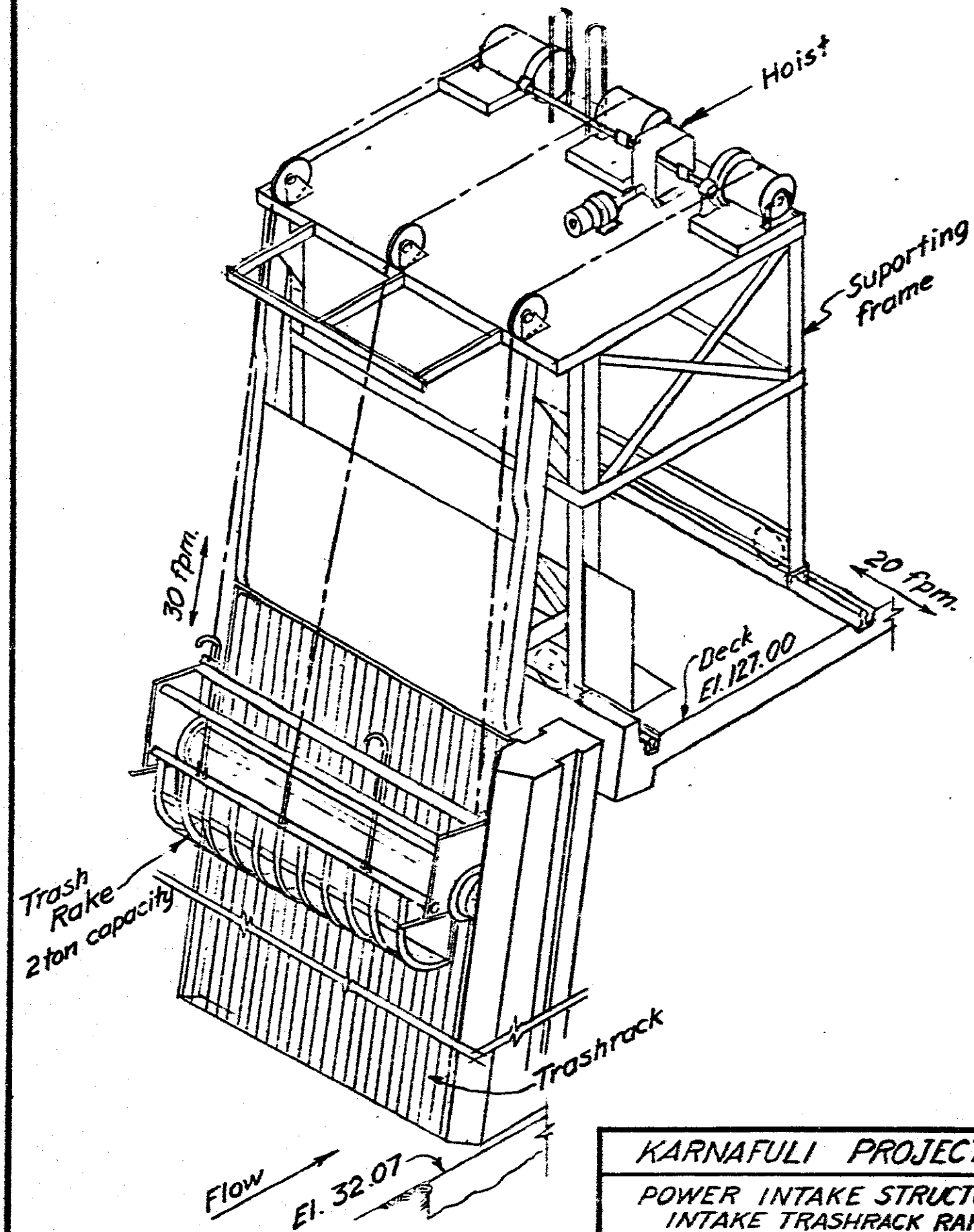
B. Design

1. Lifts and Speeds - The trashrack rake was designed for a hoist speed of 30 feet per minute, a lift of 110 feet, a travel speed of 20 feet per minute, and a travel distance of 210 feet.

2. Loads and Stresses - The trashrack rake was designed to be stable and rigid to prevent undue deflections and vibrations for the following loads:

Dead Load	- Total weight of supporting frame, apron, chute, supporting frame trucks, and hoist and drive machinery.
Live Load	- Total weight of rake, wire rope, and 2-ton rated trash load.
Wind Load	- 10 pounds per square foot while traveling and 30 pounds per square foot while stationary.
Impact Load	- 10 percent of the wheel loads.
Travel Traction	- 10 percent of the total weight, 75 percent of this applied to one side.

Mechanical and structural parts were designed for a safety factor of five, at rated load, based on the ultimate strength of the materials used. The wire rope was selected with a safety factor of five, at rated load, based on the breaking strength of the rope with due allowance for reeving. Increased stresses were permitted for an increase of 25 percent of the rated trash load. The stresses induced under the maximum torque of the hoist drive motor will not exceed 80 percent of the yield point of the materials. The maximum deflection of any part under any load does not exceed 0.00125 inch per inch of unsupported span. The



KARNAFULI PROJECT		
POWER INTAKE STRUCTURE INTAKE TRASHRACK RAKE DESIGN REPORT		
INTERNATIONAL ENGINEERING CO., INC. SAN FRANCISCO, CALIF.		
DR. BCN CK.	RECOMMENDED	APPROVED
DATE Nov. 4, 1959	FIG. A	

2. Description - Four gates, two each for Penstock No. 1 and Penstock No. 2, are provided for shutting off flow from the reservoir. Penstock No. 3 is closed with temporary concrete bulkheads.

Each of the fixed-wheel gates is 33 feet-10 inches high by 13-feet-5 inches wide and weighs approximately 45,500 pounds. The gates are normally suspended from the hoist cables in the "Open" position with the bottom of the gate just above the penstock opening. The intake structure is provided with gate hanger beams for suspension of the gate in a "dogged" position in the maintenance chamber above Elevation 92 feet.

The gate is composed of six independent units. Each unit is made up of three built-up horizontal girders, diaphragm stiffeners, a continuous downstream skinplate, end plates, and four wheels and axles. Units are connected together by a welded vertical strap on the upstream flange of the girder and by a horizontal, bolted splice plate on the skinplate. The gate has a total of 24 wheels. The intake structure is provided with gate slots, side and sill seal plates, and guide and bearing rails. The gates are equipped with rubber seals.

The gates will be normally positioned directly above the intake opening in order to close off the flow to the penstocks in the minimum amount of time.

B. Design

1. Criteria and Assumptions - The fixed-wheel gates were designed for normal steel stresses (basic tensile stress being 18,000 pounds per square inch) under full hydrostatic head with a normal pool water surface at Elevation 118 feet. A 33 percent increase in stresses was allowed for the combination of girder and skinplate bending stresses. The hoisting connection was designed to resist twice the hoist capacity at normal working stresses.

An allowance for corrosion was made by adding 1/16 inch of extra thickness to the skinplate and upstream flange and using a minimum thickness of 3/8 inch for all members.

2. Analysis - Each of the gates was designed as six independent units sized such that each unit resists approximately one sixth of the total hydrostatic load. The hydrostatic load is resisted by the horizontal girders. The downstream flange of the girders was assumed to consist of a section of skinplate with a width of 24 times the skinplate thickness. The vertical diaphragms were provided to reduce the bending stress in the skinplate between the girders. The load on the girders is transferred to the end plate and into the cantilevered axles and wheels. Because of the flexibility of the assembled gate, all wheels will be in contact with the bearing rail and no moments due to distortion will be induced in the gate. Complete details of the design analysis are given in Exhibit V-2.

righting moment is equal to 110 percent of the overturning moment caused by the maximum torque of the hoist motor with a jammed rake plus the wind load.

3. Mechanical Features - The hoisting machinery consists of a motor-driven worm gear speed reducer, drums, gearing, and shafting with supporting bearings. The worm gear speed reducer is self-locking and functions as a load brake during lowering operation. The drums will receive all the wire rope in a single layer. The maximum rope fleet angle is 3-1/2 degrees. Guards prevent the rope from leaving the sheaves.

The travel drive machinery consists of a motor driven speed reducer, gearing, and shafting to the drive wheels.

The rake basket is self-dumping and is large enough to accommodate a 3-foot diameter log. The rake wheels ride on, and are guided by the trashbars. Rake tooth penetration is 1-1/2 inches. The trash rake has a lever-controlled emergency load release.

4. Structural Features - The trashrack rake has an apron and hinged chute to permit dumping. A shielded operator's platform, located to provide an optimum view of all operations, is supported by the frame. A ladder permits access to the hoist machinery deck. Supporting frame trucks are equipped with rail sweeps and safety lugs.

5. Electrical Features - All electrical equipment is suitable for outdoor operation and for the power source of 230/400 volts, grounded wye, 3-phase, 50 cycle, alternating current. The trashrack rake is equipped with an automatic take-up cable reel with capacity for 100 feet of 4-conductor, 600 volt cable. All operations are pushbutton controlled; each push button has a nameplate designating its function. Each separate operation has independent control with mechanical interlocks to prevent any two buttons from being pressed simultaneously.

The hoist machinery is equipped with an electrically operated holding brake, mounted on the hoist drive motor shaft extension. The travel drive machinery is equipped with an electrically-operated travel and parking brake. The brakes set upon de-energizing motor circuits.

The limits of the position of the rake are controlled by electrical limit switches in the control circuits. A separate, directly activated, limit switch is provided for the extreme upper rake position.

The rake, dumping area, and operator's platform are illuminated by lighting fixtures mounted on the supporting frame.

5.04 INTAKE GATES

A. General

1. Selection - The intake gates will normally be operated under balanced head conditions but must be capable of quick closure under an unbalanced head in an emergency. Fixed-wheel gates were selected to provide for this dual requirement because of their economy.

5.05 STOPLOGS

A. General

Two sets of structural steel stoplogs are provided to permit inspection and maintenance of the intake gate, track, and guides. Each set consists of a top panel approximately 14 feet wide by 12 feet high by 2 feet thick and an intermediate and bottom panel approximately 14 feet wide by 13 feet high by 2 feet thick. The intermediate and bottom panels are interchangeable. Total weight of both sets of stoplogs is approximately 60,000 pounds.

B. Loads and Stresses

The stoplog panels were designed to withstand full hydrostatic head due to a normal pool water surface at Elevation 118 feet. An allowable stress of 120 percent of AISC allowable stresses was used for the girders. An increase of 1/3 was allowed for the skinplate. A corrosion allowance of 1/32 inch was added to the skinplate and the upstream flange of the girder.

C. Design

Design of the stoplogs was similar to that of the intake gate as the hydrostatic load is resisted by horizontal girders composed of an upstream flange, a web, and a downstream flange with an assumed width of 24 times the skinplate thickness. The hydrostatic load is passed from the skinplate through the webs, end plates and wood seals into the concrete.

D. Bearings, Seals and Guides

The intake structure is provided with stoplog slots and embedded metal seal plates. Bolted timbers bear and seal against the downstream face of the stoplog slot. Sealing at the sill and between stoplogs is also accomplished by timber seals. Timber bumpers at each end of the stoplog panel act as guides and prevent damage to the concrete.

E. Handling

Stoplog panels are both lowered and raised by a lifting beam operated from a portable crane. The lifting beam is automatic for both operations. A spring-activated latch pin will either connect or disconnect the lifting beam and the stoplog panels. The beam is provided with a tag line to release the pin in case of a premature connection. The lifting beam has timber guides on each end which travel in the same guide slot as the stoplogs, assuring positive alignment with the stoplog lifting connection.

5.06 BULKHEADS

A. General

As previously mentioned, the two intake openings for Unit No. 3 are closed by reinforced concrete bulkheads. These bulkheads, which were cast in place, are each approximately 36 feet high by 14.5 feet wide by 2.5 feet thick. The gate slots had all required embedded metal work for later installation of fixed-wheel guides, rails, etc. in place before placing the bulkhead concrete. A metal water stop was installed in the joint between the gate slot concrete and the concrete of the bulkhead.

B. Loads and Stresses

The bulkheads were designed to withstand full hydrostatic head due to a normal pool water surface at Elevation 118 feet. Allowable compressive stress in the concrete was 1350 pounds per square inch. Allowable tensile stresses in the reinforcement were 20,000 pounds per square inch.

C. Design

The bulkheads were designed as simply supported one-way slabs spanning in the horizontal direction. This one-way action was ensured by placing asphaltic filler cushions at the top and bottom bulkhead bearing surfaces.

5.07 HOISTS

A. General

1. Purpose - Four intake gate hoists are provided for raising and lowering the fixed-wheel gates for routine inspection and maintenance and for emergency closure of the penstocks.

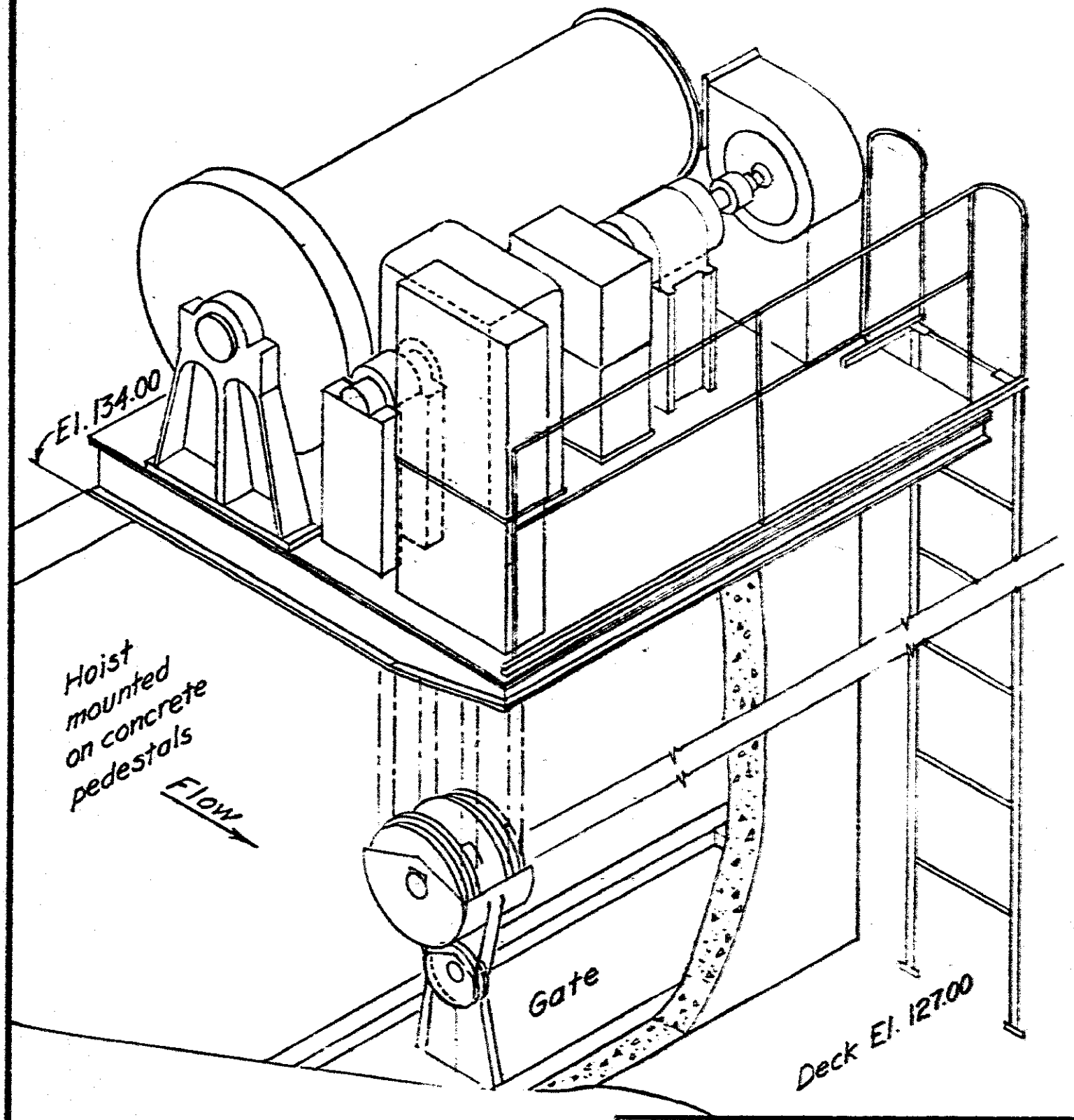
2. Location - The intake gate hoists are mounted directly above the gate slots on concrete pedestals which rise from the intake structure deck at Elevation 127 feet.

3. Description - The intake gate hoists are 40-ton (2,000 pounds per ton) nominal capacity, reeved wire rope, single drum, single pickup, pedestal-mounted, outdoor type as shown in Figure B and are electrically driven with provisions for emergency closure.

4. Selection of Type - Since the intake gates are to be used for emergency closure, individual hoists were selected. Selection of this type hoist was based on experience of successful use in other installations which have similar requirements.

B. Design

1. Lifts and Speeds - The intake gate hoists were designed for a lift of 68 feet with a normal raising and lowering speed of 1.5 feet per minute and an emergency lowering speed of 4.5 feet per minute.



HOIST

Capacity ---------- 40 Tons
Lift ---------- 68 ft.
Normal Speed
Raising ---------- 1.5 fpm.
Lowering ---------- 1.5 fpm.
Emergency Closure Speed --- 4.5 fpm.

KARNAFULI PROJECT

POWER INTAKE STRUCTURE
INTAKE GATE HOIST
DESIGN REPORT

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR.	RECOMMENDED	APPROVED
CK.		

DATE Nov. 4, 1959 | FIG 18

2. Loads and Stresses - The hoist capacity at the gate connection was determined from the gate weight (submerged or dry), wheel friction, seal friction, hydraulic loads, and down pull during emergency closure against unbalanced head. The component hoist parts were designed with a safety factor of five at rated load and based on the ultimate strengths of the materials used. The wire rope was selected with a safety factor of six at rated load and based on the rope breaking strength, with due allowance for reeving efficiencies. The stresses induced under the maximum torque of the hoist drive motor did not exceed 80 percent of the yield point of the materials. Under maximum loading, the hoist base bearing loads on the concrete did not exceed 600 pounds per square inch.

3. Mechanical Features - Mechanical features of the intake gate hoists include the motor-driven speed reducer, gearing, drum, and wire rope with sheaves and load block. The drum is large enough to receive all the wire rope in a single layer. The maximum fleet angle is 3-1/2 degrees. The load block was designed to ensure that the ropes will not leave the sheaves at any operating position. A fan brake will control the speed during emergency closure of the gate. A gate position indicator is mounted near the hoist control panel. All sliding and meshing surfaces are adequately lubricated.

4. Structural Features - The hoist base is a rigid structural steel unit and is equipped with lifting eyes to facilitate handling of the hoist. An access ladder from the intake structure deck at Elevation 127 feet to the hoist base, and hand rails on the base are provided.

5. Electrical Features - The electrical equipment is suitable for outdoor operation and for the power source of 230/400 volts, grounded wye, 3-phase, 50-cycle alternating current. The electrical wiring is enclosed in rigid metal conduits, tagged and marked for permanent identification.

Hoist operations are push-button controlled with nameplates designating the function of each push-button. The normal speed for lowering gates is controlled by the regenerative braking action of the electric motor. During emergency lowering of gates, the motor turns freely with no power applied to its terminals, with the fan brake controlling the lowering speed. Each hoist is equipped with an automatic electric holding brake, mounted on the hoist motor shaft. For emergency gate lowering a push-button at the powerhouse releases the holding brake. The holding brakes are also equipped with a manual release to be used in event of electrical power failure. Electrical limit switches are provided in the hoist control circuit to control gate travel and, as an additional safety measure, a direct activated upper limit switch is provided.

Instructions for operating the hoists and lubrication charts and electrical wiring diagrams are mounted in the control cabinet and protected against the weather.

5.08 OTHER EQUIPMENT

A. Reservoir Elevation Gage

A continuous record of the reservoir water surface elevation is made by means of a float-well and transmitter at the intake structure and a recorder in the powerhouse. A 14-inch diameter steel pipe float well, extending from Elevation 63 feet to Elevation 129.50 feet, is located in the intake structure air vent shaft for Unit No. 2. The float well is connected to the forebay by a two-inch diameter pipe at Elevation 70 feet. The inlet of this pipe is screened to prevent the entrance of foreign material. For cleaning the screen, a one-inch diameter air blow-off connection is provided. The transmitter, which is mounted at Elevation 129.50 feet, is actuated by a float, counterweight, and pulley arrangement. Electrical impulses are transmitted to a recorder in the powerhouse control room.

B. Penstock Filling Gates and Valves

Each penstock is equipped with a 24-inch diameter slide gate for filling the penstock and spiral case. The slide gates are mounted at Elevation 67.50 feet on the wall between the fixed-wheel gate chamber and the air vent shaft. The gates are operated by hand-cranks on enclosed gear pedestal lifts mounted at Elevation 119 feet. With the slide gates wide open, approximately 2 hours are required to fill the penstock and spiral case.

Each intake is also equipped with an 8-inch line for filling the space between the stoplog and fixed-wheel gate. The line is located at Elevation 71.50 feet, and has an 8-inch filling valve with a stem extending up to a floor stand at Elevation 119 feet. Each line also has a guard valve to permit maintenance of the filling valve. Approximately two hours are required to fill the space between the stoplog and the fixed-wheel gate.

5.09 ELECTRICAL POWER AND LIGHTING

A. General

Power is provided to the intake structure from the power center in the powerhouse through underground conduit and cable. Direct-current service of 250 volts for operation of the holding brakes of the intake gate hoists is furnished from the station control battery in the powerhouse through underground conduit and cable.

Gate hoist motors and trashrack rake motors are provided with 230/400 volts, grounded wye, 3-phase, 50 cycle alternating current. Lighting and convenience outlets are provided with 230 volt, single-phase, 50 cycle service.

B. Secondary Protection

To provide secondary protection for branch circuits to motors, power outlets, and lighting and convenience outlets, a standard convertible 3-phase, 4-wire, distribution type circuit breaker panelboard is installed at the power intake structure. This arrangement is simple, safe, and economical.

C. Controls

All motors are equipped with magnetic starters. Remote control of the holding brakes of the gate hoists from the powerhouse control room is provided for emergency closure of the gates.

D. Lighting

Lighting is furnished by standard incandescent industrial type fixtures. The illumination level for the intake structure deck house is 10 foot-candles and for the deck itself is 3 foot-candles. The trashracks are illuminated for night raking operations by fixtures mounted on the trashrack rake.

E. Grounding

A copper grounding system is provided to protect personnel from danger of high voltage due to faulty equipment or short circuits. All exposed metalwork is connected to the main ground cable to keep all such metal at equal potential. The main ground cable is grounded by connection to driven ground rods and to metal constantly in contact with water in the forebay channel.

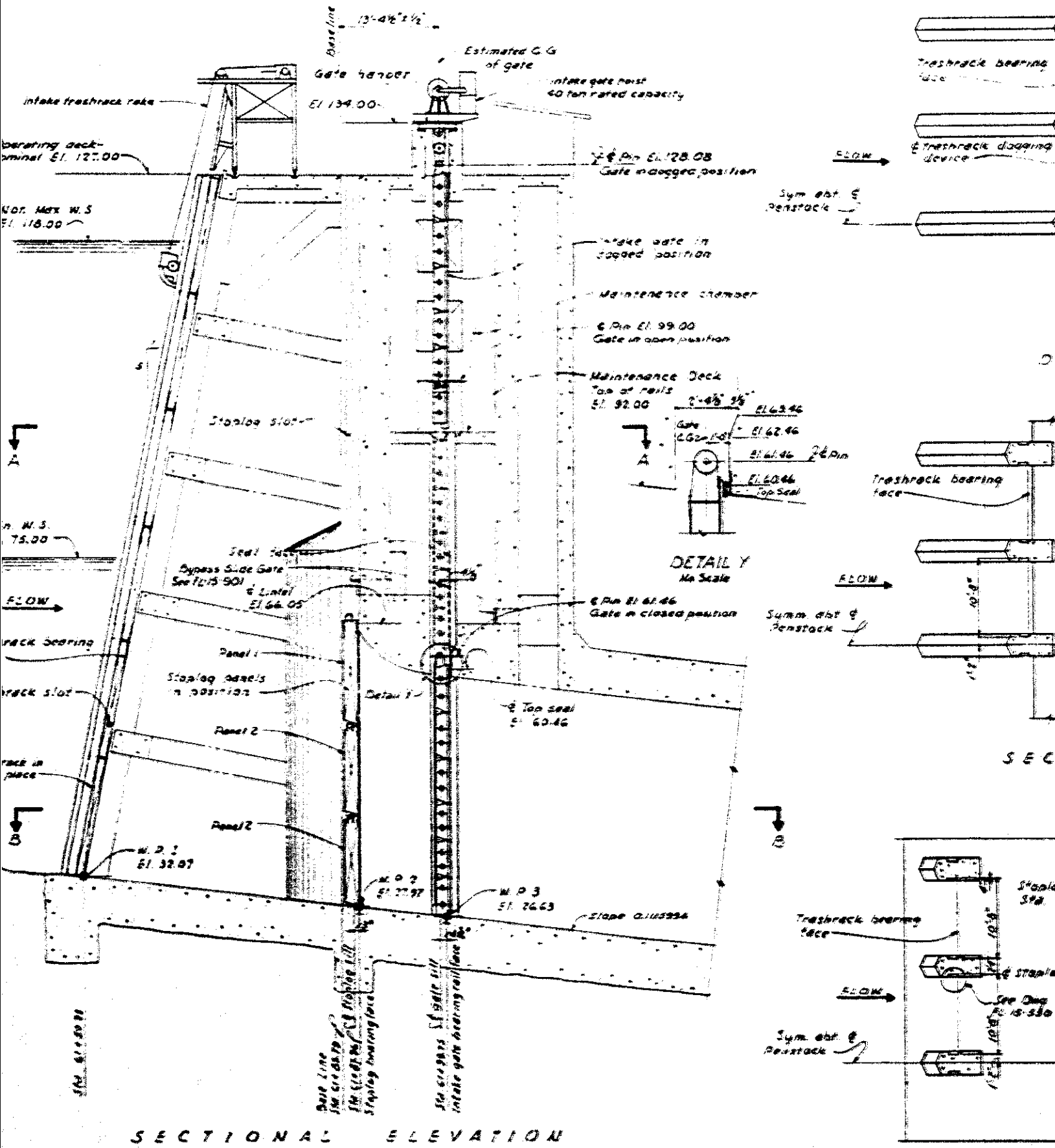
F. Telephone

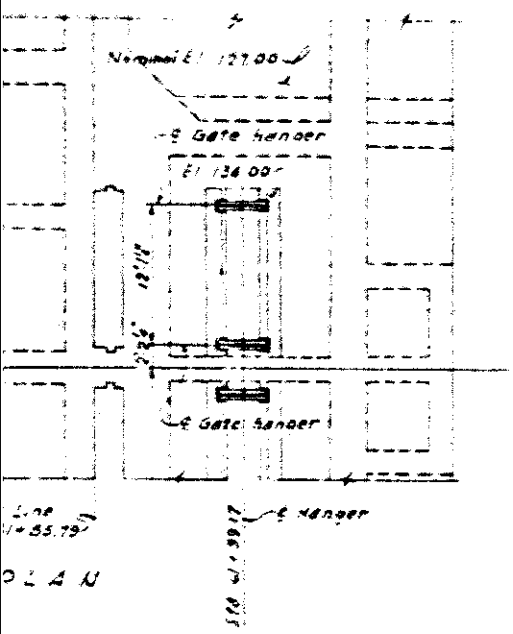
A telephone is installed at the intake structure deck house for communication with other project areas.

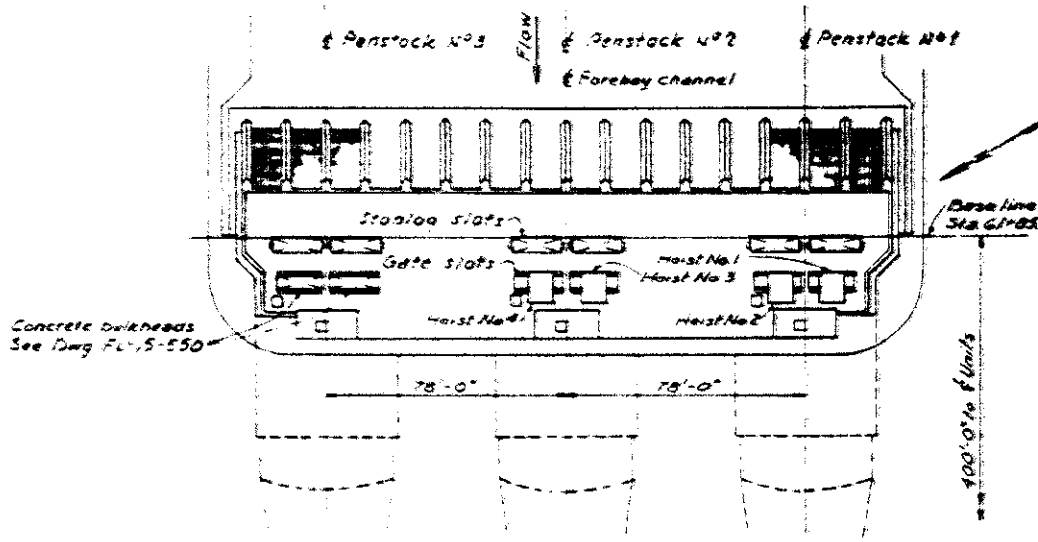
PART V - CHAPTER V

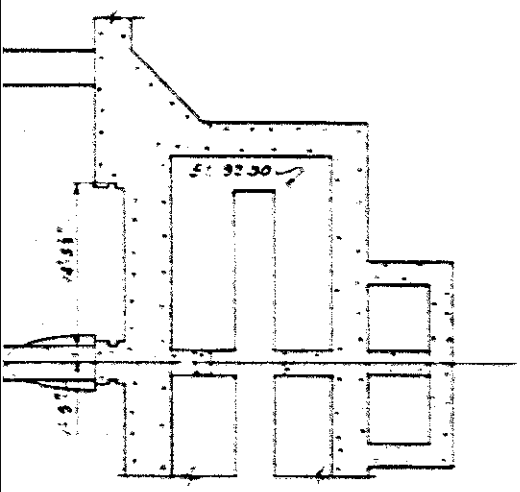
LIST OF EXHIBITS

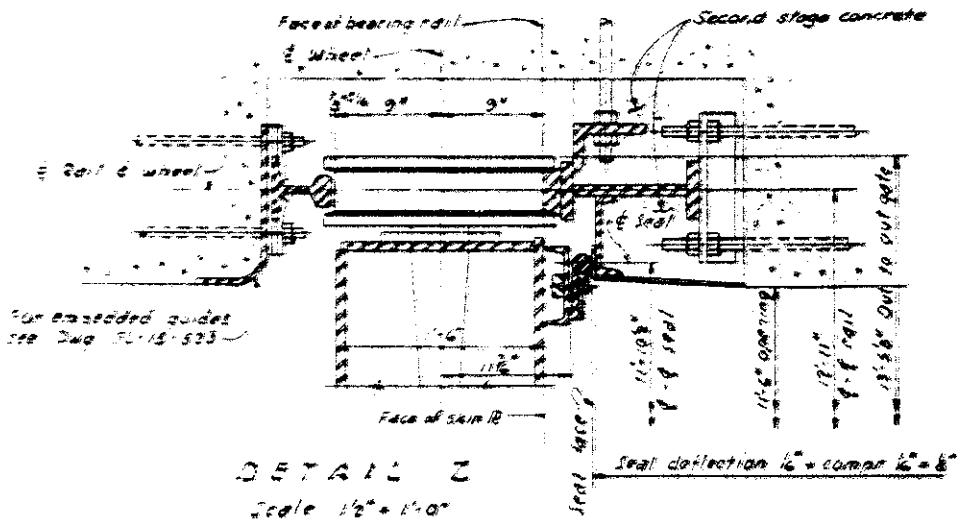
Exhibit	Title
V-1	General Arrangement
V-2	Intake Gate Design Analyses

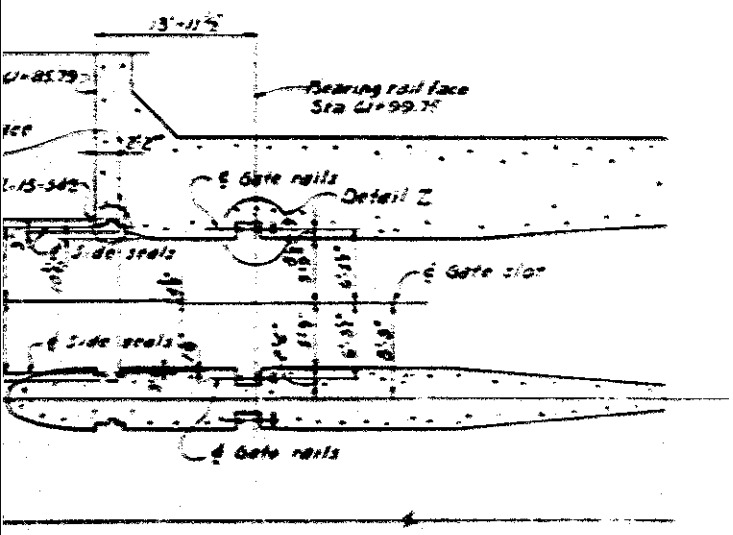

Base line
13'-4½" ± ½"
Estimated C.G. of gate
Gate hanger
Intake gate hoist 40 ton rated capacity
Intake trashrack rake
El 134.00
Operating deck-nominal El. 127.00
℄ Pin El. 128.08
Gate in dogged position
Max. W.S. El. 118.00
Intake gate in stopped position
Maintenance chamber
℄ Pin El. 99.00
Gate in open position
Maintenance Deck
Top of rails
El. 92.00
Staplog slot
Gate C.G.
El. 63.46
El. 62.46
El. 61.46
℄ Pin
El. 60.46
Top Seal
DETAIL Y
No Scale
Seal face
Bypass Slide Gate
See Dwg. No. 15-901
℄ Lintel
El. 66.05
℄ Pin El. 61.46
Gate in closed position
Panel 1
Staplog panels in position
Detail 1
℄ Top seal
El. 60.46
Panel 2
Panel 2
W. P. 1
El. 32.87
W. P. 2
El. 27.97
W. P. 3
El. 26.63
Slope 0.1025994
Base line
Staplog bearing plate
Intake gate bearing rail
FLOW
Sym. abt. ℄ Penstock
Trashrack bearing face
℄ Trashrack cleaning device
Trashrack bearing face
Trashrack bearing face
Trashrack bearing face
Staplog slot
Staplog panels in place
SECTIONAL ELEVATION
SEC







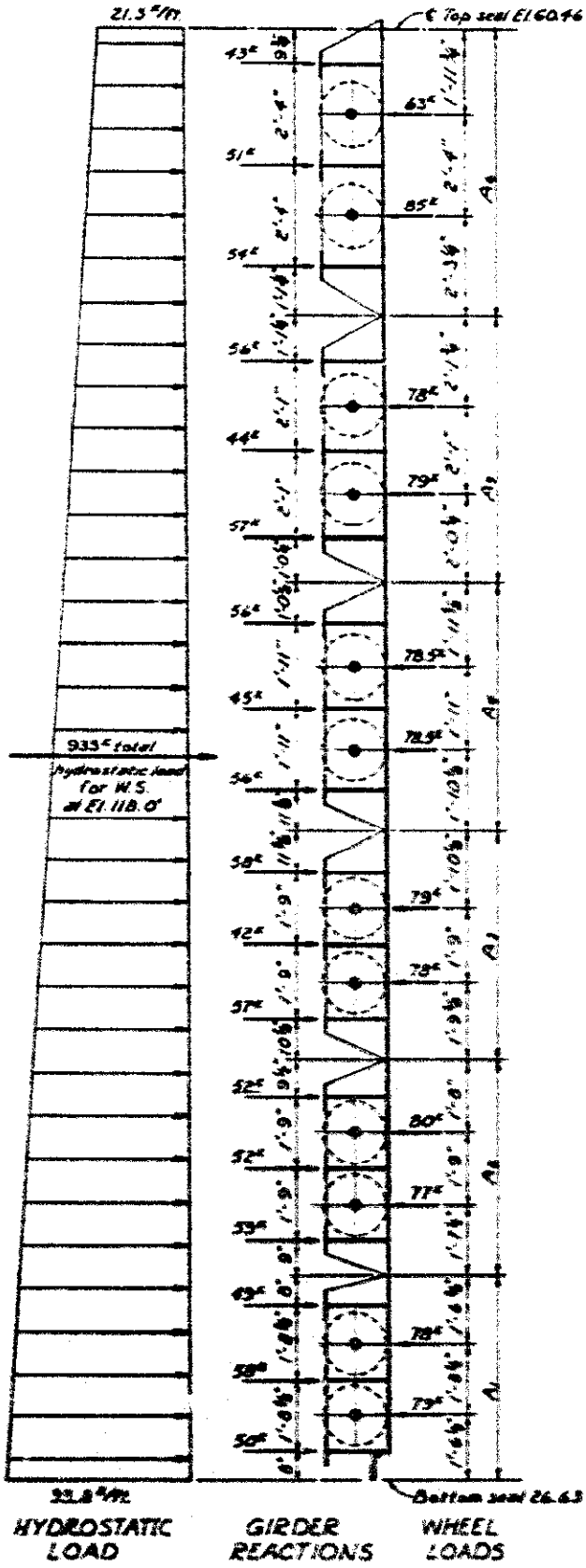




REFERENCE DRAWINGS:
POWER INTAKE STRUCTURE
Intake Gate - Framing & Seals FL-15-5
"Trashracks" Dogging Devices - Embedded Frames FL-15-5
Stoplogs - Framing & Details FL-15-5
Third Unit Bulkhead FL-15-5
POWER INTAKE
Intake Structure - General Arrangement FL-31-3
- 40-Ton Intake Gate Hoist General Arrangement FL-15-50

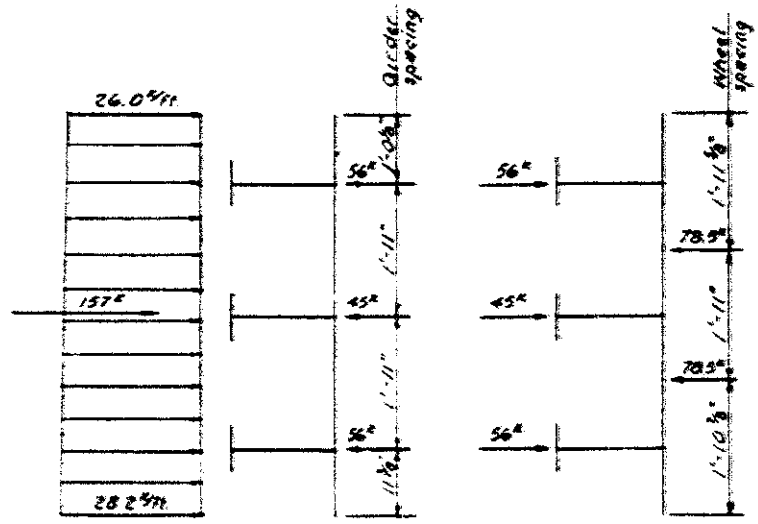
Scale 0 8 16 Feet
1/8" = 1'-0"
Except as noted

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES
KARNAFULI PROJECT
POWER INTAKE STRUCTURE
EQUIPMENT
GENERAL ARRANGEMENT
INTERNATIONAL ENGINEERING CO. INC. - ENGINEERS



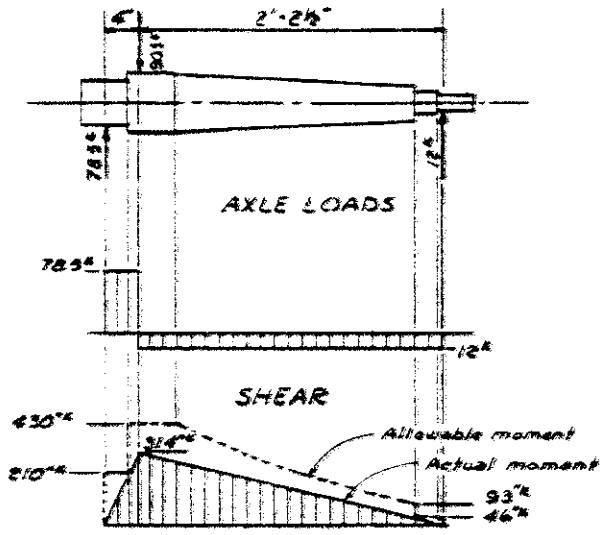
LOAD DIAGRAM

For half gate

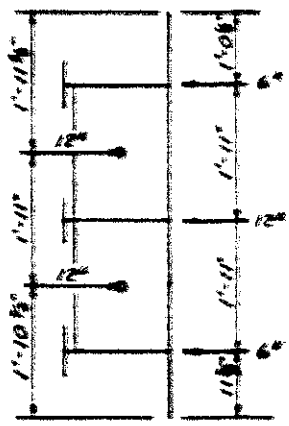


HYDRO LOAD - GIRDER REACTIONS WHEEL LOADS

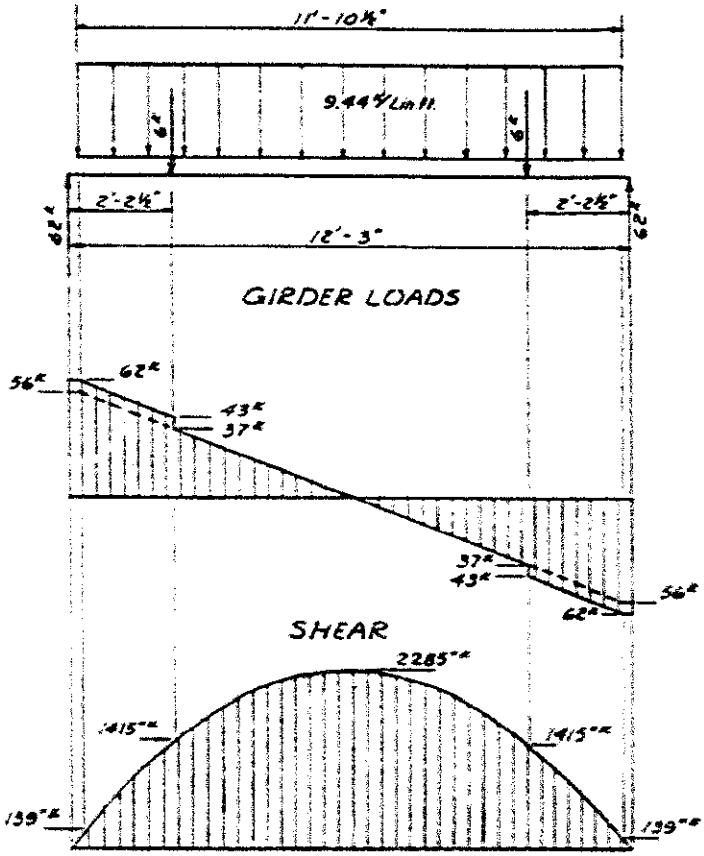
Panel A-4 - Selected for typical analysis



AXLE ANALYSIS

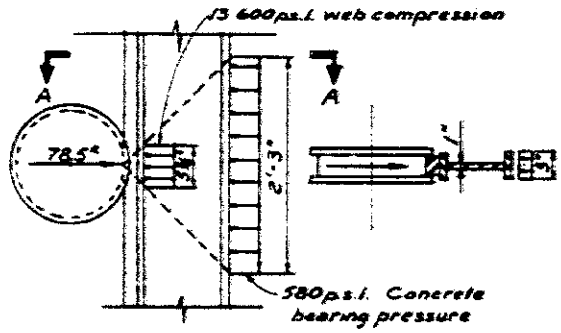


GIRDER REACTIONS-AXLE LOAD



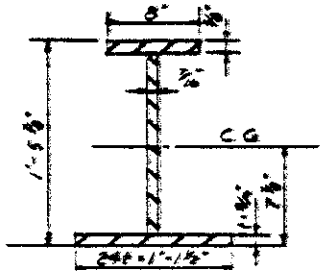
MOMENT

GIRDER ANALYSIS



SIDE ELEVATION SECTION A-A

EMBEDDED TRACK ANALYSIS



Moment of Inertia = 1100 in⁴

ASSUMED GIRDER SECTION

NOTES:

Gate designed in accordance with AISC Manual except that basic unit stresses are limited to 90% of the manual's values.

Estimated hoist loads:

Normal Closure

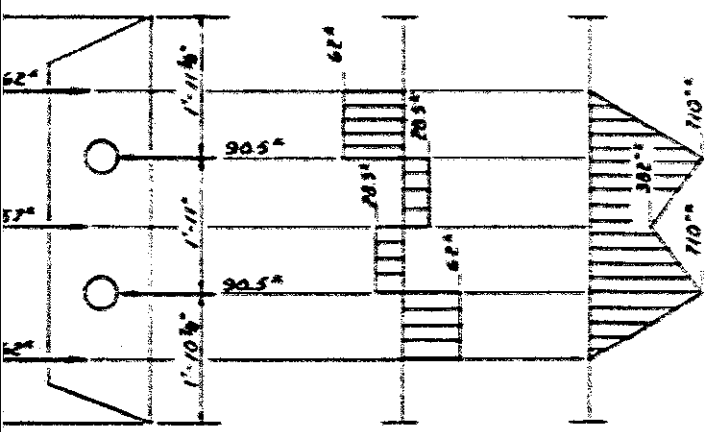
Dry weight of gate 45k

Submerged weight of gate 40k

Emergency Closure

Maximum closure force 10.4k

Final closure force 20k



END PLATE ANALYSIS

REFERENCE DRAWING:

POWER INTAKE STRUCTURE

Intake Gate

Framing and Seals FL-15-521

No Scale

GOVERNMENT OF PAKISTAN MINISTRY OF INDUSTRIES	
KARNAFULI PROJECT POWER INTAKE STRUCTURE INTAKE GATE DESIGN ANALYSIS	
INTERNATIONAL ENGINEERING CO., INC. — ENGINEERS	
ISSUED / REF.	
KA-X-1	FL-15-525

PART V

CHAPTER VI

SWITCHYARD

6.01 GENERAL

The switchyard equipment protects the generators, main transformers and transmission lines, and provides a number of possible different connections between the generators and the transmission lines. Additional equipment, also located at the switchyard, couples the communication facilities at the powerhouse to the conductors of the transmission lines which are used as carrier for communication and relaying coordination with other stations.

The electrical arrangement of the switchyard is shown on Exhibit VI-1. The physical arrangement of equipment and structures appears on Exhibit VI-2.

Presented in this chapter is information pertaining to the selection and design of the electrical equipment in the switchyard. Design of the supporting structures is discussed in Part III, Chapter VI.

6.02 DESIGN CHARACTERISTICS

A. General

The electrical scheme which was adopted for the switchyard is of the single-bus type, improved by the addition of bus sectionalizing breakers.

The equipment is arranged in a number of parallel, identical bays, each consisting of: (1) one generator oil circuit breaker, (2) a section of bus, and (3) one transmission line oil circuit breaker. The sections of bus in adjacent bays are connected through a bus sectionalizing oil circuit breaker.

The line oil circuit breakers are provided with two air disconnecting switches and one air break by-pass switch to allow operation of the line while the oil circuit breaker is out of service.

The sectionalizing oil circuit breaker is provided with air disconnecting switches. It is not equipped with a by-pass switch as this breaker may be removed from service without interruption to normal power flow.

The generator oil circuit breakers are provided with an air disconnecting switch on the bus side. No disconnecting switch is provided on the generator side as it is not feasible to operate a generator without the protection of its breaker.

The initial installation consists of two bays, with interposed bus sectionalizing breaker. Space is provided for a third bay and a second sectionalizing breaker which will be added to the switchyard when the third generator is installed in the powerhouse.

Additional equipment installed at the switchyard consists of lightning arrestors on each line, to protect equipment and insulation from surges on the transmission lines, and of grounding switches to ground the lines when out of service.

The design, manufacture, rating, testing, and installation of all electrical equipment and material was made in accordance with United States standards applicable to this type of equipment.

Power circuit breakers weigh approximately 37,600 pounds each and air disconnecting switches weigh approximately 3,000 pounds each.

B. Flexibility

The arrangement of the switchyard equipment allows the following connections to be made:

With only one generator running

Generator No. 1 feeding transmission line No. 1 (sectionalizing breaker open)

or Generator No. 2 feeding transmission line No. 2 (sectionalizing breaker open)

or Generator No. 1 feeding transmission line No. 1 and No. 2 (sectionalizing breaker closed)

or Generator No. 2 feeding transmission line No. 1 and No. 2 (sectionalizing breaker closed)

With two generators running, non-paralleled (Sectionalizing breaker open)

Generator No. 1 feeding transmission line No. 1

and Generator No. 2 feeding transmission line No. 2

With two generators running, paralleled (Sectionalizing breaker closed)

Generators No. 1 and No. 2 feeding transmission line No. 1

or Generators No. 1 and No. 2 feeding transmission line No. 2

or Generators No. 1 and No. 2 feeding transmission lines No. 1 and No. 2.

The above connections are satisfactory in view of the number of generators and transmission lines involved, the type of operation foreseen for the plant, and the location of the plant in the system. The additional expense of providing a higher degree of flexibility was considered to be unjustified.

C. Dependability

Any section of the system outside the switchyard which develops a fault will be automatically excluded from the system. Generator or transformer faults will trip the generator breaker in the corresponding bay and will result in the plant being operated on one of the connections shown above for one generator running. Faults on a transmission line will trip the appropriate line breaker and disconnect the line from the bus.

Faults in the equipment at the switchyard can take place either in one of the breakers or in one of the bus sections. Generator breaker faults will result in the exclusion of the breaker and related generator. Bus sectionalizing breaker faults will require the exclusion of the breaker and operation of the plant with one of the connections shown above for one generator running or for two generators running, non-paralleled. Faults at a transmission line breaker will result in excluding that breaker from service and connecting the line to the bus through the by-pass switch. Finally, a fault in a section of bus will trip the sectionalizing breaker and the two breakers of that bay. Operation can be continued with the generator and the line in the bay not affected by the fault. It will be necessary to clear the fault before the other bay can be brought into service again. The likelihood of a fault in a bus section, however, is extremely remote and if it should occur the fault could be cleared in a very short time.

The degree of dependability provided by the modified single-bus scheme is considered satisfactory for the type of loads present in the system, the inherent dependability of the equipment specified for the project, which in itself greatly reduces the possibility of faults, and the short time required to clear an extremely unlikely fault at a bus section.

6.03 RATINGS

The switchyard was designed for the following ratings:

Voltage - 132 KV - Determined by transmission line design.

Power - 150,000 KVA - Determined by installation of two 50,000 KVA generators initially and one 50,000 KVA generator in the future.

Interrupting capacity of Power Circuit Breaker - 3,500 MVA - Determined by short circuit study on basis of four 50,000 KVA generators and two 132 KV 60 MVA transmission lines to Chittagong.

6.04 INTERCONNECTIONS

The switchyard is connected to the main power transformer at the powerhouse by overhead transmission lines and to the next station in the system also by overhead transmission lines.

Low voltage power is brought to the switchyard from the powerhouse by way of the underground ducts and the cable gallery. This route is also followed by the control cables connecting the equipment in the switchyard to the duplex switchboard, and by the coaxial cable connecting the carrier communication equipment in the powerhouse to the coupling devices in the switchyard.

6.05 CONTROLS

Local manual control is provided for opening and closing of each air break disconnecting switch.

Local manual and remote electrical control at the powerhouse duplex switchboard is provided for opening and closing of each power circuit breaker.

6.06 INTERLOCK SCHEMES

The grounding switches on the transmission lines have a mechanical key operated interlock to prevent the closing of the grounding switches if the OCB by-pass or line side disconnecting switches are closed.

The OCB by-pass and line side disconnecting switches are equipped with a mechanical key operated interlock to prevent their closing if the line grounding switch is closed.

6.07 SIGNALING DEVICES

Visual-audible alarm is provided on the duplex switchboard in the powerhouse control room for the following conditions:

Bus Differential Relay Trip

Line OCB Relay Trip

OCB No-voltage and Low Air Pressure.

Lights are provided on the duplex switchboard in the powerhouse control room for each power circuit breaker to indicate open, closed, or tripped position of the breaker.

Indicating instruments are provided on the duplex switchboard in the powerhouse control room for:

Bus voltage

Line voltage (each line)

Line watts (each line)

Line watt-hours (each line)

6.08 ELECTRICAL POWER AND LIGHTING

Electrical power is provided at the switchyard from the powerhouse 230/400 volts wye system for operation of oil circuit breaker air-compressors, space heaters, carrier communication equipment, switchyard lighting, and convenience outlets. An outdoor weatherproof convertible 3-phase, 4-wire circuit breaker type distribution panel will protect the various branch circuits to the above equipment.

Conventional switchyard lighting is provided with structure-mounted or separately pole-mounted fixtures as required for good illumination.

6.09 COMMUNICATIONS

A telephone located at the switchyard will permit communication with other project areas during routine operation and maintenance activities. The telephone is installed in a weatherproof enclosure and has a separate loud ringing bell.

Carrier communication coupling equipment consisting of line traps, capacitor potential devices and line tuning units provides a communication and relaying channel over the transmission lines to other locations on the transmission system that are similarly equipped. The carrier system was designed as part of the design of the transmission lines by others.

6.10 SAFETY PROVISIONS

A copper grounding system protects personnel and equipment from danger of high voltages due to lightning, faulty equipment or short circuit conditions. Exposed metal work is connected to the main ground cables in order to keep all such metal at equal potential. Ground potential gradient is kept at a minimum by proper spacing of main mat connections and ground rods.

Protective relay equipment is provided for switchyard equipment as follows:

Bus differential current on each bus section

Line impedance, directional distance relays

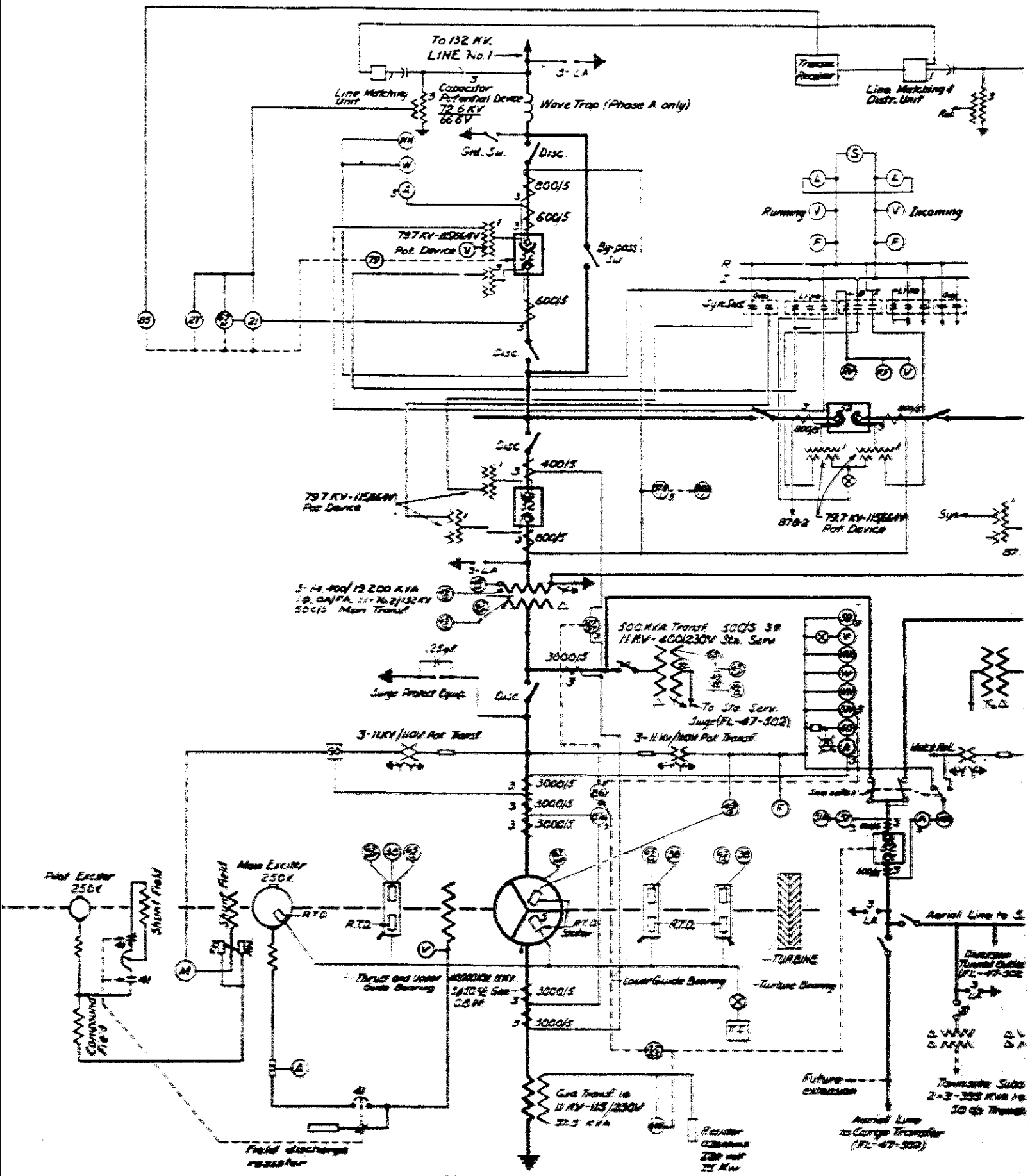
Line directional ground relays, current polarized.

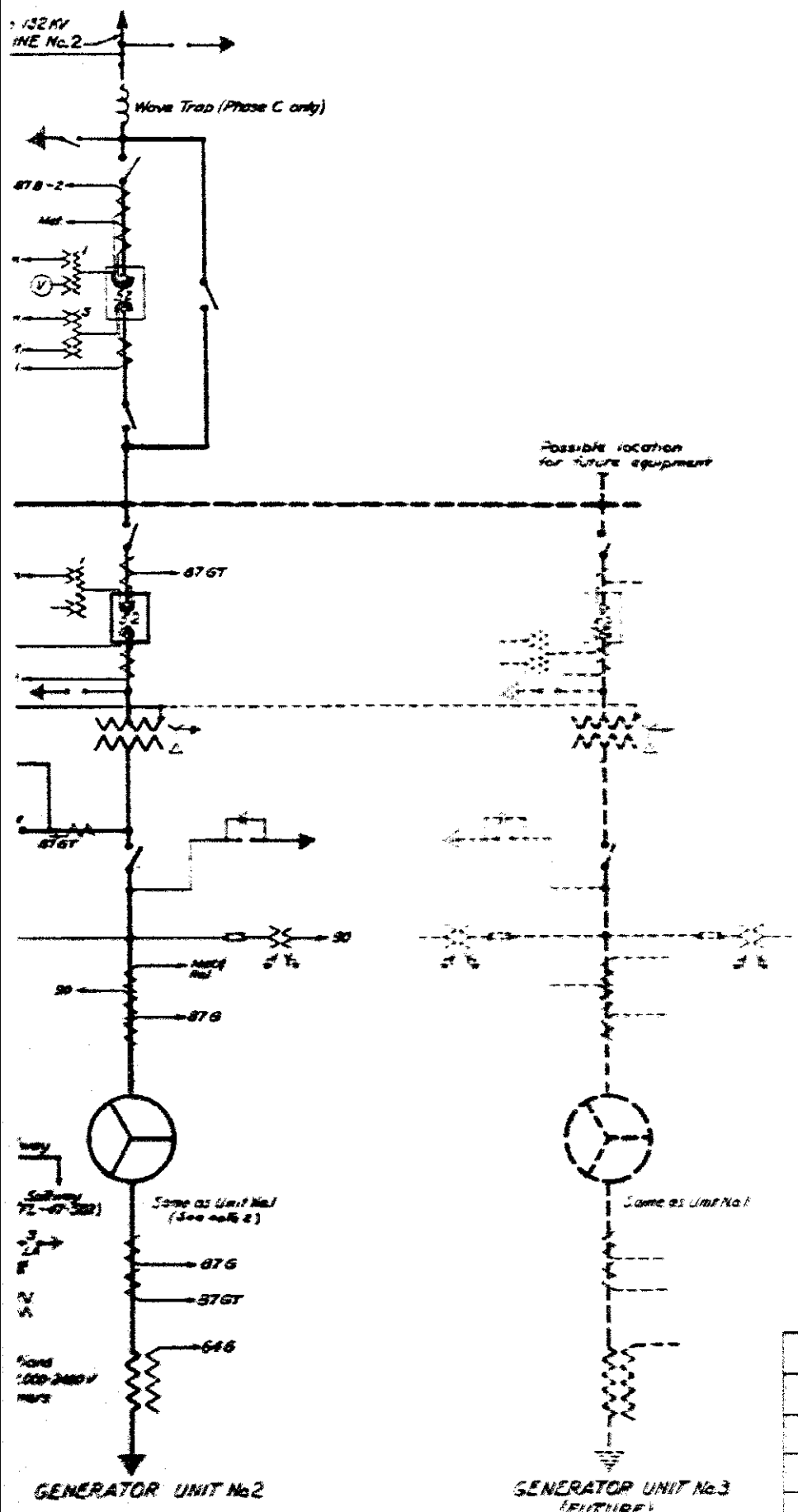
The line relay system was designed as part of the design of the transmission lines by others.

PART V - CHAPTER VI

LIST OF EXHIBITS

Exhibit	Title
VI - 1	Main Single Line Diagram
VI - 2	Switchyard Electrical Equipment Plan


To 132 KV. LINE No 1
Line Matching Unit
Capacitor Potential Device
Wave Trap (Phase A only)
Transm. Receiver
Line Matching & Distr. Unit
Grd. Sw.
Disc.
800/5
600/5
By-pass Sw.
Running
Incoming
Syn. Sel.
400/5
Pot. Device
Syn.
3-14 400/13 200 KVA
Main Transf.
500 KVA Transf. 500/5 3φ
11 KV-400/230V Sta. Serv.
3000/5
To Sta. Serv.
Surge Protect Equip.
3-11KV/110V Pot. Transf.
3-11KV/110V Pot. Transf.
Field Exciter 250V
Shunt Field
Main Exciter 250V
Series Field
RTD
Stator
TURBINE
Thrust and Upper Guide Bearing
Lower Guide Bearing
Turbine Bearing
Aerial Line to S.
Future extension
Aerial Line to Cargo Transfer
Field discharge resistor
SENERATOR UNIT No.1



LEGEND

ASA standard device numbers and symbols are used for relays and instruments.
Relay suffix letters indicate location or condition to which device responds as follow

B	Bus
G	Generator
GF	Gas Pressure
GT	Generator-Transformer Unit
N	Neutral
O	Oil
V	Voltage Restraint
W	Winding
WF	Water Flow

Instrument letters are defined as follows:

A	Ammeter
F	Frequency Meter
L	Lamp
M	Motor
PMG	Permanent Magnet Generator
RF	Recording Frequency-Meter
RV	Recording Voltmeter
S	Synchroscope
SI	Speed Indicator
TI	Temperature Indicator
V	Voltmeter
VAR	VARmeter
W	Wattmeter
Wh	Watthour-meter
⊗	Instrument Switch
⊗	Indicating Lamp, blue cap

NOTES

1.—Air break transfer switches connecting the 11 kV to the generator buses shall be interlocked prevent simultaneous closing of both switch
2.—Details of protective, metering and control circ are not shown for unit No.2 unless differen unit No.1.

REFERENCE DRAWINGS

Auxiliary Power Single Line Diagram
Misc. Small A.C. & D.C. Single Line Diagram

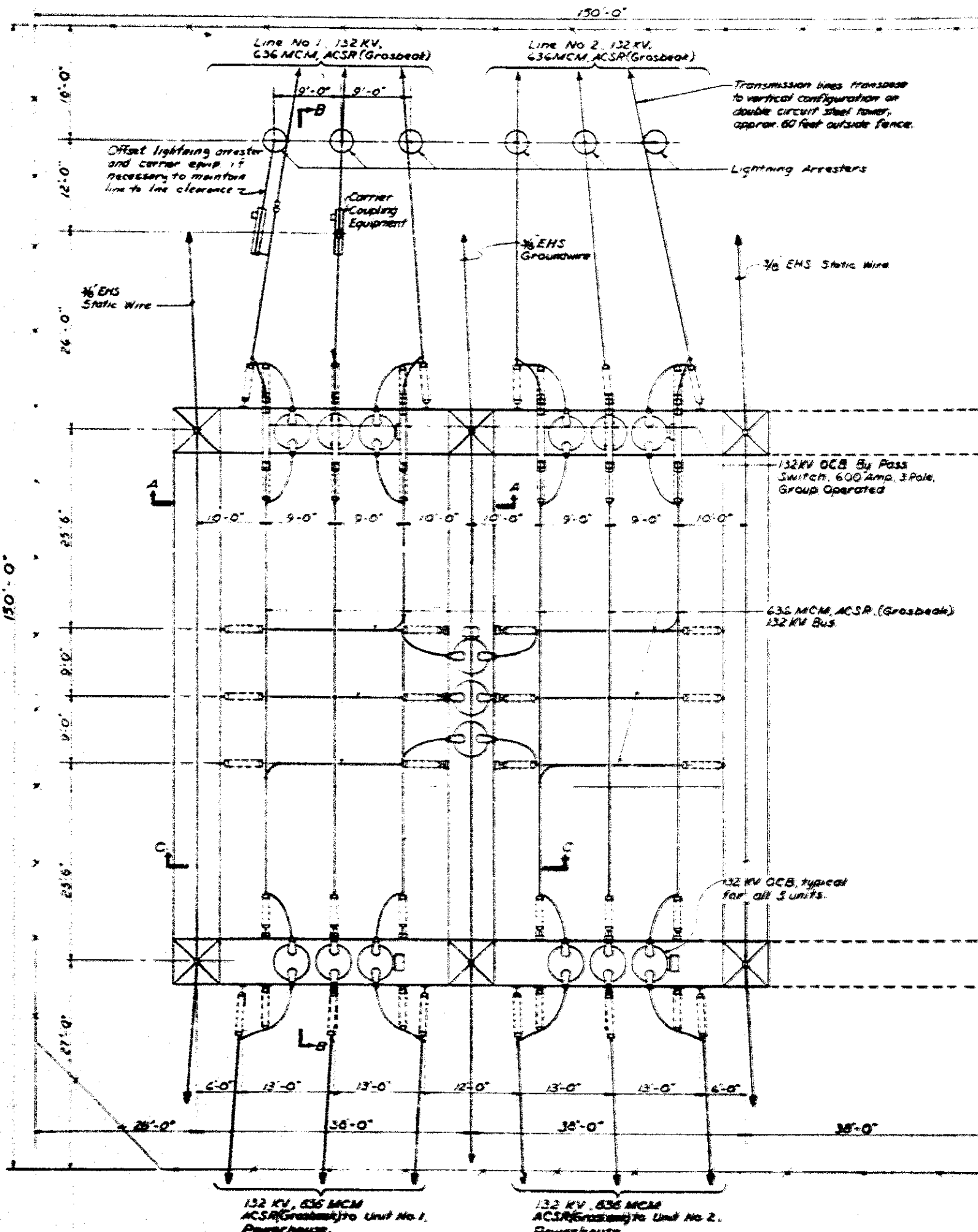
No Scale

GOVERNMENT OF PAKIS
MINISTRY OF INDUSTRI
KARNAFULI PROJEC

MAIN SINGLE LINE D

INTERNATIONAL ENGINEERING CO. INC.

5	March 21, 1960	RC	JA
	As Built		
4	May 13, 1959	RC	JA
	General Changes		
3	March 5, 1959	JA	KK
	Chg PT & BPO, V Rel		
2	Sept 24, 1958	AL	JA
	Add V.Lines Lime & Meters		
1	Sept 3, 1958		JA


150'-0"
Line No 1, 132 KV, 636 MCM, ACSR (Grosbeak)
Line No 2, 132 KV, 636 MCM, ACSR (Grosbeak)
Transmission lines transpose to vertical configuration on double circuit steel tower, approx. 60 feet outside fence.
Offset lightning arrester and carrier equip. if necessary to maintain line to line clearance
Lightning Arresters
Carrier Coupling Equipment
3/8" EHS Groundwire
3/8" EHS Static Wire
3/8" EHS Static Wire
132 KV OCB By Pass Switch, 600 Amp, 3 Pole, Group Operated
636 MCM, ACSR, (Grosbeak) 132 KV Bus
132 KV OCB, typical for all 5 units.
132 KV, 636 MCM ACSR (Grosbeak) to Unit No. 1, Powerhouse
132 KV, 636 MCM ACSR (Grosbeak) to Unit No. 2, Powerhouse

GENERAL NOTE

Transmission line take-off assumed 11° West of North.

Minimum clearances shall be as follows:

Line to Line	8'-0"
Line to ground	4'-9"
Line above earth over roads or vehicular access	30'-0"

REFERENCE DRAWINGS

Powerhouse - Electrical Equipment Outdoors - Plan	FL-57-201
Switchyard - Electrical Equipment Sections	FL-57-202
Powerhouse - Switchyard Transmission Line Layout	FL-59-203

Scale 0 _____ 8 Feet

GOVERNMENT OF PAKISTAN
MINISTRY OF INDUSTRIES

KARNAFULI PROJECT

SWITCHYARD
ELECTRICAL EQUIPMENT
PLAN

INTERNATIONAL ENGINEERING CO., INC. — ENGINEERS

Designed J.A.	Checked KK	Approved	
Drawn F.L.P.E.N.J.	Traced RCM RAC		
ICA-X-1	San Francisco, Cal. Date June 2, 1959	FL-57-201	

No.	Date	Revision	Checked	Approved
4				
3				
2				
1				

PART V

CHAPTER VII

CARGO TRANSFER SYSTEM TROLLEYS

7.01 GENERAL

A. Purpose

Three monorail type electric driven trolleys are provided to transport bamboo or wood rafts and small river boats between the reservoir and the river below the dam. Two trolleys will normally be sufficient to meet the general requirements of traffic, permitting the third to be placed on standby service. Two trolleys may be operated in tandem when individual loads to be transported are in excess of the capacity of a single trolley unit.

B. Description

Each trolley consists of a supporting frame suspended from a single monorail track by two 4-wheel trucks. Each unit is completely self-contained with motivating drives, a two-drum hoist, operator's cab, anti-collision device, and controls. Power is furnished to the trolleys from an overhead conductor system mounted on the top of the monorail track. Complete control of the functions of the trolley can be effected from the cab. See Figure A.

7.02 DESIGN

A. Lifts and Speeds

The trolley hoists were designed for a lift of 125 feet with excess capacity to lift 139 feet from the extreme raised position. The normal full load hoisting and lowering speed is 120 feet per minute. The full load trolley traveling speed is 420 feet per minute.

B. Loads

Each trolley hoist is rated at 4 tons capacity, effected from two 2-ton hooks. The capacity is sufficient for individual lifts and, when correlated with the hoist and trolley speeds, is adequate to handle the design rate of 150,000 lbs. of cargo per hour with two units. The component parts of the trolleys are designed in consistence with good practice for equipment of this type.

C. Mechanical Features

The trolleys are driven by the four wheels of the rear truck, these comprising one-half of the total trolley wheels. The drive bogie

consists of two 2-horsepower motors, each with gear reductions to the driven wheels and conical rotor brakes built-in to the motors.

The two-drum hoist is rated at 4000 lbs. at each hook. The hoist mechanism consists of a single 40-horsepower motor with a laminated disc-type brake and a two-stage speed reduction unit coupled directly to the drums. High and low position limit switches are driven directly from the forward drum and interrupt the respective hoist circuits. Wire ropes of 15-millimeter diameter extend from each drum to a lifting beam. The lifting beam is equipped with automatic disengaging devices.

The aluminum panel operator's cab is weatherproof. It has adequate window area for the operator's observance of hoisting and traveling. Entry to the cab is from a platform located in the vicinity of the side track or from the catwalk.

Each trolley is fitted with an anti-collision device consisting of a separately towed four-wheel truck suspended from the monorail. The towed trucks are equipped with self-actuated brakes to maintain the correct relationship with the trolley frames. Contact between a trolley and a towed truck in front interrupts the drive motor current to the back trolley, thereby braking the unit.

D. Structural Features

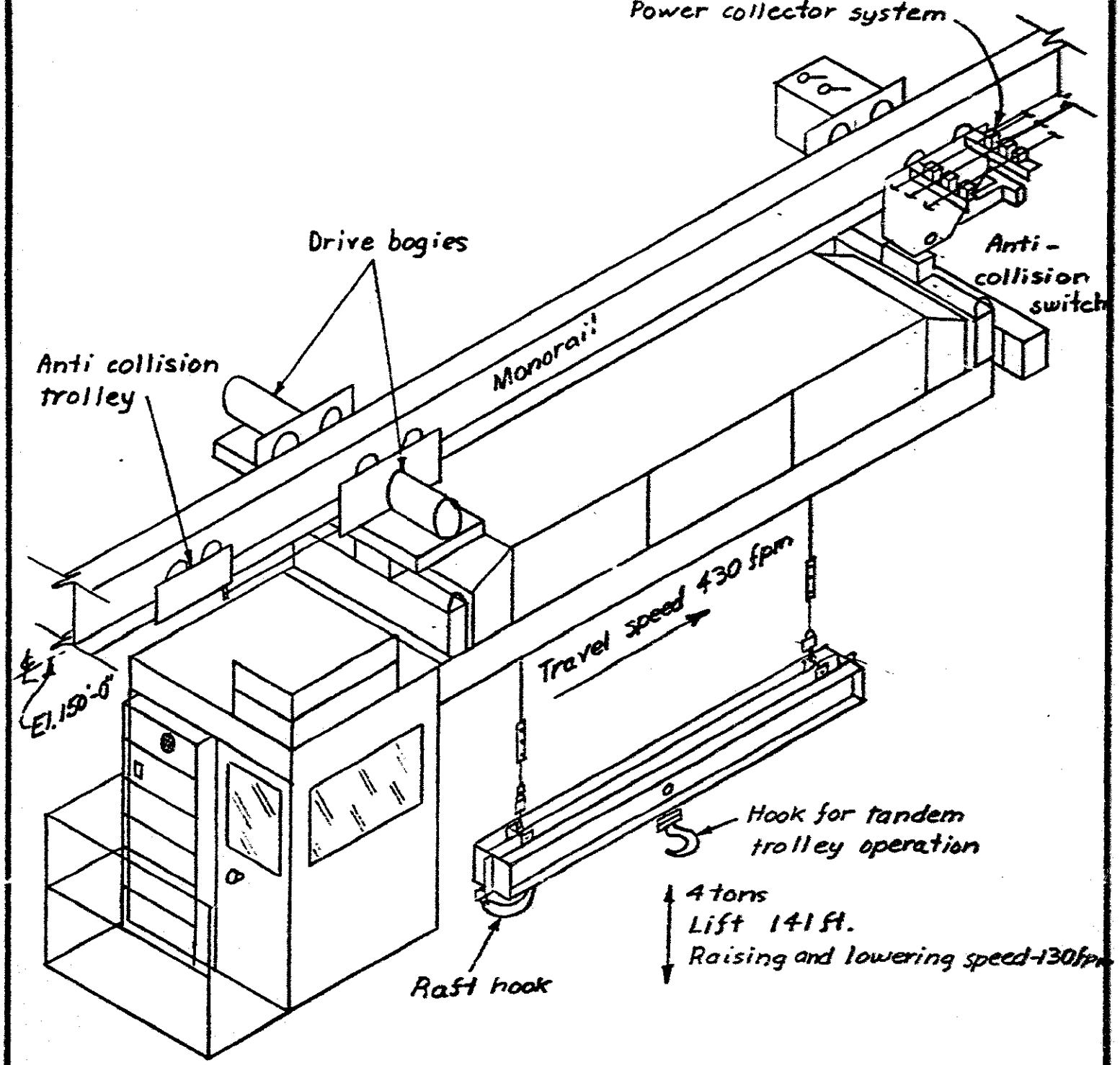
The main frame for the trolleys is a rigid welded steel unit, supporting the hoisting machinery and the cab. The frame is suspended from the trucks by double-pivoted joints to maintain uniform wheel contact on the monorail while traversing on the curved portion of the track.

E. Electrical Features

The electrical equipment is suitable for outdoor operation and for the power source of 230/400 volt, 3 phase, 50 cycle current. The hoist and drive motors are of the totally enclosed, slip-ring type, 1500 revolutions per minute, ball-bearing fitted, with one hour rating. Holding brakes for hoist and trolley drives are an integral part of the motors with special provisions to accommodate them. The lowering speed of the hoist is controlled by under-synchronous braking, affording greatly reduced lowering speeds when desired. The hoist control is of the variable speed, reversing manual type with four speed points in each direction. The traveling control is similar.

Cab lighting and ventilation is provided and each trolley is equipped with a tail light and a 50-watt flood light.

All components and materials for the electrical systems are suitably designed for tropical exposure.



CARGO TRANSFER SYSTEM
TROLLEY ASSEMBLY
DESIGN REPORT

INTERNATIONAL ENGINEERING CO., INC.
SAN FRANCISCO, CALIF.

DR. G.D. CK.	RECOMMENDED	APPROVED

DATE April 28, 1960 | FIG. A

KARNAFULI PROJECT

DESIGN REPORT

PART VI

ROADWAYS, CHANNELS, SURFACE DRAINAGE

and

MISCELLANEOUS FEATURES

CHAPTER I GENERAL

CHAPTER II ROADWAYS

2.01 General
2.02 Gutters and Culverts

CHAPTER III SURFACE DRAINAGE AND LANDSCAPING

3.01 General
3.02 Special Areas

CHAPTER IV MISCELLANEOUS FEATURES

4.01 Fencing
4.02 Guard Rails and Curbs
4.03 Miscellaneous Buildings
4.04 Tailrace Channel Protection

PART VI

CHAPTER I

GENERAL

This part of the Design Report discusses the design of roadways, channels, surface drainage, and miscellaneous features. These items, though not major, are quite important in that they provide essential protection or access to the major project features.

The design of the roadways, channels, surface drainage, and miscellaneous features. while in no way difficult or complex, necesarily required full attention to numerous details to achieve adequate protection and convenient access at an economical cost.

Details of the design, including purpose, location, and description of these various items, are given in the chapters which follow.

PART VI

CHAPTER II

ROADWAYS

2.01 GENERAL

A. Purpose

The roadways provide access between the various structures of the project to existing housing and office buildings and to existing roads.

B. Location

The main access road begins at the northwest entrance to the project area where it connects to the existing road to Chandraghona and Chittagong. From this point it is routed under the cargo transfer structure, southeast along the reservoir rim, across the ridge between the powerhouse and the intake structure, southeast to the center of the main camp area, north to the diversion tunnel gate house where it swings to the east to cross the main dam and connect to the spillway access bridge.

The access road to the powerhouse leaves the main access road in a westerly direction and swings north around the high ground between saddle dam No. 2 and the river to connect to the powerhouse parking area at Elevation 49 feet. Two access roads are provided to the switchyard from the main access road. One road is routed to the north side of the switchyard, and one to the east side. An access road is provided to the northeast side of the intake structure from the main access road at a point where it swings to cross the ridge between the powerhouse and the intake structure. A short access road is provided from the main access road at the left abutment of the main dam to connect to an existing townsite road.

Access turnouts and driveways are provided where existing townsite roads intersect the main access road. (Exhibit 2, Part I shows layout of access roads.)

C. Description

Concrete was selected for the pavement because it will require minimum maintenance under the severe weather conditions of East Pakistan. A flexible pavement would require more maintenance, and it would have been necessary to purchase special equipment for its construction. No additional equipment was required to construct the concrete pavement.

The pavement is placed directly on the sub-grade where the sub-grade is sand. In areas where the sub-grade is compacted clay or shale, the pavement is placed on a layer of compacted sand to provide positive drainage under the concrete and to prevent pumping action in the sub-grade under traffic.

A width of 22 feet was selected for the pavement on the main access road and the powerhouse access road to provide safe passing widths for traffic. A width of 15 feet was selected for all other roads for single lane construction as there was either no requirement for passing or the road connected to an existing narrow road.

All roads, except that across the main dam, have 6-foot wide shoulders which are sodded to prevent erosion by monsoon rains. The shoulders provide parking areas for disabled vehicles and a protective zone for drainage structures which normally parallel the road.

In cross-section the pavement slopes down 3 percent on both sides of the centerline for quick runoff of rain water to the gutters. The sub-grade slopes are parallel to the pavement slopes to provide under-drainage. All horizontal curves have been super-elevated from 5 to 8 inches. Changes of grades greater than one percent are made by 100-foot vertical curves.

D. Design

The rigid pavement was designed as an unreinforced slab with adequately restrained joints and edge thickening. Two thicknesses of pavement were selected. The roadway from the cargo transfer structure to the approach of the ridge between the powerhouse and intake structure (where heavy lumber hauling may be encountered) is 7-1/2 inches thick, with an edge thickness of 11 inches. This section is adequate for 10,000 to 12,000-pound wheel loads, and is the equivalent of heavily traveled rural roads in the United States of America. The remainder of the access roads are 6 inches thick, with an edge thickness of 9-1/2 inches. This section is adequate for 7,000 to 8,500-pound wheel loads, and is the equivalent of roads and streets carrying only occasional wheel loads of 8,000 pounds and over in the United States of America. The thickness was determined from the following formula:

$$d_1 = \sqrt{\frac{1.92\ WC}{S}}, \quad ti = 0.85\ d_1, \quad te = 1.5\ ti$$

where d_1 = nominal thickness of pavement

ti = center thickness of pavement

te = edge thickness of pavement

W = wheel load in pounds

C = coefficient of sub-grade - taken as 1.00

$$S = \frac{\text{modulus of rupture}}{\text{factor of safety}} \text{ - taken as } \frac{650}{2} = 325$$

Expansion joints were placed at 120-foot intervals, and contraction joints at 15-foot intervals. Details of joints were as recommended by the American Concrete Institute standards "Design of Concrete Pavements". Exhibit II-1 shows typical road sections and construction details.

Where the roadway sub-grade is placed on fill, the fill is compacted to a density of 90 percent of Modified AASHO standards.

2.02 GUTTERS AND CULVERTS

A. General

Gutters are provided to take care of rain water runoff and to provide drainage of roadway bases. In general, they parallel the road except in locations where the ground slopes away from the roadway and the presence of runoff water is acceptable. Culverts are provided where it is necessary for drainage to cross the roadway.

B. Description

Two types of gutters were selected; local brick plastered with Portland cement-sand mortar, and precast concrete blocks. The plastered brick gutters were selected during the early part of construction when it was expedient to install permanent gutters for protection and other materials were not available.

Precast block gutters were selected later, as they are similar to the gutters and drains used on the downstream slope of the main dam and casting forms were available. Two types of precast block gutters were used: type A, vee-shaped with 2.5-to-1 side slopes; and type B, trapezoidal in shape with 1-1/2-to-1 side slopes and a one-foot wide base. In general, the gutters follow the road profile. Where the road profile is flat or reverse to the flow in the gutters, the minimum slope of the gutter is 0.005 feet to the foot.

The corrugated galvanized metal culverts are of either 18-inch diameter or 30-inch diameter, depending on the estimated runoff. In areas where it was necessary to maintain the grade in gutters and outfalls so high that proper burial depths for protection of the culverts was not possible, 30-inch reinforced concrete box culverts were specified. All culvert entrances have drop inlets and are protected by head-walls at their outlets. The minimum slope on culverts is 3/16-inch to the foot.

PART VI

CHAPTER III

SURFACE DRAINAGE AND LANDSCAPING

3.01 GENERAL

With the exception of the powerhouse area, the majority of landscaping in the project was determined by requirements for surface drainage. The intention was to give the job a neat and pleasing appearance, to provide controlled surface drainage, and to protect runoff slopes from erosion by heavy monsoon rains.

The typical hills and deep gullies of the Chittagong Hill Tracts in the area were leveled and filled to elevations varying from Elevation 120 to 170 feet. Cuts for permanent features, other than hydraulic channels, were made as steep as practical to prevent erosion by rain. The leveled areas at the tops of the cuts were graded to carry surface drainage away from the cut. These areas drain into the gutters paralleling the access roads, or into special gutters provided to collect surface water. Discharge from the gutters is conveyed by concrete chutes into the reservoir or the Karnafuli River. Where the chutes do not discharge directly into water, the outlet end is provided with a small stilling basin to dissipate the energy of the water. The ground at the outlet is further protected by riprap unless the material is hard shale. On steep slopes the chutes are anchored to prevent movement.

The areas graded for surface drainage are sodded in the vicinity of permanent structures to prevent unsightly erosion of the topsoil.

3.02 SPECIAL AREAS

A. Powerhouse

The ridge between the powerhouse and the intake structure is graded to drain into the gutters paralleling the main access road to prevent surface drainage from flowing down the cut faces into the powerhouse or intake structure. The sandy material on top of the ridge is covered with a minimum of 12 inches of impervious top soil, and sodded to present a pleasing appearance and to prevent ingress of rain water into the ridge. The berms around the powerhouse excavation are graded to the inside, and either provided with plastered-brick gutters or ditches to carry drainage away from the cuts to areas of shale outcrop.

Special attention was given to the landscaping of the powerhouse area in order to harmonize with the architectural features of the powerhouse. The entire area between the tailrace and the river is graded for surface drainage to the river, sodded and planted with a number of local trees. The concrete parking area is surrounded with a shrub border, which defines the area and complements the planted areas at either side of the main entrance (see Part IV, Chapter II).

B. Spillway

The right abutment of the spillway rests against a cut in shale. To prevent ingress of water and limit fall-outs from the steep slope, the surface at the top of the cut is sealed with impervious material and sodded. The area is graded to drain away from the cut into a gutter which discharges into the reservoir.

C. Cargo Transfer

The river bank under the cargo transfer structure consists of sandy materials with occasional shale outcrops. Construction berms in this area were graded to drain into plastered-brick gutters on the inside of the berm to prevent runoff from eroding the structure foundations. The gutters discharge on shale outcrops.

CHAPTER IV

MISCELLANEOUS FEATURES

4.01 FENCING

A. Description

Two types of fencing were selected for use on the project. Galvanized chain-link fences on galvanized pipe frames are installed in areas requiring maximum security. Barbed-wire fences on concrete posts are used in all other fenced areas.

The chain-link fences are 2-inch mesh with 9-gage galvanized wire stretched tightly over, and clamped to, 2-inch minimum diameter galvanized steel posts, nominally spaced at 10-foot centers. The bottoms of the posts are encased in concrete for maximum rigidity. Chain-link fences are provided with gates where required for access.

The barbed-wire fences are 5 feet high with galvanized 12-gage, 4-point hog wire stretched, at 6-inch spaces, on triangular concrete posts set nominally at 12-foot intervals. The posts are set 3 feet deep in the ground. Strain, corner, and angle posts are set in concrete.

B. Location

The switchyard is completely fenced with a 6-foot high chain-link fence, with angle arms at the tops of the fence posts suspending 3 strands of barbed-wire. A 14-foot wide gate is provided for access on the north and east sides of the yard. This fence prevents unauthorized persons from entering the dangerous high-voltage area of the switchyard.

A 5-foot high chain-link fence is placed across the powerhouse access road near the powerhouse parking area to regulate entry into the area. The fence is provided with a 20-foot wide gate for access of vehicles, and a 3-foot wide gate for access of pedestrians. A barbed-wire fence is connected to each end of the chain-link fence at the approach to the powerhouse, and is extended to enclose the entire powerhouse area to prevent entry of animals and unauthorized persons.

Four-foot high chain-link fences were placed at each end of the transformer deck to deter entry into the dangerous high-voltage area by unauthorized persons. Three-foot wide gates are provided in each fence for access to the transformer deck, and a 14-foot wide gate is provided in the south fence to allow movement of the gantry crane and the transformers.

A barbed-wire fence runs from the diversion tunnel gate shaft house across the main dam to the spillway right abutment to prevent unauthorized entry into the project in this area, and especially to prevent handling of cargo over the main dam. A barbed-wire fence is placed at the top of the cut on the right abutment of the spillway as a safety precaution.

4.02 GUARD RAILS AND CURBS

A guard rail is provided on the ridge between the powerhouse and the intake structure to prevent vehicles out of control from plunging into the powerhouse. The guard rail is of the standard flexible-beam type made from 12-gage galvanized metal. The rail is bolted to 8-inch square reinforced concrete posts set 4 feet deep at 12.5-foot centers. The center of the rails is 18 inches above ground level.

Eighteen-inch high reinforced concrete curbs are provided on both sides of the main access road across saddle dam No. 2 to a point south of the intake structure. The fill in this area is too narrow to allow placement of the usual side gutters. The curbs prevent road drainage from eroding the embankment. They also provide protection against vehicles running off the embankment.

4.03 MISCELLANEOUS BUILDINGS

Miscellaneous small buildings have walls of local brick and floors and roofs of reinforced concrete. The walls are plastered inside and out with Portland cement-sand mortar. Four of the buildings house fire-fighting equipment, two at the powerhouse and two at the switchyard. Two of the buildings on the downstream slope of the dam house the horizontal piezometer gages and related equipment. One building houses the emergency diesel generator near the spillway right abutment.

4.04 TAILRACE CHANNEL PROTECTION

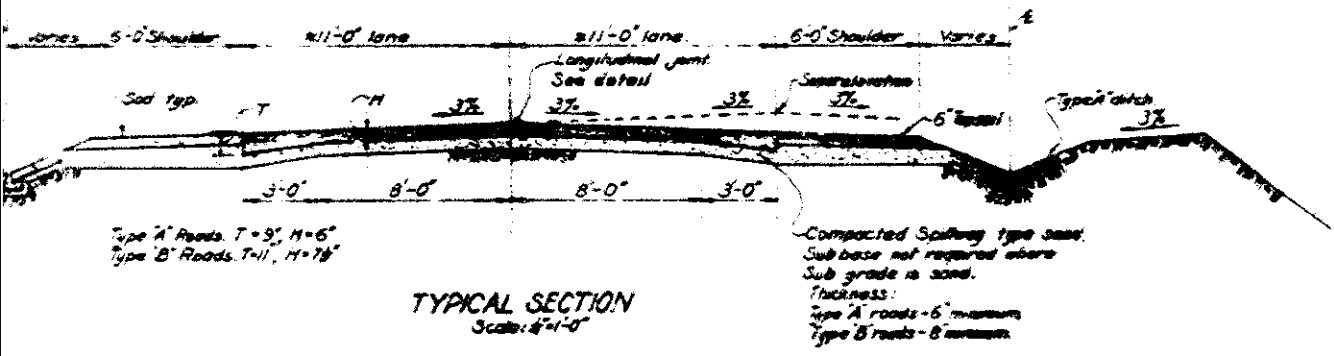
The tailrace channel was excavated in shale with near-vertical sides. The channel was lined with concrete for a distance of 50 feet downstream from the powerhouse for protection against erosion of the shale surfaces. The base slab of the lining is 12 inches thick and is divided by longitudinal contraction joints at 35.5-foot spacing. The top face of the slab is nominally reinforced to prevent cracking due to temperature and shrinkage stresses.

The lining on the side cuts is approximately 12 inches thick and extends from the base slab to Elevation 26.0 feet. It is anchored by No. 8 reinforcing bar grouted into the shale at 6.0-foot centers. Weep holes at 4.0-foot centers were formed in the lining to prevent unbalanced hydrostatic pressure between the shale and the lining. Both faces of the side lining were nominally reinforced.

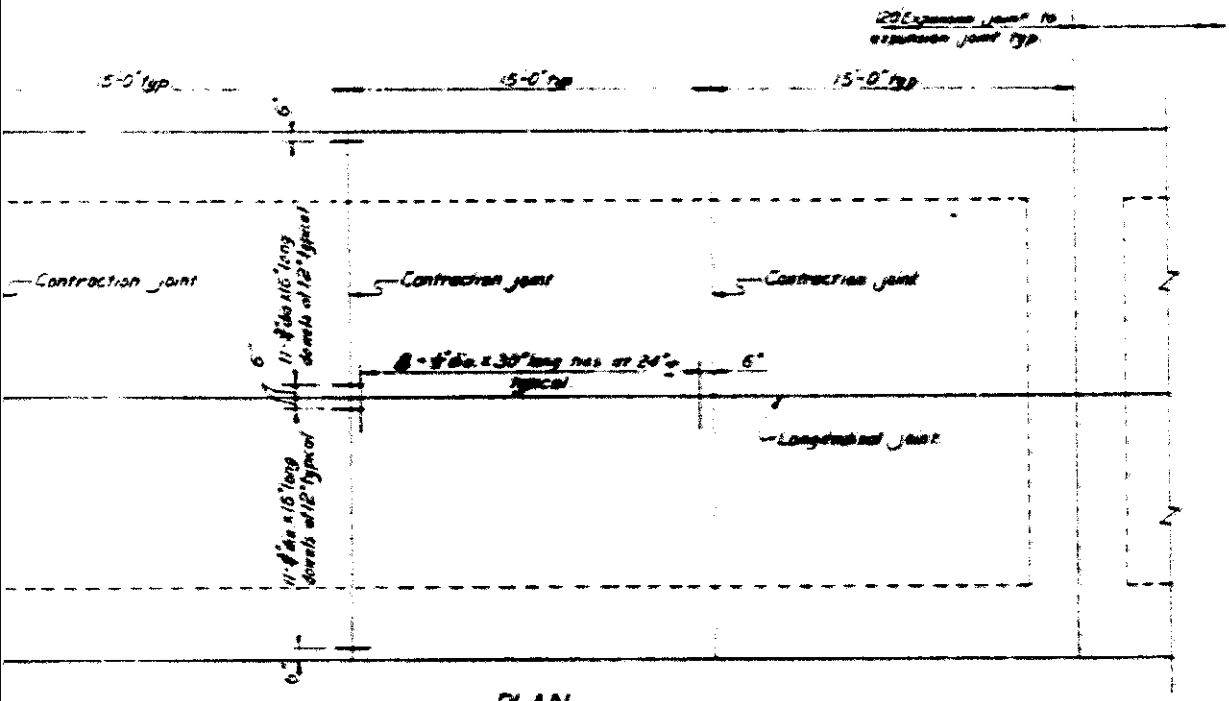
On the left side of the channel above the lining, rock anchors were installed at 6.0-foot centers up to Elevation 46.0 feet to prevent fallouts. The full shale face was similarly anchored for a distance of 100 feet downstream from the end of the lining.

On the right side of the channel, rock anchors were installed at 6.0-foot centers up to Elevation 22.0 feet also for a distance of 100 feet downstream from the end of the lining.

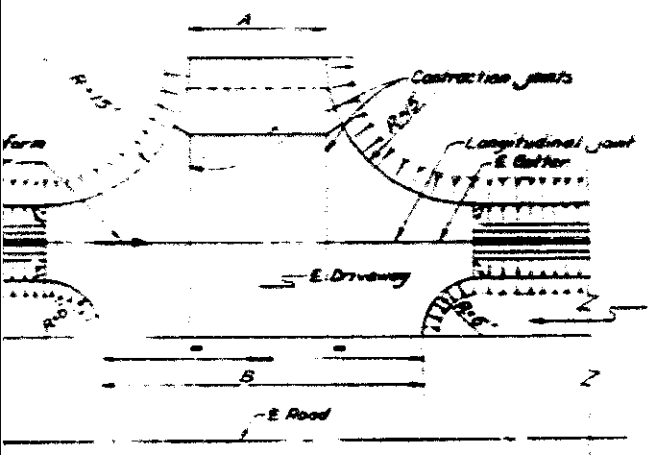
* Main access Road lane 12'-6", Sta. 261+50 to 269+00 typical Section Dwg. FL-81-1213
Intake access Road and Switchyard access Roads single lane 15'-0" wide see typical Section Dwg. FL-81-1220



TYPICAL SECTION
Scale: ¼"=1'-0"

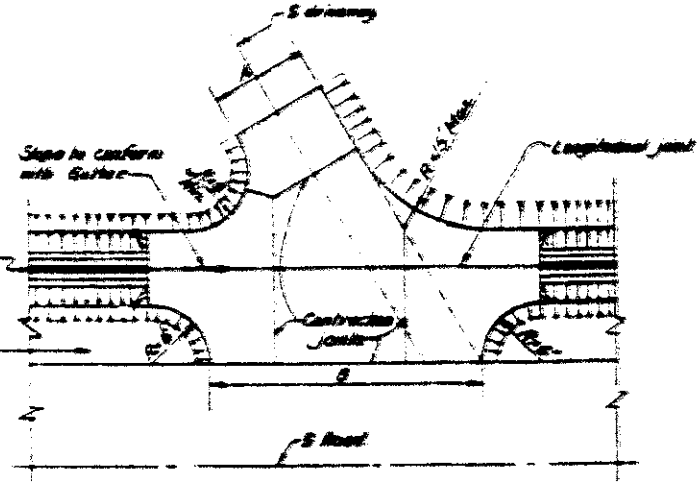


PLAN
TYPICAL PAVEMENT
Scale: ¼"=1'-0"

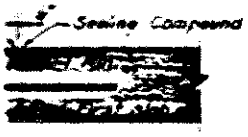


PLAN AT RIGHT ANGLE
TYPICAL ARRANGEMENT
DRIVEWAYS OVER GUTTERS
Scale: 1"=10'-0"

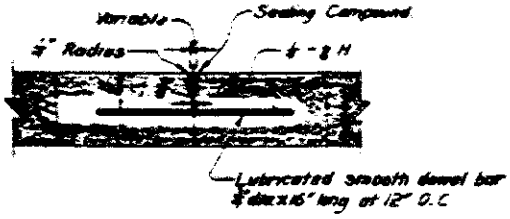
B=A+18'



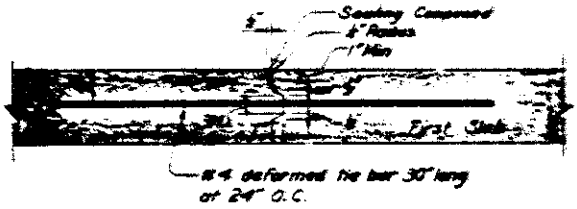
PLAN AT ANGLE
TYPICAL ARRANGEMENT
DRIVEWAYS OVER GUTTERS
Scale: 1"=10'-0"



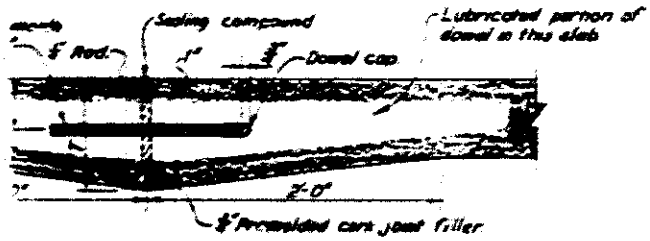
TAIL
ION JOINTS
T TYPE



DETAIL
CONTRACTION JOINTS
GROOVE TYPE.



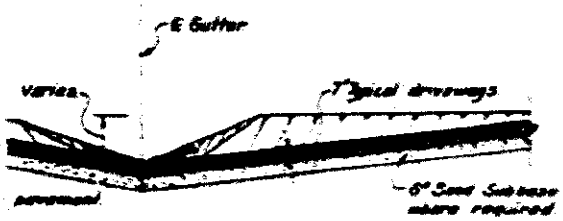
DETAIL
LONGITUDINAL JOINTS



DETAIL
EXPANSION JOINT

NOTES:

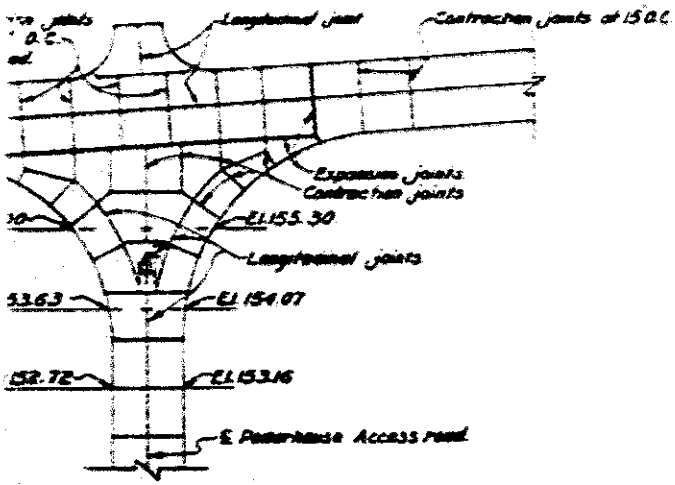
Road pavement shall be class "A" concrete
Road shall be of two types. Type "A" standard, Type "B" heav
thickened edges and joints as detailed
Expansion joints shall be nominally at 120 feet intervals
Contraction joints shall be nominally at 15 feet intervals
Contraction joints may be of the "butt" type or "groo
Construction for 22 feet and 25 feet wide roads shall
construction" with longitudinal joint as detailed
Roads with widths 15 feet and below may be of single la
For layout of roads see drawings - FL-81-1202 and 1203
to 1212, FL-81-1217 and 1220.



ICAL SECTION DRIVEWAYS
Scale: 1/4"=1'-0"

REFERENCE DRAWINGS:

Powerhouse access Road Plan and profile Sheet 1 FL-
Powerhouse access Road Plan and profile Sheet 2 FL-
Main access Road Sta 198+20.51 to 211+06.75 Plan & Profile
Main access Road Sta 211+06.75 to 220+00.00 Plan & Profile FL-
Main access Road Sta 220+00.00 to 231+80.25 Plan & Profile FL-
Main access Road Sta 231+80.25 to 243+13.71 Plan & Profile FL
Main access Road Sta 243+13.71 to 254+91.02 Plan & Profile
Main access Road Sta 254+91.02 to 268+79.44 Plan & Profile
Main Dam Sta 224+00 to Spillway Plan & Sections - - - - - - FL
Intake Structure access Road Plan and Sections - - FL-



DETAIL INTERSECTION
N ACCESS ROAD AND
WERHOUSE ACCESS ROAD
Scale: 1"=30'-0"

Scale: As noted.

GOVERNMENT OF PAKISTA
MINISTRY OF INDUSTRIES

KARNAFULI PROJECT

ACCESS ROADS
SECTIONS AND DETAILS

INTERNATIONAL ENGINEERING CO. INC. — ENGI

R.C.B.K.
A.A.K.

KA-X-1

Nov 26, 1969

FL-81-12