Proposed Construction Sequence Documents

for the

Natick Dam Hydroelectric Project

West Warwick, Rhode Island

April 18, 1990

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I) Construction Sequence

0) Preliminaries:

a) Contractor to breach the existing floodwall at the location of the proposed powerhouse. Contractor to excavate to elevation 20 msl and create a hole at this elevation approximately 60' long and 30' wide. The side slopes of the excavation should not exceed 3' H to 5' V. When the excavation reaches 3' above tailwater (approximately 34.5 msl), contractor should construct a tailwater coffer dam on the river bed. Said coffer dam shall be constructed of "Jersey" barriers in a semicircular arc with a 70' radius centered at the center of the proposed powerhouse's back wall. Cofferdam shall then have sand bags placed along its outside edge. The bottom row of bags shall be four wide tapering to one bag wide along the top edge of the concrete barriers. Once the coffer dam is completed, the contractor shall complete the powerhouse excavation as described. Borings indicate that approximately 3' to 5' of rock will need to be removed from the base of the excavation before any drilling and pinning can begin.

b) Contractor shall dig a 5' deep sump hole in one corner of the excavation and install a pump or pumps of sufficient capacity to dewater the hole. Once the hole is dewatered, an engineer shall inspect the side slopes and seepage rates and determine if it is safe for workmen to access the excavation. If the seepage rates are excessive, contractor shall drive sheet piling across the mouth of the canal to refusal or to a minimum of 20'. Piling to extend perpendicular, from the existing stonewall along Water Street, to the granite dam abutment. Sheet piling to be parallel to and approximately 15' easterly of the existing shoreline which forms the filled in mouth of the canal.

c) Contractor shall be responsible for shoring up the masonry opening cut through the granite flood wall to prevent stones from dislodging and injuring the workmen.

d) Contractor shall drill 1 1/2" diameter pin holes , 18" deep, as described on the construction sequence drawings. Holes shall be flushed and blown clear with compressed air. Rebar pins to be epoxy nonshrink grouted into the holes.

1) Construct the Discharge Pit Side and Center Walls:

Walls to begin at and be perpendicular to the existing river retaining wall. Walls to be level, at top of wall (TOW), elevation 39.0 for 18' from the riverside. Walls require pockets for the 30" beam. Vertical rows of rebar to be left projecting normal to their top faces to tie the waterbox floor to. (note: tapered 2x4 s to be inserted to create horizontal keys). Also horizontal double rows of rebar left projecting to tie rear waterbox walls into. Six inch drains to be installed at approximate elevation 21.5 msl along outside walls. Walls to be backfilled with crushed gravel to elevation 31.5 msl. A plastic liner is to be placed over the top (only) of the gravel before final backfilling. Backfilling is to be done only after 21 days of curing have passed. See drawings No. ND 101 to ND 104 inclusive for details.

2) Construct Discharge Pit Rear Walls:

Hang rebar, set forms and pour rear waterbox walls. Vertical rebar to be left projecting 18" to tie waterbox floor slab into.

3) Set Lower 30" Beam:

Sling beam into place with ends placed into and supported by the lower pockets. Beam to be aligned and leveled to top of beam el. 39.0. Previously the beam should be sandblasted, primed and painted with an epoxy cold tar based paint. See drawing ND 105 for details

4) Set the Four North-South 8"x24# Turbine Support Beams:

These beams to be set with the top of their flanges at el. 40.0 msl, the top of the waterbox floor. Since the waterbox floor is 12" thick, the beams must be supported at this elevation by some acceptable means. If they are to be supported at their ends, only steel blocking is acceptable. Suggest using short sections of 4" channel or I beams. See drawing No. ND 105 for details.

5) Construct Waterbox Floor Slab :

Rebar to consist of two sets of horizontal grids. Rebar to be #8 bar at 6" spacing in the short (north-south) direction and #6 bar at 18" spacing in the long (east-west) direction. Lower horizontal grid to be at el. 39.25 (ie: 3" of bottom cover) and upper horizontal grid to be at el. 39.75 (ie: 3" of top cover). Draft tubes to be set in place, one each, between each set of turbine support beams. Water box floor drains (to be supplied by Contractor) to be installed. Before pouring walls, 2x4 waterstop forms to be removed and "Duraweld C" epoxy bonding agent or equivalent shall be liberally applied to waterstop joints. Then concrete should be poured into forms. Floor requires a set of horizontal row of rebar projecting normal to its face at elevation 39.25 msl to tie the forebay floor into. Vertical rows of rebar to be left projecting normal to the floor at the locations of the waterbox walls. Tapered 2x4s to be inserted in fresh concrete between vertical rows of projecting rebar at location of waterbox walls. This to create waterstop keys to pour the waterbox and divider walls onto. See Drwg. No. ND-105 for locations and dimensions.

6) Construct the Waterbox Side and Center Walls :

Walls to begin at and be perpendicular to the existing river retaining wall. Walls to be level, at top of wall (TOW), elevation 53.0 for 18' from the riverside. Walls require pockets for the 30" beam. Vertical rows of rebar to be left projecting normal to their top faces to tie the generating room floor to. (note: tapered 2x4 s to be inserted to create horizontal keys). Walls to be backfilled with crushed gravel to elevation 44.5 msl. A plastic liner is to be placed over the top (only) of the gravel before final backfilling. Backfilling is to be done only after 21 days of curing have passed. See drawings No. ND 106 to ND 108 inclusive for details.

7) Construct Waterbox Rear Walls:

Hang rebar, set forms and pour rear waterbox walls. Vertical rebar to be left projecting 18" to tie generating room floor slab into. Contractor to form up and pour a patch to tie the powerhouse to the existing river retaining wall. Patch to be pinned to the powerhouse and masonry.

8) Set Lower 30" Beam:

Sling beam into place with ends placed into and supported by the upper pockets. Beam to be aligned and leveled to top of beam el. 53.0. Previously the beam should be sandblasted, primed and painted with an epoxy cold tar based paint. See drawing ND 110 for details

9) Set the Four North-South 12"x92# Turbine Support Beams:

These beams to be set with the top of their flanges at el. 54.1 msl, the top of the generating room floor. Since the generating room floor is 13" thick, the beams must be supported at this elevation by some acceptable means. If they are to be supported at their ends, only steel blocking is acceptable. Suggest using short sections of 4" channel or I beams. See drawing No. ND 110 for details.

10) Construct Generating Room Floor Slab :

Rebar to consist of two sets of horizontal grids. Rebar to be #7 bar at 6" spacing in the short (north-south) direction and #7 bar at 18" spacing in the long (east-west) direction. Lower horizontal grid to be at el. 53.25 (ie: 3" of bottom cover) and upper horizontal grid to be at el. 53.75 (ie: 3" of top cover). Before pouring walls, 2x4 waterstop forms to be removed and "Duraweld C" epoxy bonding agent or equivalent shall be liberally applied to waterstop joints. Then concrete should be poured into forms. Floor requires a set of horizontal row of rebar projecting normal to its face at elevation 53.25 msl to tie the trash raking platform into. Eight inch dia. pipe should be set over the location of the water box drains to create access holes through the generating room floor (this to provide an opening to insert actuating rods to pull tapered drain plugs). Generator high and low voltage conduits should be installed to switch gear cabinet location. Then concrete should be poured into forms to el. 54.1 msl. Floor should be mechanically floated to a smooth finish. See Drwg. No. ND-110 for locations and dimensions.

11) Construct Forebay Footings:

Once all concrete has cured for 21 days, the walls should be backfilled. Contractor to then excavate out the hole, for the forebay to be located in. Footings as shown in ND-109 should be poured at 38.5 msl.

12) Construct Forebay Side and Center Walls:

Walls to be tied into and set onto new footings. Trashrack support beams act to support racks and act as spreader columns during dewatering of a single bay. Beams to be installed and aligned in the forms by the contractor. Contractor to insure the beam locations and that their flanges are 45 degrees to the horizontal. 14" platform support beams to be installed with their top flange at 53.7 msl.

13) Construct Forebay Floor:

Contractor to fill the floor space with crushed gravel 6" thick from elevation 39.0 to 39.5 msl. Floor to be constructed with #6 bar on 18" grid. Floor to be 6" thick and machine floated to a smooth finish.

14) Backfill Forebay Walls:

Contractor to install french drains parallel to the forebay walls and running their length to discharge into the river at the rear of the powerhouse. French drains to be 8" diameter and placed in the gravel backfill as shown on ND 115.

Design of Powerhouse Substructure
Natick Hydroelectric Project
West Warwick, Rhode Island
FERC License Project No. 3013-RI

W. Fay P.E.
3/28/90

Stamp

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I) Methodology:

A) Floor slabs to be designed as one-way floor slabs using the "ACI-Ultimate Strength Design Methods"

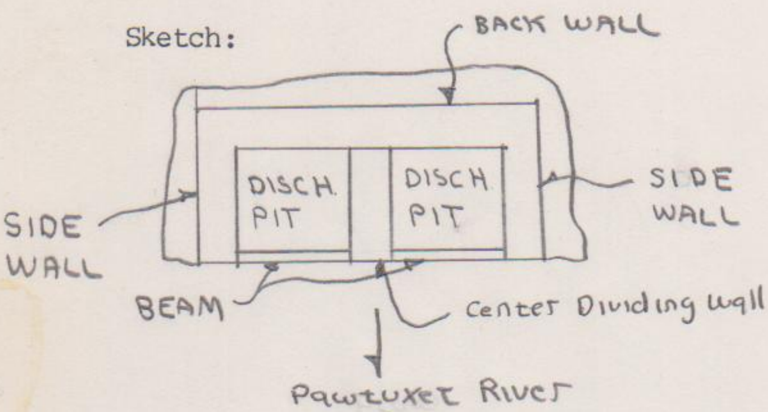
B) Walls to be designed as one-way floor slabs using the "ACI-Ultimate Strength Design Methods" and assuming hydrostatic load distributions for saturated soils.

C) Beams to be designed using ASI Code.

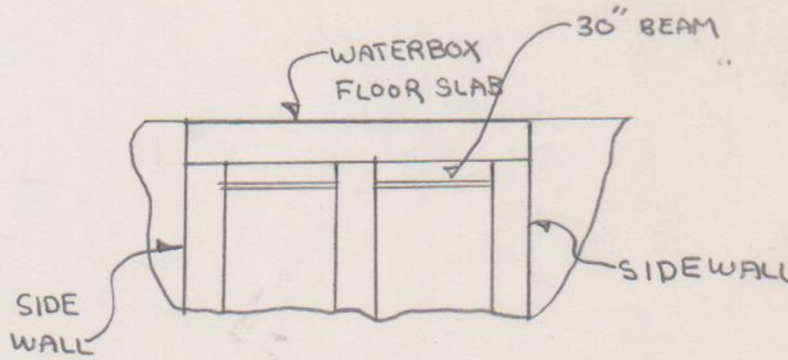
II) Design Approach:

A) Design assumes waterbox floor slab is supported on three sides by reinforced concrete walls and by a steel beam on the downstream, tailrace side. Additionally, the discharge pit will be subdivided into two equal sized compartments by a wall running in the streamwise direction and this reinforced wall will support the midspan of the waterbox floor.

Sketch:



PLAN VIEW



ELEV. VIEW

B) Waterbox floor slab loads will considst of uniformly distributed dead loads of water, rebar and concrete and the point loads of the turbine gate cases. The design will incorporate four (4) transverse beams, one on each side of the gate cases to support the gate case point load.

C) The discharge pit walls will be designed for external loads for when the pit may occasionally be dewatered by a coffer dam to maintain the draft tubes.

D) Waterbox walls will be designed for the internal pressure due to the headwater and for external pressure due to dewatering and emergency repair work during highwater.
E) Generator floor slabs will be designed for a uniform floor loading of 300 lb/sq.ft and for the point loads of the generator, turbine shaft, turbine runner and hydraulic thrust. This point load will be supported by transverse beams similiar to B) above. These beams will be designed according to ASI Code to have proper strength, with minimal deflection.

F) The upstream side of the generating floor will be supported by a steel beam spanning the waterbox inlets.

G) The waterbox will be divided into two (2) seperate chambers by a structural wall.

H) The main steel beam supporting the waterbox floor slab will be designed conservatively to support the weight of the waterbox rear wall, powerhouse rear wall and one half (1/2) the weights of the waterbox floor slab, the generating room floor slab, the turbine/generator hydraulic thrust point load and the overhead crane point load (assumed to be in the most compromising position).

I) The main steel beam supporting the generating room floor slab and spanning the waterbox inlets will be designed conservatively to support the weight of the generating room upstream wall and one half (1/2) the weights of the generating room floor slab, the turbine/generator hydraulic thrust point load and the overhead crane point load (assumed to be in the most compromising position).

J) The following elevations will be assumed:

****(Datum is USGS-MSL)****

1) Top of discharge pit floor slab - 21.0 msl

2) Mean tailwater elevation - 31.5 msl

3) Waterbox floor - 40.0 msl

4) Mean headwater elevation - 50.5 msl

5) Top of generating room floor slab - 54.0 msl

Assume main beams forming the discharge pit/tailrace and water box inlet arches are WF-30"x 124 #/ft.

III) <u>Design of the Waterbox Floor Slab:</u>

A) Dimensions- from Fay Engineering Services layout dated 12/27/89 entitled "Powerhouse Longitudinal Section". Assume these dimensions are correct.

1) External dimensions:

a) length = 40.Ø'

b) width = 18'

2) Internal dimensions:

a) length = 36.Ø'

b) width = 16.Ø'

B) Loading:

1) Water, total load = 62.4 #/ft3 * 16' * (38'* (5Ø.5-4Ø.Ø))

= 398,361 lbs

2) Water, load per foot width of box = 1Ø,483 lbs/ft-width of flume

3) Water, load per foot width of box per foot width of beam = 655 lb/ft

4) Concrete, total load = 15Ø #/ft3 * 16' * 38.Ø'*

(9"/(12"/ft)) = 68,4ØØ lbf

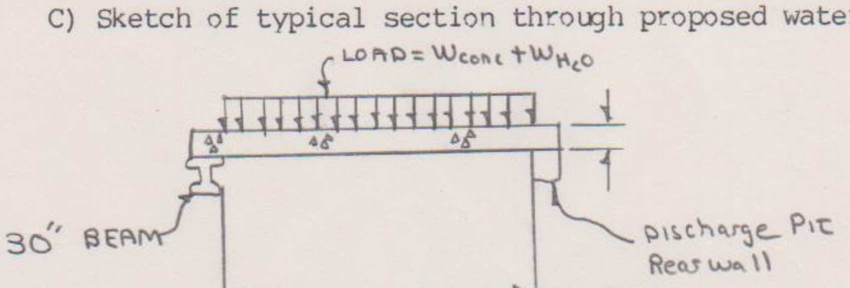
5) Concrete, load per foot width of box = 68,4ØØ lbf/38.Ø' = 18ØØ lbf/ft width of box

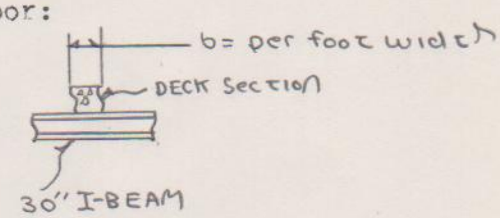
6) Concrete, load per foot width of box per foot width of beam =

18ØØ lbf/ft-width/16' = 113 lbf/ ft

7) Since the waterbox will be periodically dewatered, treat the water as a live distributed load and use the larger live load overload factor ACI ultimate strength multipliers.

C) Sketch of typical section through proposed waterbox floor:





D) Design Calculations: - reference, "Design of Concrete Structures", 9th ed., Winter & Nilson.

1) Assume the yield strength of the steel reinforcing is 30,000 psi and that the compressive strength of the concrete is 3000 psi.

2) Select the trial thickness of the slab, use L/20 from Table 5.1, p.206 in Winter & Nilson.

T= (12 "/ft * 18')/20 = 10.8" approximately = 11"

3) The slab weight is 150 #/ft3 * (11/12) = 138 PSF

4) Apply the ACI load multipliers and obtain the factored load:

Dead Load = 113 PSF * 1.4 = 158.2 PSF

Live Load = 655 PSF * 1.7 = 1113.5 PSF

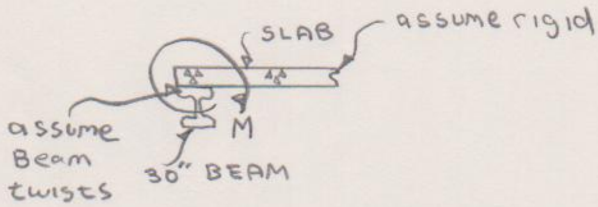
Total Factored Load = 1272 PSF

5) Use the ACI moment coefficients to determine the design moments at the critical sections:

a) Since the floor slab is being designed as a one-way slab in the short direction (ie: from the inlet end to the tailrace end), the slab will be resting on the main 30" beam which acts as the arch at the rear of the poerhouse over the tailrace and will be built into the top of the rear (upstream wall) of the discharge pit. At the tailrace end, the floor slab is simply supported and the beam is free to twist and cannot be assumed to be rigid, so use:

(1/11)*Wu*ln^2 >>>>>>>>from Table 8.1 W&N

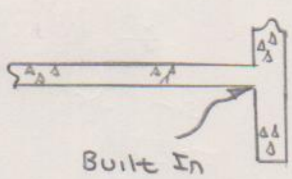
Sketch:



b) At the upstream end, the slab is to be built into the discharge pit wall and can be assumed to be rigid, so use:

(1/14)*Wu*ln^2>>>>>>>>from Table 8.1 W&N

Sketch:



c) At the interior span use:

(1/14)*Wu*ln^2>>>>>>>>from Table 8.1 W&N

d) At the tailrace: -M = 1/11*1.27 KSF *18'^2 = 37.4 ft-kips

e) At the upstream end: -M = 1/14*1.27 KSF *18'^2 = 29.4 ft-kips

d) At the midspan: -M = 1/16*1.27 KSF *18'^2 = 25.7 ft-kips

5) Determine the maximum steel ratio permitted by the ACI Code:

Pmax = 0.75*Pbalanced = 0.75*0.85*B1*(fc'/fy)*(87,000/(87,000+fc'))

this formula for fc'<4000 psi and B1=0.85

Pmax=0.75*0.85*0.85*(3000 psi/30,000 psi)*(87,000/(87,000 + 30,000))

= 0.04

6) Determine the minimum required effective depth: (This is controlled by the largest moment at the tailrace)

d^2 = Mu/(phi*p*fy*b*(1-(0.59*p*fy/fc)))   note: phi=0.9 for bending

= (37.4 ft-kips*(12"\ft))/(0.9*.04*30*12*(1-.59*0.04*(30,000/3000)))

= 45.3 in^2

Therefore, d = 6.7 inchs

7) Determine the minimum effective depth using code restrictions:

dm= 11"- 1" = 10"

8) Since the calculated value of 6.7 inches is less then the coded effective depth, use d= 10 inches

9) At the tailrace end, assume the stress block depth a = 1.00 inch. Then the area of steel required per foot width in the top of the slab is:

As= Mu/(phi*fy*(d-a/2))= (37.4 ft-kips*12"/ft)/0.9*30*(10-1/2)=

1.75 in^2

10) Check the assumed depth:

a= As*fy/(0.85*fc'*b)= 1.75 in^2 *30,000/(0.85*3000*12"/ft)= 1.72 in

11) The assumed area of steel and the calculated area of steel are reasonably close so use 1.72 in^2 of rebar per foot width of floor slab.

12) At the other critical sections use the same lever arm to determine the required cross sectional areas of steel rebar:

a) at the midspan: As= 25.7 ft-kips*12/(0.9*30*(11-1.72/2))= 1.13 in^2

b) at the upstream wall: As= 29.4 ft-kips*12/(0.9*30*(11-1.72/2))= 1.29 in^2

13) The minimum reinforcement required to control shrinkage is: see p. 207, W&N.

As= 0.002*12*11= 0.26 in^2/ 12" with strip

The required steel necessary for shrinkage is met by the steel required to meet the externally applied loads.

14) Determine the factored shear force:

Vu= 1.15 * (1272*18/2)-1272*(11/12)= 13,165-1166 = 11,999 lbs

15) The nominal shear strength of the slab is:

Vn= Vc= 2*b*d*fc'^0.5

= 2*12*11*(3000 psi^0.5)

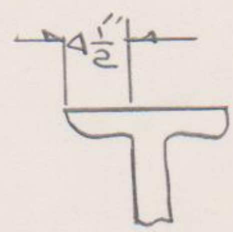
= 14,460 lbs

16) The design shear strength is:

phi*Vc= 0.85*14,460 lbs= 12,291 lbs

17) Since the design shear strength is above the required shear strength, no additional steel is necessary to resist the internal shear forces.

IV) <u>Design the Steel Main Support Beam at the Tailrace</u>



A) Data for proposed Beam:

1) height= 30"
weight= 124 lbs/ft-width
length= 55 feet
section modulus= 355 in^3
moment of inertia= 5360 in^4
depth= 30.16"
width= 10.521"
flange thickness= 0.93"
web= 0.585"

2) Flat of flange= (10.521"-0.585")/2= 4.97"
90 % of the flat is 4.5"

B) Determine the Beam Loading:

1) Point Loads:

a) generator= 225 rpm, 32 pole, 197 kva= 15,800 lbs

b) exciter= 1400 lbs

c) Shipping weight of turbine= 14,000 lbs

d) Hydraulic thrust= 21,636 #

2) Crane Load - assume a point load in the middle of the beam of 5 tons for a five ton bridge crane, this gives a 2.0 safety factor on the crane load.

3) Total point load is:

(15,800 # + 1400 # + 14,000 # + 21636 #)/2 + 5000 #= 31,418 lbs

4) Dead Loads:

a) Powerhouse backwall:

(40'*((69.0-54.0)*0.67) + 38*((53.0-40.0)*1.0))*150 lbs/ft^3= 134,400 #

b) Waterbox floor:

(18'*40'*1')*150 lbs/ft^3= 108,000 #

c) Generating room floor:

(18'*40'*1')*150 lbs/ft^3= 108,000 #

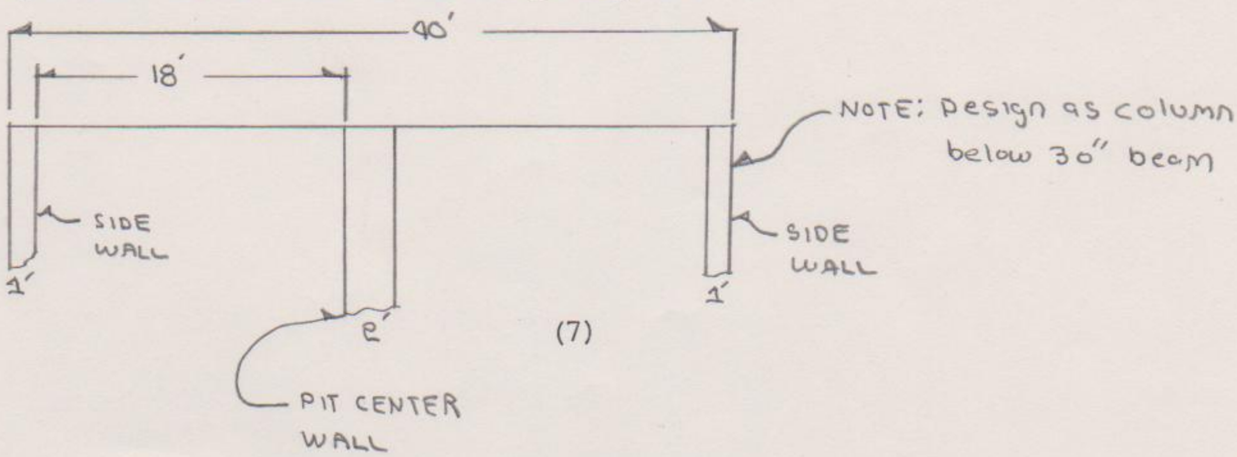
d) Water:

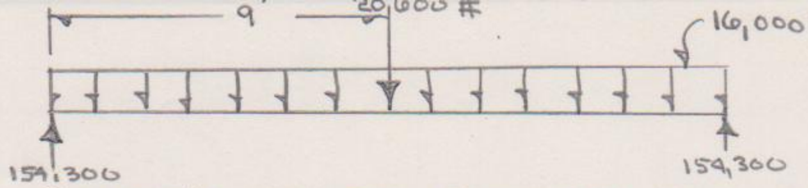
18'*34'*(50.5-31.5)*62.4 lbs/ft^3= 725,587 lbs

5) Total Distributed Load:

(134,400 # + 31,418 # + 216,000 #/2 + 725,587 #/2)/40'= 15,915 lbs/ft-width beam

6) Sketch the beam and the loading diagram:





C) Determine the extreme fiber stress and the maximum deflection:

1) Even though the beam will be embedded into the walls on either side, the weight of the wall immediately above the beam will not counter act the loading and one should assume that the beam is simply supported. This is a conservative assumption and will result in the largest moment.

2) Maximun Moment:

15,915 lb/ft * (18.Ø'^2)/8 = 644,569 ft-lbs

3) The extreme fiber stress is:

Sigma = M/Z = (644,569 ft-lbs * 12"/ft)/355 in^3 = 21,788 psi

4) From the AISC "Manual of Steel Construction", 7th edition, P. 5-124, section 1.5.1.4.1, the allowable bending stress for W shapes is Ø.66*Fy, where Fy is 36,ØØØ psi.

Therefore, the recommended maximum load is:

Sigma max= Ø.66*36,ØØØ psi= 23,76Ø psi <<<<<<<<*****

15) Since the calculated stress is less then the maximum recommended by AISC Code, the 3Ø" beam is alright for strength.

16) Determine the maximum deflection:

a) For uniformly distributed load with built in ends:

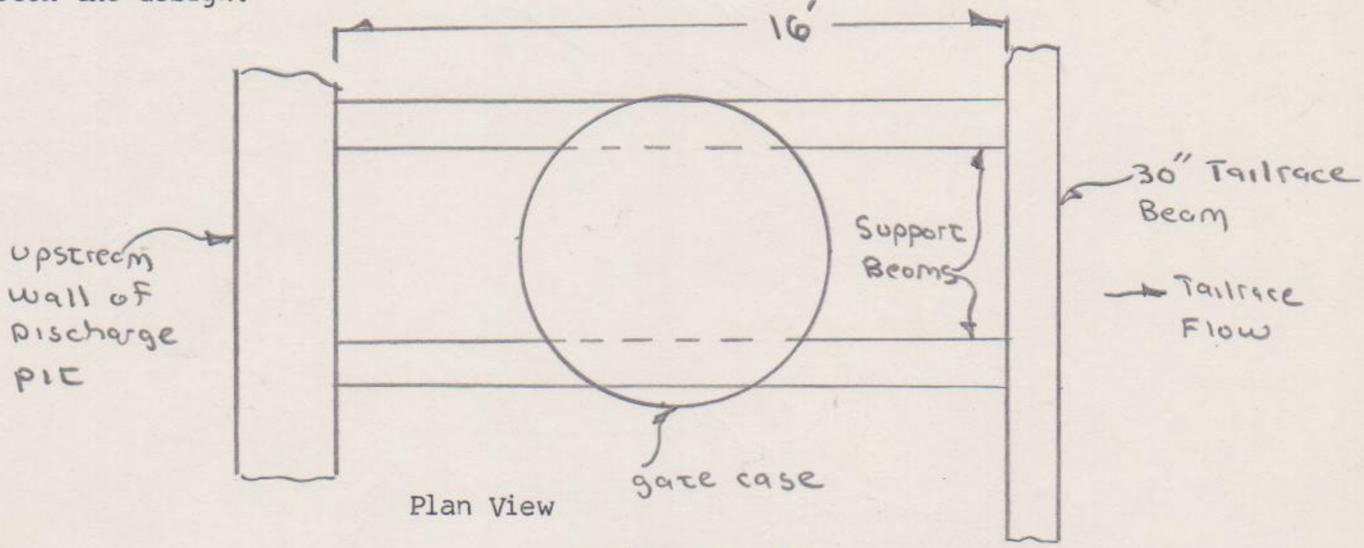
delta max= w*l^4/(384*E*I)

= 15,915 lb/ft * (18.Ø'*12")^4/(384*29,ØØØ,ØØØ psi* 536Ø in^4*12)= Ø.58 inch

17) The calculated deflection is negligible and should not effect the concrete resting upon the beam, especially since the major deflection will take place when the concrete is wet, before it dries.

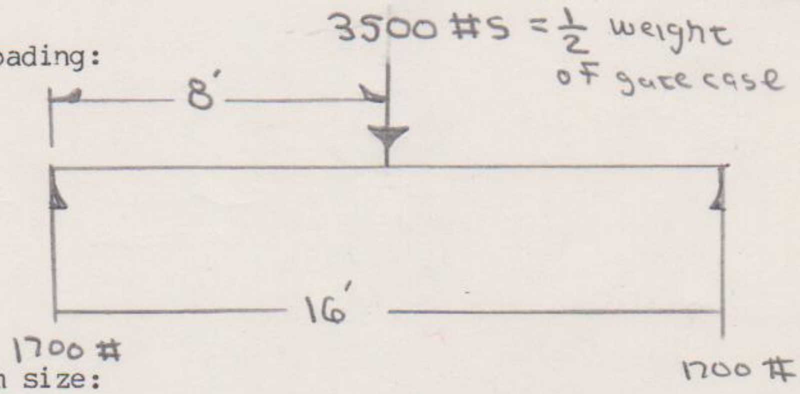
V) <u>Design the turbine gate case support beams</u>:

A) Sketch the design:



Plan View

B) Sketch the beam loading:



C) Determine the beam size:

1) Size the beam at 80 % of the AISC Codes maximum allowable stress or

0.8 * 21,600 psi= 17,280 psi

2) Maximum moment is 8' * 3500 # = 28,000 ft-lbs

3) Z= M/sigma= (28,000 ft-lbs * 12 in/ft)/17,280 psi= 19.5 in^3

D) From the AISC Handbook, 7th ed., P. 1-42, tentatively choose a W8"x24 lbs/ft

E) Beam Properties:

height= 7.93"
weight= 24 lbs/ft-width
length= 18 feet
section modulus= 20.8 in^3
moment of inertia= 82.5 in^4
depth= 7.93"
width= 6.50"
flange thickness= 0.398"
web= 0.245"

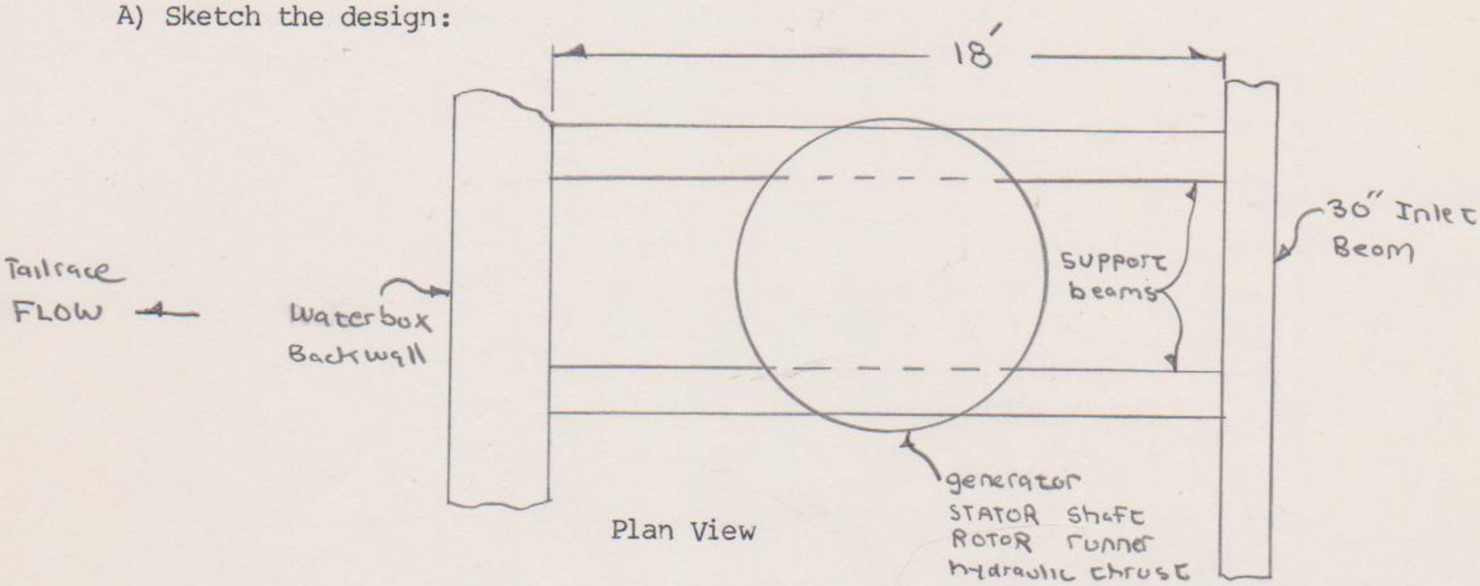
F) Check the deflection:

Sigma max= P*L^3/(48*E*I)= 3500# * (15.5'*(12"/ft)^3)/(48 * 29,000,000 psi * 82.5 in^3= 0.00006 inch <<<< okay

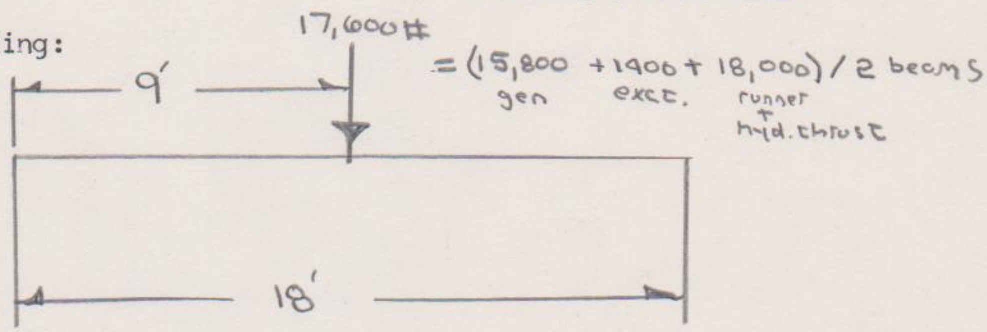
The choosen beam will meet the design requirements.

VI) Design the generator support beams:

A) Sketch the design:



B) Sketch the beam loading:



C) Determine the beam size:

1) Size the beam at 80 % of the AISC Codes maximum allowable stress or

0.8 * 21,600 psi= 17,280 psi

2) Maximum moment is 9.0' * (21,636 #+ 17,600 #)/2 = 176,562 ft-lbs

3) Z= M/sigma= (176,562 ft-lbs * 12 in/ft)/17,280 psi= 123 in^3

D) From the AISC Handbook, 7th ed., P. 1-42, tentatively choose a W12"x92 lbs/ft

E) Beam Properties:

height= 12.62"
weight= 92 lbs/ft-width
length= 18 feet
section modulus= 125 in^3
moment of inertia= 789 in^4
depth= 12.62"
width= 12.155"
flange thickness= 0.856"
web= 0.545"

F) Check the deflection:

Sigma max= P*L^3/(48*E*I)= 19618# * (18'*(12"/ft)^3)/(48 * 29,000,000 psi * 125 in^3= 1.1 inch <<<< okay

The choosen beam will meet the design requirements. The deflection is high, but in reality, the beams will be supported along their edges by the rigid generating room floor

VII) Design the generator floor:

A) Dimensions- - from Fay Engineering Services layout dated 12/27/89 entitled "Powerhouse Longitudinal Section". Assume these dimensions are correct.

1) External dimensions:

a) length = 40.0'

b) width = 18'

2) Internal dimensions:

a) length = 36.0'

b) width = 16.0'

B) Loading:

1) Concrete, total load = 150 #/ft3 * 16' * 36'*

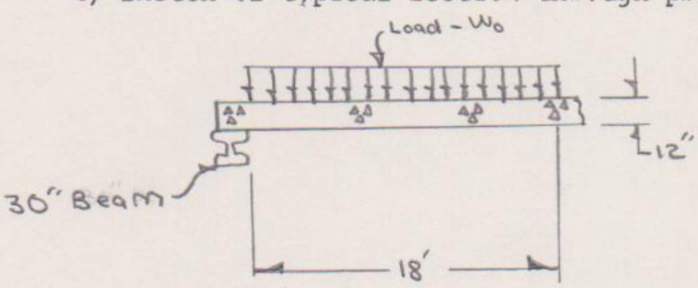
(12"/(12"/ft)) = 86,400 lbf

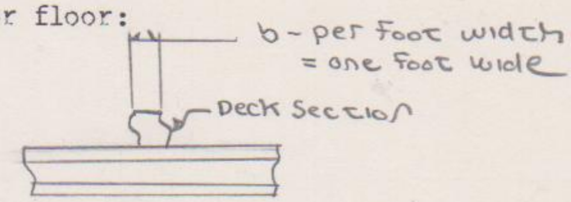
2) Concrete, load per foot width of box = 86,400 lbf/36' = 2400 lbf/ft width of box

3) Concrete, load per foot width of box per foot width of beam =

2400 lbf/ft-width/16' = 150 lbf/ ft

C) Sketch of typical section through proposed generator floor:





D) Design Calculations: - reference, "Design of Concrete Structures", 9th ed., Winter & Nilson.

1) Assume the yield strength of the steel reinforcing is 30,000 psi and that the compressive strength of the concrete is 3000 psi.

2) Select the trial thickness of the slab, use L/20 from Table 5.1, p.206 in Winter & Nilson.

T= (12 "/ft * 16')/20 = 9.6" approximately = 12"

3) The slab weight is 150 #/ft3 * (12/12) = 150 PSF

4) Apply the ACI load multipliers and obtain the factored load:

Dead Load = 150 PSF * 1.4 = 210.0 PSF

Live Load = 300 PSF * 1.7 = 510.0 PSF

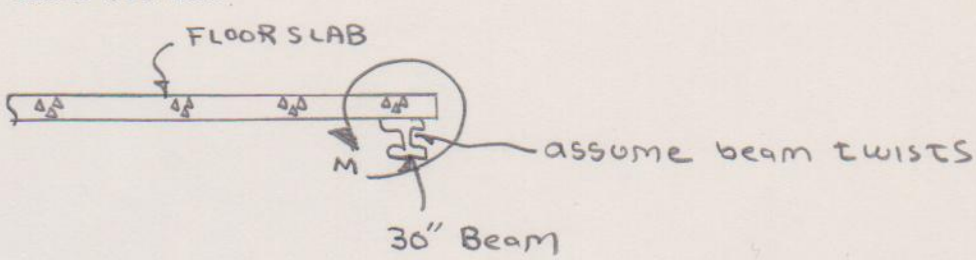
Total Factored Load = 720 PSF

5) Use the ACI moment coefficients to determine the design moments at the critical sections:

a) Since the floor slab is being designed as a one-way slab in the short direction (ie: from the inlet end to the tailrace end), the slab will be resting on the main 30" beam which acts as the arch at the front of the powerhouse, over the inlets and will be built into the top of the rear (downstream wall) of the water boxes. At the inlet end, the floor slab is simply supported and the beam is free to twist and cannot be assumed to be rigid, so use:

(1/11)*Wu*ln^2 >>>>>>>>from Table 8.1 W&N

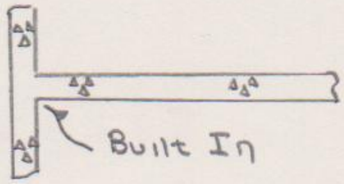
Sketch:



b) At the upstream end, the slab is to be built into the discharge pit wall and can be assumed to be rigid, so use:

(1/14)*Wu*ln^2>>>>>>>>from Table 8.1 W&N

Sketch:



c) At the interior span use:

(1/16)*Wu*ln^2>>>>>>>>from Table 8.1 W&N

d) At the inlets: -M = 1/11*0.720 KSF *14'^2 = 12.8 ft-kips

e) At the downstream end: -M = 1/14*0.720 KSF *14'^2 = 10.1 ft-kips

f) At the midspan: -M = 1/16*0.720 KSF *14'^2 = 8.8 ft-kips

5) Determine the maximum steel ratio permitted by the ACI Code:

Pmax = 0.75*Pbalanced = 0.75*0.85*B1*(fc'/fy)*(87,000/(87,000+fc'))

this formula for fc'<4000 psi and B1=0.85

Pmax=0.75*0.85*0.85*(3000 psi/30,000 psi)*(87,000/(87,000 + 30,000))

= 0.04

6) Determine the minimum required effective depth: (This is controlled by the largest moment at the inlet end)

d^2 = Mu/(phi*p*fy*b*(1-(0.59*p*fy/fc))) note: phi=0.9 for bending

= (12.8ft-kips*(12"\ft))/(0.9*0.04*30*12*(1-.59*0.04*(30,000/3000)))

= 15.51 in^2

Therefore, d = 3.9 inchs

7) Determine the minimum effective depth using code restrictions:

dm= 12"- 1" = 11"

8) Since the calculated value of 4.7 inches is less then the coded effective depth, use d= 11 inches

9) At the tailrace end, assume the stress block depth a = 1.00 inch. Then the area of steel required per foot width in the top of the slab is:

As= Mu/(phi*fy*(d-a/2))= (12.8 ft-kips*12"/ft)/.9*30*(11-1/2)= 0.54 in^2

10) Check the assumed depth:

a= As*fy/(0.85*fc'*b)= 0.54 in^2 *30,000/(0.85*3000*12"/ft)= 0.53 in

11) Reiterate assuming a=0.53 in:

As=Mu/(phi*f*(d-a/2))=(12.8 ft-kips*12"/ft)/0.9*30*(11-.53/2)= 0.52in^2

12) Reiterate assuming As=0.52 in^2:

a= As*fy/(0.85*fc'*b)= 0.52 in^2 *30,000/(0.85*3000*12"/ft)= 0.51 in

13) Reiterate assuming a=0.51 in:

As=Mu/(phi*f*(d-a/2))=(12.8 ft-kips*12"/ft)/0.9*30*(11-0.51/2)= 0.53 in^2

14) The assumed area of steel and the calculated area of steel are reasonably close so use 0.53 in^2 of rebar per foot width of floor slab.

15) At the other critical sections use the same lever arm to determine the required cross sectional areas of steel rebar:

a) at the midspan: As= 10.1 ksi*12/(0.9*30*(11-0.51/2))= 0.42 in^2

b) at the upstream wall: As= 8.8 ksi*12/(0.9*30*(11-0.51/2))= 0.36 in^2

16) The minimum reinforcement required to control shrinkage is: see p. 207, W&N.

As= 0.002*12*12= 0.288 in^2/ 12" with strip

The required steel necessary for shrinkage is met by the steel required to meet the externally applied loads.

17) Determine the factored shear force:

Vu= 1.15*(720*18/2)-720*(12/12)= 7452-720 = 6372 lbs

15) The nominal shear strength of the slab is:

Vn= Vc= 2*b*d*fc'^0.5

= 2*12*12*(3000 psi^0.5)

= 15,774 lbs

16) The design shear strength is:

phi*Vc= Ø.85*15,774 lbs= 13,4Ø8 lbs

17) Since the design shear strength is above the required shear strength by 15 %, no additional steel is necessary to resist the internal shear forces.

VIII Design Waterbox Walls:

A) Dimensions- - from Fay Engineering Services layout dated 12/27/89 entitled "Powerhouse Longitudinal Section". Assume these dimensions are correct.

1) External dimensions:

a) height= 14'

b) width = 18'

2) Internal dimensions:

a) length = 14'

b) width = 18'

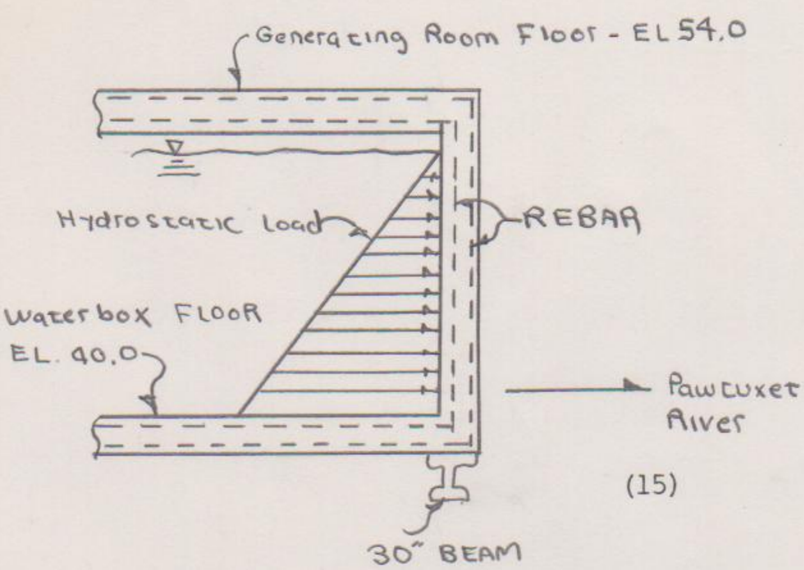
B) Loading:

1) Concrete, total load = 15Ø #/ft3 * 14' * 18'* (12"/(12"/ft)) = 37,8ØØ lbf

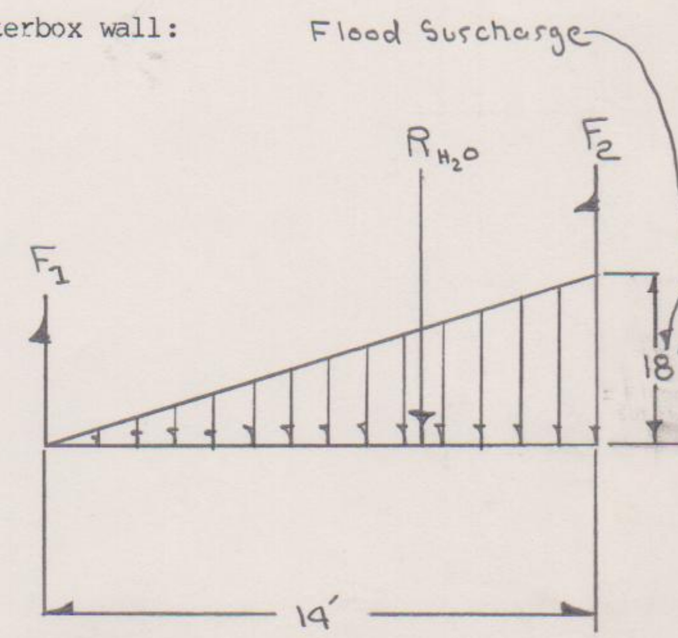
2) Concrete, load per sq. ft. = 37,8ØØ lbf/4Ø' = 15Ø lbf/ft^2

3) Water Load= 62.4 lbf/ft^3 * 14' wide * 18' deep * 1' thick/ 14'wide
= 1123 PSF. This is the maximum load at the base of the hydrostatic load.

C) Sketch of typical section through proposed waterbox wall:



Sketch



Loading Diagram

D) Design Calculations: - reference, "Design of Concrete Structures", 9th ed., Winter & Nilson.

1) Assume the yield strength of the steel reinforcing is 30,000 psi and that the compressive strength of the concrete is 3000 psi.

2) Select the trial thickness of the slab, use L/20 from Table 5.1, p.206 in Winter & Nilson.

T= (12 "/ft * 18')/20 = 10.8" approximately = 11"

3) The slab weight is 150 #/ft3 * (12/12) = 150 PSF

4) Apply the ACI load multipliers and obtain the factored load:

Dead Load = 150 PSF * 1.4 = 210 PSF

Live Load = 1123 PSF * 1.7 = 1910.0 PSF

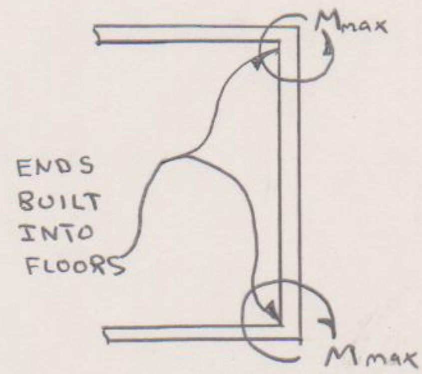
Total Factored Load = 2120 PSF

5) Use the ACI moment coefficients to determine the design moments at the critical sections:

a) The wall is rigidly built into the floor slabs at the top and bottom, so use:

(1/14)*Wu*ln^2 >>>>>>>>from Table 8.1 W&N

Sketch:



b) At the interior span use:

(1/16)*Wu*ln^2>>>>>>>>from Table 8.1 W&N

c) At the top: -M = 1/14*2.2 KSF *14'^2 = 30.8 ft-kips

d) At the bottom: -M = 1/14*2.2 KSF *14'^2 = 30.8 ft-kips

e) At the midspan: -M = 1/16*2.2 KSF *14'^2 = 27.0 ft-kips

5) Determine the maximum steel ratio permitted by the ACI Code:

Pmax = 0.75*Pbalanced = 0.75*0.85*B1*(fc'/fy)*(87,000/(87,000+fc'))

this formula for fc'<4000 psi and B1=0.85

Pmax=0.75*0.85*0.85*(3000 psi/30,000 psi)*(87,000/(87,000 + 30,000))

= 0.04

6) Determine the minimum required effective depth: (This is controlled by the largest moment at either the top or the bottom)

d^2 = Mu/(phi*p*fy*b*(1-(0.59*p*fy/fc))) note: phi=0.9 for bending

= (30.8ft-kips*(12"\ft))/(0.9*0.04*30*12*(1-.59*0.04*(30,000/3000)))

= 35.2 in^2

Therefore, d = 5.9 inchs

7) Determine the minimum effective depth using code restrictions:

dm= 12"- 1" = 11"

8) Since the calculated value of 5.9 inches is less then the coded effective depth, use d= 11 inches

9) At the tailrace end, assume the stress block depth a = 1.00 inch. Then the area of steel required per foot width in the top of the slab is:

As= Mu/(phi*fy*(d-a/2))= (30.8 ft-kips*12"/ft)/.9*30*(11-1/2)= 1.3 in^2

10) Check the assumed depth:

a= As*fy/(0.85*fc'*b)= 1.3 in^2 *30,000/(0.85*3000*12"/ft)= 1.27 in

11) Reiterate assuming a=1.27 in:

As=Mu/(phi*f*(d-a/2))=(30.8 ft-kips*12"/ft)/0.9*30*(12-1.27/2)= 1.2 in^2

12) Reiterate assuming As=1.20 in^2:

a= As*fy/(0.85*fc'*b)= 1.20 in^2 *30,000/(0.85*3000*12"/ft)= 1.18 in

13) Reiterate assuming a=1.18 in:

As=Mu/(phi*f*(d-a/2))=(30.9 ft-kips*12"/ft)/.9*30*(12-1.18/2)= 1.20in^2

14) The assumed area of steel and the calculated area of steel are reasonably close so use 1.20 in^2 of rebar per foot width of floor slab.

15) At the other critical sections use the same lever arm to determine the required cross sectional areas of steel rebar:

a) at the midspan: As= 30.8 ksi*12/(0.9*30*(12-1.18/2))= 1.20 in^2

b) at the bottom: As= 27 ksi*12/(0.9*30*(12-1.18/2))= 1.05 in^2

16) The minimum reinforcement required to control shrinkage is: see p. 207, W&N.

As= 0.002*12*12= 0.288 in^2/ 12" with strip

The required steel necessary for shrinkage is met by the steel required to meet the externally applied loads.

17) Determine the factored shear force: Note that the dead weight of the vertical concrete does not add to the shear component.

Vu=1.15*1910*14/2-2120*(12/12)= 15,375-2120 = 13,255 lbs

15) The nominal shear strength of the slab is:

Vn= Vc= 2*b*d*fc'^0.5

= 2*12*12*(3000 psi^0.5)

= 15,774 lbs

16) The design shear strength is:

phi*Vc= 0.85*15,774 lbs= 12,897 lbs

17) The design shear strength is slightly less then the required shear strength. However, the differential is small and no additional steel is necessary to resist the internal shear forces.

18) Use 1.2 in^2 of steel for the vertical reinforcement.

IX) <u>Design the discharge pit walls:</u>

A) Assume the pit is dewatered and drained externally. The height of water is 31.5-20.5=11'

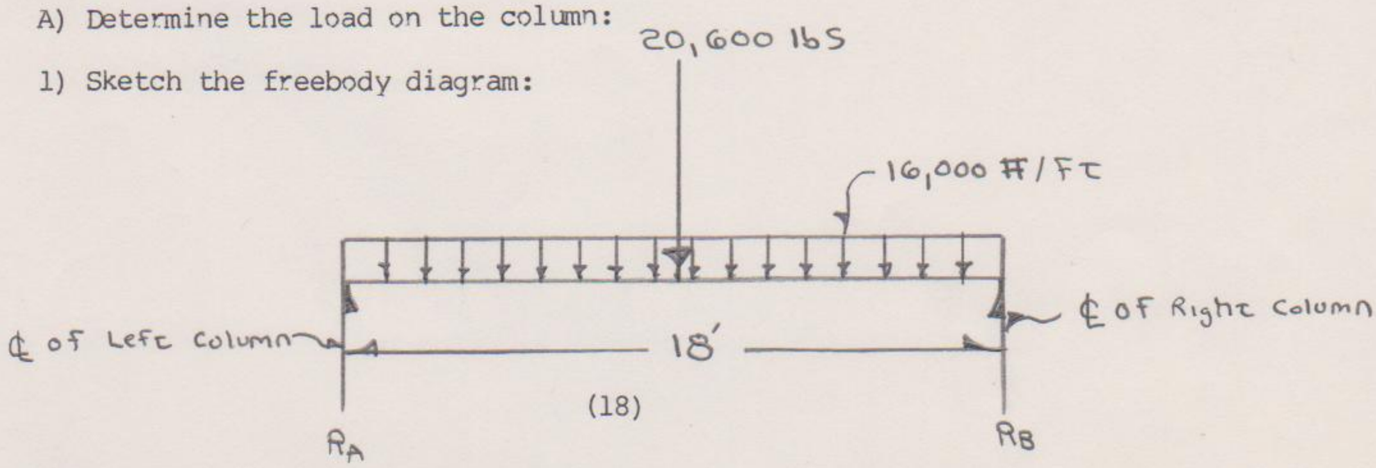
B) This design is similiar to the waterbox walls.

C) Use 1.2 in^2 of steel per foot width of wall.

X) <u>Design main 30" beam support column:</u>

A) Determine the load on the column:

1) Sketch the freebody diagram:



Ra=((20,600 lbs + (16,000*18'))/2 = 154,300 lbf

2) the factored ACI Code load is:

Pu=((1.4*(16,000 lbf * 18')/2) + (1.7 * 20,600/2)

= 201,600 lbf + 17,510 lbf = 219,110 lbf

B) Determine the nominal axial load strength of the column, Po, assuming minimal eccentricity:

1) Po= 0.85 * fc' * Aconc + fy * Ast

2) By ACI Code, the ratio of the longitudinal steel area to the gross column area must be:

0.01 <= Pg <= 0.08. Try 0.025 to start.

3) Assume the column is 12" square, built into the discharge pit walls and has a steel plate between the WF beam flange and the concrete to transmit the load.

4) The gross area of the column, Ag = 144 sq. in.

5) Areas of steel are:

Ast = Pg * Ag = 0.025 * 144 in. sq. = 3.6 in. sq.

6) Po = 144 in. sq. * (0.85*(1-0.025) * 3000 psi + 30,000 psi * 0.025

= 466,020 lbf

C) Determine the ACI factored design strength:

Pdesign = phi*Po = 0.80 * 0.70 * Po

= 0.8 * 0.7 * 466,020 lbf

= 260,971 lbf

D) Since the factored ACI design strength is greater then the factored ACI load, this design will work

phi * Po > Pu >>>>>>>>>>>> 260,971 lbf > 219,110 lbf

E) Determine the number of bars and their size:

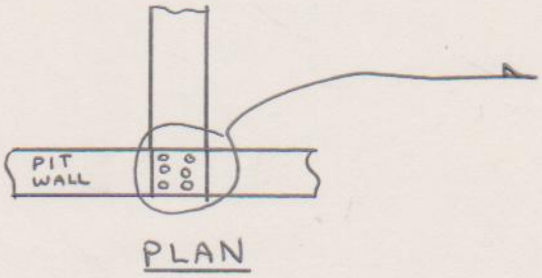
1) Assume six bar design

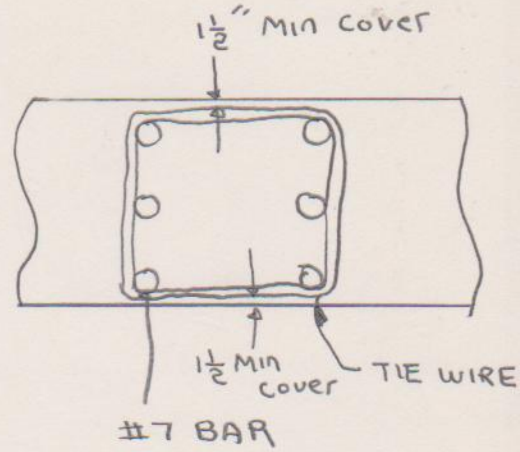
2) Area of single bar = Ast/# bars

= 3.6 in^2/6 = 0.60 sq. in.

3) From ASTM rebar, table #7 bars are 0.6 in. sq.

F) Sketch design:





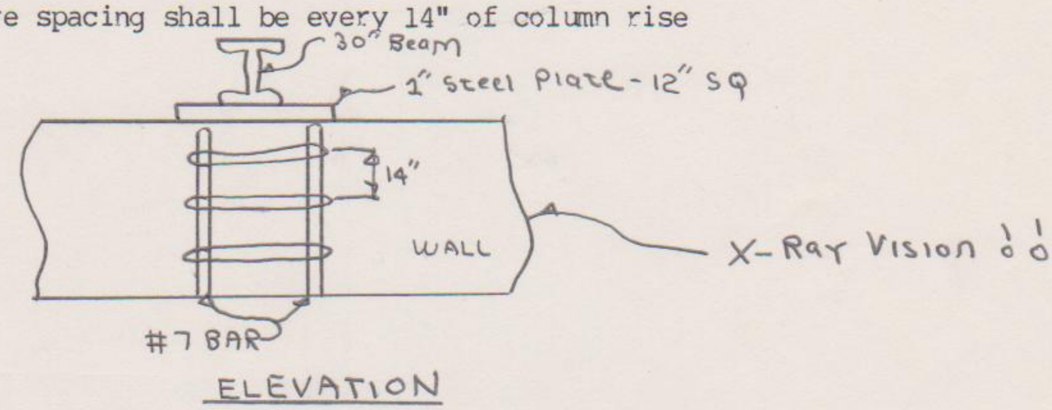
1) minimum spacing is 1.5 * 0.875 = 1.31"

2) minimum tie wire size is #4

3) concrete cover must be 1 1/2" thick

4) every corner to be supported by a tie wire

5) tie wire spacing shall be every 14" of column rise



G) Check the slenderness ratio of the column:

1) SR = K*Lu/r = K*Lu/0.3*W

K = 1 for unbraced columns
Lu = 39 - 21 = 18'
w = 12"

SR = 1 * 18'/(0.3*1') = 60

Since 60>22 this design is slender

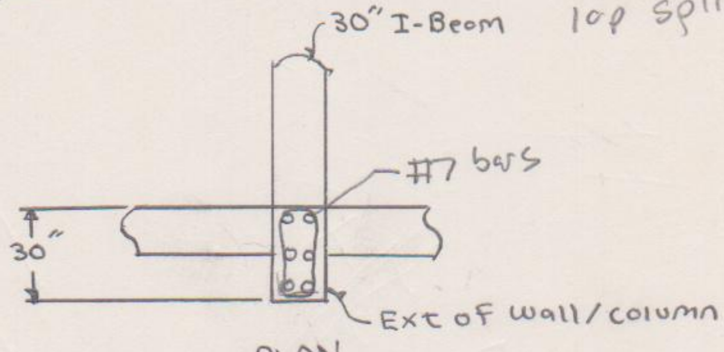
2) Determine w for SR= 22 minimum

W = KLu/(0.3*SR) = 18'/(0.3*22) = 2.73' = 33"

H) Retrofit design so that wall is 33" thick for the 12" length of the wall which the column is embedded into.

I) Sketch design:

10Ø splice lengths to be ?



PLAN

XI) Check the rock foundation bearing capacity:

A) Total weight of the powerhouse and equipment:

Water + 2 floors + machinery + back & front wall + side walls =

400,982 lbf + 100,000 lbf + 36,200 lbf + 432,000 lbf + 388,000 lbf =

1,357,182 lbf

B) Total surface bearing area is:

(40' + 2*18') * 1' = 76 sq. ft.

C) The stress on the rock is:

1,357,182 lbf/76 sq. ft. = 17,858 lbf/ft^2 = 8.92 tons/sq. ft.

D) "American Civil Engineer's Handbook", Merriman & Wiggin, 5th edition, P. 711, table lists the allowable soil pressures in short tons per sq. ft. for very hard native bedrock at 9 tons/sq.ft.. A short ton is 2000 lbs. Since the calculated pressure of 4.5 tons/sq.ft. is much less then 15 tons/sq.ft. This design should be all right if the walls are poured directly on the rock excavation. The footing should be chipped square and level before the forms are set up.

XII) Rebar size and spacing selection:

A) Waterbox Floor:

1) To obtain 1.72 in. sq./ft. width slab, use one #8 bar every six inches.

2) Use #6 bar at 18 inch spacing in the longitudinal spacing.

B) Generating Room Floor:

1) To obtain 0.53 in. sq./ft. width slab, use one #7 bar every 12 inches.

2) Use #5 bar at 18 inch spacing in the longitudinal spacing.

C) Walls:

1) To obtain 1.20 in. sq./ft. width slab, use one #7 bar every 6 inches.

2) Use #5 bar at 12 inch spacing in the longitudinal spacing.