Johnsonville Fossil Plant (JOF) Ash Disposal Area 2 Drawdown and Permanent Spillway Installation

Stantec

Design report detailing design methodology for pool drawdown and permanent spillway installation along the perimeter dike of Ash Pond C at Ash Disposal Area 2.

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1.0 Introduction

The purpose of this project is to allow TVA to abandon/permanently close the existing concrete pipe riser/outlet spillways in Pond C at Ash Disposal Area 2, and to allow the facility to operate at a lower than current pool level. This will eliminate the current tall, unsupported spillway structures that have questionable joint integrity. It will also allow an increased freeboard and reduction of seepage pressures on the dikes. All of these enhance the safety of the dikes.

2.0 Preliminary Design Summary

The current elevation in Pond C is approximately 387.5'. Pool drawdown will be accomplished by the installation of a temporary siphon outlet consisting of four (4) 18" nominal diameter HDPE pipes. These pipes will be installed over the existing dike and discharge into Kentucky Lake. They will be regulated using gate valves near the downstream end. The siphon outlet will be operated to draw down the elevation in the lake to approximately 378.0' over a period of no more than ten days. The siphons will then be used to maintain this elevation during construction of the new permanent spillway system and decommissioning of the current outlet structures.

The new permanent spillways will consist of six (6) precast concrete inlet boxes, in which removable fiberglass stoplogs will be used to control the normal pool elevation. These boxes will be placed on the upstream slope of the Pond C dike, and will discharge through six (6) 30" nominal diameter HDPE pipes. These pipes will extend through the dike and run along the downstream slope before discharging through an energy dissipater structure into Kentucky Lake. The permanent spillway structures were designed to maintain the permanent pool elevation at approximately 384.5' while passing the 6-hour PMP storm through the pond with over 1' of freeboard.

3.0 Final Design

3.1 TEMPORARY SIPHON DRAWDOWN

The initial step in the design process was to design the drawdown mechanism. A siphon spillway was selected because:

1. It is a simple design with relatively quick construction.

2. Pumps are only required for priming, siphon is driven by potential energy during operation so it is energy efficient.

3. It can be removed and reused once the permanent spillways are in place.

4. Only minor excavation is required on the dikes.

3.1.1 Criteria

The temporary siphon spillway structures were designed to convey the typical daily flow of 32 MGD from Ash Pond C to Kentucky Lake, as well as provide additional conveyance capacity to draw down the water level in Ash Pond C from 387.5’ to 378.0’ over a period of no more than ten days. Also considered in the design was the need to maintain a free water volume of 267,000 cubic yards (CY) in Ash Ponds A, B, and C of Ash Disposal Area 2.

3.1.2 Layout

The siphon outlet will be placed over the southwest dike of the ash pond approximately 250’ north of the northernmost existing spillway. This will allow it to pull water from Pond C. Since Pond C is the most downstream pond in the stilling area, most ash will have settled out, allowing water with the lowest TSS levels to be discharged into Kentucky Lake.

3.1.3 Design

The low water drawdown elevation in Pond C of 378.0’ was selected so that construction on the permanent spillways can occur without the need for cofferdams on the upstream face of the dike, and to provide immediate relief to seepage pressures on the exterior dikes. The drawdown amount is limited since TVA is required to maintain a free water volume of 267,000 CY in Ponds A, B, and C combined. A recent hydrographic survey of the ponds was provided by TVA. This information was used to check the free water volume at proposed drawdown elevations. Drawdown to elevation 378.0’ leaves approximately 244,000 CY of free water volume remaining in the ponds. Based on estimates from TVA, approximately 3,000 CY of free water volume is lost each month due to settled ash. If the water is drawn down for an assumed 8 months, this would result in a loss of 24,000 CY of free water volume. This leaves approximately 220,000 CY of free water volume, resulting in a deficit of 47,000 CY when compared to the required free water volume. TVA is currently performing dredging operations in the ponds that will result in removal of greater than 47,000 CY of material to provide the required 267,000 CY of free water volume.

The siphons will be constructed of high density polyethylene (HDPE) pipe. This material was chosen because a siphon must be airtight. HDPE pipe joints are fused to create an airtight seal. HDPE pipe is also flexible and can conform to existing ground contours, reducing the required number of joints when compared to other pipe materials. It is also relatively lightweight, allowing smaller equipment to be used during installation.

The siphon drawdown system at JOF will consist of four (4) 18" DR17 HDPE pipe runs. This configuration was selected based on several criteria, including cost, pool drawdown and maintenance ability, and resistance to pipe collapse.

The siphon system was designed to complete pool drawdown within ten (10) days of operation and maintain a normal base flow of 32 million gallons per day (MGD). Drawdown from the current elevation of 387.5' to 378.0' will require removal of approximately 280 acre-feet (ac-ft) of water in addition to the base flow. The drawdown will be gradual (over a period of 7-10 days) to allow pore pressures in the dikes to dissipate. However, depending on relative water surface elevations in the ash pond and Kentucky Lake, the four (4) 18" DR17 HDPE pipe runs may have additional capacity beyond the required baseflow and drawdown volume. Therefore, flows out of these pipes will need to be regulated using the gate valves near the downstream end.

The water surface elevation in the ash pond should be monitored during drawdown to verify that it is not being lowered by more than 0.5" per hour. A staff gage with alternating colored marks at 0.1' intervals and whole foot call-outs will be installed and calibrated. This gage will be used to measure the rate of drawdown and also when the pond elevation has dropped to 378.0' ±0.5'. In addition, flow velocities in the siphon system should be limited to approximately 15 feet per second to avoid flow separation in the lines. To regulate flow through the siphons, the valves on multiple pipe runs should be partially closed, if possible, as opposed to completely closing one pipe run. This will eliminate the need to reprime the run if additional conveyance is needed due to a large storm event or above average inflow. However, complete closure of one or more lines may be necessary depending on relative water surface elevations. Once initial drawdown is completed, the siphon valves should be adjusted and the water elevation in the ash pond should be observed to verify that the siphons are maintaining the pool level at 378.0' ± 0.5'. Maintenance of the pool will likely involve complete shutdown of one siphon run and regulating two of the runs by ½ closures of the gate valves.

A pipe collapse check was performed to verify that the wall thickness selected would be able to withstand the negative pressures over the dike crest during operation. Because a siphon flows up from the ash pond over the dike, negative pressures result. The worst case scenario occurs when the water in the ash pond is at its lowest drawdown level. Under the worst case scenario (Ash Pond C at elevation 378.0'), the maximum negative pressure in the pipe over the dike crest was approximately 10 psi. DR17 HDPE pipe is able to withstand a vacuum pressure of 14 psi for 1 year. Supporting calculations can be found in Appendix A.

The siphons will be installed over the existing dike. They will run through 24" steel casing over the dike crest and will be covered with aggregate to protect the pipes from traffic. The pipes will lay along the downslope where they will discharge into Kentucky Lake. The outlet of the pipe will be fitted with a 45 degree elbow so that discharge sprays up at the outlet and does not shoot directly on the slope below. The outlet elevation was selected so that it discharges above the summer pool easement elevation for Kentucky Lake. Downstream of the outlet, a 3.5' thick layer of Class C Machine Grade riprap will be placed to provide scour protection. On the ash pond side of the dike, the siphon pipe will lie along the bed of the pond and will be held in place with concrete anchors near the outlet. Keeping the pipe submerged will limit the amount of priming necessary at startup. Excavation will be necessary in the pond so that the pipe can stay submerged when the pond is drawn down to 378.0'. Currently, hydrographic surveys of the pond indicate ash levels are above the 378.0' elevation for a distance of approximately 100' from the dike. An area will need to be excavated for placement of the siphon pipes.

The upstream inlet of each siphon run will extend into the deeper part of the pond and be fitted with a torpedo strainer to prevent vortex formation as water is drawn into the siphon. In typical siphon installations, the inlet is kept near the bottom of the reservoir to prevent pulling of surface air into the siphon. In this application, however, it is equally important to not pull fly ash from the bottom of the pond into the siphon. Velocities in the siphon for the design conditions will be in the range of 15 feet per second; this would likely pull material off the bottom of the lake if the inlet was a simple pipe stub. The torpedo strainer consists of several drilled holes along the most upstream 30' of pipe. This serves to increase the inlet cross sectional area so that the maximum inlet velocity is limited to 1.5 feet per second. This will significantly reduce the potential for the siphon to pull fly ash off the pond bed. Supporting calculations for the torpedo strainers can be found in Appendix A.

3.1.4 Hydrologic / Hydraulic Considerations

Flow through each pipe run is primarily dependent on the relative water surface elevations in the ash pond and Kentucky Lake. The head differential between these two bodies of water is what drives flow through the siphon. With increased head differential, the flow rate through each pipe is increased. Therefore, siphon performance was evaluated under current and post-drawdown levels in the ash pond.

The Bernoulli equation was used to estimate flows through the siphon system. Head losses in the siphon system were represented by frictional losses along the pipe walls and minor losses due to valves, etc. The Darcy-Weisbach formula was used to predict frictional losses. Minor loss coefficients were included for entrance and exit losses, as well as minor losses due to pipe bends and the gate valves near the exit. The computations and assumptions included in computing flows through the siphon can be found in Appendix A. Also included in Appendix A is a summary of the various head and valve conditions under which the siphon may potentially be operated.

3.1.5 Operation

Operation of the siphon spillways will first require priming of the siphon. A Dri-Prime pump will be needed during this process. These pumps are fitted with a venturi mechanism that allows air to be sucked out of the siphon. The steps for priming of the siphon are:

1. Completely close gate valve at siphon outlet.

2. Connect hose to quick connect fitting on fill port and pump water (using Dri-Prime pump) from ash pond into downstream section of the siphon. Fill downstream section completely.

3. Once downstream section is filled, reverse direction of pump and suck air out of upstream end of pipe (this will simultaneously pull water from the ash pond into the upstream section of the siphon).

4. Once the Dri-Prime pump begins to discharge water instead of air, close the ball valve on the fill port. The entire siphon should now be filled with water.

5. Remove hose from quick connect at fill port.

6. Ensure no workers are present downstream of the outlet.

7. Open gate valve near outlet. This will start the siphon.

8. Regulate gate valves for drawdown and pool maintenance. During initial pool drawdown, monitor pool elevation hourly and adjust valves so that pool is not lowered more than 0.5” per hour. During pool maintenance phase, monitor water surface elevation and siphon performance daily to verify maintenance of water level at 378.0’ ± 0.5’. Based on initial estimates, this will likely entail complete closure of one run and half closure of the valves on two runs.

3.2 PERMANENT SPILLWAY

The new permanent spillway structures were designed to replace the current spillways which are tall and unsupported. The new spillway structures will consist of precast inlet boxes with stoplogs to control the water surface elevation in the pond. These will discharge through HDPE pipe into Kentucky Lake.

3.2.1 Criteria

The new permanent spillway structures were designed to pass the typical daily flow of 32 MGD while maintaining a normal pool elevation of approximately 384.5’. The design was also checked against the 6-hour Probable Maximum Precipitation (PMP) event. It is desired to pass this storm with 1 foot of freeboard on the perimeter dike.

3.2.2 Layout

The new permanent spillway structures will be located approximately 120 feet north of the northernmost existing spillway along the southwest dike. The spillway will consist of a series of inlet box structures with stoplogs at the upstream face. Stoplogs were utilized to allow for regulation of the water surface elevation in the ash pond. These boxes will transition to 30”

DR17 HDPE pipes through the dike, which will then run along the downstream face of the dike and discharge into a concrete energy dissipater located at Kentucky Lake. A 3.5' thick layer of grouted riprap will be placed downstream of the energy dissipater to provide scour resistance downstream of the structure.

3.2.3 Design

The permanent spillway structures will be precast concrete box structures located on the ash pond side of the perimeter dike. Six (6) inlet structures will be used at JOF. During normal flow conditions, these structures will have approximately 0.6' of head over the crest. Supporting calculations for this determination can be found in Appendix B. These structures will have a guide on the front in which 7' wide fiberglass stoplogs will be placed using a hand crane. The stoplogs were utilized in this application so that the water surface elevation in the ash pond could be regulated. By removing stoplogs, the water surface elevation in the pond could be drawn down to take pressure off of the perimeter dike if necessary. The top of the concrete inlet box will be fitted with steel grating to help prevent debris from clogging the structure during large storm events, while also allowing access to the hand crane to remove or place stoplogs. A 4' tall skimmer attachment constructed of corrugated metal will be installed on the face of the inlet box using steel angle to prevent cenospheres from being pulled over the stoplogs and into Kentucky Lake. The skimmer will span the entire width of the inlet structures and extend approximately 5' from the face. Computations are included in Appendix B detailing the factor of safety against overturning of the inlet box structures, as well as design calculations for the skimmer structure.

Flow will be conveyed through the dike through 30" DR17 HDPE pipe attached to the back of each inlet box. This pipe will extend through the dike and run along the downslope to an energy dissipater structure. HDPE pipe was selected due to its ease in handling for placement along the downslope. Each run will be fitted with three anti-seep collars to reduce seepage along the pipes through the dike. The energy dissipater structure will be a cast-in-place concrete stilling basin intended to create a hydraulic jump within the basin. This will serve to dissipate energy from the flow coming out of the downstream pipe sections (at a 45% slope). A 3.5' thick layer of grouted riprap will be placed downstream of the energy dissipater structure to provide scour resistance as flows transition out of the dissipater structure and into Kentucky Lake.

3.2.4 Hydrologic / Hydraulic Considerations

The concrete inlet box has potential hydraulic controls at (1) the stoplog crest, (2) the pipe inlet, and (3) the pipe section through the dike. It was desired to have control at the stoplogs during normal pool conditions with approximately 6" of head on the stoplog crest. A rating curve was developed for the inlet box using standard hydraulic equations to represent weir flow and inlet flow (including weir flow and orifice flow equations), and standard HDS-5 culvert nomographs (in HY-8) to represent flow through the discharge pipes. The equation for weir flow takes the form:

$$Q_w = C_w L_w (H - Z_w)^{3/2}$$

Where: C_w = weir discharge coefficient (2.7 to account for losses due to contraction); L_w = length of weir (ft); H-Z_w = head over weir crest (ft).

When the water surface elevation at the pipe inlet is below the crown of the pipe, the inlet behaves as a weir and is governed by the equation[1]:

$$Q_0 = C_w (\pi D)[H - E_0]^{3/2}$$

Where: Q_0 = outflow; C_w = weir coefficient; D = inlet diameter; H-E_0 = head over center of inlet.

When the inlet crown is submerged, orifice hydraulics govern in the form:

$$Q_0 = C_0 A_0 \sqrt{2g(H - E_0)}$$

Where: Q_0 = outflow; C_0 = orifice coefficient; g = gravitational acceleration; H-E_0 = head over center of inlet.

Culvert flows were computed using Federal Highway Administration software HY-8[2], which is based on Federal Highway Administration publication HDS-5[3]. The rating curve developed using this methodology is shown in Figure 1 alongside a rating curve developed in HEC-RAS. The HEC-RAS rating curve was slightly more conservative and thus used in routing the PMP storm.

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[1] Mays, Larry W. “Stormwater Collection Systems Design Handbook” McGraw Hill, 2001.
[2] Federal Highway Administration “HY-8 Culvert Hydraulic Analysis Program – Version 7.1” July 2008.
[3] Federal Highway Administration “Hydraulic Design of Highway Culverts, Second Edition” HDS No. 5, September 2001, 376 p.

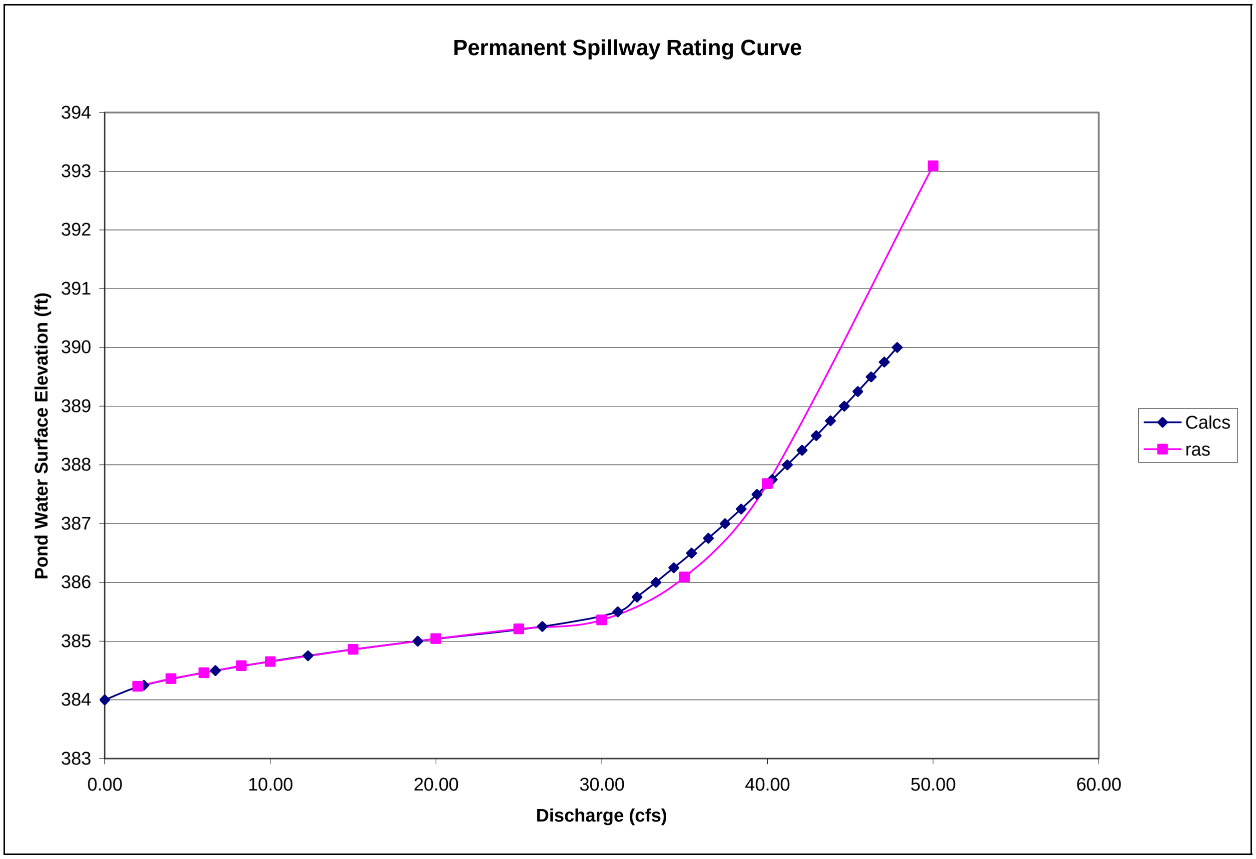


Figure 1 - Permanent Spillway Rating Curve

As can be seen from both curves, weir flow controls until an elevation of 385.5' (corresponding to a discharge of approximately 34 cfs), at which time pipe flow begins to control. The results of this analysis showed that six (6) inlet boxes were required to convey the 32 MGD over the stoplogs with roughly 6" of head on the stoplogs. This flow would be carried through each pipe segment under open-channel conditions. Both the section of pipe through the dike and the section along the downstream slope will flow under supercritical conditions due to the steep slopes of each section. Flow velocities approach 20 feet per second just upstream of the pipe outlet to the energy dissipater structure, where a hydraulic jump forms. A profile of normal flow through one inlet box system is shown in Figure 2.

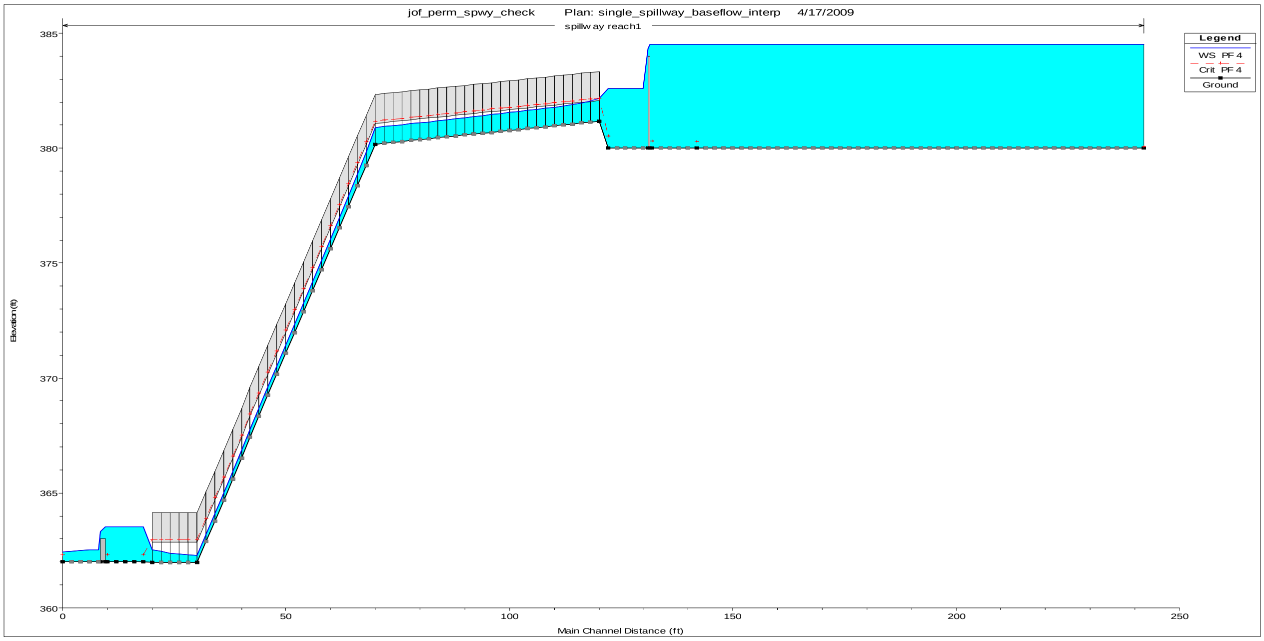


Figure 2 - HEC-RAS Profile Plot (8.25 cfs)

The permanent spillway was also evaluated relative to performance during the 6-hour PMP storm event to determine the maximum normal pool elevation and hence the stoplog crest elevation. A hydrologic model was assembled to compute runoff from this storm event. The stoplog crest elevation was adjusted in iterative model runs to determine the maximum elevation of the stoplog crest while still passing the 6-hour PMP storm with available freeboard on the perimeter dike. Maintaining the pool at the highest potential elevation yields the largest free water volume, thus allowing the maximum amount of ash to be stored in the pond.

The 6-hour PMP rainfall for Humphreys County, Tennessee is approximately 35 inches[4]. The total drainage area to the spillway structure is approximately 87 acres. This includes the majority of Ash Disposal Area 2, including Ash Ponds A, B, and C and the interior slopes. The SCS distribution was used to convert this rainfall into runoff. A composite curve number of 99 was assumed, since most rainfall will be falling into the ponds and be converted to runoff. The lag time of the watershed was assumed to be a conservative 10 minutes. Stage-storage relationships for the interior ponds (Ponds A, B, and C and Cell Ponds) were taken from free water volume data provided by TVA (see Appendix B). The rating curve developed from HEC-RAS for the inlet structure and pipe system was incorporated into a HEC-HMS model and the PMP storm routed through the spillway. Iterative models runs showed that with a normal pool elevation of 384.5’, the spillway structures were able to pass the PMP storm with over 1’ of

[4] Zurndorfer, E.A., Schwarz, F.K., and Hansen, E.M. “Probable Maximum and TVA Precipitation Estimates With Areal Distribution for Tennessee River Drainages Less than 3000 Mi^2 in Area” Hydrometeorological Report No. 56, Office of Hydrology, National Weather Service, October 1986.

freeboard on the perimeter dikes. The inflow and outflow hydrographs for a stoplog crest elevation of 384.0’ are shown in Figure 3.

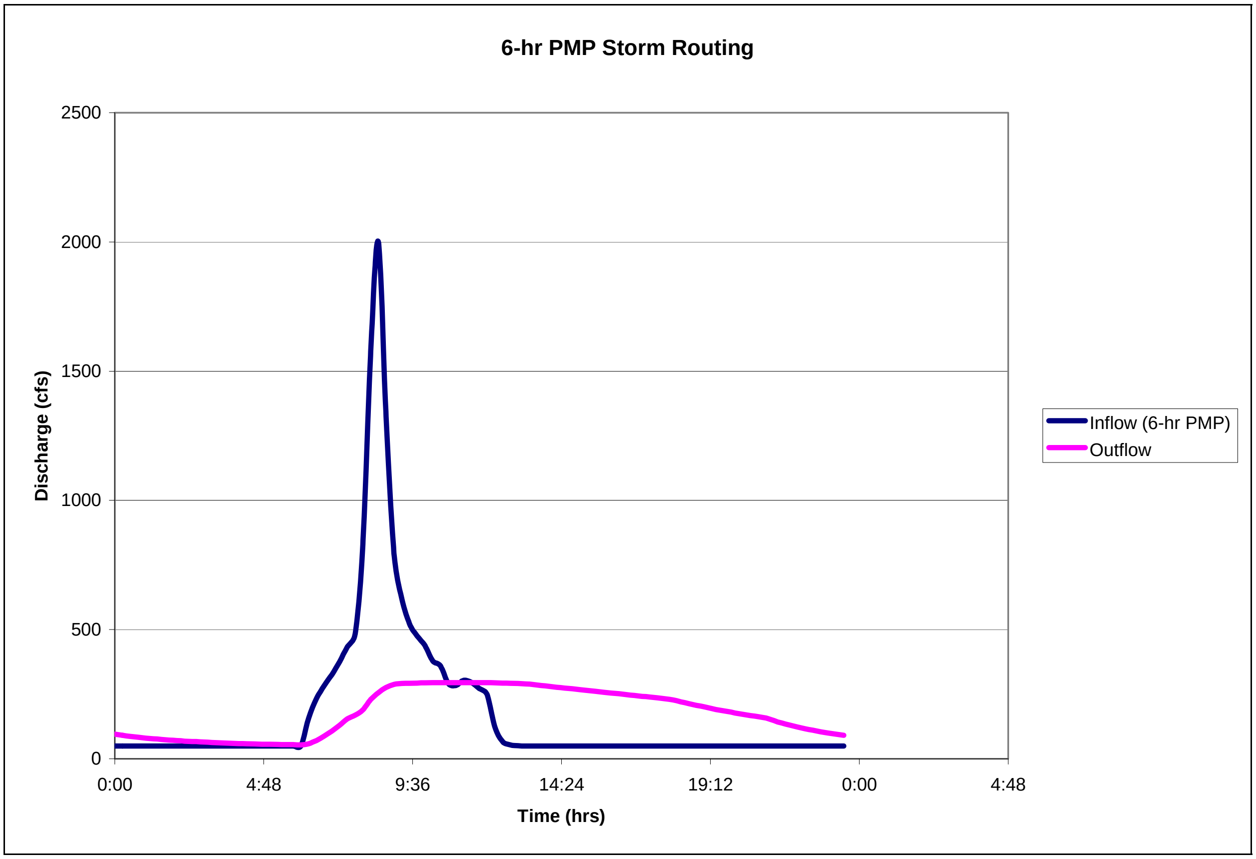


Figure 3 - 6-hr PMP Inflow and Outflow Hydrographs

With a stoplog crest elevation of 384.0’, the water surface in the pond reached a maximum elevation of 388.7’. The crest elevation of the perimeter dike is approximately 390.0’. Therefore, the 1 foot freeboard objective is satisfied. The peak discharge from the combined six structures during this event is estimated at approximately 280 cfs.

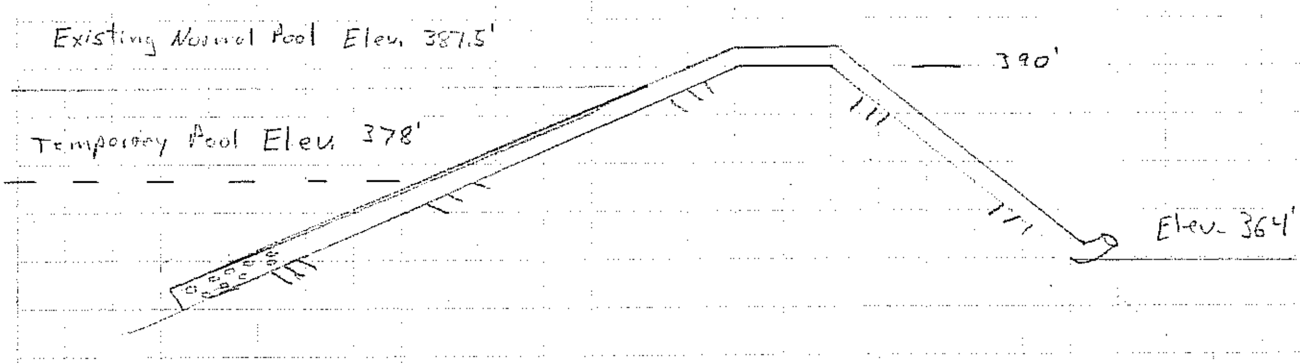
3.2.5 Operation

Operation of the permanent spillway structures will involve placing the stoplogs in the stoplog guide using a hand crane attached to the top of the inlet structures. The crest elevation of the top stoplog should be set at approximately 384.0’.

Appendix A: Temporary Siphon Supporting Calculations

Design Temporary Siphon Spillway to lower stilling pond elevation.

- The existing stilling pond elevation needs to be lowered in order to construct a new permanent spillway at the active ash pond of the TVA Johnsonville Fossil Plant. This will be accomplished by a temporary siphon spillway that will be used to lower the pond elevation to a temporary level and then to maintain that level until the new permanent spillway can be constructed.



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Determine Drawdown Time

- To drain the stilling ponds from elevation 387.5' to 378', approximately 280 ac-ft of water will need to be removed, plus the inflow to the pond of 32 MGD (49.5 cfs) will need to be maintained. It is desirable to drawdown the pond at a rate of one foot per day.
- Assume that four (4) runs of 18" DR 17 HDPE pipe will be used.

Inside Diameter = 15.755" = 1.31' ✓
Length of one run = 270'

- Using the mid-point (elev. 382.75') elevation to determine an average flow during drawdown, the flow through one pipe can be calculated using Bernoulli's Equation.

$$z_1 + \frac{p_1}{\gamma} + \frac{v_1^2}{2g} = z_2 + \frac{p_2}{\gamma} + \frac{v_2^2}{2g} + h_L$$ ✓

If points are taken on the pond + lake surfaces:
$p_1 = p_2 = 0$ (atmospheric conditions) ✓
$v_1 = v_2 = 0$ (Since surface area lake/pond >> area of pipe)

$$z_1 = z_2 + h_L$$ ✓

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- Using Darcy-Weisbach to determine h_L

$$h_L = \frac{v^2}{2g}\left[\frac{fL}{D} + \sum k_m\right]$$

Minor Loses

Fitting	k_m
Entrance	0.8
Joints	0.4
Priming Connection	0.2
Gate Valve (Fully Open)	0.2
Exit	1.0
	$\sum k_m = 2.6$ ✓

Assume initial friction factor, $f = 0.013$

$$v = \sqrt{\frac{(z_1 - z_2)2g}{\frac{fL}{D} + \sum k_m}}$$ ✓ $g = 32.2 \frac{ft}{s^2}$

$$v = \sqrt{\frac{(382.75 - 364)(2)(32.2)}{\left[\frac{(0.013)(270)}{(1.31)} + 2.6\right]}}$$ ✓

$v = \underline{15.12 \frac{ft}{s}}$ ✓

Check assumption of $f = 0.013$

kinematic viscosity of water @ 50°F = $1.41 \times 10^{-5} \frac{ft^2}{s}$

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- Reynolds Number, $R = \frac{Dv}{\nu} = \frac{(1.31)(15.12)}{1.41\times10^{-5}} = 1.41\times10^{6}$

- Roughess factor for HDPE, $\varepsilon = 7\times10^{-5}$ ft

  $\varepsilon/D = \frac{7\times10^{-5}}{1.31} = 5.3\times10^{-5}$

- From Moody Diagram $f = 0.0127 \approx 0.013$ OK ✓

- In order to reduce the possibility of water separation in the downstream section of the siphon, the maximum velocity allowed will be 15 ft/s. Therefore the draw down time will be based on 15 ft/s.

- Flow through one siphon, $Q_1 = vA = 15[\pi(\frac{1.31}{2})^2]$
  $= 20.2$ cfs ✓

- Flow through four, $Q_{Total} = 20.2(4)$
  $= 80.8$ cfs ✓

- Subtract out the inflow to the pond
  $Q_{drawdown} = 80.8 - 49.5 = 31.3$ cfs ≈ 61.5 $\frac{ac\text{-}ft}{day}$ ✓

- To drain 280 ac-ft of water

  Time to Drain $= \frac{280}{61.5} = 4.6 \approx 5$ days < 10 days OK ✓

The four (4) 18" DR17 HDPE pipes will draw down the stilling pond at a rate of at least one foot per day.

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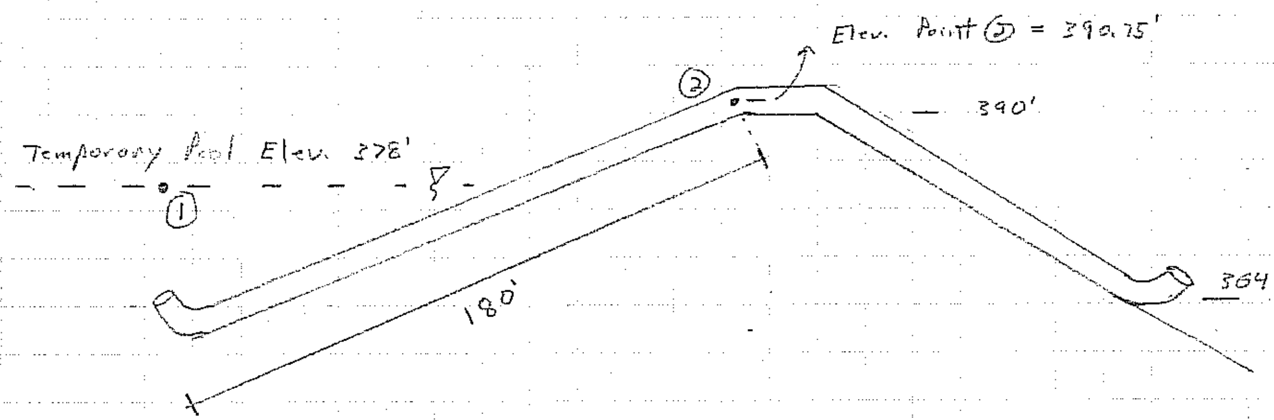
Check Pipe Collapse



- Use Bernoulli's to determine negative pressure at worst case scenario, with pond elevation at its lowest (378') and the crest of siphon.

$$z_1 + \frac{P_1}{\gamma} + \frac{v_1^2}{2g} = z_2 + \frac{P_2}{\gamma} + \frac{v_2^2}{2g} + h_L$$

- At Pond elev. = 378', $v_2 = 13.1$ ft/s ✓
- Assume atmospheric pressure at Point 1, $P_1 = 0$ ✓
- Assume $v_1 = 0$ ✓

$$h_L = \frac{v_2^2}{2g}\left[\frac{fL}{D} + \Sigma K_m\right] = \frac{(13.1)^2}{2(32.2)}\left[\frac{(0.013)(180)}{1.31} + (0.8 + 0.1)\right]$$ ✓

$$= 7.16'$$

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$$P_2 = \gamma\left[(z_1 - z_2) - \frac{v_2^2}{2g} - h_L\right]$$

$$P_2 = 62.4\,\tfrac{lb}{ft^3}\left[(378 - 390.75') - \frac{(13.1)^2}{2(32.2)} - 7.16'\right] \checkmark$$

$$P_2 = -1408.6\,\tfrac{lb}{ft^2} = -9.78\ psi \checkmark$$

- A DR 17 pipe can sustain a vacuum pressure of 14 psi for 1 year. Siphon will be in operation for less than 1 year, so DR 17 is <u>OK</u>. ✓

<u>Maintain Temporary Elevation</u>

- It is desired to only close a gate valve at most half-closed (other than full closure) to prevent water hammer issues. Minor loss coefficient for a half-closed gate valve is $K_m = 2.1$. Known inflow is 49.5 cfs.

- At the temporary pond elevation of 378', four (4) fully open siphons will discharge

$$Q_{Total} = (13.1)(1.35)(4) = 70.6\ cfs. \checkmark$$

- By completely shutting down one siphon + throttling two down to half-closed, new flow will be:

$$v_{1/2} = \sqrt{\frac{(378-364)(2)(32.2)}{\frac{0.013(270)}{1.31} + (2.4 + 2.1)}} = 11.2 \text{ ft/s}$$

$$Q_{1/2} = (11.2)(1.35) = 15.1 \text{ cfs}$$

$$Q = 2(15.1) + 17.6 = 47.8 \text{ cfs} \approx 49.5 \text{ cfs} \quad \underline{\text{OK}}$$

To maintain the temporary pond elevation of 378', completely shut down one siphon and throttle two down to half-closed. This will provide an outflow of 47.8 cfs which will counter the inflow of 49.5 cfs within tolerance of ±0.5'.

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Design Torpedo Strainer Inlet

It is critical to keep the siphon inlet deep in the stilling pond to ensure that a vortex will not form. However, the closer the inlet is to the bottom of the pond the more likely it is to suck up ash and discharge it into the Kentucky Lake. Therefore a torpedo strainer inlet will be used to reduce the inlet velocities thereby reducing the tendency of the siphon to suck ash from the pond bottom.

- Max. Allowable Dischage in one Pipe is 20.2 cfs based on max. velocity of 15 ft/s.
- Diameter of inlet orifice will be 4"
- Max. velocity through each orifice will be limited to 1.5 ft/s ✓

Q for one hole $= (1.5 \frac{ft}{s}) \pi (\frac{0.33}{2})^2 = 0.131$ cfs ✓

Number of holes required, $N_H = \frac{20.2}{0.131} = 154.3 \approx 155$ ✓

- 4 rows parallel to the length of the pipe across the top half only with 8" seperation center to center and staggered 4".
- Number of holes per row, $N_{H,R} = \frac{155}{4} = 38.8 \approx 39$ ✓

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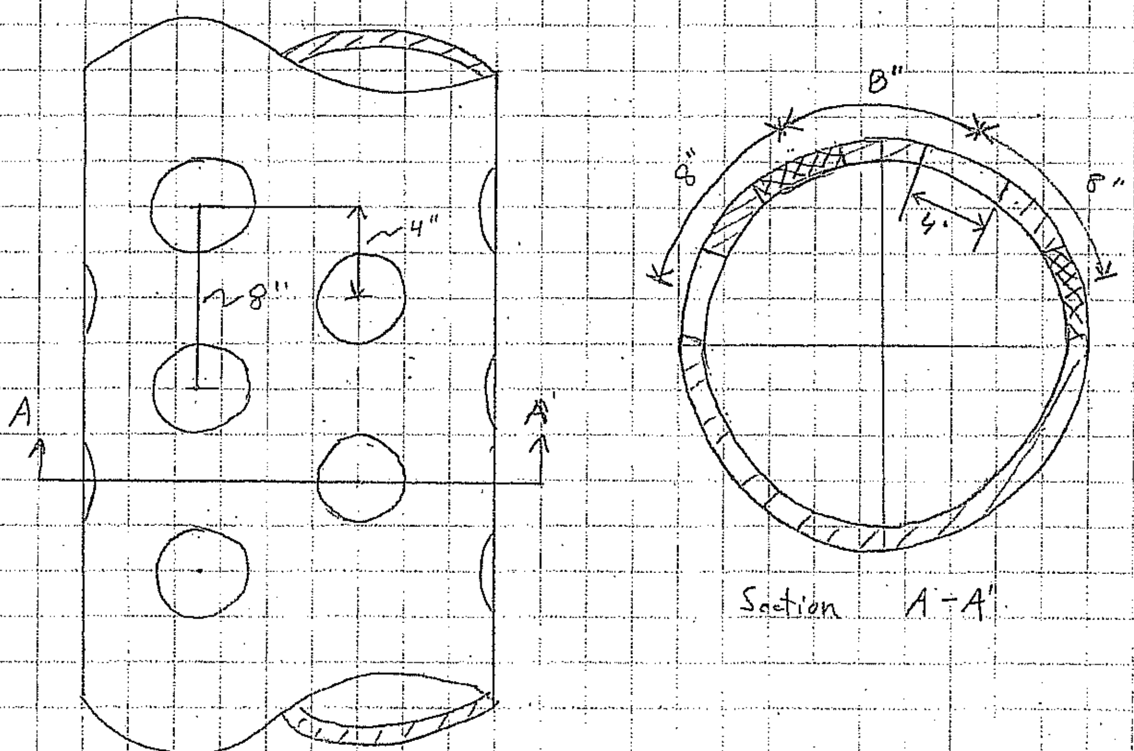
- Using 8" center to center spacing for holes parallel to length of pipe, length of pipe required,

$L = 39\,(0.67') = 26.1'$ ✓

Add 2' to each end, $L_T \approx 30'$ ✓



Section A - A'

Space holes as shown. Each row is to have a minimum of 39 holes each.

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JOF Siphon Rating
18" DR17 HDPE
4 Runs Total

Losses

Fitting	k_M	
Entrance	0.80	
Bend 1	0.40	
Bend 2	0.00	
Bend 3	0.00	
Bend 4	0.00	
Tee	0.20	
Gate Valve	0.15	Fully Open
Exit	1.00	
Total	2.55	

Siphon Run 1 - Gate Valve Fully Open

Outlet Elevation (ft)	Stilling Pond Elevation (ft)	Δ Elevation (ft)	Velocity (ft/s)	HL (ft)	Discharge (cfs)
364	378	14	13.1	14	17.8
364	379	15	13.6	15	18.4
364	380	16	14.0	16	19.0
364	381	17	14.5	17	19.6
364	382	18	14.9	18	20.2
364	383	19	15.3	19	20.7
364	384	20	15.7	20	21.3
364	385	21	16.1	21	21.8
364	386	22	16.5	22	22.3
364	387	23	16.8	23	22.8
364	387.5	23.5	17.0	23.5	23.0

Total Flow - 4 Runs (Valves Fully Open)

Outlet Elevation (ft)	Pond Elevation	Discharge (cfs)
364	378	71.1
364	379	73.6
364	380	76.1
364	381	78.4
364	382	80.7
364	383	82.9
364	384	85.0
364	385	87.1
364	386	89.2
364	387	91.2
364	387.5	92.2

Fitting	k_M	
Entrance	0.80	
Bend 1	0.40	
Bend 2	0.00	
Bend 3	0.00	
Bend 4	0.00	
Tee	0.20	
Gate Valve	0.26	3/4 Open
Exit	1.00	
Total	2.66	

Siphon Run 1 - Gate Valve 3/4 Open

Outlet Elevation (ft)	Stilling Pond Elevation (ft)	Δ Elevation (ft)	Velocity (ft/s)	HL (ft)	Discharge (cfs)
364	378	14	13.0	14	17.6
364	379	15	13.5	15	18.2
364	380	16	13.9	16	18.8
364	381	17	14.3	17	19.4
364	382	18	14.7	18	20.0
364	383	19	15.1	19	20.5
364	384	20	15.5	20	21.0
364	385	21	15.9	21	21.6
364	386	22	16.3	22	22.1
364	387	23	16.7	23	22.6
364	387.5	23.5	16.8	23.5	22.8

Notes:

32 MGD = 49.5 cfs

Gate Valve Kms

0.15	Fully Open
0.26	3/4 Open
2.1	1/2 Open
17	1/4 Open

Fitting	k_M	
Entrance	0.80	
Bend 1	0.40	
Bend 2	0.00	
Bend 3	0.00	
Bend 4	0.00	
Tee	0.20	
Gate Valve	2.10	1/2 Open
Exit	1.00	
Total	4.50	

Siphon Run 1 - Gate Valve 1/2 Open

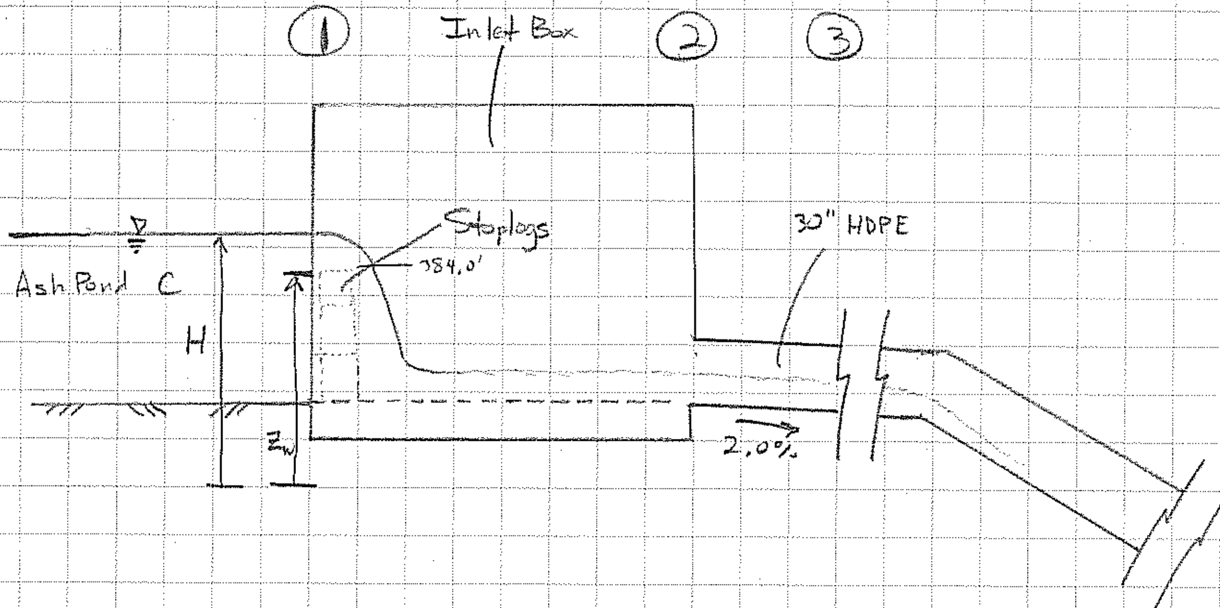
Outlet Elevation (ft)	Stilling Pond Elevation (ft)	Δ Elevation (ft)	Velocity (ft/s)	HL (ft)	Discharge (cfs)
364	378	14	11.2	14	15.2
364	379	15	11.6	15	15.7
364	380	16	12.0	16	16.2
364	381	17	12.4	17	16.7
364	382	18	12.7	18	17.2
364	383	19	13.1	19	17.7
364	384	20	13.4	20	18.1
364	385	21	13.7	21	18.6
364	386	22	14.1	22	19.0
364	387	23	14.4	23	19.5
364	387.5	23.5	14.5	23.5	19.7

Fitting	k_M	
Entrance	0.80	
Bend 1	0.40	
Bend 2	0.00	
Bend 3	0.00	
Bend 4	0.00	
Tee	0.20	
Gate Valve	17.00	1/4 Open
Exit	1.00	
Total	19.40	

Siphon Run 1 - Gate Valve 1/4 Open

Outlet Elevation (ft)	Stilling Pond Elevation (ft)	Δ Elevation (ft)	Velocity (ft/s)	HL (ft)	Discharge (cfs)
364	378	14	6.4	14	8.7
364	379	15	6.6	15	9.0
364	380	16	6.8	16	9.2
364	381	17	7.0	17	9.5
364	382	18	7.2	18	9.8
364	383	19	7.4	19	10.1
364	384	20	7.6	20	10.3
364	385	21	7.8	21	10.6
364	386	22	8.0	22	10.8
364	387	23	8.2	23	11.1
364	387.5	23.5	8.3	23.5	11.2

Appendix B: Permanent Spillway Supporting Calculations



Potential hydraulic controls at: (1) Stoplog crest
(2) Pipe Inlet
(3) Pipe system

Stoplogs behave as sharp-crested weir. Each stoplog is 7 ft wide.

Discharge over stoplog:

$$Q = C_w L_w (H - Z_w)^{3/2}$$

C_w = weir discharge coefficient = 2.7
L_w = weir length = 7 ft
H = water surface elevation
Z_w = elevation of weir crest

Designed by: Matt Hoy Checked by:

Typical daily flow = 32 MGD = 49.5 cfs

Assume 6 inlet structures. Each will need to convey:

$$Q = 49.5 \text{ cfs} / 6 = 8.25 \text{ cfs}$$

Solving for H-Zw in weir equation

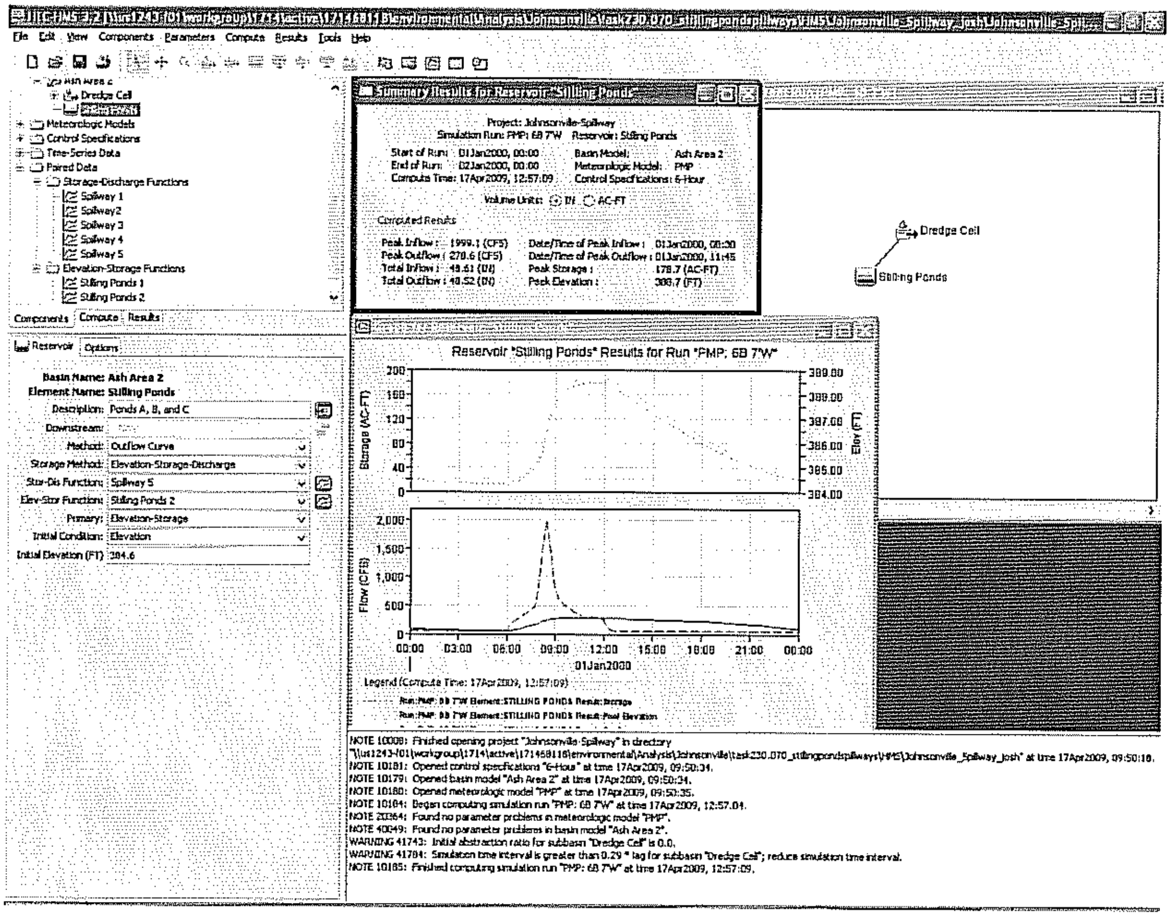
$$H - Z_w = \sqrt[3/2]{\frac{C_w L_w}{Q}} = \boxed{0.58 \text{ ft}} \hat{=} 7 \text{ inches}$$

$\therefore$ ~0.6ft of head over the stoplog crest is acceptable

Invert of 30" HDPE outlet pipe on back of box is approximately 381.2ft. HEC-RAS indicates inlet controlled headwater elevation of 382.6'. Control is at stoplog.

Designed by: Matt Hoy Checked by:

HEC-HMS Model Summary
6-hr PMP Storm
Johnsonville Fossil Plant – Ash Disposal Area 2



JOF Ash Disposal Area 2
Free Water Volume

Elevation (US Survey Feet)	Free Water Volume (Cubic Yards)						Total (ac-ft)
	Pond A	Pond B	Pond C	Cell Pond 1	Cell Pond 2	Total	
391.0	333,713	195,150	187,017	104,963	116,326	937,169	581
390.0	314,123	180,189	187,017	90,990	99,625	871,944	540
389.0	295,356	166,128	187,017	77,561	84,835	810,897	503
388.0	277,658	153,180	172,852	65,197	71,551	740,438	459
387.8	274,229	150,722	167,168	59,493	65,548	717,160	445
387.0	260,751	141,132	159,129	53,788	59,545	674,345	418
386.4	250,829	134,195	151,179	48,437	54,221	638,861	396
386.0	244,298	129,689	146,133	43,085	48,897	612,102	379
385.0	228,256	118,839	133,975	33,143	39,555	553,768	343
384.0	212,608	108,579	122,191	24,053	30,889	498,320	309
383.0	197,359	98,913	110,966	15,931	22,760	445,929	276
382.0	182,533	89,760	100,275	9,130	15,182	396,880	246
381.0	168,025	80,997	90,005	4,081	8,752	351,860	218
380.0	153,816	72,530	80,105	1,045	4,546	312,042	193
379.0	139,888	64,343	70,551	37	2,132	276,951	172
378.0	**126,256**	**56,449**	**61,324**		**676**	**244,705**	**152**
377.0	112,971	48,876	52,434		52	214,333	133
376.0	100,069	41,681	43,933			185,683	115
375.0	87,557	35,029	35,906			158,492	98
374.0	75,520	29,309	28,393			133,222	83
373.0	64,033	24,410	21,550			109,993	68
372.0	53,015	19,951	15,580			88,546	55
371.0	42,483	15,838	10,430			68,751	43
370.0	32,724	12,008	5,672			50,404	31
369.0	24,535	8,605	1,649			34,789	22
368.0	17,486	5,942	79			23,507	15
367.0	11,770	3,568	0			15,338	10

JOF Stage-Storage
HMS Model Supporting Calculations

Using surveyed volumes

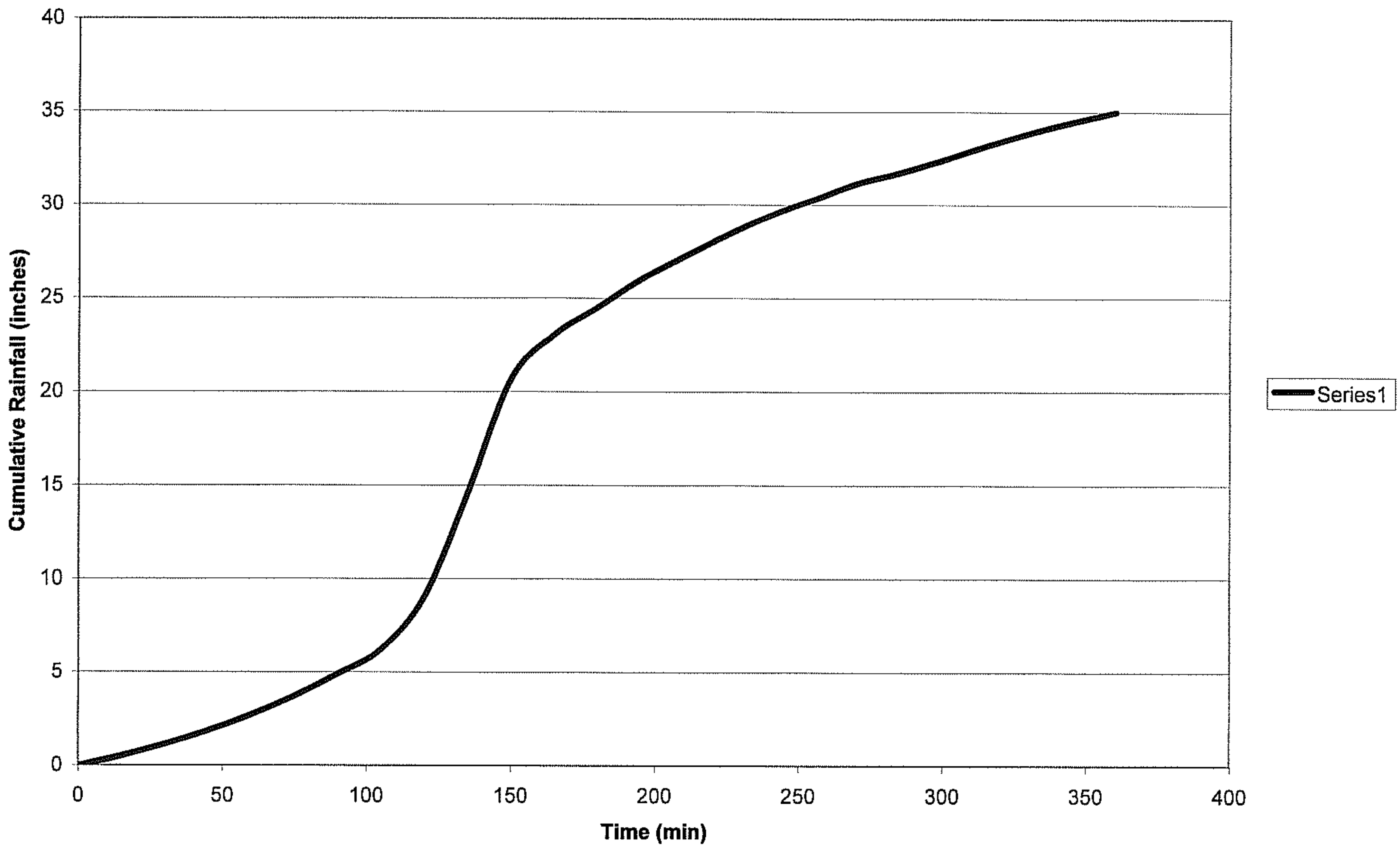
With Cell Pond Volumes Included

Elevation (US Survey Feet)	Absolute Total (ac-ft)	Incremental Volume (ac-ft)	Above weir Volume* (ac-ft)
384.0	309	0	0
385.0	343	34	34
386.0	379	36	71
387.0	418	39	109
388.0	459	41	150
389.0	503	44	194
390.0	540	38	232
391.0	581	40	272

6-hr PMP Rainfall
SCS Distribution
Humphreys County, TN

Storm Duration (min)	Cumulative Storm Rain (in)	Incremental Rain (in)	Rainfall Intensity (in/hr)
0	0.00	0.00	0.00
15	0.50	0.50	1.99
30	1.12	0.62	2.48
45	1.85	0.74	2.94
60	2.70	0.85	3.40
75	3.72	1.01	4.04
90	4.90	1.18	4.74
105	6.21	1.31	5.25
120	9.05	2.83	11.33
135	14.61	5.57	22.26
150	20.68	6.07	24.28
165	23.02	2.34	9.36
180	24.50	1.48	5.92
195	25.98	1.48	5.91
210	27.20	1.22	4.87
225	28.37	1.18	4.70
240	29.43	1.05	4.22
255	30.28	0.85	3.41
270	31.15	0.87	3.48
285	31.73	0.58	2.32
300	32.41	0.68	2.72
315	33.18	0.76	3.06
330	33.86	0.68	2.73
345	34.46	0.60	2.41
360	35.00	0.54	2.16

6-hour PMP SCS Storm Rainfall Distribution (JOF)



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                                   HEC-RAS Version 4.0 Beta
                                 U.S. Army Corp of Engineers
                                Hydrologic Engineering Center
                                      609 Second Street
                                     Davis, California

            X     X  XXXXXX   XXXX        XXXX     XX     XXXX
            X     X  X       X    X       X   X   X  X   X
            X     X  X       X            X   X  X    X  X
            XXXXXXX  XXXX    X      XXX   XXXX   XXXXXX   XXXX
            X     X  X       X            X X    X    X       X
            X     X  X       X    X       X  X   X    X       X
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PROJECT DATA
Project Title: jof_perm_spwy_check
Project File : jof_spwy.prj
Run Date and Time: 4/17/2009 8:45:38 AM

Project in English units

********************************************************************************

PLAN DATA

Plan Title: single_spillway_baseflow_interp
Plan File : Z:\tva_jville\ras\jof_spwy.p02

           Geometry Title: single_spillway_interp
           Geometry File : Z:\tva_jville\ras\jof_spwy.g02

           Flow Title    : single_spillway
           Flow File     : Z:\tva_jville\ras\jof_spwy.f01

Plan Summary Information:
Number of:  Cross Sections =  122    Multiple Openings  =    0
            Culverts       =    0    Inline Structures  =    2
            Bridges        =    0    Lateral Structures =    0

Computational Information
    Water surface calculation tolerance  =  0.01
    Critical depth calculation tolerance =  0.01
    Maximum number of iterations         =  20
    Maximum difference tolerance         =  0.3
    Flow tolerance factor                =  0.001

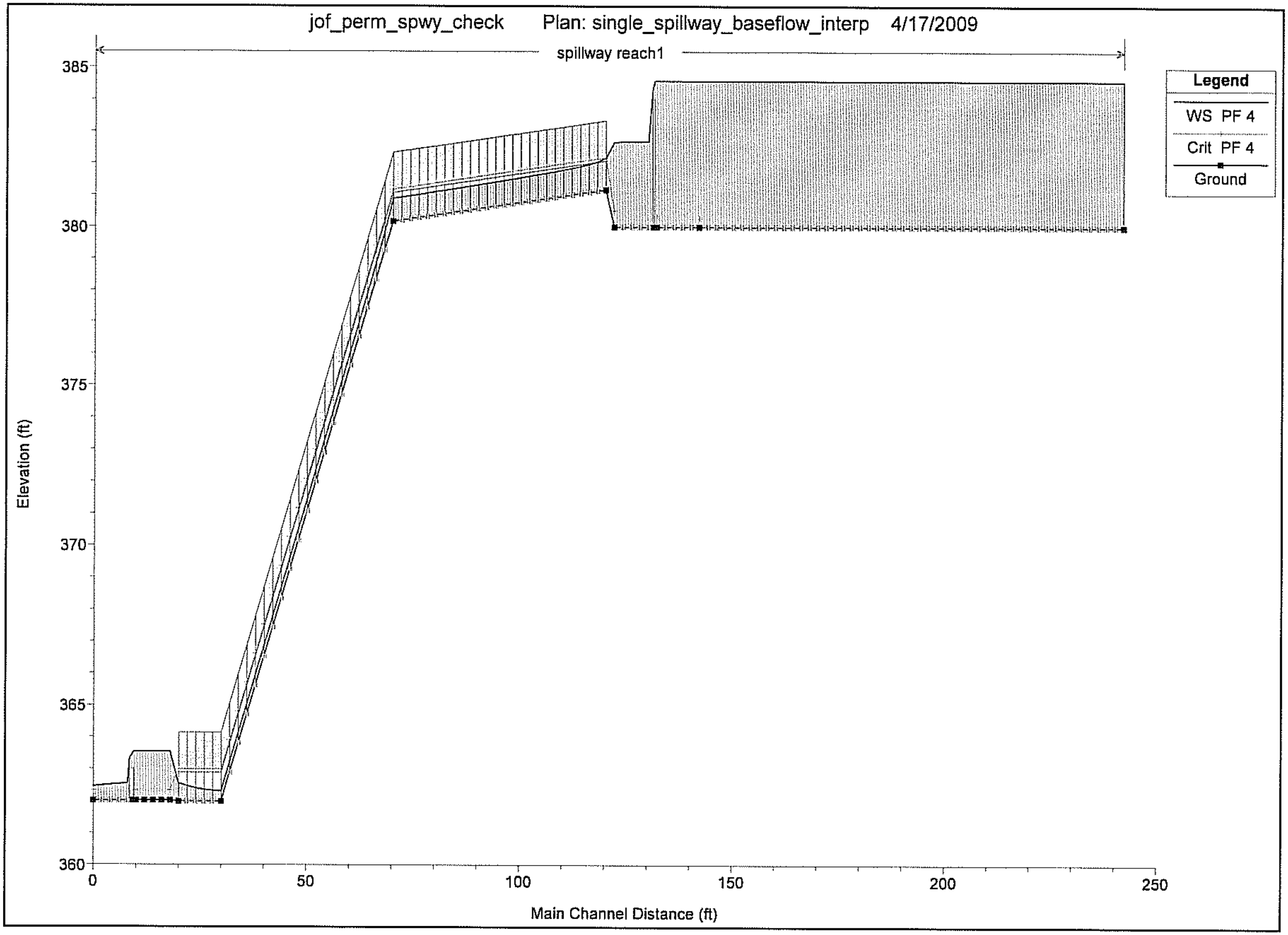
Computation Options
    Critical depth computed only where necessary
    Conveyance Calculation Method: At breaks in n values only
    Friction Slope Method:         Average Conveyance
    Computational Flow Regime:     Mixed Flow
```

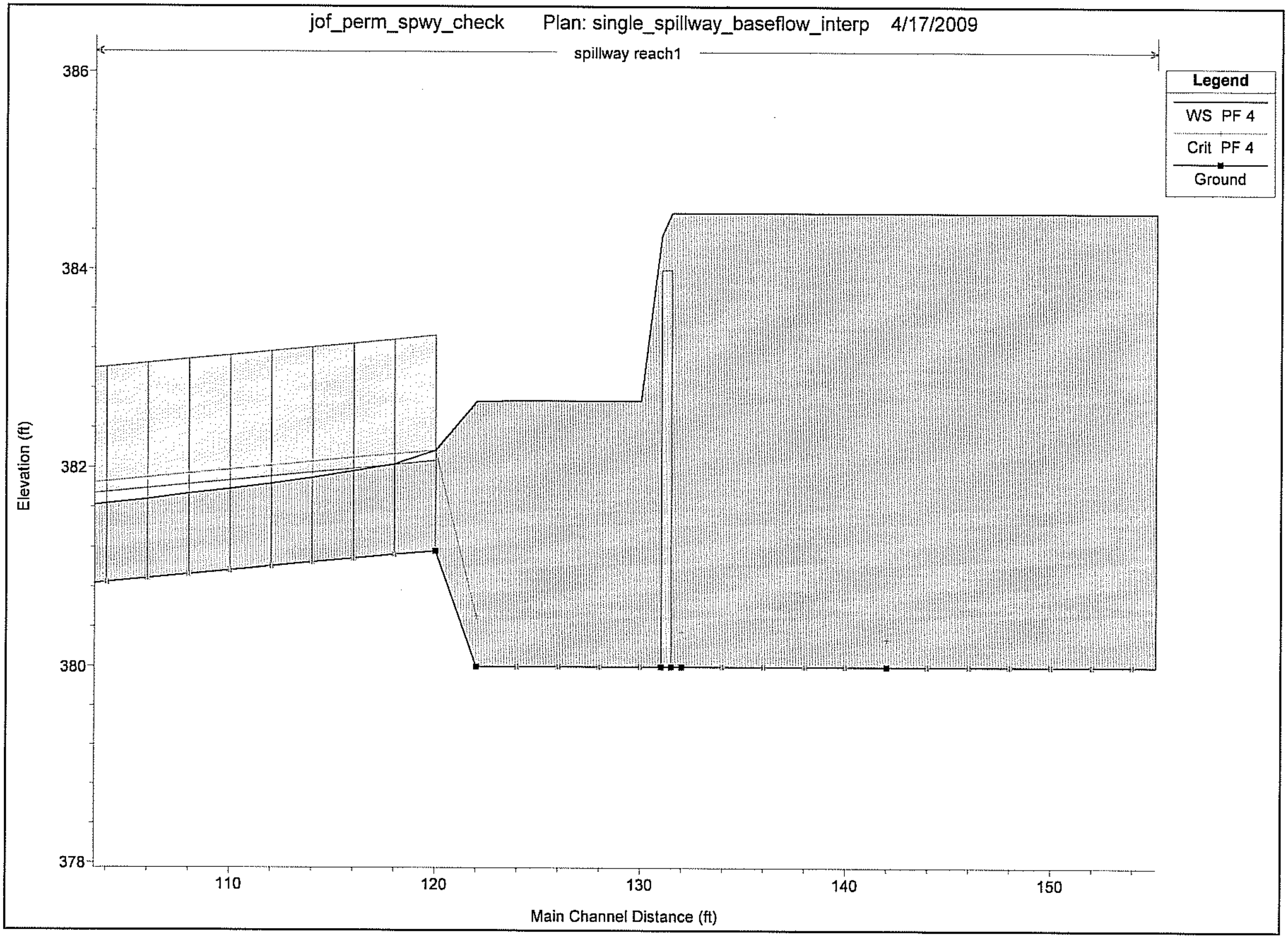
HEC-RAS Plan: sing_spwy_int River: spillway Reach: reach1 Profile: PF 4

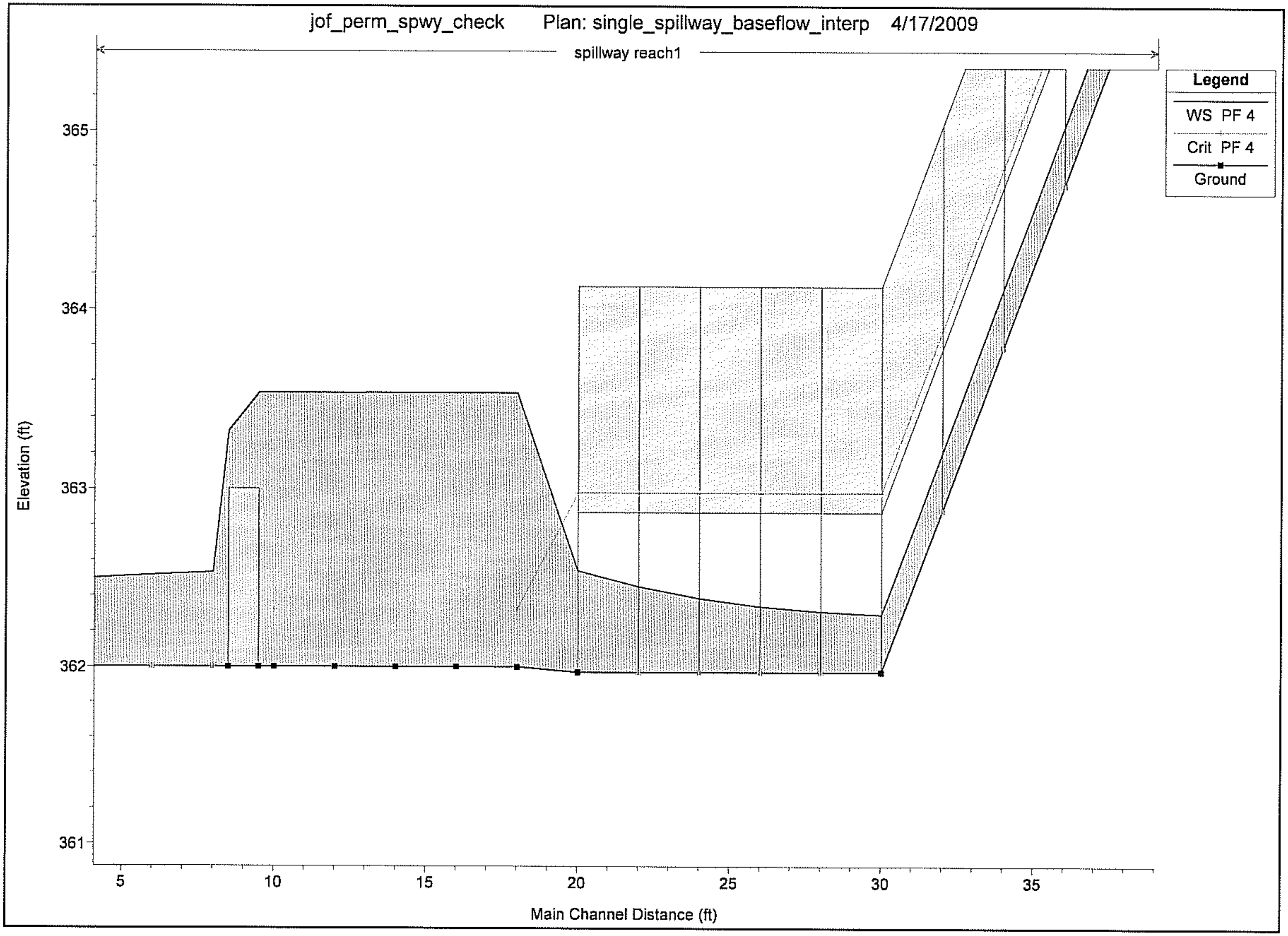
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	
reach1	1100	PF 4	8.25	380.00	384.58	380.08	384.58	0.000000	0.02	387.08	0.00
reach1	1098.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1096.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1094.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1092.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1090.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1088.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1086.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1084.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1082.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1080.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1078.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1076.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1074.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1072.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1070.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1068.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1066.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1064.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1062.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1060.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1058.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1056.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1054.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1052.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1050.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1048.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1046.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1044.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1042.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1040.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1038.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1036.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1034.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1032.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1030.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1028.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1026.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1024.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1022.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1020.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1018.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1016.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1014.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1012.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1010.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1008.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1006.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1004.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1002.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.02	387.08	0.00
reach1	1000	PF 4	8.25	380.00	384.58	380.28	384.58	0.000002	0.18	45.76	0.01
reach1	998.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.03	290.35	0.00
reach1	996.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.05	175.78	0.00
reach1	994.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.07	113.49	0.01
reach1	992.*	PF 4	8.25	380.00	384.58		384.58	0.000000	0.11	75.05	0.01
reach1	990	PF 4	8.25	380.00	384.58	380.35	384.58	0.000002	0.26	32.03	0.02
reach1	989		Inl Struct								
reach1	988.*	PF 4	8.25	380.00	382.67		382.68	0.000009	0.44	18.71	0.05
reach1	986.*	PF 4	8.25	380.00	382.67		382.68	0.000009	0.44	18.71	0.05
reach1	984.*	PF 4	8.25	380.00	382.67		382.68	0.000009	0.44	18.71	0.05
reach1	982.*	PF 4	8.25	380.00	382.67		382.68	0.000009	0.44	18.71	0.05
reach1	980	PF 4	8.25	380.00	382.66	380.51	382.67	0.000012	0.77	10.65	0.08
reach1	970	PF 4	8.25	381.17	382.17	382.17	382.56	0.005079	5.00	1.65	0.88
reach1	968.*	PF 4	8.25	381.13	382.04	382.14	382.54	0.006723	5.71	1.44	1.23
reach1	966.*	PF 4	8.25	381.09	381.97	382.10	382.52	0.007620	5.97	1.38	1.31
reach1	964.*	PF 4	8.25	381.05	381.90	382.06	382.50	0.008607	6.24	1.32	1.39
reach1	962.*	PF 4	8.25	381.01	381.84	382.02	382.48	0.009353	6.44	1.28	1.45
reach1	960.*	PF 4	8.25	380.97	381.78	381.98	382.46	0.010016	6.60	1.25	1.50
reach1	958.*	PF 4	8.25	380.93	381.74	381.94	382.43	0.010378	6.68	1.23	1.53
reach1	956.*	PF 4	8.25	380.89	381.68	381.90	382.41	0.011175	6.87	1.20	1.59
reach1	954.*	PF 4	8.25	380.85	381.64	381.86	382.38	0.011464	6.93	1.19	1.61
reach1	952.*	PF 4	8.25	380.81	381.59	381.82	382.35	0.011773	7.00	1.18	1.63

HEC-RAS Plan: sing_spwy_int River: spillway Reach: reach1 Profile: PF 4 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	
reach1	950.*	PF 4	8.25	380.77	381.54	381.78	382.32	0.012085	7.06	1.17	1.66
reach1	948.*	PF 4	8.25	380.73	381.50	381.74	382.29	0.012401	7.13	1.16	1.68
reach1	946.*	PF 4	8.25	380.69	381.45	381.70	382.26	0.012713	7.19	1.15	1.70
reach1	944.*	PF 4	8.25	380.65	381.41	381.66	382.23	0.013024	7.26	1.14	1.72
reach1	942.*	PF 4	8.25	380.61	381.36	381.62	382.20	0.013328	7.32	1.13	1.74
reach1	940.*	PF 4	8.25	380.57	381.32	381.58	382.17	0.013628	7.38	1.12	1.76
reach1	938.*	PF 4	8.25	380.53	381.28	381.54	382.13	0.013920	7.43	1.11	1.78
reach1	936.*	PF 4	8.25	380.49	381.23	381.50	382.10	0.014206	7.49	1.10	1.80
reach1	934.*	PF 4	8.25	380.45	381.19	381.46	382.07	0.014484	7.54	1.09	1.82
reach1	932.*	PF 4	8.25	380.41	381.15	381.42	382.04	0.014752	7.59	1.09	1.83
reach1	930.*	PF 4	8.25	380.37	381.10	381.38	382.01	0.015011	7.64	1.08	1.85
reach1	928.*	PF 4	8.25	380.33	381.06	381.34	381.98	0.015260	7.68	1.07	1.87
reach1	926.*	PF 4	8.25	380.29	381.02	381.30	381.94	0.015502	7.73	1.07	1.88
reach1	924.*	PF 4	8.25	380.25	380.98	381.26	381.90	0.015457	7.72	1.07	1.88
reach1	922.*	PF 4	8.25	380.21	380.94	381.22	381.86	0.015416	7.71	1.07	1.88
reach1	920	PF 4	8.25	380.17	380.90	381.18	381.82	0.015377	7.70	1.07	1.87
reach1	918.*	PF 4	8.25	379.26	379.83	380.27	381.68	0.040408	10.92	0.76	3.03
reach1	916.*	PF 4	8.25	378.35	378.85	379.36	381.49	0.066334	13.05	0.63	3.86
reach1	914.*	PF 4	8.25	377.44	377.90	378.45	381.26	0.093012	14.72	0.56	4.54
reach1	912.*	PF 4	8.25	376.53	376.96	377.54	380.98	0.119638	16.09	0.51	5.14
reach1	910.*	PF 4	8.25	375.62	376.03	376.63	380.66	0.145943	17.26	0.48	5.66
reach1	908.*	PF 4	8.25	374.71	375.10	375.72	380.28	0.171165	18.25	0.45	6.11
reach1	906.*	PF 4	8.25	373.80	374.18	374.81	379.86	0.195601	19.12	0.43	6.52
reach1	904.*	PF 4	8.25	372.89	373.26	373.90	379.40	0.218788	19.89	0.41	6.88
reach1	902.*	PF 4	8.25	371.98	372.34	372.99	378.90	0.240562	20.55	0.40	7.20
reach1	900.*	PF 4	8.25	371.07	371.43	372.08	378.36	0.260724	21.13	0.39	7.49
reach1	898.*	PF 4	8.25	370.16	370.51	371.17	377.79	0.279345	21.65	0.38	7.74
reach1	896.*	PF 4	8.25	369.25	369.59	370.26	377.18	0.296856	22.11	0.37	7.97
reach1	894.*	PF 4	8.25	368.34	368.68	369.35	376.55	0.312764	22.51	0.37	8.17
reach1	892.*	PF 4	8.25	367.43	367.77	368.44	375.88	0.327189	22.86	0.36	8.35
reach1	890.*	PF 4	8.25	366.52	366.85	367.53	375.19	0.340255	23.17	0.36	8.51
reach1	888.*	PF 4	8.25	365.61	365.94	366.62	374.48	0.352174	23.45	0.35	8.65
reach1	886.*	PF 4	8.25	364.70	365.03	365.71	373.75	0.363093	23.70	0.35	8.78
reach1	884.*	PF 4	8.25	363.79	364.12	364.80	373.00	0.373097	23.92	0.34	8.90
reach1	882.*	PF 4	8.25	362.88	363.20	363.89	372.23	0.381786	24.11	0.34	8.99
reach1	880	PF 4	8.25	361.97	362.29	362.98	371.44	0.389488	24.27	0.34	9.08
reach1	878.*	PF 4	8.25	361.97	362.31	362.98	370.14	0.531615	22.46	0.37	8.15
reach1	876.*	PF 4	8.25	361.97	362.34	362.98	368.49	0.627435	19.90	0.41	6.88
reach1	874.*	PF 4	8.25	361.97	362.39	362.98	366.80	0.546405	16.86	0.49	5.48
reach1	872.*	PF 4	8.25	361.97	362.45	362.98	365.39	0.437075	13.75	0.60	4.14
reach1	870	PF 4	8.25	361.97	362.54	362.98	364.36	0.285730	10.82	0.76	2.99
reach1	868	PF 4	8.25	362.00	363.54	362.32	363.54	0.000218	0.67	12.28	0.10
reach1	866	PF 4	8.25	362.00	363.53		363.54	0.000218	0.67	12.28	0.10
reach1	864	PF 4	8.25	362.00	363.53		363.54	0.000218	0.67	12.28	0.10
reach1	862	PF 4	8.25	362.00	363.53		363.54	0.000218	0.67	12.27	0.10
reach1	860	PF 4	8.25	362.00	363.53	362.32	363.54	0.000219	0.67	12.27	0.10
reach1	859		Inl Struct								
reach1	858.*	PF 4	8.25	362.00	362.53		362.59	0.005752	1.94	4.25	0.47
reach1	856.*	PF 4	8.25	362.00	362.52		362.58	0.006322	2.00	4.12	0.49
reach1	854.*	PF 4	8.25	362.00	362.50		362.56	0.007068	2.07	3.98	0.52
reach1	852.*	PF 4	8.25	362.00	362.48		362.55	0.008109	2.16	3.81	0.55
reach1	850	PF 4	8.25	362.00	362.45	362.32	362.53	0.009999	2.31	3.57	0.61


jof_perm_spwy_check Plan: single_spillway_baseflow_interp 4/17/2009
spillway reach1
Legend
WS PF 4
Crit PF 4
Ground
Elevation (ft)
385
380
375
370
365
360
0
50
100
150
200
250
Main Channel Distance (ft)

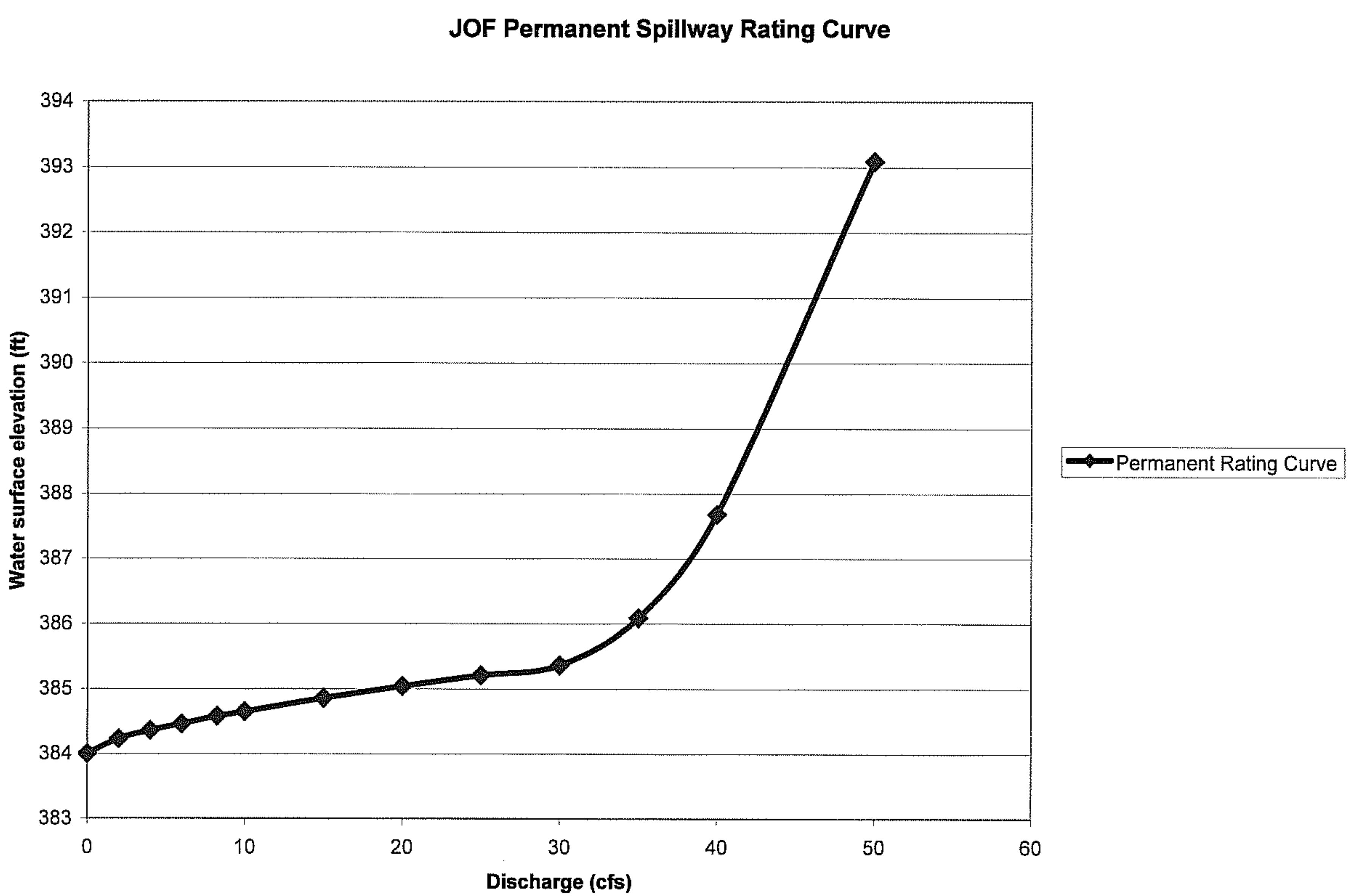

jof_perm_spwy_check Plan: single_spillway_baseflow_interp 4/17/2009
spillway reach1
Legend
WS PF 4
Crit PF 4
Ground
386
384
382
380
378
Elevation (ft)
110
120
130
140
150
Main Channel Distance (ft)



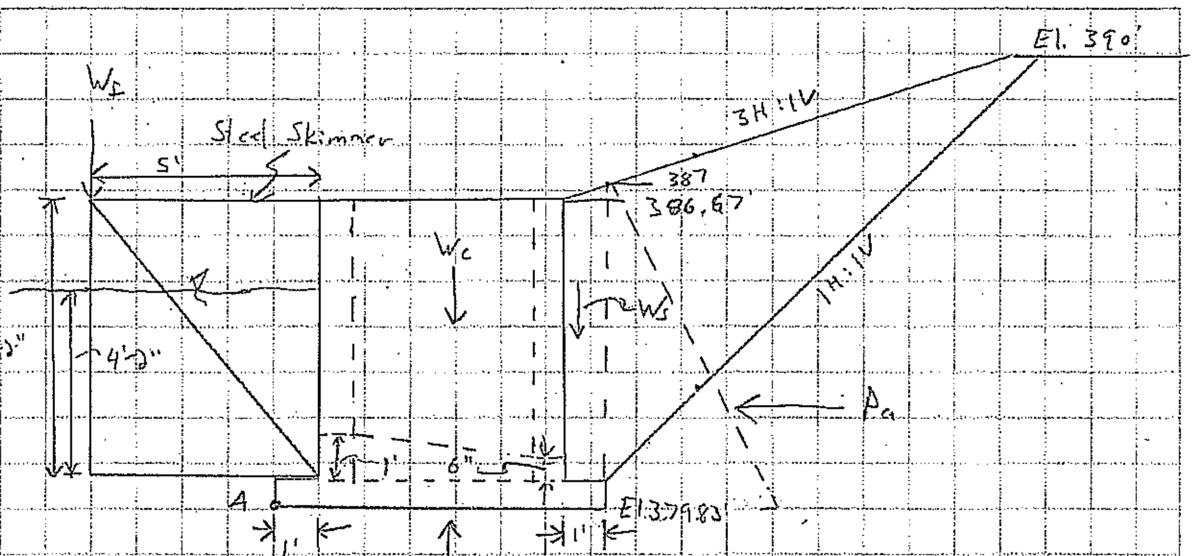
jof_perm_spwy_check Plan: single_spillway_baseflow_interp 4/17/2009
spillway reach1
Legend
WS PF 4
Crit PF 4
Ground
Elevation (ft)
365
364
363
362
361
5
10
15
20
25
30
35
Main Channel Distance (ft)

JOF Permanent Spillway Stage-Storage-Discharge Table

Stage	Storage	Discharge (each outlet structure)	Cumulative Discharge (six outlet structures)
(ft)	(ac-ft)	(cfs)	(cfs)
384	0	0.00	0.00
384.25	6.875	2.27	13.60
384.5	13.75	6.73	40.36
384.75	20.625	12.35	74.11
385	27.5	18.86	113.14
385.25	34.375	26.33	157.97
385.5	41.25	32.99	197.96
385.75	48.125	34.64	207.86
386	55	34.68	208.07
386.25	61.875	35.84	215.02
386.5	68.75	36.89	221.31
386.75	75.625	37.70	226.18
387	82.5	38.35	230.10
387.25	89.375	38.93	233.58
387.5	96.25	39.52	237.13
387.75	103.125	40.20	241.19
388	110	40.89	245.36
388.25	116.875	41.56	249.39
388.5	123.75	42.21	253.28
388.75	130.625	42.84	257.03
389	137.5	43.44	260.64
389.25	144.375	44.02	264.11
389.5	151.25	44.57	267.44
389.75	158.125	45.11	270.64
390	165	45.62	273.70


JOF Permanent Spillway Rating Curve
Water surface elevation (ft)
394
393
392
391
390
389
388
387
386
385
384
383
0
10
20
30
40
50
60
Discharge (cfs)
Permanent Rating Curve

Johnsonville Fossil Plant
Overturning + Eccentricity for Inlet Structures
Page 1 of 2



Width of 2 boxes is 18.7'.

- Assume no drainage and an unsupported soil condition.
- Assume $\gamma_{wsoil} = 125$ pcf + $\phi = 28°$

Determine Equivalent Fluid Pressure (EFP)

EFP for drained soil with 3:1 back slope = 58 pcf

So, for undrained, $EFP = \frac{58}{2} + 62.4 = 91.4$ pcf
Use EFP = 90 pcf ✓

$W_s = \frac{1}{2}(1')(18.7')(0.33')(0.125\ kcf) + (1')(6.17')(18.7')(0.125) = 14.8k$ ✓

✓
$W_c = (155.8\ ft^3)(0.150\ \frac{k}{ft^3})(2) = 46.7$ kips ✓

$W_f = 2.5$ kips (from Steel Skimmer Calc Sheets) ✓

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Checked by: NAB

$P_a = \frac{1}{2}[90(7.17')](7.17')(18.7')$
$= 43.3$ kips ✓

Area	Force	Arm	Moment
W_s	14.8 k ✓	6.83' ✓	C 101.1 ft·k ✓
W_c	46.7 k ✓	3.67 ✓	171.4 ft·k ✓
W_f	2.5 k ✓	4.0' ✓	− 10.0 ft·k ✓
P_a	43.3 k ✓	2.4' ✓	− 103.9 ft·k ✓
			Σ 158.6 ft·k ✓

$FS_{overturning} = \frac{C\Sigma M}{C\Sigma M} = \frac{101.1+171.4}{10+103.9} = 2.4 > 1.5$ OK ✓

$e = \frac{L}{2} - \frac{\Sigma M}{\Sigma V} = \frac{7.33}{2} - \frac{158.6}{14.8+46.7+2.5} = 1.19' < \frac{L}{6} = 1.22$ ✓ OK

Resultant inside middle third

$q_{max} = \frac{64}{(7.33')(18.7')}\left(1 + \frac{6(1.19)}{7.33}\right) = 0.92$ ksf ✓

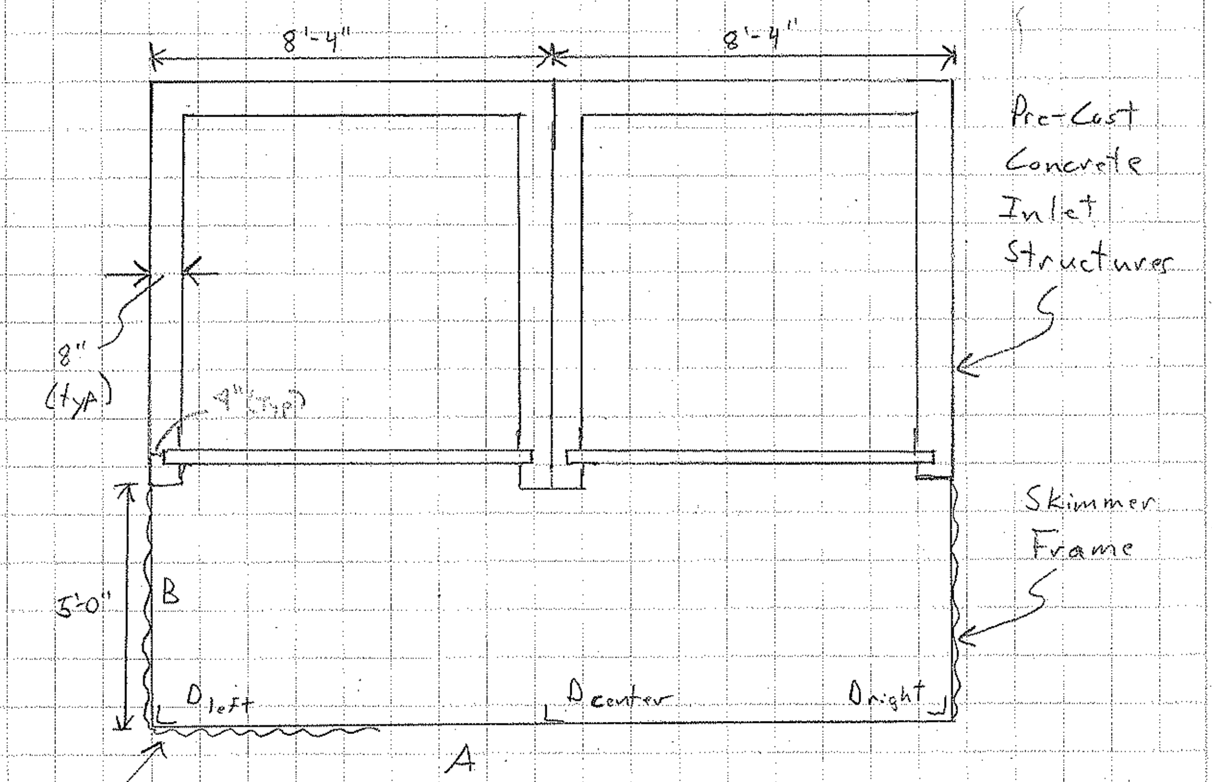
$q_{min} = \frac{64}{(7.33)(18.7)}\left(1 - \frac{6(1.19')}{7.33}\right) = 0.01$ ksf ✓

Check Sliding

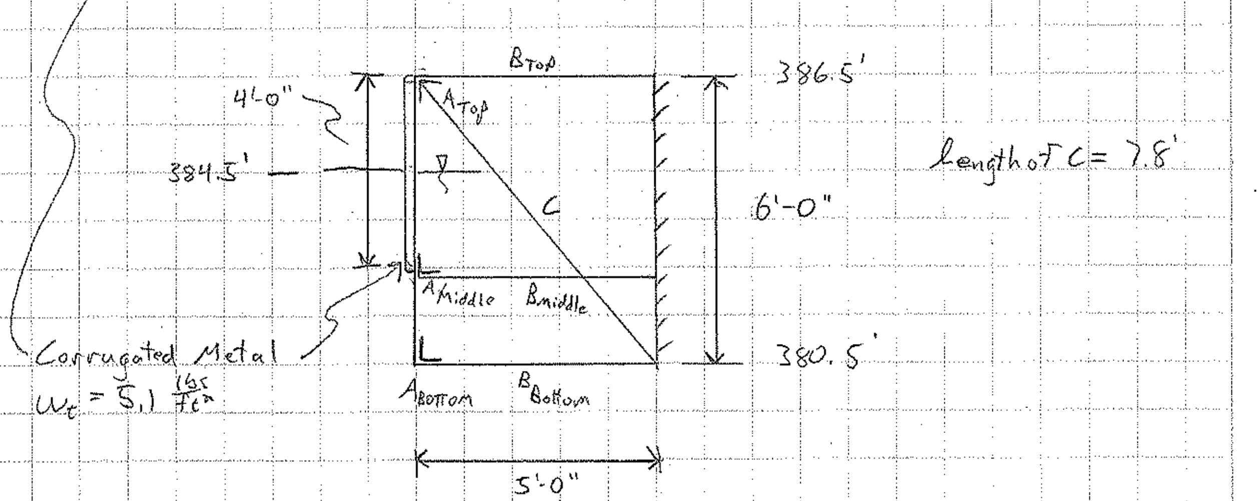
$F_{RESISTING} = \Sigma V \tan\delta + C_a B L = 64k(0.35) + 0.5\text{ ksf}(7.33\text{ ft})(18.7\text{ ft}) = 90.9$ k
$F_{ACTING} = 43.3$K

$FS_{SLIDING} = 90.9k / 43.3k = 2.1 > 1.5$ OK

Plan View



Elevation View



Loads: Loads acting on the structure will be dead load, wind load, and wave load. Since sluiced byproduct enters stilling ponds at higher temperature and the short duration in the ponds, it is assumed no ice loads will be exerted on the structure.

Wind load is determined from ASCE 7-02 6.4.2.1. The structure is classified as Category I since it is a low hazard to human life.

The wind pressure on the frame is calculated by (ASCE 7-02 pg. 27)

$$p_s = \lambda I p_{s30}$$

- Basic Wind Speed = 90 mph, $p_{s30} = 12.8$ psf ✓
- Adjustment Factor for Height + Exposure, $\lambda = 1.47$ ✓
  (Height < 15', Exposure D)
- Importance Factor, $I = 0.87$ ✓

$$p_s = (1.47)(0.87)(12.8 \text{ psf}) = 16.4 \text{ psf}$$ ✓

Wave load is determined from combination of ASCE 7-02 5.3.3.4 and Design of Small Dams, US Dept. of the Interior (1987).

- Using Table 6.7 in Design of Small Dams, a quadratic equation for wave heights for wind speeds of 100 mph was determined as a function of fetch length. (pg. 258)

- For a fetch length of 1,500 ft, wave height $H_b = 7.6'' \approx 8''$

- Wave pressure from ASCE Eq. 5-7

$$F_t = 1.1 C_p \gamma_w d_s^2 + 1.9 \gamma_w d_s^2 \qquad \text{(ASCE 7-02 pg 19)}$$

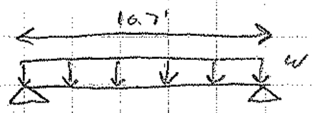
- $d_s = \frac{H_b}{0.78} = \frac{0.67'}{0.78} = 0.86'$ ✓
- $C_p = 1.6$ ✓
- $\gamma_w = 62.4 \frac{lb}{ft^3}$ ✓

$$F_t = 1.1(1.6)(62.4 \tfrac{lb}{ft^3})(0.86')^2 + 1.9(62.4)(0.86')^2$$
$$= 169 \tfrac{lbs}{ft} \checkmark$$

Dead load will be determined after initial sizing of members.

Estimate M_{max} on Member A (Live Load)

Worst case scenario occurs with wind + wave load acting together.



Assume simply supported beam w/ uniform load.

$$w = (16.4 \tfrac{lb}{ft^2})(2 ft) + 169 \tfrac{lb}{ft}$$
$$= 202 \tfrac{lbs}{ft} \checkmark$$

$$M_{max} = \frac{wl^2}{8} = \frac{202(16.7)^2}{8} \qquad \text{(LRFD 4-190)}$$
$$= 7042 \text{ ft·lbs} \checkmark$$
$$= 7.042 \text{ ft·kips} \checkmark$$

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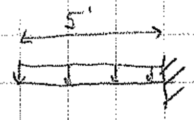
$M_{max\,(factored)} = 7.042\,(1.7) = 12.0$ ft·kips ✓

$S_{req} = \frac{M_{max}}{\phi F_y} = \frac{(12.0\text{ ft·kips})(\frac{12\text{ in}}{1\text{ ft}})}{(0.9)(36\text{ ksi})} = 4.44\text{ in}^3$ ✓ (Conservative since load is split with middle angle too)

Use L 6×6×½ for Members A_{TOP} & A_{middle} ($S = 4.61\text{ in}^3$, LRFD 1-58)

Estimate M_{max} on Member B (Live Load)

Worst case scenario occurs with wind + wave load acting together.



$M_{max} = \frac{w l^2}{2} = \frac{(202)(5)^2}{2}$ (LRFD 4-196)

$= 2{,}525$ ft·lbs ✓ Assume cantilever beam.

$= 2.525$ ft·kips ✓

$M_{max\,(factored)} = 2.525\,(1.7) = 4.293$ ft·kips ✓

$S_{req} = \frac{(4.293)(12)}{(0.90)(36)} = 1.59\text{ in}^3$ ✓ (Conservative since load is split with middle angle)

Minimum angle desired is L 4×4×½ with $S = 1.97 > S_{req}$. Therefore use L 4×4×½ for all other members. ✓ (LRFD 1-60)

Estimate M_{max} on Member B_{Top} (Dead Load)

$P_{ult} = 2(16.7\text{ ft})(19.6\,\frac{lb}{ft}) + 12.8\,\frac{lb}{ft}\left[16.7 + 3(6) + 4(5) + (2)7.8\right] + (16.7 + 2(5))(4)(5.1\,\frac{lb}{ft^2})$

$= \cancel{2{,}127}$ lbs $\cancel{2{,}227}$ lbs 2,099 lbs ✓

$P_{ult,\,factored} = \cancel{2{,}127}\ 2{,}099\,(1.4) = \cancel{2{,}978}\ 2{,}939$ lbs ✓

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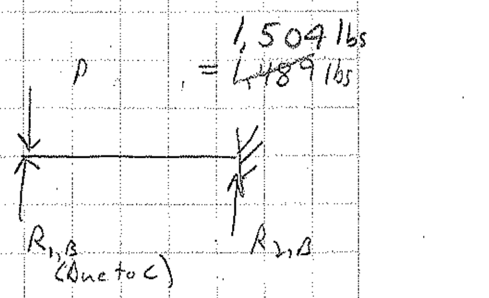
- Since there are two B_{Tops}, only half of load will act on each.

$\frac{1}{2} P_{ult,factored} = \cancel{1,489}$ 1,470 lbs ✓

$P = P_{wt} + \frac{3(12.8^{\#}/ft)(5ft)}{8}(1.4) = 1,504$ lbs

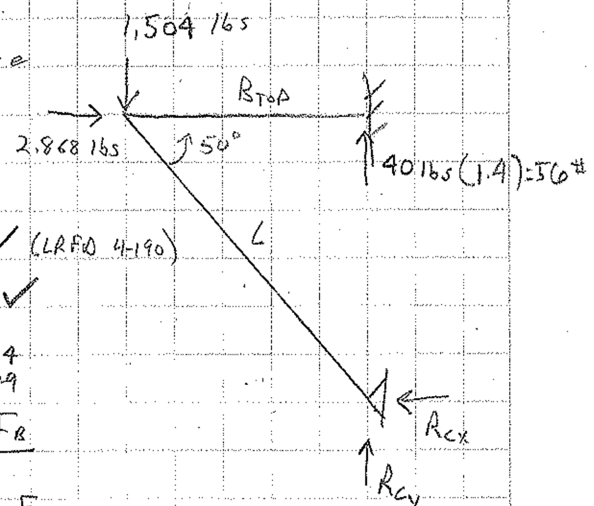
- For dead load, B_{Top} can be assumed to be fixed at one end and simply supported (by Member C) at other with $\frac{1}{2} P_{wt}$ acting as point load at the end. Therefore, no moment is caused by dead load since load passes through support.



$R_2 = \frac{5(12.8)(5)}{8} = 40$ lbs $(1.4) = 56^{\#}$

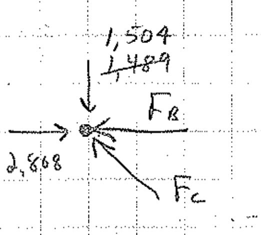
Check Buckling of Members C + B_{Top} (Dead + Live Load)

- The reaction at the point where B_{Top} + C meet is due to the live load on Member A.



$R_A = \frac{wl}{2} = \frac{202(16.7)}{2} = 1,687$ lbs ✓ (LRFD 4-190)

$R_{A\,Factored} = 1,687(1.7) = 2,868$ lbs ✓



$\Sigma F_y = \left(\frac{6}{7.8}\right)F_c - \cancel{1,489}\ 1,504 = 0$

$\Sigma F_y = 2,868 - F_B - \left(\frac{5}{7.8}\right)F_c = 0$

$F_c = \cancel{1936}$ 1,955 lbs

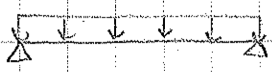
$F_B = 1,615$ lbs

Designed by: JDK Checked by: NAB

By inspection, compression in Members C & B_{top} is less than axial strength of $\angle\, 4'' \times 4'' \times \frac{1}{2}''$ of 21 kips (LRFD 3-115).

Estimate M_{max} on Member A (Dead Load)

$M_{max,factored} = \frac{w\ell^2}{8}(1.4)$ (LRFD 4-190)



$= \frac{(40\frac{lb}{ft})(16.7)^2}{8}(1.4)$

$w = 19.6\frac{lb}{ft} + (4\,ft)(5.1\frac{lb}{ft^2})$
$= 40\frac{lb}{ft}$

$= 1952$ ft·lbs $< 7{,}042$ ft·lbs

OK

Check Connections

Welds: Worst case shear occurs at connection of Members A & B during live load.

$R_{A,factored} = V_{max} = 2{,}868$ lbs ✓

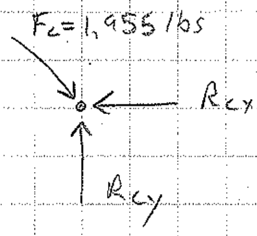
Weld Strength for $\frac{3}{16}''$ weld $= \phi(0.6\,F_{exx})\,A_e$ (LRFD 6-78)
$= (0.75)(0.6)(70)(\frac{3}{16}'')$
$= 5.9\ ^{k}/_{in}$ ✓

Use $\frac{3}{16}''$ fillet welds with minimum length of 1" at all connections. ~~except at connections to seats use $\frac{1}{4}''$ weld per LRFD J2.4. Seats will be 3/4" thick.~~

Designed by: JDK Checked by: NAB

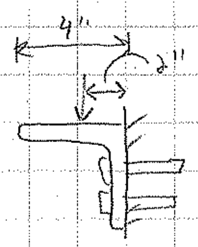
Angle Seat to Riser:

$$\Sigma F_y = R_{cy} - F_c\left(\frac{6}{7.8}\right) = R_{cy} - 1{,}955\left(\frac{6}{7.8}\right) = 0$$
$$\Sigma F_x = F_c\left(\frac{5}{7.8}\right) - R_{cx} = 1{,}955\left(\frac{5}{7.8}\right) - R_{cx}$$



$R_{cy} = \cancel{1{,}489}\ 1{,}504$ lbs
$R_{cx} = \cancel{1{,}241}\ 1{,}254$ lbs

- Worst moment on seat will be due to compression from Member C.



$M_n = \cancel{1.489}\ 1{,}504$ kips (2.0in)
$= \cancel{2.978}$ in.k 3.01 in.k ✓

$$S_{req} = \frac{\cancel{2.978}\ 3.01}{(0.9)(36)} = 0.093 ✓$$

Assume load is inadvertently positioned 2" from wall.

- Assume overhanging leg of angle will be 4" x ½"

$$S = \frac{bd^2}{6} = \frac{4(\frac{1}{2})^2}{6}$$ (LRFD 7-17)
$$= 0.167 \text{ in}^3 > 0.09 \quad \underline{OK}$$

Overhanging leg of seat should be minimum of 4" + ½" thick. It should also be placed a maximum of 2" away from wall.

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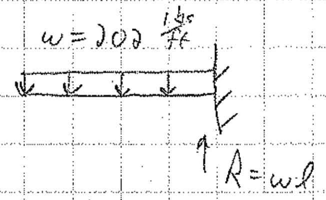
Check Concrete Breakout Strength at Anchor Bolts

Breakout Strength Due to Shear:

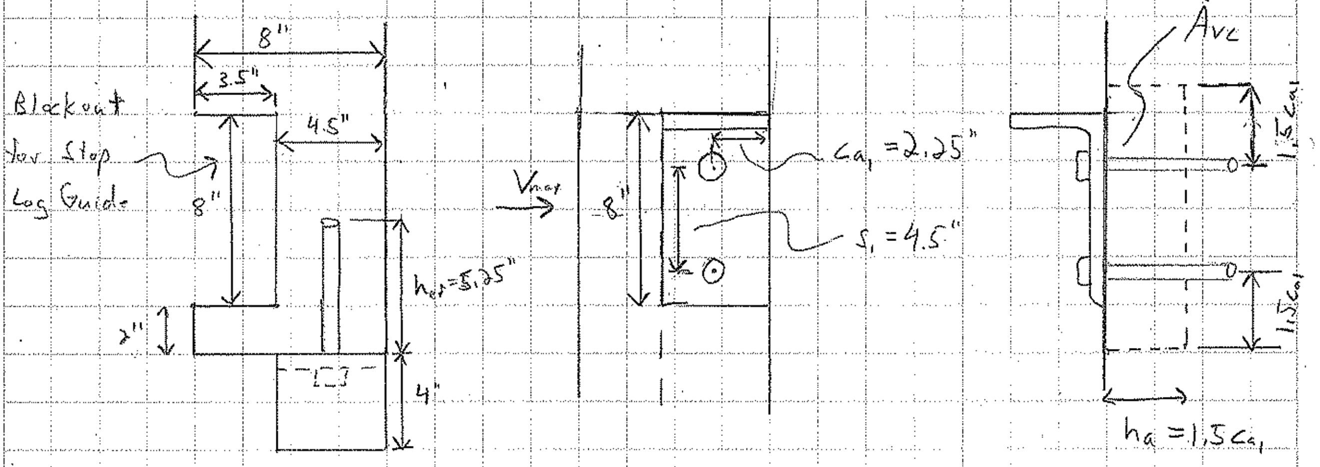


- Maximum shear is due to live loads.

$R = V_{max} = wl = 202(5) = 1{,}010$ lbs (LRFD 4-196)

$V_{max,factored} = 1{,}010\,(1.7) = 1{,}717$ lbs ✓

Maximum shear is split between B_{top} & B_{middle} hence $\frac{1}{2} V_{max,factored} = 859$ lbs



The concrete breakout strength of a group of anchors in shear is:

$$\phi V_{cbg} = \frac{A_{vc}}{A_{vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} V_b \quad \text{(ACI 318 D6.2)}$$

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$$A_{Vc} = (2(1.5c_{a1}) + s_1) \times 1.5c_{a1}$$
$$= [2(1.5)(2.25") + 4.5"] \times (1.5)(2.25)$$
$$= 37.97 \text{ in}^2$$

$$A_{Vco} = 4.5c_{a1}^2 \quad \text{(ACI 318 pg 398)}$$
$$= 4.5(2.25)^2 = 22.78 \text{ in}^2$$

- $\Psi_{ec,V}$ is modification factor for eccentric loading
  $\Psi_{ec,V} = 1/(1 + \frac{2(3.75)}{3(2.25)}) = 0.47$

- $\Psi_{ed,V}$ is modification factor for edge effect.
  $\Psi_{ed,V} = 1.0$ for $c_{a2} \geq 1.5c_{a1}$ ($c_{a2} \simeq 1.5c_{a1}$)

- $\Psi_{c,V}$ is modification factor for cracking of concrete at service loads. Assume concrete will crack but there will be bars between edge and anchor $\Psi_{c,V} = 1.4$

$$V_b = 7\left(\frac{l_e}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f'_c}\,(c_{a1})^{1.5} \quad \text{(ACI 318 pg 399)}$$

- Use $d_o' = \frac{c_{a,min}}{6} = \frac{2.25}{6} = 0.375"$ (per ACI 318 D.8.4)
- Use $l_e' = 8d_o' = 8(0.375) = 3"$ (per ACI 318 D.6.2.2)
- $f'_c = 3000$ psi

$$V_b = 7\left(\frac{3}{0.375}\right)^{0.2}\sqrt{0.375}\sqrt{3000}\,(2.25)^{1.5}$$
$$= 1{,}201 \text{ lbs}$$

$$\phi V_{cbg} = \left(\frac{37.97}{22.78}\right)(1.4)(1)(0.47)(1{,}201)(0.75)$$

$$= 988 \text{ lbs} > \tfrac{1}{2}V_{max,factored} = 859 \text{ lbs} \quad \underline{ok}$$

By inspection, shear in y-direction of 988 lbs can be carried by two bolts at either connection (per ACI 318 D.6.2 (c))

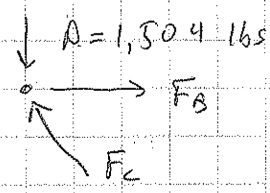
$$\phi V_{cbg}(2) = 988(2) = 1{,}976 \text{ lbs} > 1{,}504 \text{ lbs} \checkmark$$

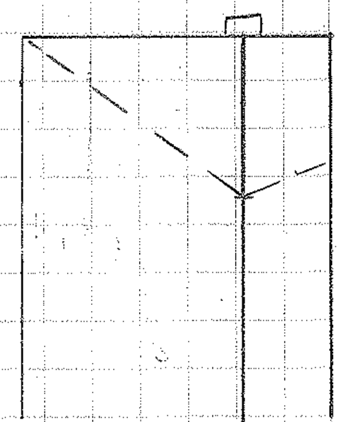
Breakout Strength Due to Tension:

- When only dead load is considered, Member B_{Top} is in tension.

$$\Sigma F_y = \left(\frac{6}{7.8}\right) F_c - 1{,}504 = 0$$

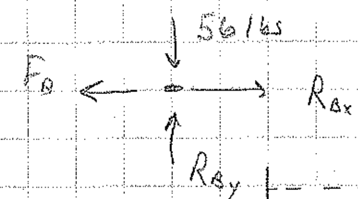
$$\Sigma F_x = F_B - \left(\frac{5}{7.8}\right) F_c = 0$$





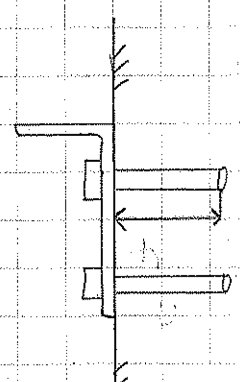
$F_c = 1{,}955$ lbs ✓

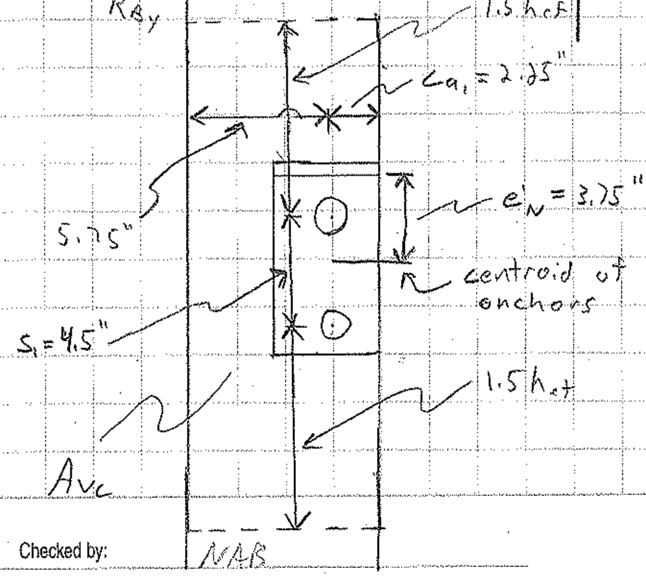
$F_B = 1{,}253$ lbs ✓



$R_{Bx} = 1{,}253$ lbs ✓

$R_{By} = 56$ lbs ✓





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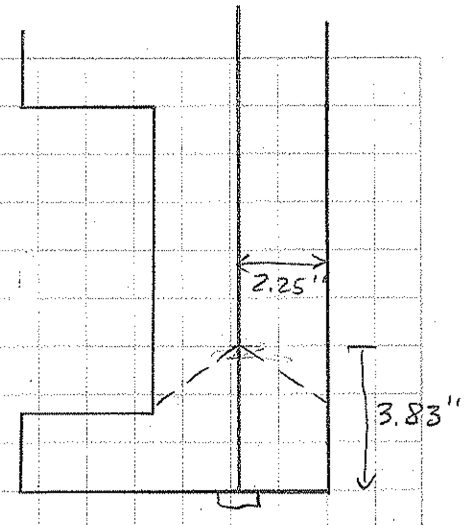


h_{ef} shall be the greater of $C_{a\,max}/1.5$ and $\frac{1}{3}S_1$

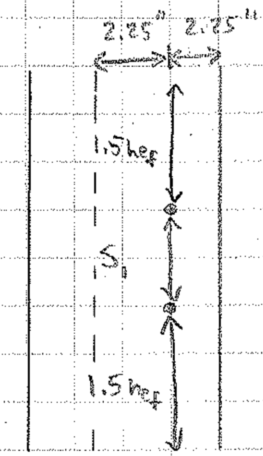
$\frac{1}{3}S_1 = \frac{1}{3}(4.5") = 1.5"$

$C_{a\,max}/1.5 = 5.75"/1.5 = 3.83"$

$h_{ef} = 3.83"$

The concrete breakout strength of the group of anchors is

$$\phi N_{cbg} = \frac{A_{NC}}{A_{NCO}}\,\psi_{ec,N}\,\psi_{ed,N}\,\psi_{c,N}\,\psi_{cp,N}\,N_B$$



$A_{NC} = 4.5"(1.5(3.83")(2) + 4.5")$
$= 72\ in^2$

$A_{NCO} = 9h_{ef}^2 = 9(3.83")^2$
$= 132\ in^2$

$\psi_{ec,N} = \dfrac{1}{\left(1 + \frac{2e'_N}{3h_{ef}}\right)}$ where $e'_N = 3.75"$

$= \dfrac{1}{\left(1 + \frac{2(3.75")}{3(3.83")}\right)} = 0.61$

$\psi_{ed,N} = 0.7 + 0.3\dfrac{c_{a\,min}}{1.5h_{ef}}$ because $c_{a\,min} = 2.25" < 1.5h_{ef} = 5.7"$

$= 0.7 + 0.3\dfrac{2.25"}{5.75"} = 0.82$

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Stantec

$\psi_{c,N} = 1$ assuming concrete cracking at service loads

$\psi_{cp,N} = 1$ assuming riser will be reinforced to control splitting

$N_B = k_c \sqrt{f'_c}\, h_{ef}^{1.5}$

$= 17 \sqrt{3000}\,(3.83)^{1.5} = 6{,}979$ lbs

Therefore:

$\phi N_{cbg} = (72/132)(0.61)(0.82)(1)(1)(6{,}979)(0.85)$

$= 1{,}618$ lbs $> 1{,}253$ lbs OK

Based on additional calculations, breakout strength of concrete in shear and tension governs strength of anchors. No further checks of shear or tensile strength of steel bolts required.

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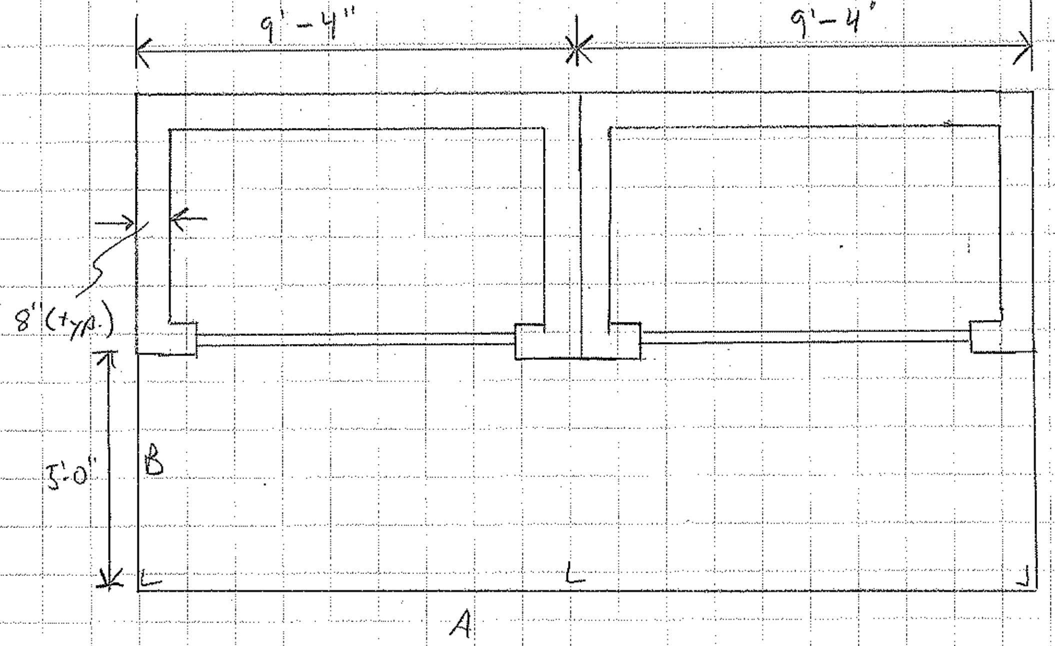
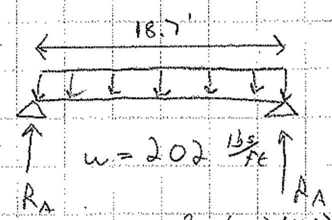
The width of each structure has increased from 8'-4" to 9'-4". The necessary load and design calculations are re-computed herein.



Estimate M_{max} on Member A (Live Load)

$$M_{max} = \frac{wl^2}{8} = \frac{202(18.7)^2}{8} = 8.83 \text{ ft}\cdot\text{kips} ✓$$



$$M_{max, Factored} = 8.83(1.7) = 15.0 \text{ ft}\cdot\text{kips} ✓$$

$$S_{req} = \frac{M_{max}}{\Phi F_y} = \frac{15(12)}{0.9(36)} = 5.56 \text{ in}^3 ✓$$

$$R_A = \frac{wl}{2} = \frac{(202)(18.7)}{2} = 1,889 \text{ lbs}$$

$$R_{A,Factored} = 1.7(1,889) = 3,211 \text{ lbs}$$

Use $L\, 6" \times 6" \times \frac{5}{8}"$ for members A_{Top} + A_{middle} (LRFD 3rd Ed. 1-34)

$S = 5.66 \text{ in}^3$

(This member replaces $L\, 6 \times 6" \times \frac{1}{2}"$ from previous calcs.)

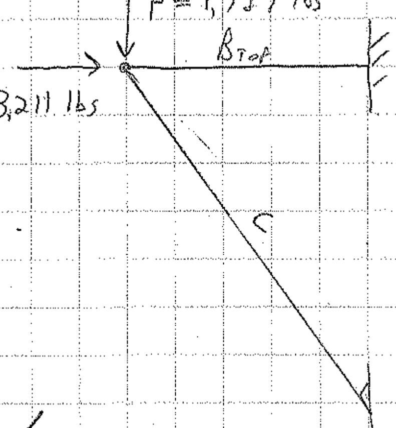
909
925
585
24

Dimensions of Members B + C did not change, so live load will be the same, so no new calcs are performed.

<u>Check Buckling of Members C + B_{Top} (Dead Load + Live Load from A)</u>

Since total weight is split between two frames.

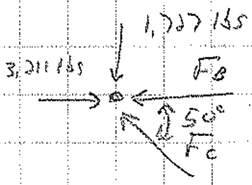
$P = \frac{1}{2} P_{wt} + R_{1A}$



$P = \frac{1}{2}\left[(2)(18.7')(24.3 \frac{lb}{ft}) + 12.8[18.7 + (3)(6) + (4)(5) + (2)(7.8)] + [18.7 + (2)(5)](4)(5.1)\right] + \frac{3(12.8)(5)}{8}$

$= 1,234$ lbs ✓

$P_{Factored} = 1.4(1,234) = 1,727$ lbs ✓



$\Sigma F_y = \left(\frac{6}{7.8}\right) F_c - 1,727 = 0$

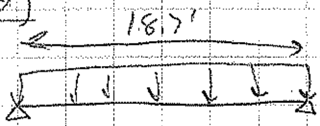
$\Sigma F_x = 3,211 - F_B - \left(\frac{5}{7.8}\right)(F_c) = 0$

$F_c = 2,245$ lbs ✓ < 21 kips (axial strength of ∠ 4"x4"x½" LRFD 3-115)

$F_B = 1,772$ lbs ✓

<u>OK</u>

<u>Estimate M_{max} on Member A (Dead Load)</u>



$M_{max, factored} = \frac{wl^2}{8}(1.4)$

$= \frac{(44.7)(18.7)^2}{8}(1.4)$

$w = 24.3 + (4)(5.1)$
$= 44.7 \frac{lb}{ft}$

$= 2,735$ ft·kips < 15.0 ft·kips <u>OK</u>

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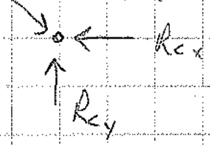
Checked by: NAR

A 1" weld will still be sufficient for maximum shear of $R_{A,Factored} = 3{,}211$ lbs. OK

Angle Seat to Riser

$\Sigma F_y = R_{cy} - F_c\left(\frac{6}{7.8}\right) = 0$
$\Sigma F_x = F_c\left(\frac{5}{7.8}\right) + R_{cx} = 0$

$F_c = 2{,}245$ lbs



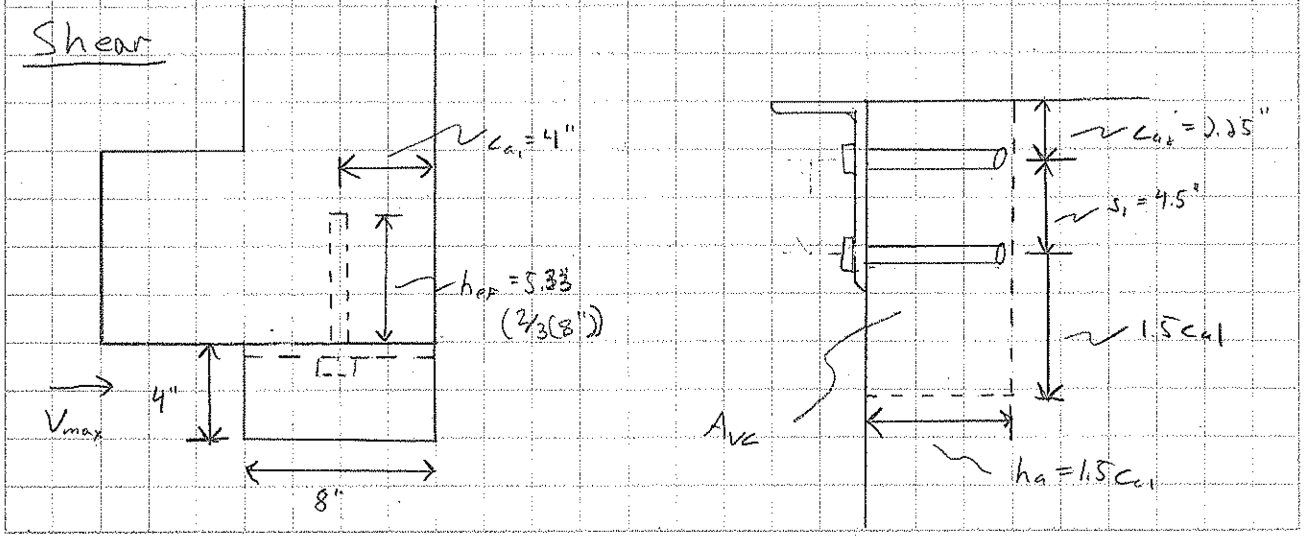
$R_{cy} = 1{,}727$ lbs ✓
$R_{cx} = 1{,}439$ lbs ✓

$M_n = 1{,}727\,(2\text{in}) = 3.454$ in-kips
$S_{req} = \frac{3.454}{0.9(36)} = 0.107\ \text{in}^3 \checkmark < 0.167\ \text{in}^3$ (S of 4" x ½" leg of angle)
OK

Check Concrete Breakout Strength at Anchor Bolts

- New connections are as shown (B_{Top})

Shear



Note: since shear load remains same and thickness of concrete increased, no check of anchor in shear required in x-direction. In y-direction, 1976 > 859 no further check required.

$A_{Vc} = [2.25 + 4.5 + 1.5(4)] \times (1.5)4 = 76.5 \text{ in}^2$

$A_{Vco} = 4.5(4)^2 = 72 \text{ in}^2$ (ACI 318 pg 398)

$\Psi_{ec,V} = 1 / \left(1 + \frac{2(3.75)}{3(4)}\right) = 0.62$ (ACI 318 pg 401)

$\Psi_{ed,V} = 0.7 + 0.3 \frac{(2.25)}{(1.5)(4)} = 0.81$ for $C_{a2} < 1.5 C_{a1}$ (ACI 318 pg 402)

$V_b = 7 \left(\frac{3}{0.375}\right)^{0.2} \sqrt{0.375} \sqrt{3000} (4)^{1.5} = 2{,}847 \text{ lbs}$?

$\phi V_{cbg} = \left(\frac{76.5}{72}\right)(0.81)(0.62)(1)(2{,}847)(0.75)$

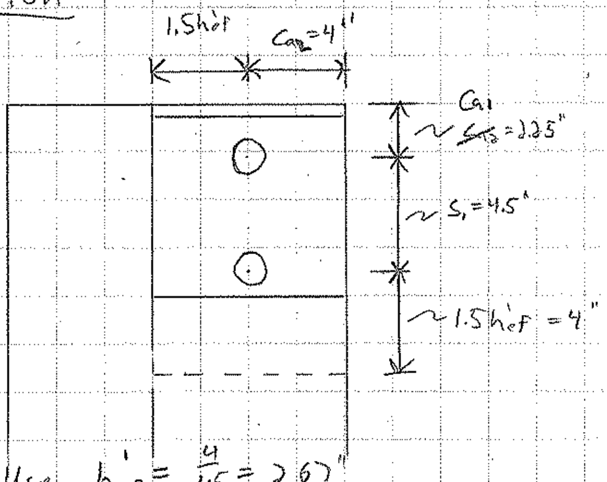
$= 1{,}139 \text{ lbs} \geq \frac{1}{2} V_{max,Factored} = 859 \text{ lbs}$ OK

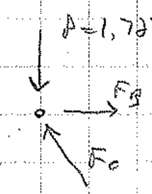
By inspection, shear in y-direction of 1,727 lbs can be carried by 2 bolts at either connection (per ACI 318 D.6.2 (c))

$\phi V_{cbg}(2) = 1{,}139(2) = 2{,}278 \text{ lbs} > 1{,}727 \text{ lbs}$ OK

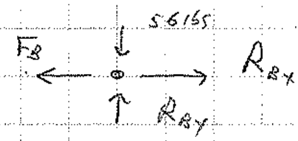
Tension

Dead Load Only





$F_c = 2{,}245 \text{ lbs}$
$F_B = 1{,}439 \text{ lbs}$



Use $h'_{ef} = \frac{4}{1.5} = 2.67"$
(per ACI 318 D.5.2.3)

$R_{By} = 56 \text{ lbs}$
$R_{Bx} = 1{,}439 \text{ lbs}$

Designed by: JOK   Checked by: NAB

$A_{Nc} = [(1.5)(2.67) + 4] \times [2.25 + 4.5 + 1.5(2.67)] = 86 \text{ in}^2$

$A_{Nco} = 9 h_{ef}'^2 = 9(2.67)^2 = 64.2 \text{ in}^2$

$\psi_{ec,N} = \dfrac{1}{\left(1 + \dfrac{2(\cancel{0.375}\ 4.44)}{3(2.67)}\right)} = \cancel{0.91}\ 0.47$

$\psi_{ed,N} = 0.7 + 0.3 \dfrac{2.25}{(1.5)(2.67)} = 0.87$ for $c_{a,min} < 1.5 h_{ef}'$

$N_b = k_c \sqrt{f'_c}\, h_{ef}'^{1.5} = 17\sqrt{3{,}000}\,(2.67)^{1.5} = 4{,}062$ lbs

$\phi N_{cbg} = \left(\dfrac{86}{64.2}\right)(\cancel{0.91}\ 0.47)(0.87)(1)(1)(4{,}062)(0.85)$

$= \cancel{3{,}662}$ lbs $> 1{,}439$ lbs OK

$1{,}891$ lbs

Designed by: JDK Checked by: NAB