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New lock at Kentucky Dam: Analysis and design of precast concrete cofferdams

- This paper discusses the structural analysis and design of precast concrete cofferdams through all construction stages using one example and brings attention to the innovative and successful use of precast concrete cofferdams.

- Design of the precast concrete cofferdam segments accounted for four different loading conditions: lifting and lowering while suspended from above; being supported by piles on the four corners; resisting lateral pressure outward from concrete placement around the bottom perimeter; and resisting the water pressing inward once the water inside the segment had been pumped out.

- A full three-dimensional finite element model built to capture the behavior of the precast concrete cofferdams was analyzed for all construction stages, with approximate hand checks being performed where possible to verify the model.

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Cofferdams are traditionally constructed with steel. Several projects in the United States, including bridges where the cofferdam provided dry space for bridge pier foundation construction[1,2] and locks where the cofferdam provided dry space for placement of mass concrete, however, have successfully been completed with precast concrete cofferdams. Analysis and design of concrete cofferdams, however, are not extensive in the literature. The goal of this paper is to discuss the structural analysis and design of precast concrete cofferdams through all construction stages using one example and bring attention to the innovative and successful use of precast concrete cofferdams.

This paper summarizes structural analysis and design for each stage of the concrete cofferdam during the construction of the new lock at the Kentucky Dam on the Tennessee River near Paducah, Ky. Also, an adequate crack control approach for concrete cofferdams is discussed, reviewing several U.S. codes and approaches.

Overview of the project

The new navigation lock at Kentucky Dam on the Tennessee River near Paducah, Ky., is currently under construction and will reduce the significant bottleneck that the current 183 m (600 ft) long lock causes on this important waterway. Because of high Tennessee River traffic levels and the current lock's size, the delay times for commercial tows going through Kentucky Lock average from 8 to 10 hours, nearly the longest delay in the country.[3]

The focus of this paper is the construction of the downstream cofferdam (**Fig. 1**), which is owned by the U.S. Army Corps of Engineers.

Ten unsymmetrical concrete cofferdams, approximately 15 × 15 m (50 × 50 ft) wide by 11 m (35 ft) high with 300 mm (12 in.) thick walls were cast on a casting barge several hundred meters away from the cofferdam's final location in the river (**Fig. 2**).

The casting barge was towed to the cofferdam's final location in the river, below the gantry barge (**Fig. 3**). The concrete cofferdam had eight cast-in post-tensioning bars, which were then connected to the spreader beam system of the gantry barge. The spreader beam system and the cofferdam were lifted by gantry-barge strand jacks, the casting barge was towed out, and finally the cofferdam was lowered down onto the riverbed. The riverbed consists of rock (limestone very close to the ground surface) with approximately level elevation. Once the riverbed was cleaned and the rock was exposed, tremie fill was placed locally to further level the river bedrock elevation. After the cofferdam was set in the water on the piles, concrete inside the cofferdam was placed. Ten cofferdams were placed next to each other, and cast-in-place concrete portions were placed on the top of each cofferdam (**Fig. 4**).

When the monolithic wall was completed, cellular steel cofferdams were continued at the end of the monolithic wall. Cellular cofferdams together with the precast concrete cofferdams constituted the downstream cofferdam. The downstream cofferdam then made it possible to excavate and then construct the new lock in dry conditions. The monolithic wall made by the precast concrete cofferdams will be part of the permanent wall of the lock. The precast concrete cofferdams have cast-in wall armors for the purpose of the final lock structure.

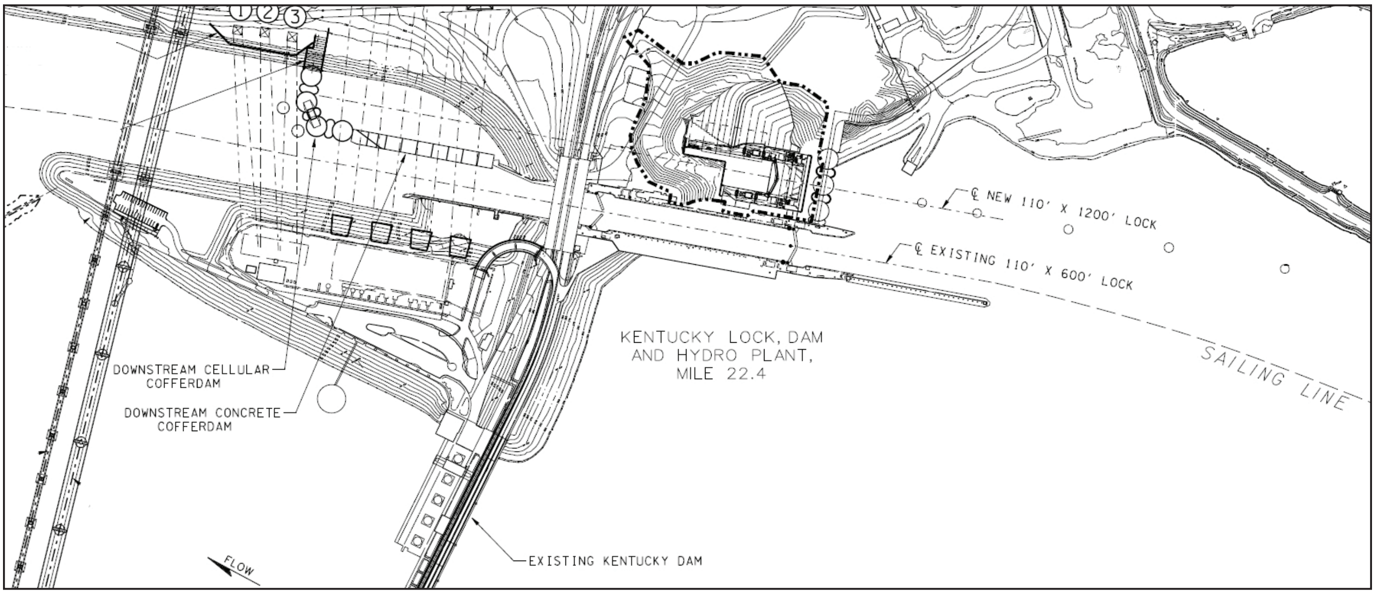

Figure 1. Layout of the Kentucky Lock addition project. Note: 1′ = 1 ft = 0.305 m.

Figure 2. Casting of the precast concrete cofferdam on the casting barge. Photo courtesy of Johnson Brothers Southland Holdings.

Figure 3. Towing the barge with the precast concrete cofferdam to the final location. Reproduced by permission from the U.S. Army Corps of Engineers.

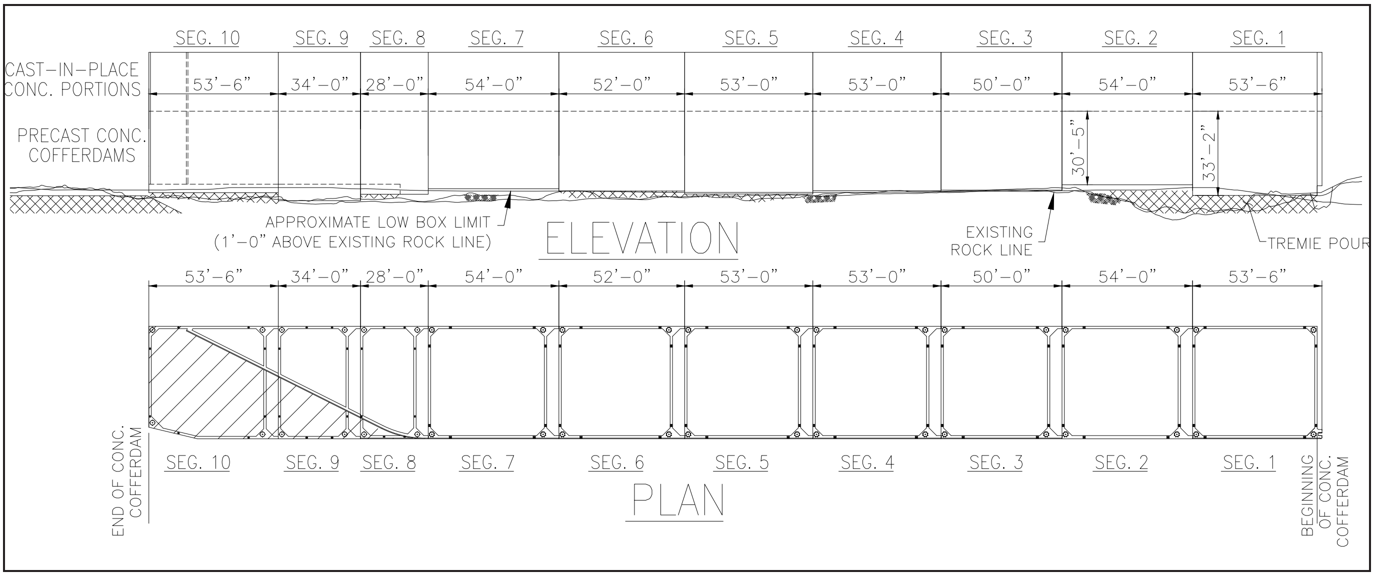

Figure 4. Plan and elevation views of all precast concrete cofferdam segments. Note: The temporary part of the monolithic wall is hatched. CONC. = concrete; SEG. = segment. 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.305 m.

Construction stages of the precast concrete cofferdams

In a structural analysis sense, precast concrete cofferdams had four different stages during construction:

1. Lifting and lowering the cofferdam: The box had eight cast-in debonded post-tensioning bars from the box bottom elevation up to the spreader beam system above the box. These post-tensioning bars were connected to the spreader beam system, which in turn was carried by two towers on the gantry barge. The cofferdam was slowly lifted and lowered into the river (**Fig. 5**).

2. The cofferdam setup on piles: When the box was approximately 300 mm (12 in.) away from the riverbed and still hanging on eight post-tensioning bars, steel pipe piles in four corners were dropped down through cast-in steel pipe sleeves and the load was slowly transferred from the post-tensioning bars to the piles.

3. Concrete placement inside the cofferdam: When the cofferdam was sitting on the piles, the concrete seal was plugged around the cofferdam's bottom perimeter. Once the concrete plug hardened, the box was sitting on the riverbed (**Fig. 6**) and the first concrete lift was placed as underwater tremie concrete. These concrete placements inside the cofferdam created outward lateral pressure on the box.

4. Water pressure (inward): Once the tremie concrete hardened, the water from the cofferdam was pumped out and a dry condition existed in the cofferdam. This stage created large inward lateral pressure from water outside of the

Figure 5. Lifting the precast concrete cofferdam. Reproduced by permission from the U.S. Army Corps of Engineers.

Figure 6. Precast concrete cofferdam placement in the river. Reproduced by permission from the U.S. Army Corps of Engineers.

box. A special strut-and-wale system inside the precast concrete segment was designed for this stage.

The focus of this paper is on the structural analysis of precast concrete cofferdams, keeping in mind their constructibility through all stages. For performing structural analysis of precast concrete cofferdams, it is essential to carefully follow the construction sequence.

Approach for evaluating the capacity of the concrete sections

The main goal of the cofferdam design was that the cofferdam concrete sections would stay impermeable so that once the inside water was pumped out, outside water would not leak in and the dry condition inside the cofferdam would exist for the placement of the higher-quality concrete.

The main criterion for the concrete cofferdam impermeability was that the width of through cracks must be less than a chosen maximum. Strictly speaking, the total width (at the level of the reinforcement) of a flexural crack is the difference in the elongation of the steel reinforcement and the concrete over a length equal to the crack spacing. The crack spacing and the variation in the steel and concrete strains are difficult to compute, and empirical equations are generally used to compute the crack width. The best known of these are the Euro-International Concrete Committee procedure and the Gergely-Lutz equation, derived statistically from a number of tests.[4] Crack width, however, is inherently subject to wide scatter, even in careful laboratory work.[5,6] Crack width can vary by an order of magnitude in the constant moment region of one test specimen.[7] Therefore, careful attention should be paid to crack control design, and further review of crack control approaches was performed.

The following U.S. codes and approaches were evaluated and are discussed herein in order to determine the most suitable method for the design of precast concrete cofferdams:

- *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*, ACI 318-14[6]
- *AASHTO LRFD Bridge Design Specifications*[8]
- *Code Requirements for Environmental Engineering Concrete Structures and Commentary*, ACI 350-06[5]
- *Control of Cracking in Concrete Structures*, ACI 224R-01[9]
- Sozen et al.[7]

Prior to 1999, ACI 318 was based on a form of the Gergely-Lutz expression corresponding to a limiting crack width of 0.41 and 0.33 mm (0.016 and 0.013 in.) for interior and exterior exposures, respectively (though the approach did not emphasize crack width itself). After 1999, ACI 318 took a different approach and considers crack widths indirectly by limiting the bar spacings s to be smaller than the following value:

$$s = \min\left(15 \times \frac{40{,}000}{f_s} - 2.5 \times c_{cl}, 12 \times \frac{40{,}000}{f_s}\right)$$

where

f_s = reinforcement steel stress

c_{cl} = clear concrete cover

The previous equation was obtained by fitting a straight line to the Gergely-Lutz equation for a flexural crack width of 0.41 mm (0.016 in.).[4]

The AASHTO LRFD specifications also control flexural cracking by limiting the spacing of reinforcement s to the following equation (rather than directly calculating the crack width).

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_{ss}} - 2 \times d_c$$

where

γ_e = exposure factor

β_s = ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face = 1 + d_c/0.7 × (h – d_c)

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto

h = overall thickness or depth of the component

f_{ss} = calculated tensile stress in nonprestressed reinforcement at the service limit state not to exceed $0.60f_y$

f_y = yield strength of reinforcing bars

This equation, however, is based on a physical model,[10] rather than the statistically based model used in previous editions of the AASHTO LRFD specifications, which used an approach similar to the pre-1999 ACI 318 approach.

Concrete cofferdam design, however, should have a more direct and stringent approach than the regular-structure serviceability requirements. ACI 350-06 states that crack width in environmental structures is highly variable but also reports that extensive laboratory work has confirmed that crack width at service loads is proportional to steel reinforcement stress. Further, it is specified that the maximum calculated stress $f_{s,max}$ in reinforcement closest to the surface in tension at service loads in normal environmental exposure areas must not exceed the limit calculated by the following:

$$f_{s,\max} = \frac{320}{\beta \times \sqrt{s^2 + 4 \times \left(2 + \frac{d_b}{2}\right)^2}}$$

where

β = ratio of distances to the neutral axis from the extreme tension fiber and from the centroid of the main reinforcement

d_b = nominal bar diameter

Further, the maximum calculated stress $f_{s,max}$ shall not exceed a maximum of 248 MPa (36,000 psi) and need not be less than 138 MPa (20,000 psi) for one-way and 165 MPa (24,000 psi) for two-way members.

The same document also reports that many structures designed by working stress methods and with low steel stress served their intended functions with very limited flexural cracking. When high-strength reinforcing steels are used at high service load stresses, however, visible cracks must be expected and steps must be taken in detailing the reinforcement to control cracking.

ACI 224R-01 gives quantitative values as a guide to reasonable crack widths in reinforced concrete structures under service loads. For water-retaining structures, a crack width of 0.1 mm (0.004 in.) is recommended. It is further stated that a portion of the cracks should be expected to exceed these values. These quantitative values are given as general guidelines for design to be used in conjunction with sound engineering judgment and are based primarily on Nawy,[11] who compiled information from several sources.

Sozen et al.,[7] due to high scatter of crack width, recommend a simple method for estimating the mean crack width based on test observations, and conclude that the main design parameters affecting crack widths are stress in the reinforcement and cover thickness.[7] The strain in the steel is assumed constant along the bar and is calculated first. The mean crack spacing was assumed to be two times the cover by referring to the experimental study by Broms.[12] The mean crack width Cr_{w_mean} is finally expressed by the following equation:

$$Cr_{w\_mean} = s_{mean} \times \varepsilon_s = \left(2 \times c_{cl}\right)\left(\frac{f_s}{E_s}\right)$$

where

s_{mean} = mean crack spacing

ε_s = reinforcement steel strain

E_s = reinforcement steel modulus of elasticity

It was also stated that the maximum crack width is likely to be less than twice the mean crack width.

After reviewing the current crack control practice, it was considered that the general crack control principles are as follows:

- It is better to have a larger number of smaller bars at smaller spacing than a smaller number of larger bars on larger distances.
- Smaller cover usually means smaller crack width.
- Smaller stress in reinforcement leads to smaller crack width.

Finally, two capacity checks were chosen to be performed for all concrete sections in each construction stage in the cofferdam:

- limiting stress in the reinforcement
- evaluating approximate mean crack width

The maximum allowable stress in the reinforcement for service unfactored load was taken according to the equation given by ACI 350-06, but an additional upper stress limit of 165 MPa (24,000 psi) was also used. Therefore, the final expression used to limit the steel stress can be symbolically expressed as follows:

$$f_{s,\max} = \max\left(20ksi, \min\left(\frac{320}{\beta \times \sqrt{s^2 + 4 \times \left(2 + \frac{d_b}{2}\right)^2}}, 24ksi\right)\right)$$

An approximate value of β of 1.2 was used. Concrete cover was defined by the owner as a minimum of 51 mm (2 in.) for this project.

Further, the approximate average crack width Cr_{w_mean} was estimated by the method proposed by Sozen et al.,[7] and it was compared with the recommended value from ACI 224-01:

$Cr_{w\_mean} \le 0.1$ mm (0.004 in.)

Given the previously mentioned variability of crack widths, this relatively simple approach seems acceptable for the design of precast concrete cofferdams. The two previously described limits (on the stress in the reinforcement and on approximate mean crack width calculated by the simple method proposed by Sozen et al.[7]) proved to be effective during the construction of the new lock at the Kentucky Dam. The construction of the cofferdam and upper cast-in-place portion is completed. The entire lock, however, is still under construction as of this paper release date.

Finite element analysis

A full three-dimensional finite element model was built to capture the behavior of the precast concrete cofferdams

(**Fig. 7**). Analysis was performed for all construction stages previously described. Approximate hand checks were done wherever possible to verify the model as the finite element method, in general, is easily "used and abused."[13]

Construction stage 1: Lifting and lowering

Eight post-tensioning bars were cast in the cofferdam for lifting and lowering purposes. Several different anchorage options were considered for this stage, and eventually the one shown in **Fig. 8** was chosen.

Post-tensioning bars were cast along the entire height of the box, and the anchor plate for post-tensioning bars was located at the bottom of the box. The end of the post-tensioning bars was cast flush with the bottom of the box for ease of construction. Because the anchor plate at the end of the post-tensioning bar was at the box bottom elevation, this avoided the lower part of the cofferdam being in tension during lifting.

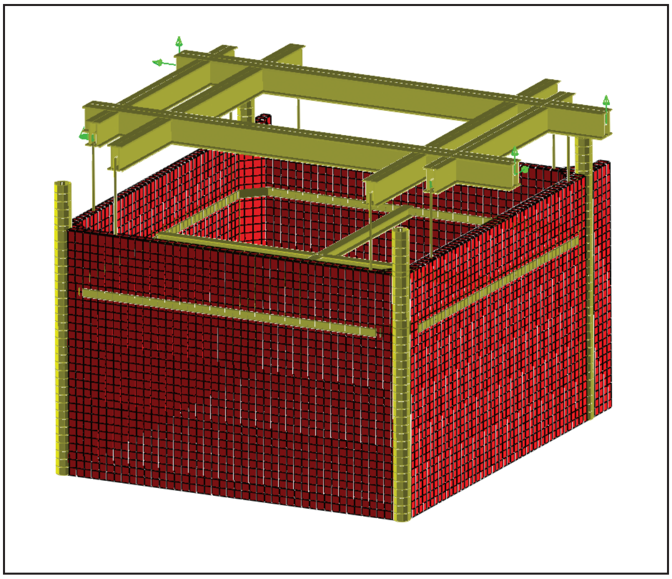

Figure 7. Finite element model of precast concrete cofferdam during the lifting stage (stage 1).

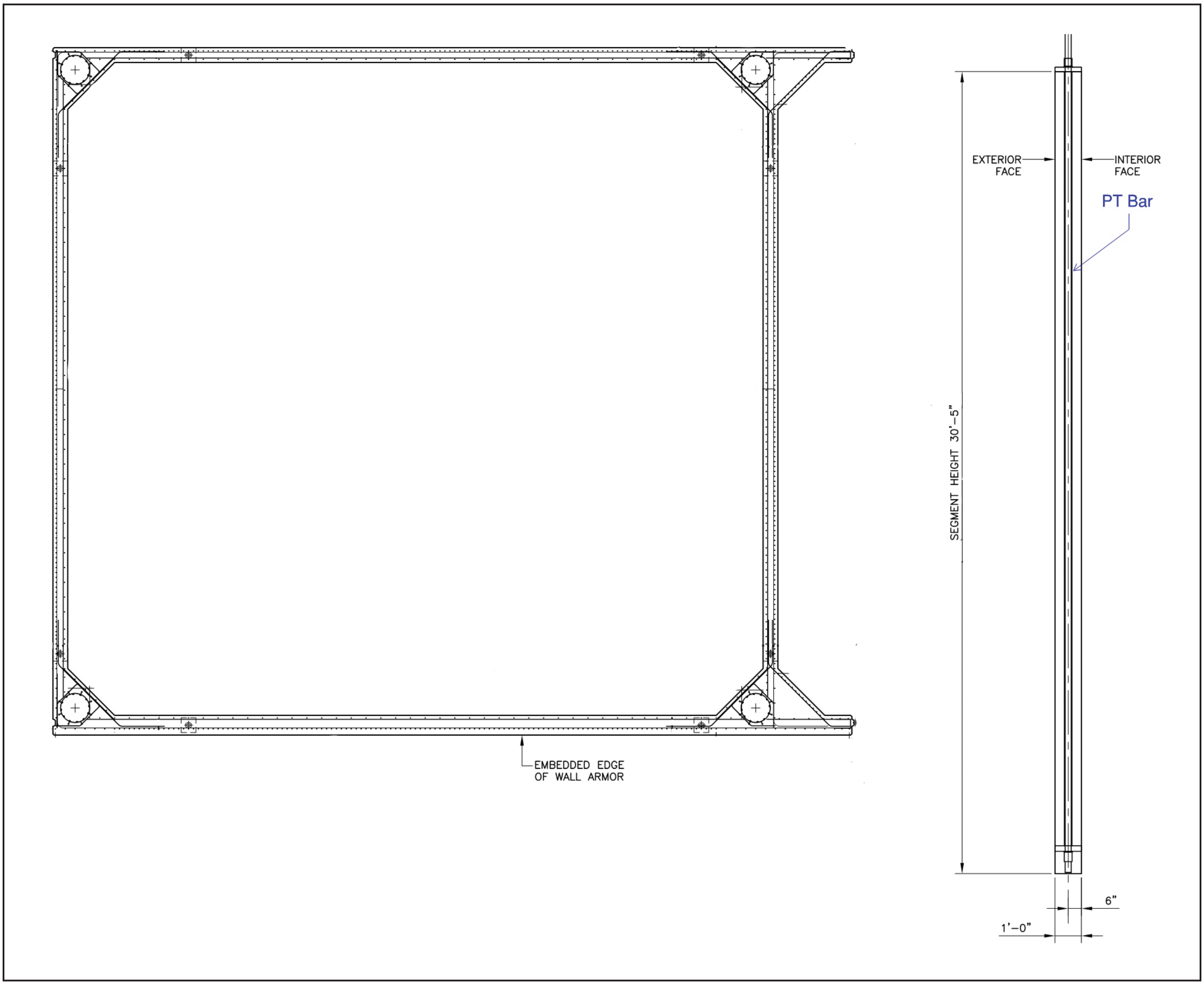

Figure 8. Plan and section view of the precast concrete cofferdam. Note: PT = post-tensioning. 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.305 m.

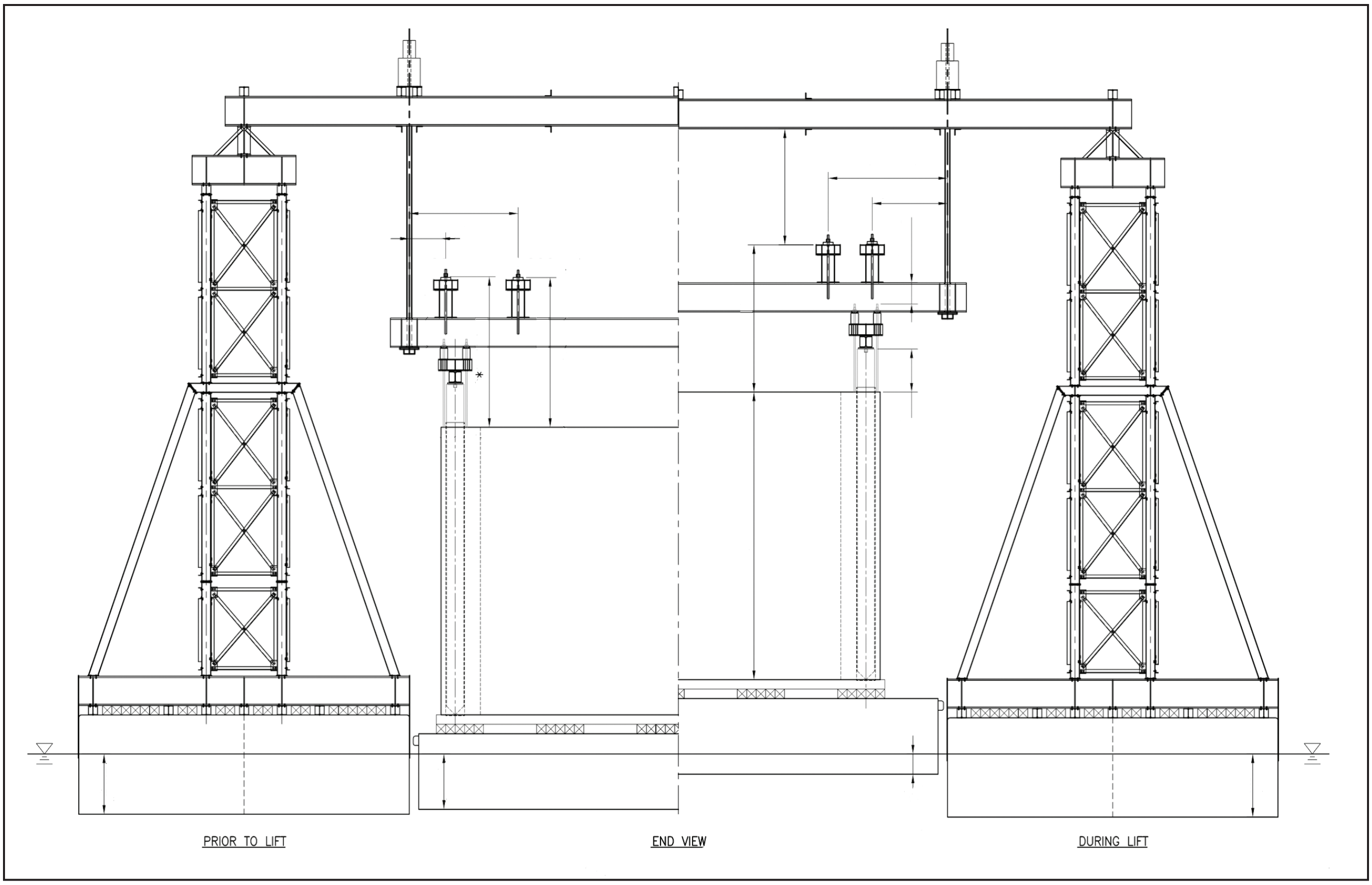

Figure 9. Gantry barge, casting barge, and precast concrete cofferdam. Note: PT = post-tensioning. 1″ = 1 in. = 25.4 mm.

Therefore, in the chosen configuration, the cofferdam structural system is similar to a concrete box or walls sitting on eight columns. To ensure this structural system where the load from the concrete box was transferred to the post-tensioning bars at the bottom elevation of the cofferdam (at anchor plates), the post-tensioning bars were debonded by a debonding layer along the full height. This eliminated any potential bond issues. The load path continued from post-tensioning bars to interior and exterior spreader beams and then to the main spreader beams. The main spreader beams were lifted by strand jacks located at the gantry beams and on top of the gantry towers and built on the gantry barge. Barge stability control was performed by changing the water level in the gantry barge chambers before, during, and after picking up the cofferdam.

The cofferdam was centered below the lifting assembly on the barge in two steps. The first step was to ensure the cofferdam weight taken by each post-tensioning bar was distributed equally. The outside post-tensioning bars (the ones closer to the gantry barge in **Fig. 9**) were computed to take significantly more load than the inside post-tensioning bars. One of the reasons is that the tips of inner beams deflect much more than the outer beam tips due to larger overhangs and deflection of the main beam. In order to make the force distribution among post-tensioning bars more equal, the nuts at the top of the outside post-tensioning bars (at the connection with the spreader beams) were turned off 25.4 mm (1 in.) away from the spreader beams such that the cofferdam was picked up first with the inside post-tensioning bars; thus, a load shift from outside to inside bars occurred. As a result, in the final configuration specified in the plans, the forces in the post-tensioning bars were almost equalized. Turning the nut off was modeled in finite element analysis as a spring that activates after an initial 25.4 mm free displacement. The spring stiffness is equal to the axial stiffness of the post-tensioning bar. Post-tensioning bars in the finite element model were connected to the concrete box at the box bottom at the anchor plate location.

The second step of stage 1 was to locate the entire box below the gantry beams such that the cofferdam centroid matched the gantry barge centroid. As a result, the reactions of the spreader beams in the finite element model (representing four strand jack reactions) were almost equalized and the strand jack reactions were measured in the field within 5% of the estimated values. The centroid match should be done with great precision because the centroid offset will make one box side or corner tilt up (which may cause construction tolerance and placement problems) if four strand jack forces are the same. The centroids will tend to match, causing the box to rotate. Due to cubelike geometry, the tilt (uneven displacement) at the corner of the box will be of the same order of magnitude as the centroid offset. Calculation of actual densities for significantly different reinforced concrete elements (instead of the usual 25 kN/m^3 [150 lb/ft^3] value) is recommended. If geometric restraints do not allow the box centroid to be positioned at the gantry barge centroid, an accurate estimate of the lifting strand jack forces (different in this case, with

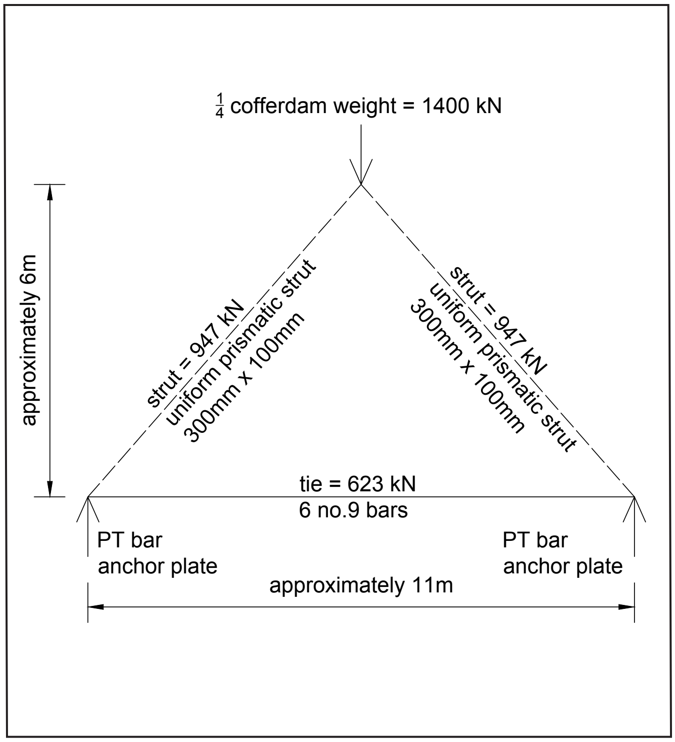

Figure 10. An approximate strut-and-tie model of one cofferdam wall. Note: PT = post-tensioning. No. 9 = 29M; 1 mm = 0.0394 in.; 1 m = 3.281 ft; 1 kN = 0.225 kip.

jacks closer to the centroid of the box pulling harder) should be provided to the erector to avoid tilting the box.

The elastic stress field obtained by finite element analysis exists only prior to any cracking. Cracking disrupts this stress field, causing a major reorientation of the internal forces. Therefore, an approximate hand check of the concrete cofferdam finite element analysis, in this stage, was done by a strut-and-tie model (**Fig. 10**). As clearly seen from **Fig. 11**, tension ties exist between post-tensioning bar anchor plates and compression fans originate from them. It was assumed that adequate anchorage for steel ties was provided because the steel reinforcement at the location of the ties was continuously present around the structure. Nodal zones were considered properly reinforced by special confinement reinforcement, discussed later in this paper (**Fig. 12**). The contribution of adjacent cofferdam walls was ignored and was assumed only to provide lateral stability for the wall in question. Strut-and-tie forces corresponded adequately to the finite element analysis results.

Special attention was given to the design of the confinement reinforcement above the post-tensioning bar anchor plate. The starting point was to provide enough reinforcement to resist bursting forces T_{burst}, present in both directions perpendicular to the applied upward concentrated tension post-tensioning bar force, based on the following equation in ACI 318-14:

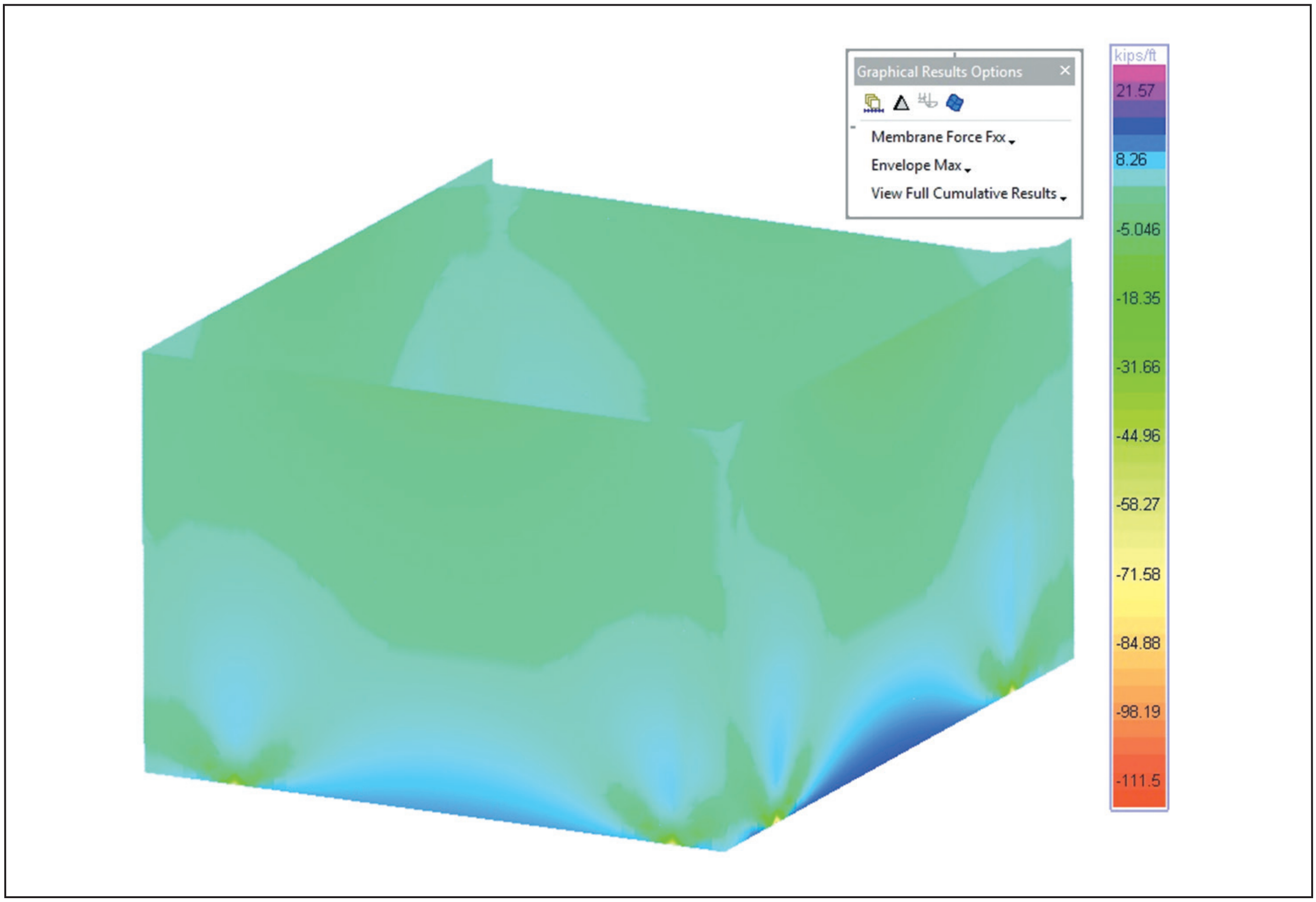

Figure 11. F_{xx} diagram of the finite element model during stage 1. Note: F_{xx} = axial force in plate elements in horizontal direction; max = maximum. 1 kip/ft = 14.593 kN/m.

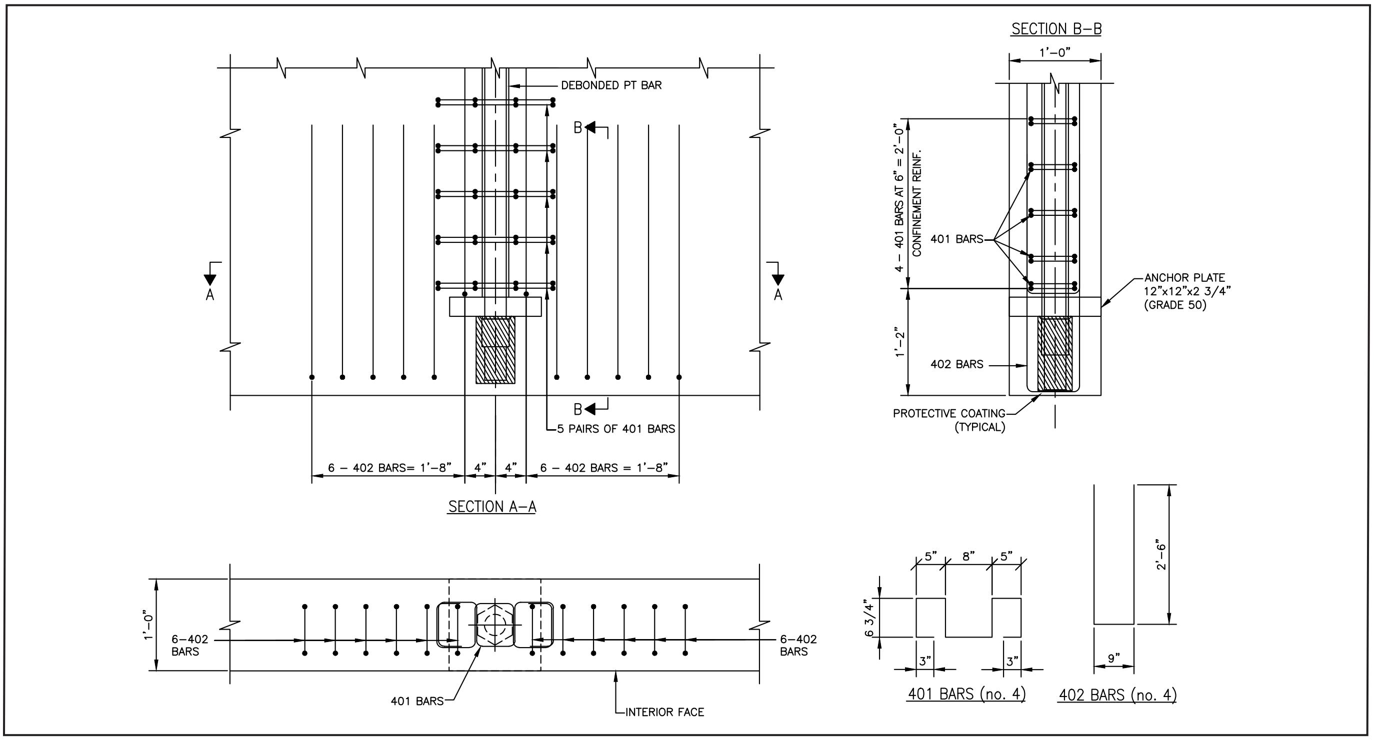

Figure 12. Confinement reinforcement above the post-tensioning bar anchor plate. Note: The main cofferdam reinforcement is not shown. PT = post-tensioning. No. 4 = 13M; 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.305 m.

$$T_{burst} \cong 0.25 \times PT_{bar\_max} \qquad \text{(ACI 25.9.4.4.2)}$$

where

PT_{bar_max} = maximum force in the lifting post-tensioning bars

This value of 0.25 can also be seen as a Poisson's ratio for applied axial compression force. The reinforcement layout was then further developed based on successful past practice. In addition, applicable concrete anchor design checks were performed according to ACI 318-14 chapter 17. The controlling limit state was side-face blowout.

Construction stage 2: Setting up the box on piles

Once the cofferdam was lowered into the river (still hanging on eight post-tensioning bars) and the bottom of the box was approximately 300 mm (12 in.) away from the riverbed or prepared tremie fill, the steel pipe piles, which were previously inserted on the casting barge inside the cast-in steel pipe sleeves in four corners of the cofferdam, were dropped down through the pipe sleeves and the load was slowly transferred from the post-tensioning bars to the piles.

The load path in this stage went from the box weight to the cast-in pipe sleeve through the activation of the shear studs on the pipe sleeve surface. This is very similar to a reinforcement pull-out test from a concrete block. Pipe sleeves by post-tensioning bars on their tops tend to be pulled out from the concrete cofferdam on the top of the box. The slope in the axial (tension) force of the pipe sleeve represented the bond transfer between the pipe sleeve and concrete. The pipe-sleeve beam elements shared the same nodes with the concrete plate elements of the cofferdam in the finite element model to capture the actual behavior (the pipe pile was not included in the model). Shear studs per unit length need to be able to transfer a shear force equal to the difference between the axial force at the end and beginning of the unit length. Most of the bond was transferred at the top of the

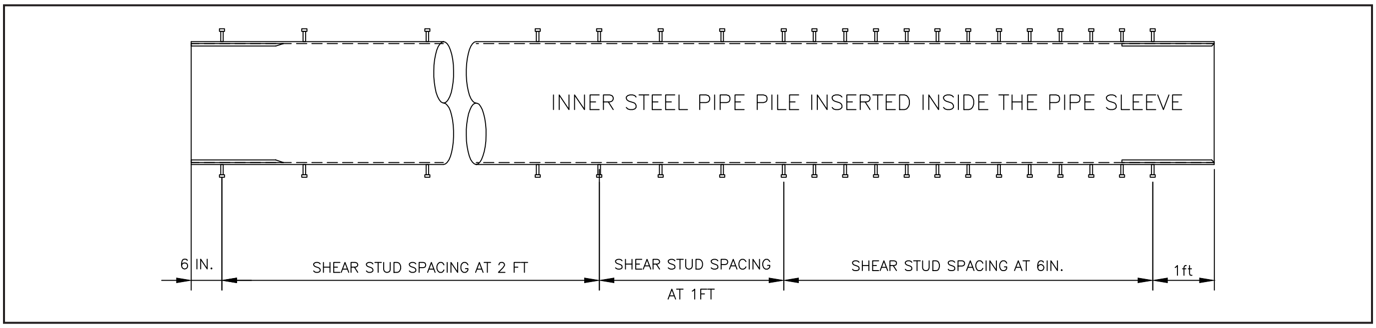

Figure 13. Pipe sleeve with shear studs.

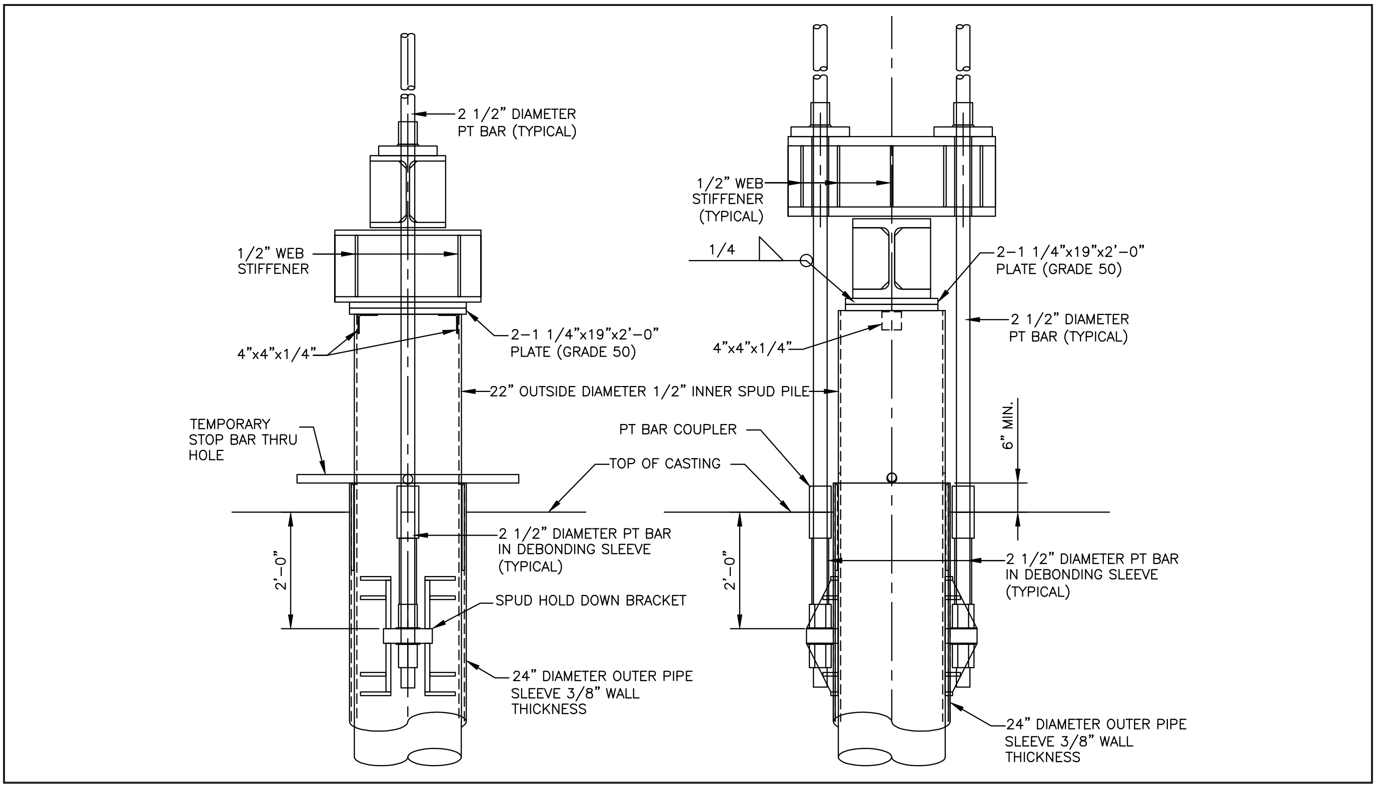

Figure 14. Top pile connection. Note: PT = post tensioning. 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.305 m.

pipe sleeve. Therefore, the shear studs were distributed more densely in the upper part of the pipe sleeve (**Fig. 13**). The cast-in pipe sleeve was, accordingly, in tension. Through the connection between the top of the outside pipe sleeve and the top of the inside pipe pile (**Fig. 14**), the load was transferred to the pipe piles, which in turn were in compression. The fact that reduced weight acted on the pipe sleeves and pipe piles because part of the cofferdam was submerged in the water was neglected due to uncertain water elevation.

The top pile connection consisted of the following elements, which are listed in order based on the load path:

1. bracket, welded onto the pipe sleeve
2. two post-tensioning bars per connection in tension
3. two I-shaped beams transferring the load to the pipe pile

The top pile connection was equipped with strand jacks, which served to level the final box elevation to a desired value with the post-tensioning bars. It was specified that in any moment, the differential elevation of the top pile corners was within 25.4 mm (1 in.). Therefore, finite element analyses were performed (**Fig. 15**) for stages when the box was hung

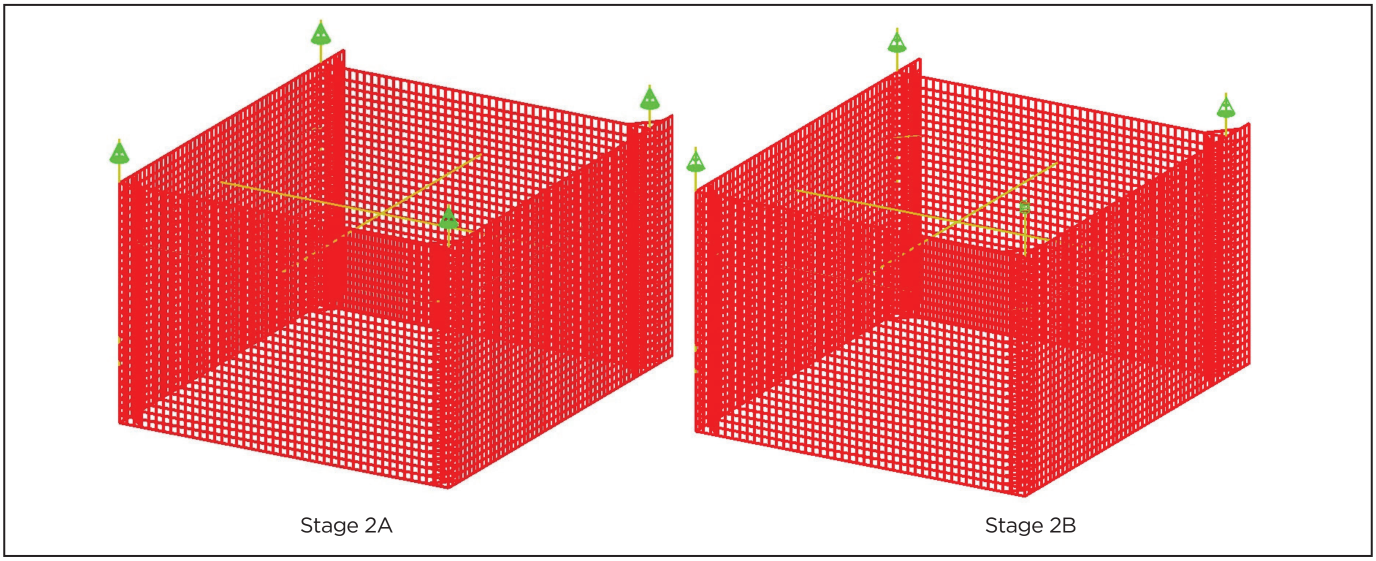

Figure 15. Finite-element models.

level on four cofferdam corners (stage 2A) and when one corner dropped down 25.4 mm (stage 2B). Unleveled displacement of one corner was modeled with a spring that allowed 25.4 mm of free movement before a stiff spring was activated (with spring stiffness being equal to axial stiffness of the pipe pile). The supports and spring in stage 2B represented the top pile connections.

Finite element analysis showed that in stage 2B, large bending moments M_x can be created due to twisting of the box when one corner is dropped down (**Fig. 16**). Analysis revealed that stage 2B was one of the controlling construction stages.

The bracket connection on the pipe sleeve was separately analyzed with the finite element method. The compression-only springs were used to represent the fact that if the pipe deformed outward, the compression force from concrete would act inward. The cohesion of concrete (tension for the spring) was neglected. *Specification for Structural Steel Buildings*[14] plate bending design checks were done for the pipe sleeve. Large local bending and buckling of the pipe sleeve (**Fig. 17**) were decreased by introducing the ribs welded onto the pipe sleeve.

Construction stage 3: Tremie concrete placement

Concrete sealing of the cofferdams was done by first placing the tremie curtain seal on the outside perimeter of the concrete box and then by pouring the tremie concrete under water, below the walls of the cofferdam. Once the concrete

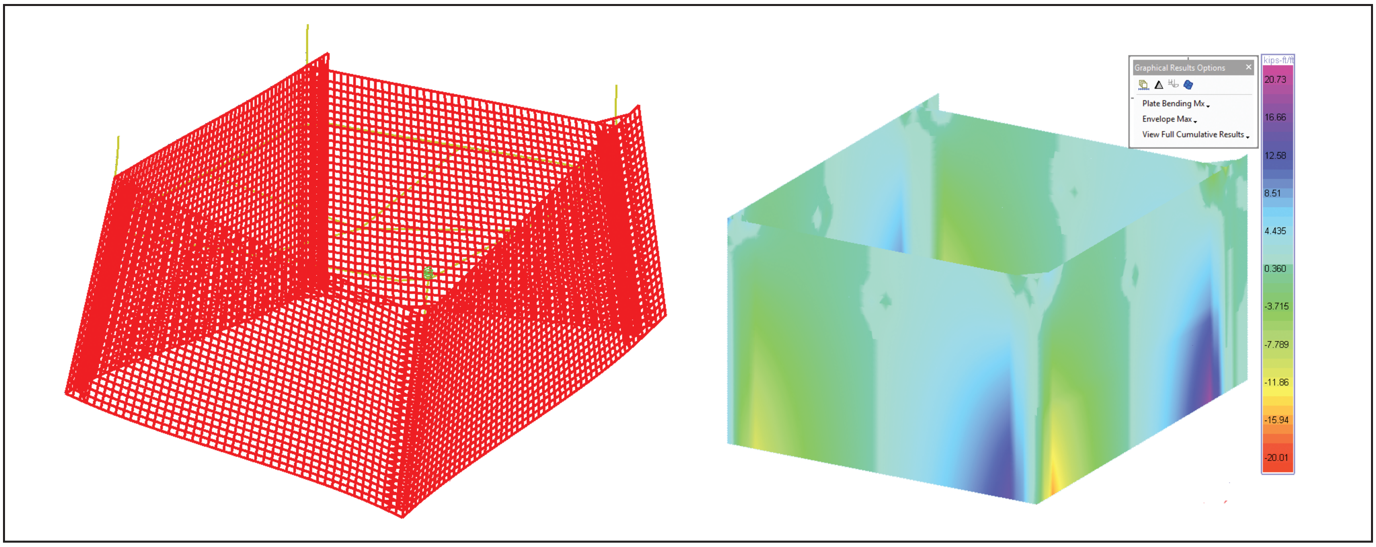

Figure 16. Deformed shape and M_x moment diagrams of the model for stage 2B. Note: max = maximum; M_x = bending moment in plate elements resisted by horizontal reinforcement. 1 kip-ft/ft = 1.365 kN-m/m.

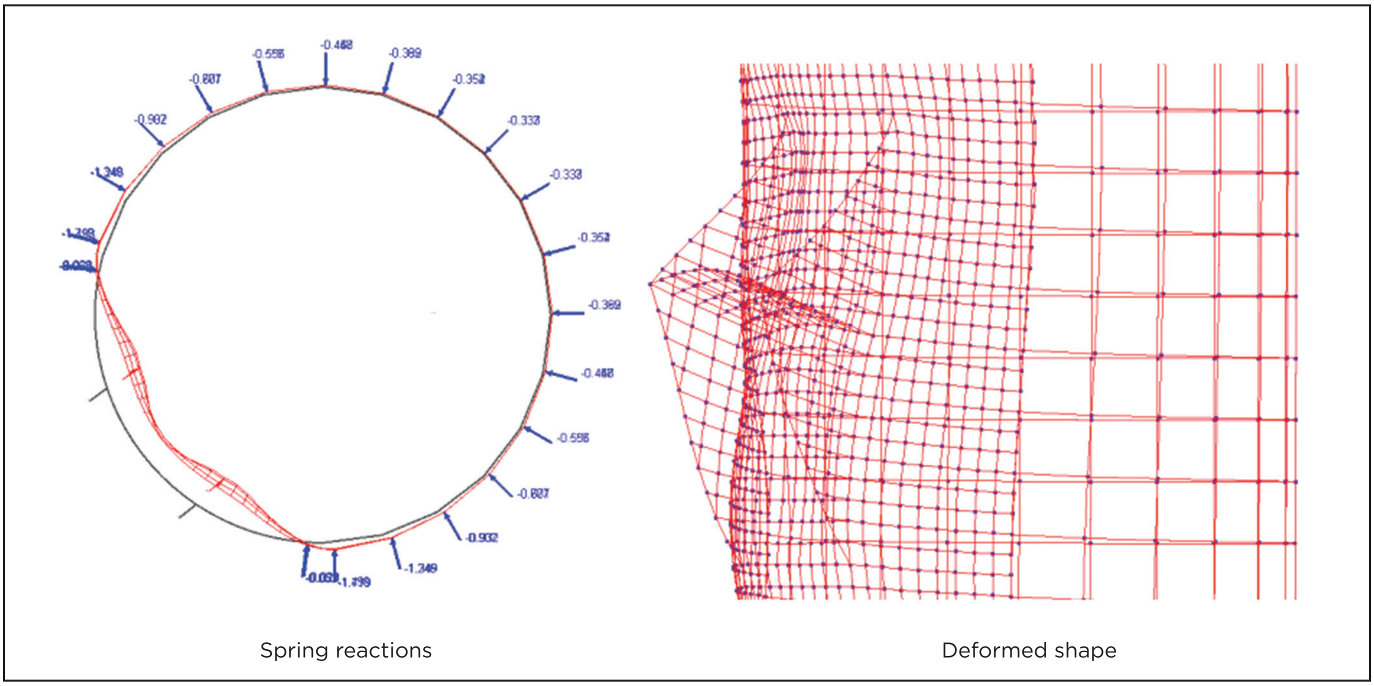

Figure 17. Spring reactions and deformed shape of the bracket on the outside pipe. Note: All units are in kip. 1 kip = 4.448 kN.

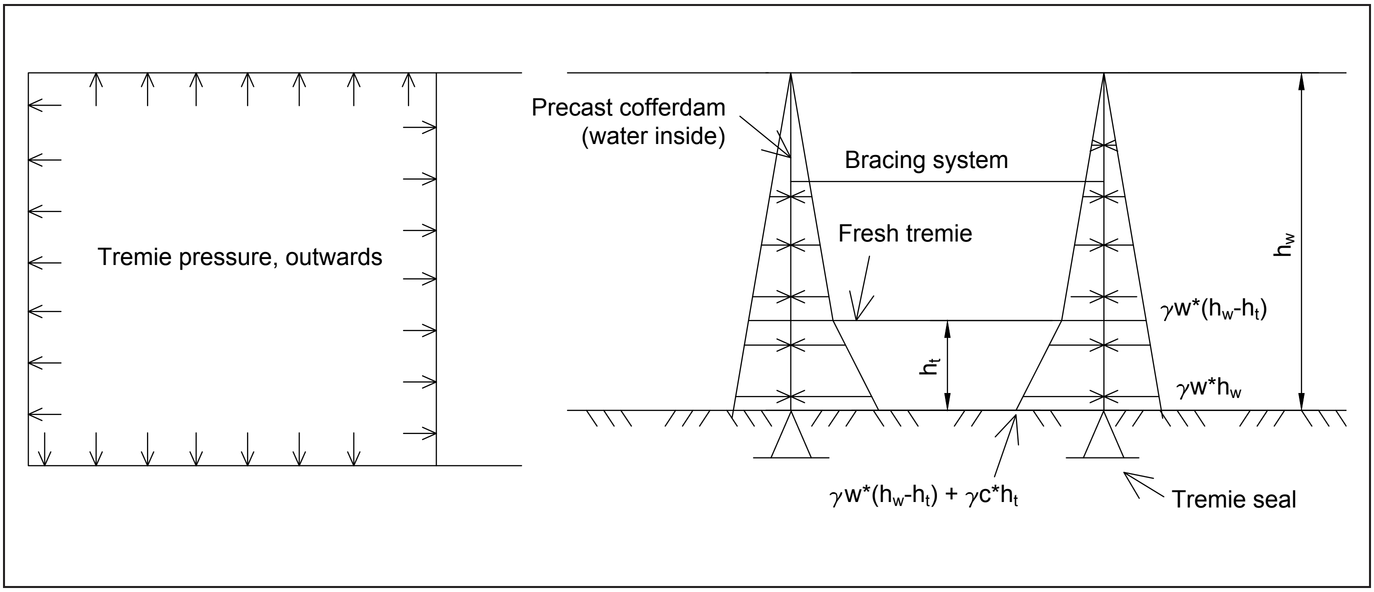

Figure 18. Schematic view of stage 3. Note: h_t = tremie height; h_w = water height; γ_c = concrete unit weight; γ_w = specific weight of the water.

plug hardened, the weight of the concrete box was no longer supported by the piles and the box then sat on the riverbed. Therefore, the cofferdam was simply supported at the box bottom elevation.

The next step was underwater placement of the tremie concrete. The fresh concrete created lateral (outward) pressure on the cofferdam and the maximum lift height in one pour was determined to be 3 m (10 ft) in order to control the structural flexural demand on the cofferdam walls (**Fig. 18**).

The total tremie pour height was also controlled by buoyancy analysis to prevent the cofferdam from moving away from its location once the water was pumped out (**Fig. 19**). Two options were proposed for the contractor:

- If some type of shear connectors between precast concrete cofferdam and tremie were provided, the total tremie height h_t would be determined based on the following equation:

  $$A_g \times h_w \times \gamma_w - W_{coff} < 0.9 \times W_{tre}$$

  where

  A_g = gross area of the cofferdam

  h_w = elevation difference between top of the water and bottom of the cofferdam

  γ_w = specific weight of the water

  W_{coff} = weight of the cofferdam

  W_{tre} = weight of the inside tremie concrete

- If shear connectors between precast concrete cofferdam and tremie were not provided, the tremie height h_t would be determined based on the following equation:

  $$A_{tre} \times h_w \times \gamma_w < 0.9 \times W_{tre}$$

  where

  A_{tre} = area of tremie concrete

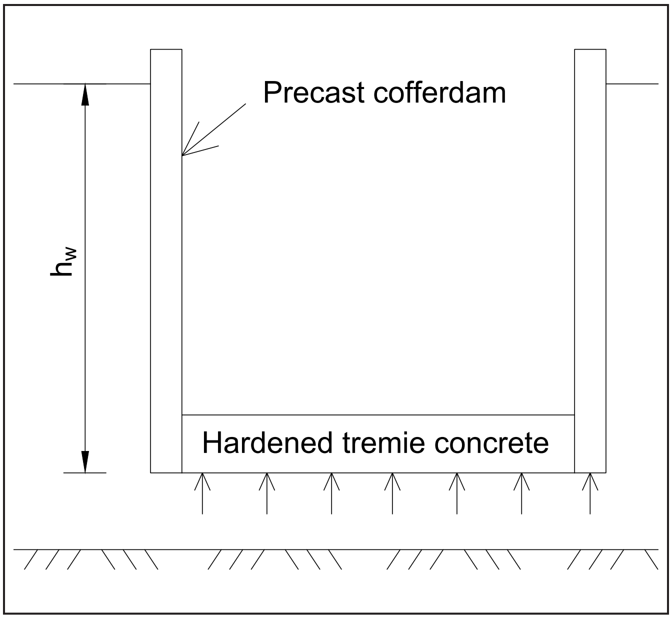

Figure 19. Buoyancy model of the cofferdam. Note: h_w = water height.

Graphs were provided for the required total tremie heights based on the actual water elevation at the time of construction. The maximum total tremie height found from the buoyancy analysis was also subject to the previous analysis for structural capacity of the box. Two tremie lifts were possible, each smaller than 3 m (10 ft).

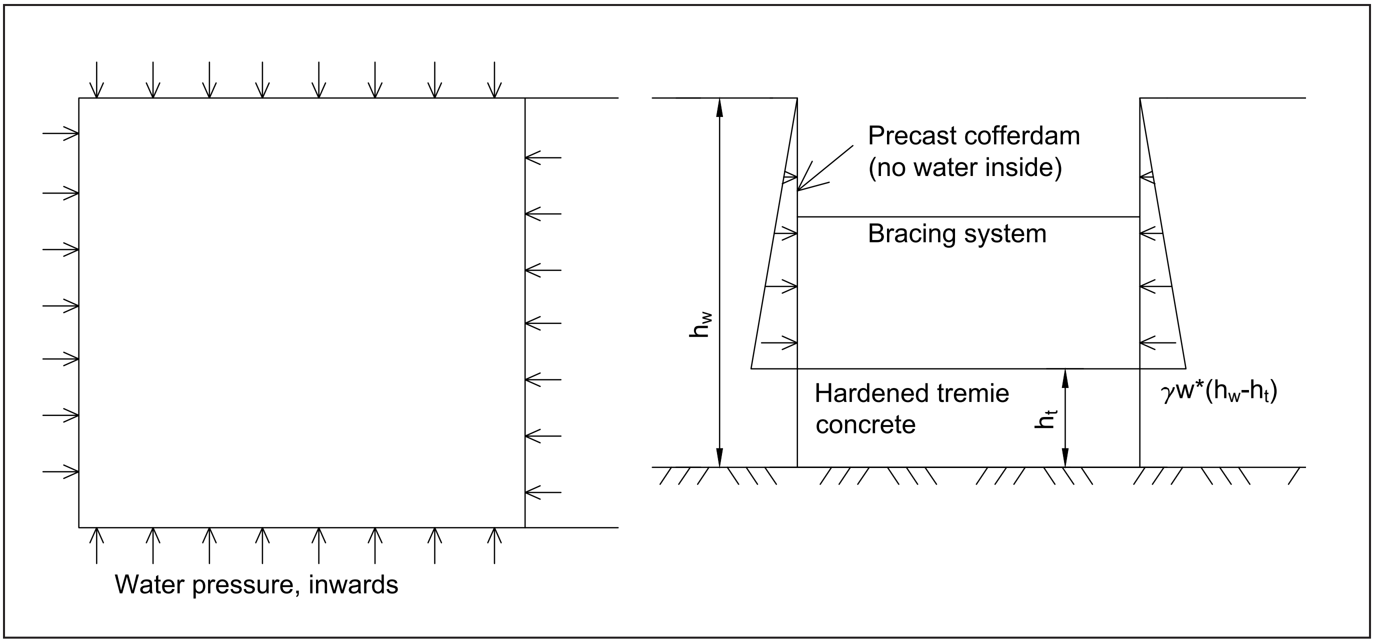

Figure 20. Schematic view of stage 4. Note: h_t = tremie height; h_w = water height; γ_w = specific weight of the water.

Construction stage 4: Outside lateral water pressure

Once the tremie concrete hardened, the water from inside the cofferdam was pumped out and dry conditions existed for the placement of higher-quality mass concrete. This stage was the main purpose of the cofferdam. Large lateral inward water pressure occurred (**Fig. 20**), creating large flexural demands in the concrete box.

To resist the lateral inward water pressure and decrease flexural demands in the cofferdam, a strut-and-wale bracing system was designed. It consisted of a W-section ring (a wale) and two perpendicular HP-section struts connecting the walls across from each other (**Fig. 21**). Based on past experience, the struts were designed for axial compression force (resisting lateral inward water pressure) together with a flexural demand taken as the larger of the following:

- self-weight moment plus 5% of the compressive force applied vertically in the middle of the strut to account for any unintended dead load, such as incidental fresh concrete during casting
- self-weight moment plus eccentricity moment taken as self-weight midspan deflection times the axial force in the strut

Figure 21. View from underneath the lifted cofferdam. Reproduced by permission from the U.S. Army Corps of Engineers.

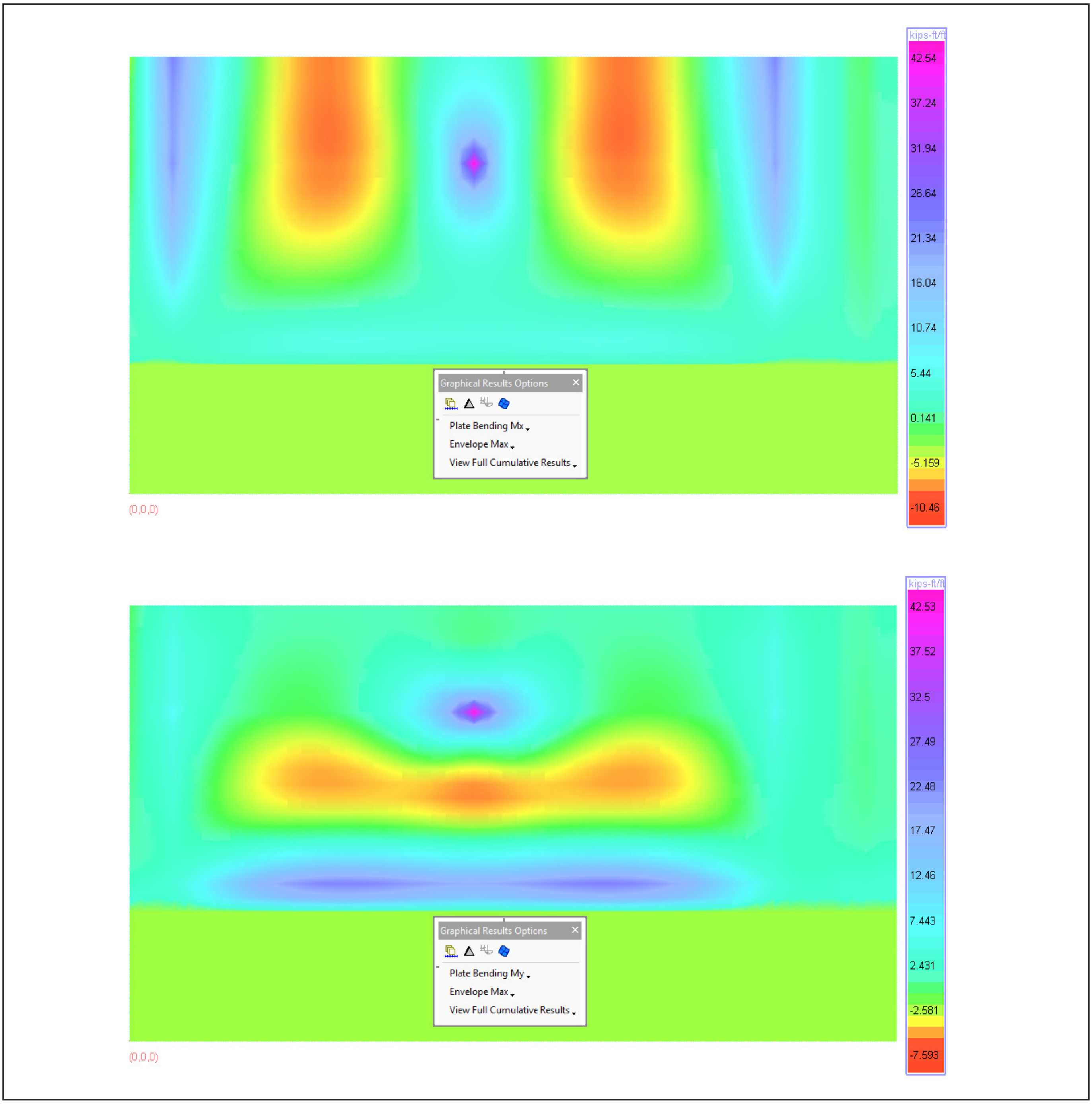

Figure 22. Mx and My moments in one cofferdam wall for stage 4. Note: max = maximum; Mx = bending moment in plate elements resisted by horizontal reinforcement; My = bending moment in plate elements resisted by vertical reinforcement. 1 kip-ft/ft = 1.365 kN-m/m.

The first case was the controlling one.

Even with the strut-and-wale bracing system, this stage created one of the largest demands in the cofferdam (**Fig. 22**). Nevertheless, 305 and 406 mm (12 and 16 in.) thick walls were designed following the previously described crack-control approach and were able to resist water-pressure demands. The cofferdams successfully served their purpose during the construction and no leakage was reported.

Once the inside concrete was poured just below the bracing system, the bracing was removed (**Fig. 23**). This stage was analyzed for fresh concrete lateral outward pressure (with no water inward pressure), as well as for inward water pressure alone. Maximum concrete pour lifts were limited based on this analysis.

Connection between temporary and permanent part of last three cofferdams

Only one part of the last three concrete cofferdams will form the eventual monolithic wall of the lock. The other part will be torn off once the entire downstream cofferdam is built in order to follow the final geometric layout of the lock.

Figure 23. First three cofferdams in place, partially filled with inside concrete. Photo courtesy of Johnson Brothers Southland Holdings.

The temporary part of the cofferdam is shown in Fig. 4 as the hatched area. A special connection between the temporary and permanent parts of the cofferdams was designed (**Fig. 24**) to make the final concrete tear off more easily and provide the minimum 51 mm (2 in.) cover throughout the permanent structure.

In the first placement, the permanent part of the cofferdam was cast together with polystyrene blockouts along the entire height of the cofferdam, except at wall armors. Reinforcement was made continuous through the permanent–temporary connection. Then, the first-pour forms were stripped, the polystyrene blockout was taken away, and a debonding layer was placed on the entire permanent–temporary connection. Finally, the second placement was cast. Once the temporary concrete is removed along the debonded layer, all protruding reinforcement will be cut at the depth of the blockout and the blockouts will be grouted with a minimum 51 mm (2 in.) of cover.

Assumptions for analysis and design were that sections were structurally continuous through the debonded layer. Tension and shear were transferred by continuous reinforcement and compression by concrete contact. Reinforcement crossing the debonded layer (minimum 29M at 150 mm [no. 9 at 6 in.] each face) was checked for shear demand as a structural steel section (allowable stress equal to $0.6 \times F_y$ divided by the factor of safety).

Conclusion

An approach for analysis and design for precast concrete cofferdams is presented using the example of the construction of the new lock at the Kentucky Dam. It can be concluded that precast concrete cofferdams represent an innovative and successful construction method with the following features and advantages:

- A wide variety of shapes can be made, as shown in the example of the last three cofferdams with temporary and permanent parts.
- Concrete can serve as future formwork and part of a permanent structure (containing mandatory wall armor elements in this case).
- Precast concrete is suitable for fast construction in the water for situations where rock is close to the surface.

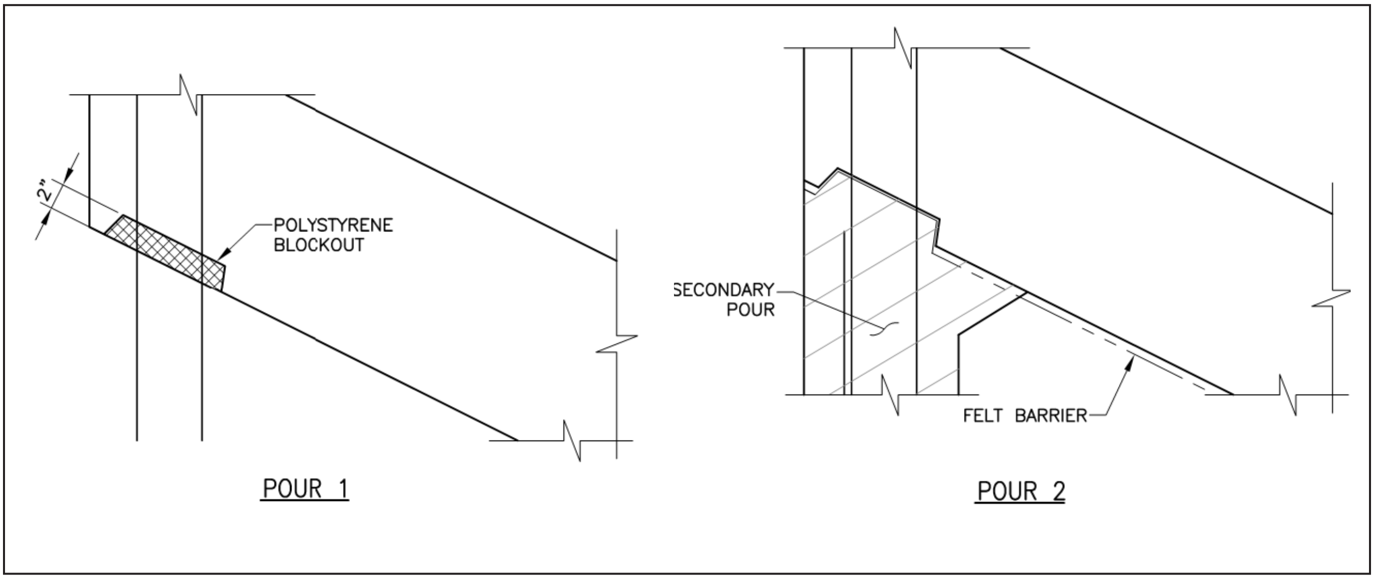

Figure 24. Connection between temporary and permanent part of the structure. Note: 1″ = 1 in. = 25.4 mm.

The main analysis and design focus for precast concrete cofferdams should include the following:

- a crack control approach
- confinement reinforcement for the purpose of erecting/lowering the cofferdam
- critical construction stages regarding the global demand in the concrete cofferdam, determined in this case to be erection (stage 1), temporary uneven settlement on the piles (stage 2B), and the stage when the inside water is pumped out and lateral water pressure inward exists (stage 4)

The solutions to these concerns presented in this paper were proved to be effective in practice. Similar precast concrete cofferdams, therefore, can be safely used for similar future endeavors.

Acknowledgments

Janssen & Spaans Engineering performed this calculation while in a design-build team with the contractor Johnson Brothers (Southland Holdings). The owner of the project is the U.S. Army Corps of Engineers, which provided valuable feedback during the design.

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Notation

A_g = gross area of the cofferdam

A_{tre} = area of tremie concrete

c_{cl} = clear concrete cover

Cr_{w_mean} = mean crack width

d_b = nominal bar diameter

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto

E_s = reinforcement steel modulus of elasticity

f_s = reinforcement steel stress

$f_{s,max}$ = maximum allowed stress in the steel at service level

f_{ss} = calculated tensile stress in nonprestressed reinforcement at the service limit state not to exceed $0.60f_y$

f_y = yield strength of reinforcing bars

F_{xx} = axial force in plate elements in horizontal direction

F_y = yield strength of the steel

h = overall thickness or depth of the component

h_t = tremie height

h_w = elevation difference between top of the water and bottom of the cofferdam

M_x = bending moment in plate elements resisted by horizontal reinforcement

M_y = bending moment in plate elements resisted by vertical reinforcement

PT_{bar_max} = maximum force in the lifting post-tensioning bars

s = spacing of deformed bars

s_{mean} = mean crack spacing

T_{burst} = bursting force in concrete as a result of applied concentrated force

W_{coff} = weight of the cofferdam

W_{tre} = weight of the inside tremie concrete

β = ratio of distances to the neutral axis from the extreme tension fiber and from the centroid of the main reinforcement

β_s = ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face

γ_c = unit weight of concrete

γ_e = exposure factor

γ_w = specific weight of the water

ε_s = reinforcement steel strain

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Abstract

This paper presents a methodology for analysis and design of precast concrete cofferdams using the example of the successful addition of a new lock at the Kentucky Dam on the Tennessee River near Paducah, Ky. Cofferdams are traditionally made from steel; however, several projects in the United States were successfully done in the past with precast concrete cofferdams. Nevertheless, analysis and design of precast concrete cofferdams are not extensively covered in the literature. For construction of the new lock at Kentucky Dam, finite element analyses and adequate capacity checks, including crack control, were performed for precast concrete cofferdams in all construction stages. Ten unsymmetrical, approximately 15 × 15 m (50 × 50 ft) wide by 11 m (35 ft) high cofferdams with 300 mm (12 in.) thick walls were placed next to each other in the river from a river barge. These precast concrete boxes will eventually create one monolithic wall (the future wall of the lock). It was concluded that precast concrete cofferdams represent an innovative and advantageous solution for future similar projects.

Keywords

Cofferdam, construction engineering, crack control, dam, lock.

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