French Modular Impoundment
Deliverable 6.4: Final Cost and Performance Evaluation
DOE Award: DE-EE0007244
Project Period: 12/01/15 – 12/31/16

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Disclaimer: Any findings, opinions, and conclusions or recommendations expressed in this report are those of the author(s) and do not necessarily reflect the views of the Department of Energy

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I. EXECUTIVE SUMMARY

This report comprises the Final Cost and Performance Report for the Department of Energy Award # EE0007244, the French Modular Impoundment (aka the "French Dam".) The French Dam is a system of applying precast modular construction to water control structures. The "French Dam" is a term used to cover the construction means/methods used to construct or rehabilitate dams, diversion structures, powerhouses, and other hydraulic structures which impound water and are covered under FDE's existing IP (Patents # US8414223B2; US9103084B2.)

It is well documented that our water infrastructure is facing a serious threat due to years of poor maintenance and underinvestment. The American Society of Civil Engineers ranks U.S. Dams a "D" (Poor) grade, and the American Society of Dam Safety Officials estimates the cost to replace just the high-hazard dams at $21B. We have seen the consequences of this failure to invest in October of 2015, when South Carolina experienced 20+ inches of rainfall over a 120 hour period. The event caused 36 dams to fail across the state, many in the Columbia suburbs area in the Gill Creek Watershed. Economists estimate the total damage of this flooding at approximately $12B. New solutions are needed to rapidly and cost-effectively rehabilitate water control infrastructure. Existing methods of dam construction and rehabilitation are simply too costly to pencil out for owners and operators, who are often unable to monetize the benefits provided by dams and diversion structures, which limits the revenue available to invest in rehabilitation.

FDE has been developing such a solution since 2009. In 40 years of heavy civil construction experience, I have seen the precast industry take the conventional construction industry by storm, as bridges, underwater tunnels, hospitals, prisons, and other structures are constructed in record time and ahead of budget. The quality of precast has proven superior to conventional mass-placed concrete, and is appropriate for projects at all levels of complexity. It is time to bring this proven technology to the dam and powerhouse construction industry, which this DOE award allowed our team to demonstrate. Through this award, we validated all aspects of the French Dam system, including segmental construction, interlocking features, underpinning support systems, and the ability to retain water without leaking. In addition, we developed best practices to apply precast construction means and methods to low/medium head dams, diversion structures and powerhouses, as well as dam rehabilitation. We have demonstrated cost reductions of 40% - 60% for precast when compared with Cast-in-Place concrete, and schedule reductions of 40%.

FDE has developed significant know-how in the field of precast dam design and construction, and has a team prepared to engage on project construction. Our role in commercial projects is to contribute this expertise to guide projects through to successful completion. However, as with any large-scale construction project, the project specifications, design, and construction sequencing should be performed by the design/build contractor, EPC firm, or other Construction Manager responsible to the project developer. Our team is deeply grateful to the U.S. Department of Energy for providing the seed funding to validate and commercialize our technology – and partnering with us to bring this technology offering to market to respond to America's infrastructure challenges.

Bill French, CEO
French Development Enterprises LLC

William S French Jr.

// II. ACKNOWLEDGEMENTS

FDE would like to acknowledge the following project participants for the critical role each played in helping this project achieve success:

Lenny Lozinsky, CFO, French Development Enterprises LLC
Peter Drown, President, Cleantech Analytics LLC
Lisa Kelley, Director of Marketing, French Development Enterprises LLC
Mike Walker, P.E., Vice President, GEI Consultants
Varoujan Hagopian, P.E., Senior Consultant, GEI Consultants
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Celeste Fay, President, Hydropower Consulting Specialists, LLC
Norm Bishop, P.E., Senior Vice President, Knight Piesold Consulting
Bill Scott, P.E., Chief Engineer, Maine Rock Drilling and Blasting
Bill Felton, Manager, Willis Towers Watson

III. PROJECT BACKGROUND

The first goal of this project was to Design, Manufacture and Test a Prototype Precast Modular Impoundment to demonstrate and de-risk the technology, accelerating the technology from TRL 3 through TRL 6. In parallel, we completed a design for a full-scale impoundment using this technology for an actual U.S. site, using site-specific parameters, and compare the resulting engineering and cost estimates with traditional dam construction methods. Key Project Objectives included:

(1) Design, manufacture and test a prototype Precast Modular Impoundment consisting of several modules with interlocking elements at Alden Test Facility
(2) Develop full engineering and cost/schedule reductions for baseline comparison using actual U.S. new hydropower site
(3) Complete dam safety evaluation and insurance consultation feedback
(4) Demonstrate scalability of proposed concept in heads of 10-50 feet

IV. DELIVERABLES BY SUBTASKS (3.1 – 6.4)

The following describes the completion of all Tasks in Budget Period 2. This consisted of the manufacture and testing of the French Dam prototype, and the delivery of a full-scale redesign and cost/schedule comparison of French Dam technology with conventional Cast in Place construction. **Deliverables for Tasks 1 and 2 are included in the Go/No-Go report in Appendix F**.

Subtask 3.1: Manufacture Precast Components (M8-M9)

Subtask Summary: Components were cast at Oldcastle's precast facility in Avon, Connecticut according to specifications from Subtask 3.1. The precast manufacturing process is 3 steps: (1) Prototype Design from Task 2 will be required to convert to manufacturing-specific drawings, considering manufacturing optimization and placement; (2) Formwork will be assembled; (3) Precasting will occur. Specific precast elements will include upper modules, lower modules, abutments, buttress walls and appurtenant structures. Any necessary embedments determined to be included during the design process (turbine mounts, bolt mounts, spillways, etc.) will be cast into place.

- **Milestone 3.1:** Precast components completed and prepared for shipment by end of Month 9

Activities Completed: During Q3 2016, the prototype French Dam was constructed at Oldcastle Precast manufacturing facility in Avon, CT. The first scheduled pour was September 7th, and modules were poured in consecutive days and completed on September 19th. Figure 1 shows the demolding of the first module in the Oldcastle facility in Avon. Figure 2 shows the completed module being prepared for shipping

[Pictures next page]

Figure 1 - Module "De-Molding"

Figure 2 - Completed Module pending shipment

Subtask 3.2: Components Shipment to Test Facility (M9-M10)

Subtask Summary: FDE was responsible for shipping components to Test Facility and delivery of modules to the test facility. Modules were inspected at the precast facility to ensure project specifications are met. Modules were then loaded on flatbed trailer with crane and driven approximately 80 miles to Test Facility. Crane was used to move modules from flatbed into Test Facility and left unassembled in preparation for Task 4.

- **Deliverable 3.2:** Prototype components delivered to Test Facility by end of Month 10

Activities Completed: Shipment to the test tank in North Billerica, MA occurred on September 27th – 29th (Figure 3)

Subtask 4.1: Precast Modular Impoundment Assembly (M11-M12)

Subtask Summary: The purpose of this task was twofold (1) test the ability of the components to form interconnection and ease of installation and (2) prepare for testing. The first goal was part of the testing process, and measured the objective of developing hydropower structures that are rapidly deployable. This Subtask demonstrated the ability to interconnect precast components into a modular structure, and disassemble when necessary to repair sections or decommission the project at end of life.

- **Milestone 4.1:** Assemble Precast Modular Impoundment in Alden Flood Wall Facility[1] by end of Month 11

Activities Completed: The test consisted of assembling the precast modules by offloading the modules and forming all interconnections. The purpose of this test was to demonstrate that an impoundment structure can be assembled in a timely fashion utilizing precast modules. The key parameter of interest for Subtask 4.1 was time of construction. The project was assembled on October 1, 2016 – in 3.5 hours. Pictures of the assembly are included below in Figures 3-8.

Parameter	Metric
Test Date	October 1, 2016
Test Time (duration)	3.5 hours
Test Condition	Heavy rain

[1] The test location was later changed from Alden to a custom test facility constructed at WLFrench Excavating in North Billerica, MA.

Figure 3 - Modules arrive on-site to be staged (09/30)_

Figure 4 - Module hoisted into position

Figure 5 - Modules placed into position

Figure 6 - Modules placed into position

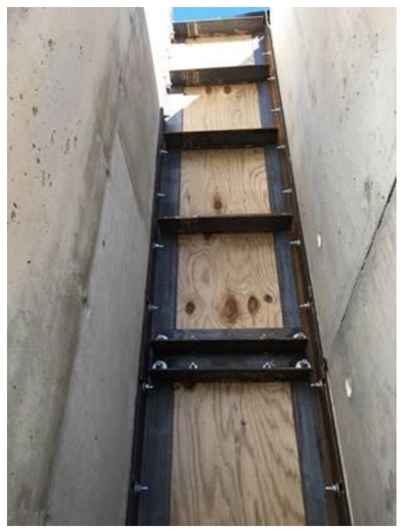


Figure 7 - Finished abutments to prevent leakage around structure

Figure 8 - Fully assembled French Dam #1

Subtask 4.2: Permeability Evaluation (M12-M13)

Subtask Summary: The potential permeability of the Interlocking Elements was one of the most critical functions of the entire modular structure and requires demonstration and prolonged evaluation. Modularity of the structure is made possible through the application of Interlocking Elements and requires water-tight seals that will not allow head loss (and corresponding energy loss) through the project's lifetime. Precast concrete is used in other water-retaining structures but the unique environment faced by hydropower structures and modular characteristics of this particular technology require a greater understanding of the potential of leakage. The dam section would seal off half of the Facility and water would be added to test seepage between adjoining modules. Test was conducted with approximately 12 ft. of head and water level was monitored over time. In addition, qualitative investigations with dye were performed to find sources of any leakage[2]. Remedies for sources of leakage were evaluated, including potential sealing material or new bolting configuration.

- **Milestone 4.2:** Complete Prototype Test Results Report.

*****See Appendix B for Independent Engineer's Report*****
*****See Appendix C for Field Observations Reports*****

Activities Completed: The permeability test was to determine whether the precast modules would be able to retain 12 feet of water without leaking. The test was performed at two different times. Water was provided by a hydrant located at the test facility. Test measurements were obtained by fabricating a staff gage on the inside of the tank to measure water levels in one-foot increments. Separately, a manometer was installed using ¼ inch clear tubing to more accurately measure the water level outside of the tank. The tubing was mounted and marked at specific locations and measured using a tape measure with accuracy to 1/16-inch. Initial fill occurred on October 12, 2016. After some leakage was observed at the abutments, the tank was drained and bulkheads were disassembled and reinforced before the second fill was performed on October 21, 2016. No leakage was observed between the modules, although a drip of water every 3-4 seconds was observed through the bulkheads. (The bulkheads were constructed out of plywood and reinforcement to fill the voids created by the difference between the modules length and the tank walls. In a "real world" application, these bulkheads would be site-specific and designed more appropriately to meet site requirements.) Critically, the modules themselves met the desired hydraulic integrity, and successfully met the desired target.

FDE retained an independent engineer, Norm Bishop, P.E., from Knight Piesold Consulting, to validate our test results and offer an Opinion letter on this technology. Norm Bishop is a Senior Executive Project Engineer based in Knight Piésold's Denver office. He has over 40 years of experience in hydraulic, heavy civil, hydroelectric, pumped storage, water resources, solar, wind, biomass and biofuel, energy storage, and thermal projects. His innovative construction and engineering solutions have resulted in significant project cost and time savings on numerous projects. The Consultant visited the site on November 3, 2016, and concluded:

"The FDE prototype precast module has been demonstrated by a test tank test that it meets the desired structural and hydraulic integrity. No visible leakage was observed by the consultant during the November 3, 2016 site visit. The consultant has also monitored the water level in the test tank for a period of four weeks and observed that there has been no water level change." (See Appendix B for complete report.)

[2] This was later changed after discussions with our engineering consultant revealed dye testing would not be accurate or necessary to determine leakage.

Figure 9 – Test Tank with 12' retained water

Task 5: Reference Site FEED and Baseline Comparison (M4-M18)

Subtask 5.1 will take place during Budget Period 1 of this project. Subtask 5.2 will take place during Budget Period 2, and is subject to the conditions of the Go/No-Go Decision. Activities under Subtask 5.1 will also serve as "Bridge Activities" to fill time during Go/No-Go Decision Process.

Task Summary: This Task required a FEED for a French Modular Impoundment at an actual site in the U.S., including Design, Cost Estimate, Schedule, Risks, etc., and comparison with an alternative configuration of the site using conventional dam construction methods (cast-in-place, RCC, gravity, etc.) This Task was conducted under supervision of Structural Dam Engineer and quantified several critical elements to determine project objectives, including: (1) ability of technology to be scalable, (2) structural loads analysis and durability, (3) ability to interface with riverbanks and foundation (4) seismic stability (5) cost & schedule reductions compared with conventional construction. The Structural Dam Engineer compared the new method of construction to baseline conventional construction methods and identified critical risks to address in full-scale implementation.

Subtask 5.2: Full-Scale FEED (M4-18)

Subtask Summary: This Subtask built off the criteria developed in Task 1 and combined with data produced during Tasks 2-4 (Prototype) to design a French Modular Impoundment for an actual hydropower site (selected in Task 1.) The Deliverable consisted of a Structural Analysis and Design document, with description and preliminary design of the impoundment with integrated generation equipment, and associated cost and schedule estimates. The Deliverable also demonstrated the appropriate range of sites and sizes (head, length, spillway type/dimensions, environmental conditions, etc.) appropriate for modular precast construction, to understand the scalability of the French Dam technology. A similar configuration using conventional dam construction methods was made and included in the Deliverable to determine final cost/schedule savings as a result of this technology. Additionally, this Subtask included an optimization spreadsheet to achieve the stated objectives of the Project, reduce cost by 60% and schedule by 4x.

- **Deliverable 5.2.1** Complete *draft* FEED by end of Month 15.
- **Milestone 5.2.2:** Final FEED delivered with Cost & Schedule Optimization incorporated to reflect identified cost savings by end of Month 18.

*****See Appendix A for Final FEED*****

Task 6: Project Management & Commercialization (M1 – M18)

Task Summary: Project Management served to coordinate all scope, schedule and budget-related aspects of the project and ensure deliverables, milestones, and miscellaneous objectives of the grant were met on time. Project Management was responsible for providing the DOE interface, and coordinating status updates and monthly reporting requirements. This Task will also managed all commercialization activity in Subtasks 6.1-6.2, including travel, conference participation, marketing, patent protection, etc. Full details are provided in the Project Management Plan.

Subtask 6.2: Marketing & Commercialization (M9-M18)

Subtask Summary: This Subtask encompassed the activities relating to commercializing this product and bringing to market. This included all necessary activities to further protect and expand patent portfolio, travel to meet with key vendors and material suppliers, internal financial controls and modeling to project costs and financing requirements, Investor Prospectus, market research, business plans, etc. Activities under this Subtask involved working closely with hydropower developers to iterate on cost/schedule information produced during this project and refining for specific site requirements. Key vendor relationships were established for specific components – for example, turbine suppliers with specific technical and physical requirements that were needed to design appropriate modules. Finally, this Subtask provided the opportunity to participate in hydropower conferences to present results of the Project (NHA, Hydrovision)

Subtask 6.3: Complete Best Practices Guide and Accompanying Spreadsheet Model (M16-18)

Subtask Summary: This Subtask included a proprietary product to accompany the licensing of this technology for developers to use when applying the Precast Modular Impoundment technology to sites, comparing this method with traditional construction. The Best Practices Guide included critical information such as Interlocking Methods, Loads Analysis, Foundation Preparation, and Precast Instructions. The Spreadsheet Model included user interface to input site-specific parameters (head, flow, climate, river width, etc.) and available materials and allowed basic costing and recommended precast element sizes and configurations.

- **Deliverable 6.3**: Complete Best Practices Guide and Spreadsheet Comparison Model by end of Month 18.

*****See Appendix E for Best Practices Guide, and Spreadsheet Comparison Model*****

Subtask 6.4: Final Cost and Performance Report (M16-M18)

Subtask Summary: This Subtask comprised the final Report submitted to DOE with cost and performance data from the design, manufacturing and testing process for the Prototype and the full Cost and Performance Baseline Comparison Report data from Subtask 5.2.

Subtask Details: This Final Report included a compilation of all prior written documentation into two documents, one prepared for public viewing and posting on OSTI, and another with proprietary information for internal purposes (and DOE review.)

- **Deliverable 6.4:** Provide Final Cost and Performance Report to DOE by end of Month 18.

*****This report comprises Deliverable 6.4*****

Appendix A. Deliverable 5.2.2 Final FEED Document

From the start of the project in December 2015, the team worked to identify an appropriate site to develop a design, cost and schedule comparison estimate for precast vs. cast-in-place methods. This has included significant industry outreach, working with our engineering partners, and market research into a large number of dams and water control structures to narrow down an appropriate sample size of dams. Our primary objective was to find a dam that would best translate from existing cast-in-place, RCC, or other conventional construction methods to our precast technology.

First, the team developed a matrix of the criteria necessary for collecting data and narrowing down appropriate candidates. The criteria evaluated are below:

- Year constructed
- Location (City, State)
- Design Availability
- Construction Schedule Availability
- Detailed Cost Availability
- Spillway Dimensions (height, length)
- Dam Dimensions (structural height, head, length)
- Primary Purpose
- New/Rehab
- Type/Material
- Project Cost (USD)
- Engineering Group
- Constructor
- Owner

Next, our team conducted significant market research of public sources (including National Inventory of Dams database and websites of dam construction contractors,) to identify plausible candidates to populate this spreadsheet. Our engineering partners, including GEI Consultants and Hydro Consulting Specialists also provided candidates of recent projects in which they have participated. Our team then identified a list of 16 dams (see Table 1) and collected available data from public and private sources, including the National Inventory of Dams and GEI's personal records of past projects. Most projects were eliminated outright due to the height or complexity of the dam, due to the DOE requirement that we demonstrate scalability in heads from 10-50 feet.

Table 1 - Candidate Dams for Precast Evaluation

Dam #	Dam Name	Year constructed	City	State
1	Labyrinth Dam	N/A	GEI Project	N/A
2	Otis Reservoir	2012	Otis	MA
3	Hickory Log Dam	2008	Canton	GA
4	Genesee No2 Dam	2007	Kittridge	CO
5	Hunting Run Dam	2002	Fredericksburg	VA
6	Folsom Dam	2012	Folsom	CA
7	Buckhorn Reservoir	1999	Wilson	NC
8	Deep Creek	2010	Yadkinville	NC
9	Franklin Dam	2006	Franklin	KY
10	Pine Brook	2006	Boulder	CO
11	Randleman Lake	2003	Randleman	NC
12	Tie Hack	1997	Buffalo	WY
13	Worumbo Hydro Station	1989	Lisbon Falls	ME
14	Moody Street Dam	?	Waltham	MA
15	Allendale Dam (GEI)	2002	Providence	RI
16	New Design	2016	N/A	N/A

Next, data was gathered for each of the dams listed. The result instantly allowed elimination of many of the structures for various reasons. Dams that were larger than the definition of "Scalability" in the DOE SOPO – between 10-50 feet of head – were ruled out, as were designs that were overly complex, too site-specific, or lacking the data required to complete the full re-design which will occur in Budget Period 2. After discussing as a team several times and reviewing all available data, the team decided to proceed with the Allendale Dam in Rhode Island, for the following reasons:

- Appropriateness to use the dam's parameters for the Prototype Test – the Allendale Dam has similar hydraulic head to the proposed prototype test.
- Appropriateness to use the dam for a Full-Scale Baseline Comparison – reconstruction of the dam was with traditional methods, cast in place concrete, and standard means and methods. No specialty equipment was needed during construction. The reconstructed dam was a very simple design that was low cost to construct. Comparison of the traditional construction methods to Precast Modular Impoundment construction will be easier to justify while making the benefits simple to visualize on larger more complex projects.
- Familiarity with the dam and the availability of needed information – GEI performed the design of the dam reconstruction. In addition, the EPA-funded project allowed for significantly greater data availability – for example, we have two 600+ page source documents for pre- and post-construction.
- Vigilant check on design – the dam design was approved by the Army Corps of Engineers
- Foundation – the concrete footings of the dam were secured to the bedrock with rock anchors embedded 20 feet into rock and spaced 5 feet on-center, similar to the Precast Modular Impoundment bedrock attachment approach.
- Schedule – preliminary discussions indicate duration of project execution will be shorter with the Precast Modular Impoundment construction verses the legacy construction, due to the speed in mobilization and needed area to execute the work.
- Cost Savings – preliminary cost comparison between the legacy construction and the Precast Modular Impoundment construction revealed modest advantages to the precast system.

Ultimately the decision was made to use the Allendale Dam in Rhode Island, for several factors. Allendale Dam is a relatively simple dam constructed with about 10 feet of head. The dam was constructed by standard means and methods – no specialty equipment was needed. Our preliminary cost comparison between this Dam and a comparable precast system revealed modest advantages to using precast system due to various factors. The primary reason this dam was chosen was GEI's familiarity with the dam and availability of needed information.

(1) Allendale Dam

The original Allendale Dam was a timber cribbing and earth embankment dam that was constructed in 1865. The dam was naturally breached in November 1991. The Army Corps authorized the reconstruction of the dam, which was completed in 2002. The reconstruction of Allendale Dam was a design-build project performed as part of a Superfund cleanup of contamination in and around the Woonasquatucket River in North Providence, Rhode Island. The cleanup has been under the jurisdiction of the Environmental Protection Agency with technical consultation and oversight provided by U.S. Army Corps of Engineers (USACE), New England District (NED).

The existing timber cribbing and earth embankment dam breached in 1991, allowing potential release of contaminants in sediments upstream of the dam. The NED prepared a repair scheme for a new dam prior to the site's designation as a Superfund site. GEI modified the NED repair scheme to minimize excavation of contaminated soils by constructing a rock-bolt-anchored concrete retaining wall dam immediately downstream of the existing dam. This fast-track design project proceeded with limited subsurface information in order to meet deadlines imposed by the EPA. The design was tailored to allow field modifications to match changed conditions found during construction. The final design included the rock-bolt-anchored concrete dam; modifications of the low level outlet control structure; a graded filter drain at dam toe; riprap scour protection; stream diversion; cofferdams; fish ladder considerations; and abutment wall stabilization. GEI also provided construction observation and rock bolt testing. Contract drawings were prepared in MicroStation, and contract specifications were developed from USACE guide specifications and SpecsIntact. Final documents delivered to the EPA were converted to AUTOCAD and Word files. Data on the reconstructed Allendale Dam is provided in Table 2.

GEI Description of Original Allendale Dam (Circa 2001)

Allendale Dam consists of timber buttresses supporting timber planking with a soil/sediment berm on top of the planking. It appears that the buttresses bear on timber cribbing/mat that in turn bears on bedrock or on dense soils above bedrock. A portion of the planking and buttresses has failed resulting in a partial breech of the dam. The dam is approximately 100 feet long, and ten feet high. Historically the dam operated as a non-gated overflow weir. The dam crest is approximately El. 93.5 ft. An existing low-level outlet consisting of a reinforced concrete structure with a wooden gate is not operational. The low-level outlet is the connection between the dam and the left abutment. The right abutment of the dam consists of a granite block retaining wall. The granite block wall is failing with large gaps visible between the granite blocks.
The gravel fill soils upstream of the dam are contaminated, and the site is currently listed as a Superfund site. The existing dam has been breached allowing the migration of contaminated sediments downstream.

Figure 10 - Original Allendale Dam Condition (EPA Document SDMS DocID 35144)

The USACE prepared a design for the reconstruction of Allendale Dam (USACE, 1995). The design would remove the existing wood timber dam and replace it with a reinforced concrete dam consisting of a footing and cantilever wall supporting an earth berm on the upstream face of the wall. The intent of the USACE design was to replicate the geometry of the original dam. The USACE design included repairs to the low-level outlet, removal of non-operational gate and installation of stop logs to replace the gate, reconstruction of the right abutment wall, and miscellaneous site work to seed and restore, the upstream berm and work area. Stream diversion in the original design consisted of phased construction using berms and the repaired outlet structure as a stream diversion channel. The upstream berm height was set at El 93.5 (historic crest height). The original stream diversion plan could pass between a one-year and two-year recurrence interval storm event.

EPA Description of Reconstructed Allendale Dam

The new Allendale Dam consists of a 105 ft concrete spillway with a concrete-set rip-rap spill pad and crushed gravel toe drain system. A new mechanically operated 60” x 48” sluice gate and a stop log system was installed to provide the means to regulate water levels in the Allendale Pond. The old gate-housing structure and existing stone wall abutment along both shores were preserved and reinforced to the extent possible. Based on a design modification, the new dam sits on four to ten feet of dense undisturbed till with rock anchor bolts installed into the bedrock. During construction, the Woonasquatucket River was diverted from the work area and controlled using cofferdams upstream and downstream of the Allendale dam location. Water was also pumped from a temporary sump and treated to remove suspended solids before being discharged downstream. All removal of sediment and debris was conducted as dry excavation. As planned, the construction of the Allendale Dam was largely completed by Spring 2002 with additional repair work on the existing gate structure performed in the Fall 2004.

Figure 11 – Reconstructed Allendale Dam

	Feature	Metric
Spillway and Dam	Dam Length	234 feet
	Concrete Spillway	106 feet
	Structural Height	19 feet
	Hydraulic Height	12 feet
	Maximum Discharge	578 cubic feet per second
	Spillway Elevation	93.5 feet
	Top of West Abutment	99.95 feet
	Top of East Abutment (Gate Structure)	100.4 feet
Reservoir	Drainage Area	40 square miles
	Maximum Storage	68 acre feet
	Normal Storage	43 acre-feet
	Normal Surface Area	13 acres
100 yr Design Flood	West side of pond upstream of Dam, Elevation	96 feet
	West side of pond downstream of Dam, Elevation	94 feet
	East side of pond upstream of Dam, Elevation	95 feet
	East side of pond downstream of Dam, Elevation	93 feet

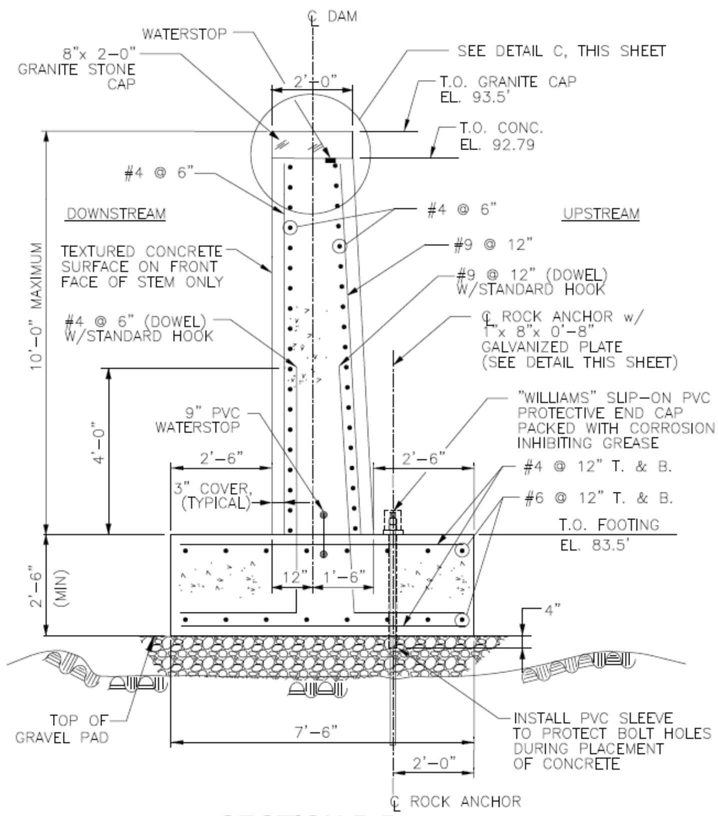
Table 2 - Reconstructed Allendale Dam Data

GEI modified the USACE design consistent within the goals of the project, as follows:

1. Relocate the dam downstream of the existing dam,
2. Reduce footing size using rock anchors for stability,
3. Replace stop logs with a sluice gate,
4. Re-design right abutment, and
5. Provide provisions for a future fish ladder.

The reinforced cast-in-place reconstruction of Allendale Dam design, (as implemented in the field,) consists of the following as seen in Figure 12 below:

1. Reinforced cast-in-place concrete width, 104.5 feet.
2. Reinforced cast-in-place concrete footing, 7.5 feet wide and 2.5 feet tall with two inch dimeter 300 kip hollow core rock bolt with 20 foot rock embedment.
3. Reinforced cast-in-place concrete stem wall, approximately 10 feet tall and 2 feet wide with granite stone cap.



SECTION B-B

SCALE: 1/2"=1'–0"

Figure 12 - Allendale Dam CIP Design

The Design Calculations for the reinforced cast-in-place reconstruction of Allendale Dam are excluded from this report due to size considerations, but considered the following elements:

1. Assessment of Subsurface Geotechnical Information
2. Stability Analysis
   a. Usual loading – normal operation conditions
   b. Unusual loading conditions – flood discharge (100-year flood)
   c. Extreme loading – normal operation with earthquake
3. Seepage Analysis
4. Evaluation of Excavation Limits

5. Rock Anchors
6. Structural Design
    a. Low Level Outlet
    b. Right Abutment Wall
    c. Foundation and Dam Wall
7. Stream Diversion
8. Future Fish Ladder

The Design Drawing for the reinforced cast-in-place reconstruction of Allendale Dam are included in appendix A.2

(2) Allendale Redesign Using French Dam Technology

The French Impoundment Dam was modified from the original Allendale Dam Rehabilitation design, consistent within the goals of the project, as follows:

1. Replace the cast-in-place dam with the Precast French Impoundment Dam.
2. Two rows of 13 precast concrete units, (8 feet wide, 8 feet long, and 7.2 feet tall) for a total width of dam 104.5 feet. A granite stone cap to match the original design.
3. Two-inch-diameter 300 kip hollow core rock bolt with 20 foot rock embedment.

The design calculations for the precast are included on pages 22-56. The precast dam design consists of following:

1. Stability Analysis
    a. Usual loading – normal operation conditions
    b. Unusual loading conditions – flood discharge (100-year flood)
    c. Extreme loading – normal operation with earthquake
2. Rock Anchors
3. Structural Design
    a. Connection between top and bottom precast units.
    b. Connection between horizontal precast units (for seismic forces)

All other elements of the original dam will remain the same, and therefore were not looked at. Full redesign of the Allendale Dam using French Dam technology is included in Section (5) of this Appendix.

(3) Schedule Comparison (Summary)

The Allendale Dam construction schedule was created by using the Photodocumentation Log presented in the "Completion of Work Report Centredale Manor Restoration Project Superfund Site," Dated April 2005 (EPA Report SDMS DocID 237558). Both the schedule and Photodocumentaion Log are presented in Appendix A.3 The Cast-in-Place reconstruction of Allendale Dam took 172 days, starting on August 13, 2001 and finished on April 9, 2002. The Surface Water Control and Diversion (Cofferdams) were in place for 142 days, from September 24, 2001 until April 9, 2002. The Dam Reconstruction including rock anchors, CIP footing and dam wall, granite capstone, and downstream Riprap took 95 days, starting October 16, 2001 and finished on January 29, 2002.

The estimated Allendale Dam construction schedule presented in Appendix B.2 "ALLENDALE DAM – FRENCH CONSTRUCTION SCHEDULE (BASED ON PHOTODOCUMENTATION REPORT)" was created based on the Photodocumentation Log presented in the "Completion of Work Report Centredale Manor Restoration Project Superfund Site," Dated April 2005 (EPA Report SDMS DocID 237558) and conservatively fixed the anchor bolt and dam footing start dates to the dates found in the report.

Modular precast reconstruction of the Allendale Dam is estimated to take 118 days, starting on August 13, 2001 and finished on January 23, 2002. The Surface Water Control and Diversion (Cofferdams) is estimated to be in place for 88 days, from September 24, 2001 until January 23, 2002. The dam reconstruction including rock anchors, working pad, precast dam, granite capstone, and downstream riprap is estimated to take 54 days, stating October 16, 2001 and finished on January 2, 2002.

The estimated Allendale Dam construction schedule presented in Appendix B.2 was created based on the Photodocumentation Log presented in the "Completion of Work Report Centredale Manor Restoration Project Superfund Site," Dated April 2005 (EPA Report SDMS DocID 237558) while allowing the anchor bolt and dam working pad start dates to move freely based on predecessor construction activities. For this schedule we have increased the rock anchors duration.

The Precast Unit installation rate in the estimated schedule is based on the rate installed during the prototype installation of 6 units in under 4 hours (or 12 units during an 8-hour shift.) The French precast reconstruction of Allendale Dam is estimated to take 88 days, starting on August 13, 2001 and finished on December 12, 2001. The Surface Water Control and Diversion (Cofferdams) is estimated to be in place for 58 days, from September 24, 2001 until December 12, 2001. The dam deconstruction including rock anchors, working pad, precast dam, granite capstone, and downstream riprap is estimated to take 34 days, stating October 16, 2001 and finished on November 28, 2001.

The precast dam could have/would have reduced the construction duration considerably. Conservatively, by fixing the start dates of the rock anchors and footing work of the precast dam to the CIP dam dates, the **overall project schedule will be reduced by approximately 31 percent**. Additional construction duration comparisons are shown in the table below.

	Allendale Dam – Construction Schedule Comparison				
Activity	**CIP Dam**	**FRENCH Dates Locked**		**FRENCH Dates Free**	
	Days	Days	% Reduction	Days	% Reduction
Cofferdams in River	142	88	38%	58	59%
Dam Reconstruction	95	54	43%	34	64%
Total Project	172	118	31%	88	49%

Table 3 - CIP vs French Dam Schedule Comparison

(4) Allendale Cost Comparison (Summary)

Based on the "Completion of Work Report - Centerdale Manor Restoration Project Superfund Site," (EPA Document SDMS DocID237558), as of April, 2005, the total cost for the activities performed in satisfying the performance criteria of the project was approximately $2,457,745. A final total cost for the completion of the activities performed will not be available until approved by EPA. A summary of project cost are:

Item	**Cost**
Engineering Cost	$157,245
Delineation and Removal of Soil & Sediment	$1,034,000
Off-site transportation and disposal of waste	$238,100
Oversight Cost with USACE	$200,000
Dam Restoration/Rebuild	$828,400
Total	$2,457,745

Table 4 - CIP Allendale Dam Costs

The dam restoration/rebuild costs include costs for mobilization/demobilization, implementing administrative and Site control, sediment excavation, dewatering and wastewater treatment, Site restoration, and additional grouting activities.

Because final or itemized costs are not available, we produced a cost estimate for the construction of the 104.5 foot wide reinforced cast-in-place concrete dam, Appendix A.4. The cost estimate only includes the main dam structure (subgrade site prep, CIP Concrete footing and wall, and rock anchors). The cost estimate was based on takeoff quantities and unit prices obtained from actual recent dam projects, and the 2014 RSMeans Heavy Construction Cost Data book.

Item	**Cost**
Site Prep	$16,345
CIP Concrete Footings and Walls	$215,967
Rock Anchors	$41,800
Project Overhead (20%)	$54,822
Total	**$328,934**

Table 5 - Allendale Costs (Primary Structure)

Other construction costs were assumed to be same regardless of the dam chosen. These items include the cofferdams, backfill, riprap, gates, concrete wingwall, granite cap, and site restoration.

The Full Cost Estimate to construct the 104.5 foot wide French Impoundment Precast dam is included in Appendix A. The cost estimate was based on takeoff quantities, prices obtained from Oldcastle, actual recent dam projects, and the 2014 RSMeans Heavy Construction Cost Data book.
The cost estimate includes the main dam structure:

- Site Prep **-** subgrade, mud mat, and concrete closure pours.
- Precast Units – production startup, precast units, hardware, shipping, and installation.
- Rock Anchors.

Item	**Cost**
Site Prep	$34,419
Precast Units	$146,466
Rock Anchors	$56,000
Project Overhead (20%)	$52,660
Total	**$315,961**

Table 6 - French Dam Allendale Costs (Primary Structure)

The French Impoundment Precast dam comparatively reduced the construction cost of the main dam structure by approximately $13,000 or about 4%. Additional cost savings would be expected due to the reduced construction schedule including owner's construction oversite costs, pumps, cofferdams, erosion protection and other duration-based costs. There is an important note here – the Allendale Dam was a fairly simple structure to cast, and smaller than conventional dam structures. As a result, the construction process was fairly simple using conventional construction methods and the difference between CIP vs precast is not as large. In the following section, we will demonstrate how scaling to more typical project configurations results in more significant cost reductions.

(5) Scalability of French Dam Results

To demonstrate the scaling capability of the Precast French Impoundment modules, GEI developed a generalized design tool to assess potential configurations and associated costs for a variety of water levels, from 10 to 50 feet of head. Only external forces were considered and assumed the precast units would act together rigidly so that the resulting structure acts as an anchored gravity structure. We analyzed the different configurations for stability against sliding and overturning, with a minimum factor of safety of 1.5. Starting at

16-feet high (3-units high), the precast modular dam was configured with two units at the crest, to provide an adequate crest width for vehicle access.

Constant exterior size was assumed for the precast units for simplicity, but variable sizes could be incorporated for different projects. Allowances were made for increasing wall thickness as the height of the structure increases from a minimum of 8-inches to a maximum of 18-inches. For cases where the reservoir was within 3-feet of the dam crest, we included a precast parapet wall to maintain freeboard. The dam was analyzed for reservoir loading with full uplift conditions, with seismic forces tied to the project test site (Billerica, MA). The seismic force was applied on a simplified basis, with a constant seismic coefficient applied to the entire mass of the structure. The seismic force was determined based on the height of the structure and the estimated corresponding approximate period on the design spectra taken from the USGS design maps website, using the 2010 ASCE 7 mapping for Billerica, MA. The maximum seismic force applied was 0.23g, and applied for structure heights less than 31 feet.

The external forces were resisted by the weight of the precast units, anchorage forces, and tailwater forces (if applicable). A friction factor of 0.7 was used, because the structure will be constructed atop a concrete mud mat. For simplicity we assumed a single anchor size, capacity, and cost for all configurations. This would be revised during design to fit any specific project. Our assumptions were a bar anchor installed to approximately 20-feet, with a nominal capacity of 60-kips each.

Figures 13 through 15 below are snapshots of the generalized design tool output showing the configuration and conceptual cost estimate for reservoir heights of 22, 30, and 50 feet. Initially, it is clear that the potential modular dam configurations resemble the shape of traditional cast-in-place structures. The main difference is that the structure is less dense, which is largely offset by the use of anchors to provide additional normal force at the base of the structure.

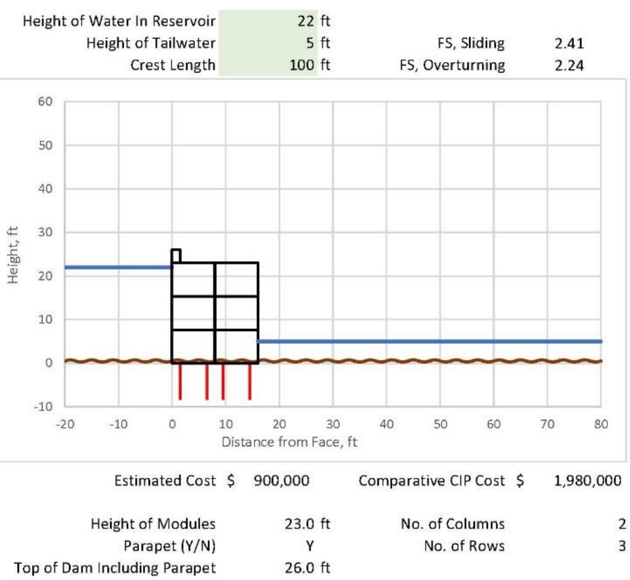
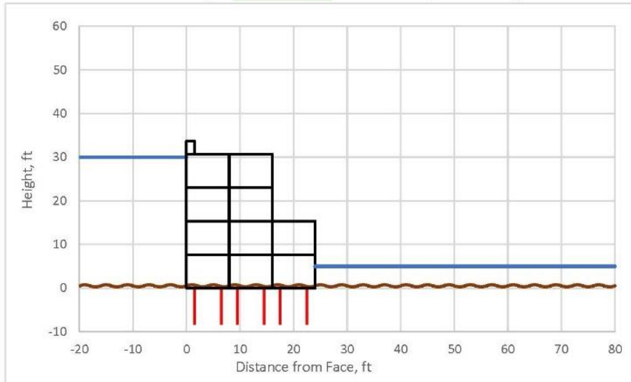


Figure 13- Output of Generalized Design Tool for 22-foot Reservoir

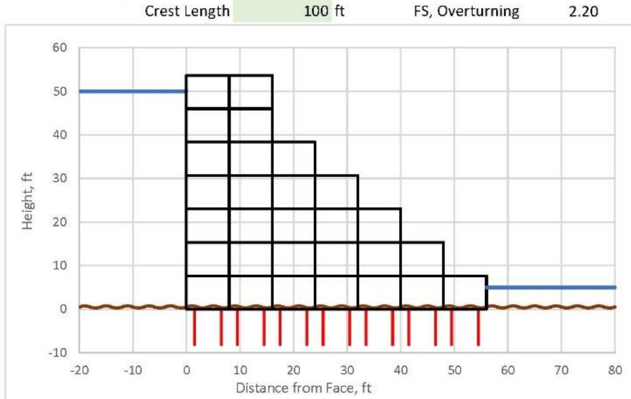
Height of Water In Reservoir	30 ft		
Height of Tailwater	5 ft	FS, Sliding	1.97
Crest Length	100 ft	FS, Overturning	2.07



Estimated Cost	$ 1,500,000	Comparative CIP Cost	$ 3,830,000
Height of Modules	30.7 ft	No. of Columns	3
Parapet (Y/N)	Y	No. of Rows	4
Top of Dam Including Parapet	33.7 ft		

Figure 14 - Output of Generalized Design Tool for 30-foot Reservoir

Height of Water In Reservoir	50 ft		
Height of Tailwater	5 ft	FS, Sliding	1.90
Crest Length	100 ft	FS, Overturning	2.20



Estimated Cost	$ 4,600,000	Comparative CIP Cost	$ 8,050,000
Height of Modules	53.7 ft	No. of Columns	7
Parapet (Y/N)	N	No. of Rows	7
Top of Dam Including Parapet	53.7 ft		

Figure 15 - Output of Generalized Design Tool for 50-foot Reservoir

For a given precast configuration, we estimated a comparative cost for a traditional cast-in-place gravity dam. The cost of the modular precast structure was estimated in the same way as described in the Allendale Dam Rehabilitation Cost Estimate section above, with a few minor changes to generalize the design. Specifically, we assumed more anchors would be necessary in the generalized design, and we assumed a slightly larger standard unit size. To compare against a cast-in-place alternative, we estimated costs for 15, 30, and 50-foot high cast-in-place concrete gravity dams based on an assumed section geometry and unit costs. The 15-foot structure was the same as the Allendale Dam, except that it was made slightly larger to accommodate freeboard, because the comparative costs are for non-overflow sections of the structure. The sections and costs for the 30 and 50-foot structures are shown below in Figures 16 and 17, respectively.

30' Cast In Place Concrete Dam			
Assumptions			
Length		100	LF
Section Area		683	SF
Conc Vol / Unit Length		25.3	CY/LF
Assumed Site Prep Time		24	Crew Hours
Crew Cost Per Hour		$681	
Site Prep Subtotal		$16,345	
Concrete Volume		2528	CY
CIP Concrete Total Unit Cost		$1,250	per CY
Concrete Subtotal (installed)		$3,159,722	
Parapet Cost Per Unit Length		$111	per LF
Parapet Subtotal (installed)		$11,111	
Subtotal		$3,187,178	
Project Overhead	20%	$637,436	
Total Price		**$3,824,614**	
Equivalent Unit Price		$1,513	per CY

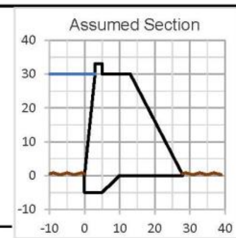


Figure 16 - Comparative Cost for 30' Cast-in-Place Concrete Gravity Dam

50' Cast In Place Concrete Dam			
Assumptions			
Length		100	LF
Section Area		1385	SF
Conc Vol / Unit Length		51.3	CY/LF
Assumed Site Prep Time		32	Crew Hours
Crew Cost Per Hour		$681	
Site Prep Subtotal		$21,793	
Concrete Volume		5130	CY
CIP Concrete Total Unit Cost		$1,300	per CY
Concrete Subtotal (installed)		$6,668,519	
Parapet Cost Per Unit Length		$111	per LF
Parapet Subtotal (installed)		$11,111	
Subtotal		$6,701,423	
Project Overhead	20%	$1,340,285	
Total Price		**$8,041,708**	
Equivalent Unit Price		$1,568	per CY

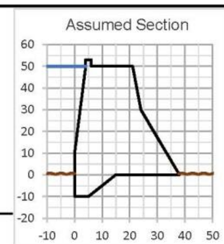


Figure 17 - Comparative Cost for 50' Cast-in-Place Concrete Gravity Dam

At larger heights, the modular precast dam is expected to become much less expensive than a comparable cast-in-place concrete dam. This does not account for the other components of the dam such as the spillway and outlet works which can be a significant proportion of the total project cost, and which are traditionally incorporated into the structure of a cast-in-place dam.

Dam Head	**CIP Cost**	**Precast Cost**	**Cost Savings (approx.)**
22'	$1,980,000	$900,000	**~60%**
30'	$3,830,000	$1,500,000	**~60%**
50'	$8,050,000	$4,600,000	**~40%**

Table 7 - Summary of Results (CIP vs Precast, 100' Crest Length)

(6) Full Design, Schedule and Cost Calculations

6.1 Full Design Calculations

6.2 Full Schedule

Gantt Chart Schedule - Anchor Bolt and Dam Footing Start Dates Fixed to Photodocumentation Log dates

Gantt Chart Schedule - Anchor Bolt and Dam Footing Start Dates Free to Move

6.3 Full Cost Analysis

6.1 Design Calculations

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Allendale Dam Reconstruction (French Dam Option)

All design assump on used for the original Allendale Cast-In-Place Dam will be use for the French Precast Dam.

STABILITY ANALYSIS:

The following calcula ons are provided to evaluate the stability of the proposed French Precast Dam.

Design Loading: Based on the 2001 Design, the stability analyses to considered:

1. Usual Loading - Normal Operating Conditions
2. Unusual Loading - Flood Discharge (100-year event)
3. Extreme Loading - Normal Operation with Earthquake

Our design assumes full fill to the top of the dam, full water pressure, and dynamic effects from both fill and water. Dynamic soil loads are determined using Mononobe-Okabe equations from USACE, 1989. The vertical acceleration is assumed to be 2/3 of the horizontal acceleration (USACE, 1989). Hydrodynamic loads are determined using Westergaard's formula (USACE, 1989). Westergaard's hydrodynamic effects from water assume a freestanding
wall and are conserva ve when applied to a wall retaining fill. We conserva vely ignored hydrodynamic effects of the tailwater and passive resistance at the toe.

STABILITY DESIGN PARAMETERS

Load Case	Headwater El. ( )	Tailwater El. ( )	Top of Till El. ( )	Top of Rock El. ( )
1. Usual Loading	93.5	82.0	93.5	78.0
2. Unusual Loading	97.7	88.9	93.5	78.0
3. Extreme Loading	93.5	82.0	93.5	78.0

The stability analyses includes the clamping ac on of the rock anchor to resist both sliding and overturning loads. Each load case was evaluated to determine the factor of safety against sliding, eccentricity, founda on bearing pressures, and concrete stress levels.

STABILITY DESIGN CRITERIA (Based on USACEE EM 1110-2-2200)

Load Case	Resultant Loca on at Base	Minimum Sliding FS	Founda on Bearing Pressure	Concrete Stress Compression
1. Usual Loading	Middle 1/3	2	Allowable	0.3 f'c
2. Unusual Loading	Middle 1/2	1.7	Allowable	0.5 f'c
3. Extreme Loading	Within Base	1.3	1.33 Allowable	0.9 f'c

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GEOMETRY & MATERIAL PROPERTIES

Eleva on of Bedrock: El_{rock} 78.0ft

Leveling Pad Thickness: H_{Pad} 0.5ft (Assumed)

Toe of Precast Eleva on: El_{toe} El_{rock} H_{Pad} 78.50 ft

Top of Dam Eleva on: El_{top} 93.5ft

Dam Height: H_{Dam} El_{top} El_{toe} 15.00 ft

Precast Width: $W_{Precast}$ 8ft

Precast Length: $L_{Precast}$ 8ft

Precast Wall Thickness: H_{c} 0.67ft

Precast Cap Thickness: $T_{Precast}$ 0.67ft

Granite Cap Thickness: $T_{Granite}$ 0.67ft

Granite Cap Width: $W_{Granite}$ 2ft

Precast Height: $H_{Precast}$ H_{Dam} $T_{Granite}$ $H_{Precast}$ 14.33 ft

Precast Height Unit: $H_{Precastunit}$ $\frac{H_{Dam} \quad T_{Precast} \quad T_{Granite}}{2}$ $H_{Precastunit}$ 6.83 ft

Unit Weight of Concrete: γ_{c} 150pcf

Unit Weight of Granite: γ_{G} 165pcf

Unit Weight of Water: γ_{w} 62.4pcf

Unit Weight of Saturated Fill: γ_{f} 125pcf

Internal Fric on Angle of Fill: ϕ_{f} 32deg

Back Face Ba er: θ 0°

Backslope Angle: β 0°

Interface Fric on Angle: δ_{f} ϕ_{f} 2 16.00 °

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Earth Pressure Coefficients

Compute the ac ve earth pressure coefficient using Rankine (horizontal backfill, ver cal face).

Earth Pressure Coefficient: $K_a \quad \frac{1 \quad \sin \phi_f}{1 \quad \sin \phi_f}$ $\boxed{K_a \quad 0.307}$

Fric on Proper es at Concrete/Bedrock Interface:

Fric onal proper es between concrete and bedrock from original design:

Fric on Angle: δ_s 30deg

Coefficient of Fric on: $\mu \quad \tan \delta_s$ μ 0.58

Apparent Cohesion: c_a 10psi

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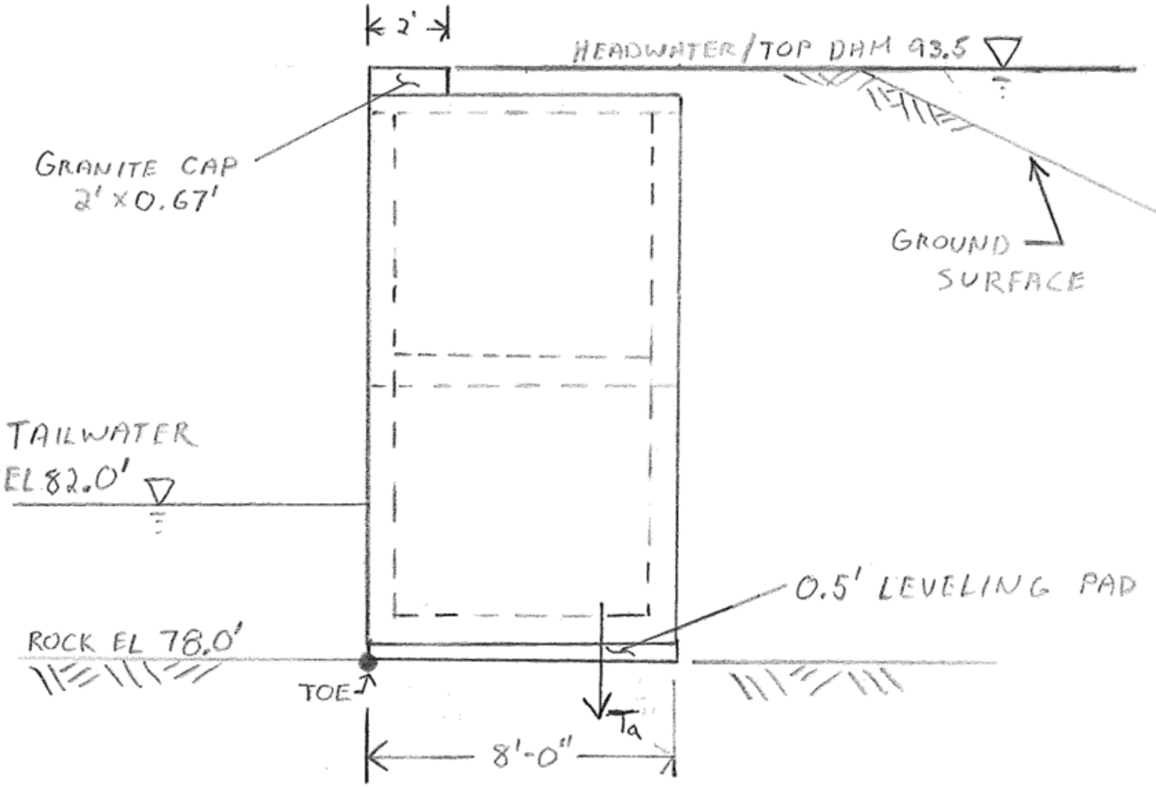
LOAD CASE 1: NORMAL POOL



DAM GEOMETRY

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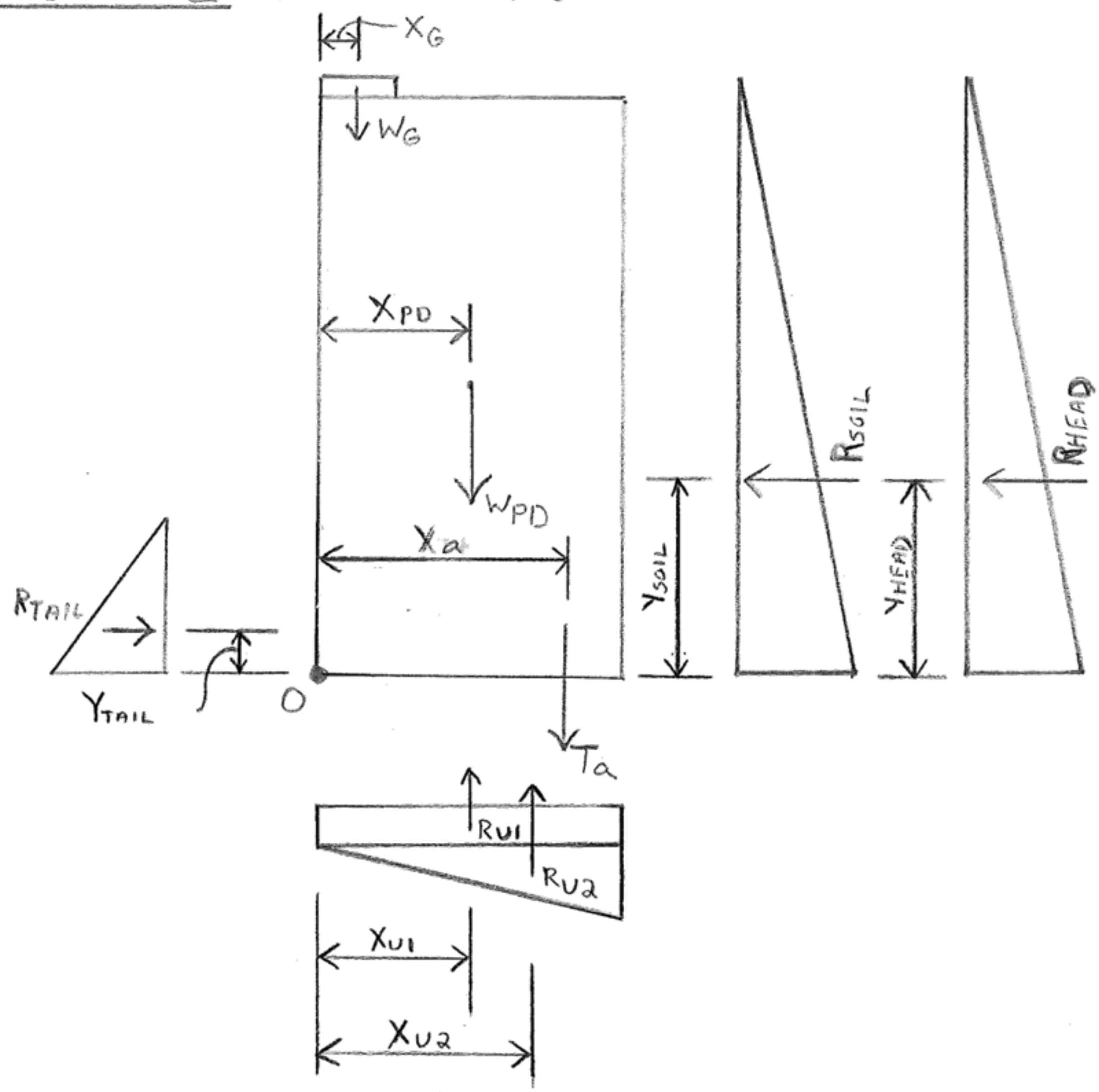
LOAD CASE 1: NORMAL POOL



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Stability Analysis for Load Case 1 - Normal Pool

Headwater Eleva on: El_{hw} 93.5ft

Tailwater Eleva on: El_{tail} 82.0ft

Compute Water Pressures and Resultant Forces:

Depth of Tailwater:	$hw_{tail}\ El_{tail}\ El_{rock}$	hw_{tail}	4.00 ft
Depth of Headwater:	$hw_{head}\ El_{hw}\ El_{rock}$	hw_{head}	15.50 ft
Depth of Soil:	$h_s\ El_{top}\ El_{rock}$	h_S	15.50 ft
Tail Water:	$P_{tail}\ \gamma_w\ hw_{tail}$	P_{tail}	249.60 psf
	$R_{tail}\ \frac{1}{2}\ hw_{tail}\ P_{tail}$	R_{tail}	499.20 plf
Loca on About Toe:	$Y_{tail}\ \frac{1}{3} hw_{tail}$	Y_{tail}	1.33 ft
Head Water:	$P_{head}\ \gamma_w\ hw_{head}$	P_{head}	967.20 psf
	$R_{head}\ \frac{1}{2} hw_{head}\ P_{head}$	R_{head}	7495.80 plf
	$Y_{head}\ \frac{1}{3} hw_{head}$	Y_{head}	5.17 ft

Upli : Upli pressure is calculated along the en re width of the precast units.

	$R_{u1}\ P_{tail}\ W_{Precast}$	R_{u1}	1996.80 plf
	$R_{u2}\ \frac{1}{2}\ W_{Precast}\ P_{head}\ P_{tail}$	R_{u2}	2870.40 plf
Loca on About Toe:	$X_{u1}\ \frac{1}{2} W_{Precast}$	X_{u1}	4.00 ft
	$X_{u2}\ \frac{2}{3} W_{Precast}$	X_{u2}	5.33 ft

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Compute Soil Pressures and Resultant Forces:

$$P_{soil} \quad K_a \ \gamma_f \ \gamma_w \ h_s$$

P_{soil} 298.1 psf

$$R_{soil} \quad \frac{1}{2} P_{soil} \ h_s$$

R_{soil} 2310.5 plf

Loca on About Toe:

$$Y_{soil} \quad \frac{1}{3} h_s$$

Y_{soil} 5.17 ft

Compute Dead Loads:

Precast Dam (2 Blocks High, and 3 bo oms/tops):

(Volume Total) (Void Height) (Void Width) (Void Length)

$$W_{PD} \quad H_{Precast} \ W_{Precast} \ L_{Precast}$$

$$H_{Precast} \quad 3H_c \quad W_{Precast} \quad 2H_c \quad L_{Precast} \quad 2\ H_c \qquad \gamma_c$$

W_{PD} 55.60 kip

Precast Dead Load Per Foot: $W_{PDF} \quad \frac{W_{PD}}{L_{Precast}}$

W_{PDF} 6950 plf

Loca on About Toe: $X_{PD} \quad \frac{1}{2} W_{Precast}$

X_{PD} 4.00 ft

Granite Cap:

$$W_G \quad T_{Granite} \ W_{Granite} \ \gamma_G$$

W_G 221.10 plf

Loca on About Toe: $X_G \quad \frac{1}{2} W_{Granite}$

X_G 1.00 ft

By inspec on rock anchors will be required to provide an adequate factor of safety against sliding, and to meet requirements for bearing pressure and loca on of the resultant within the base of the Dam.

Compute Tension Forces:

Total Rock Anchor: T_a

Loca on About Toe: X_a 6ft

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Check Overturning:

For overturning the resultant force should be in the middle third of the founda on.

Eccentricity from the centerline needs to be equal or less than: $\frac{W_{Precast}}{6}$ 1.33 ft

Anchor Load: T_a 15.47 $\frac{kip}{ft}$

Check required anchor load to resist sliding and overturning.

Anchor Loca on: X_a 6.00 ft

Sum the Moments:

$M_0 \quad R_{soil} \; Y_{soil} \quad R_{u1} \; X_{u1} \quad R_{u2} \; X_{u2} \quad R_{head} \; Y_{head}$ 74.0 $\frac{kip\ ft}{ft}$

$M_R \quad W_{PDF} \; X_{PD} \quad W_G \; X_G \quad T_a \; X_a \quad R_{tail} \; Y_{tail}$ 121.5 $\frac{kip\ ft}{ft}$

$M_{Total} \quad M_R \quad M_O$ 47.54 kip $\frac{ft}{ft}$

Sum Ver cal Forces:

$R_v \quad W_{PDF} \quad W_G \quad R_{u1} \quad R_{u2} \quad T_a$ 17.77 $\frac{kip}{ft}$

eccentricity from Toe: $e_O \quad \left| \frac{M_R \quad M_O}{R_V} \right|$ e_O 2.67 ft

eccentricity from Center line: $e_r \quad \left| \frac{W_{Precast}}{2} \quad e_o \right|$ e_r 1.33 ft

OK equal or less than

$\frac{W_{Precast}}{6}$ 1.33 ft

Resultant Loca on: $x_{Ra} \quad \frac{M_{Total}}{}$ 2.67 ft

Resultant Ra o: $r_{Ra} \quad \frac{X_{Ra}}{W}$ 0.33

Base in Compression (%): $C_r \quad \min 3 \, r_{Ra} \; 1.0$ 1.00

100% of the Base in Compression. Resultant is in the middle third
- OK

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Check Bearing Capacity:

$\sigma_{allowable}$ 0.3 4000psi 1200.00 psi

Find the Bearing Pressure at the toe and heel:

Effec ve Width: W_e $W_{Precast}$ 2 e_r 5.35 ft

Bearing Pressure: q_{toea} $\frac{R_v}{W_{Precast}}$ 1 $\frac{6\ e_r}{W_{Precast}}$ 30.76 psi

Bearing Heel: q_{heela} $\frac{R_v}{W_{Precast}}$ 1 $\frac{6\ e_r}{W_{Precast}}$ 0.10 psi

Concrete is OK.

Check Factor of Safety Against Sliding:

N Force: N_F μR_v c_a $W_{Precast}$ N_F 21.78 $\frac{kip}{ft}$

T Force: T_F R_{soil} R_{head} R_{tail} T_F 9.3 $\frac{kip}{ft}$

FS_s $\frac{N_F}{T_F}$ 2.34 Greater than or equal to 2, therefore: Okay

Find the Load required per rock anchor connec on:

S_a 4ft (Rock Anchors at 4 feet oc) F_{anchor} T_a S_a F_{anchor} 62 kip

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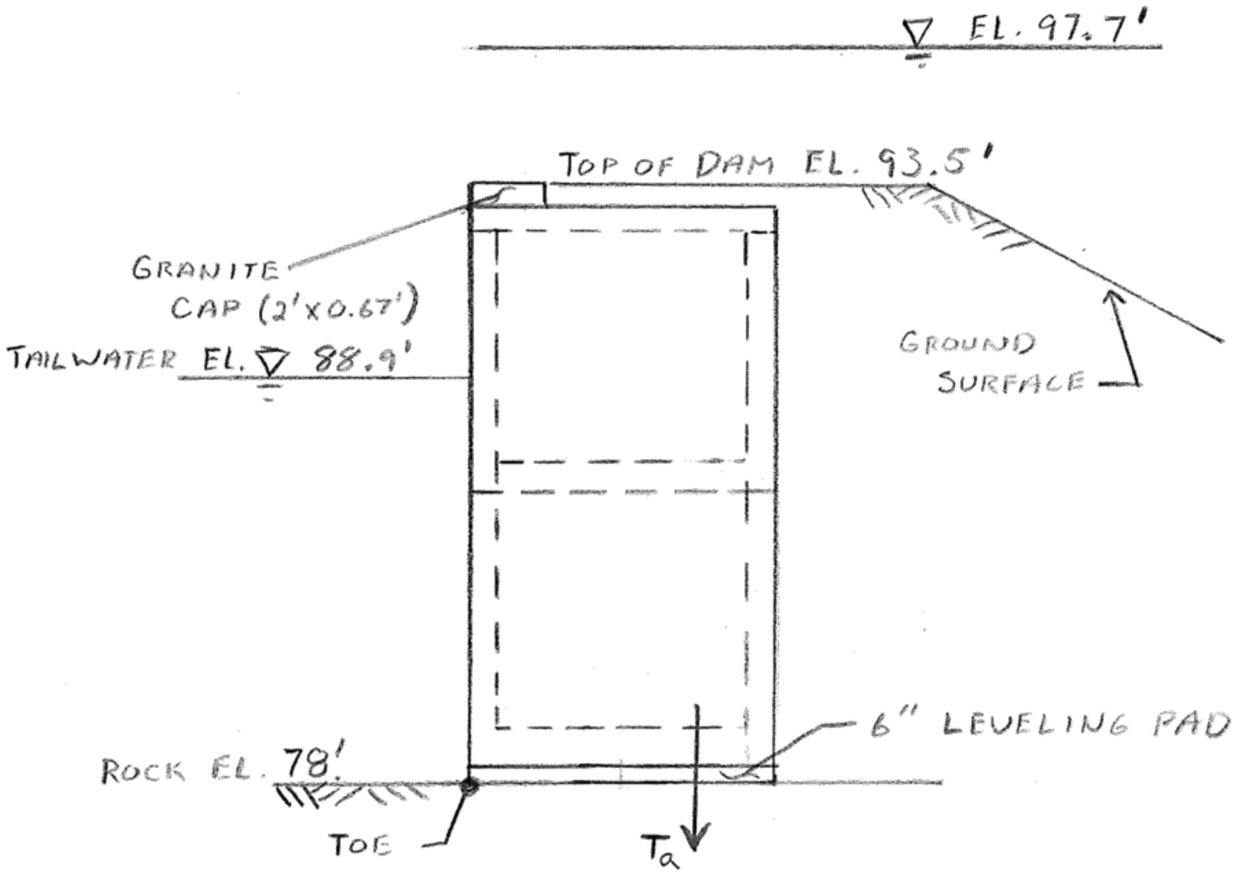
Subject	DAM GEOMETRY - LOAD CASE 2					

LOAD CASE 2: 100-YEAR FLOOD



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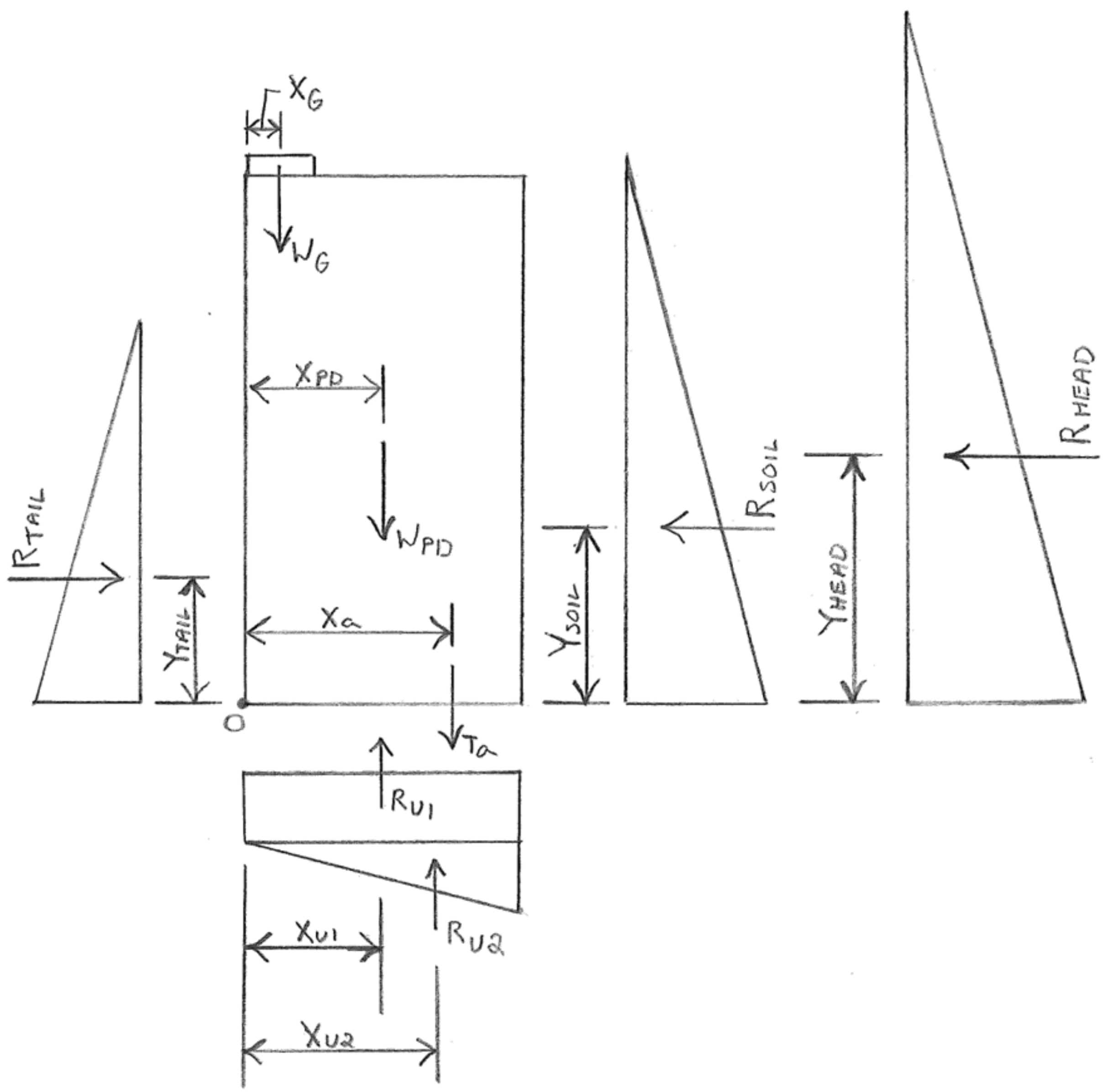
LOAD CASE 2: 100 YEAR FLOOD



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Stability Analysis for Load Case 2 - 100 Year Flood

Headwater Eleva on: El_{hw} 97.7ft

Tailwater Eleva on: El_{tail} 88.9ft

Compute Water Pressures and Resultant Forces:

Depth of Tailwater:	$hw_{tail} \; El_{tail} \; El_{rock}$	hw_{tail} 10.90 ft
Depth of Headwater:	$hw_{head} \; El_{hw} \; El_{rock}$	hw_{head} 19.70 ft
Depth of Soil:	$h_s \; El_{top} \; El_{rock}$	h_s 15.50 ft
Tail Water:	$P_{tail} \; \gamma_w \; hw_{tail}$	P_{tail} 680.16 psf
	$R_{tail} \; \frac{1}{2} \; hw_{tail} \; P_{tail}$	R_{tail} 3706.87 plf
Loca on About Toe:	$Y_{tail} \; \frac{1}{3} hw_{tail}$	Y_{tail} 3.63 ft
Head Water:	$P_{head} \; \gamma_w \; hw_{head}$	P_{head} 1229.28 psf
	$R_{head} \; \frac{1}{2} hw_{head} \; P_{head}$	R_{head} 12108.41 plf
	$Y_{head} \; \frac{1}{3} hw_{head}$	Y_{head} 6.57 ft

Upli : Upli pressure is calculated along the en re width of the precast units.

	$R_{u1} \; P_{tail} \; W_{Precast}$	R_{u1} 5441.28 plf
	$R_{u2} \; \frac{1}{2} \; W_{Precast} \; P_{head} \; P_{tail}$	R_{u2} 2196.48 plf
Loca on About Toe:	$X_{u1} \; \frac{1}{2} W_{Precast}$	X_{u1} 4.00 ft
	$X_{u2} \; \frac{2}{3} W_{Precast}$	X_{u2} 5.33 ft

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Compute Soil Pressures and Resultant Forces:

P_{soil} K_a γ_f γ_w h_s — P_{soil} 298.1 psf

R_{soil} $\frac{1}{2} P_{soil} h_s$ — R_{soil} 2310.5 plf

Loca on About Toe: Y_{soil} $\frac{1}{3} h_s$ — Y_{soil} 5.17 ft

Compute Dead Loads:

Precast Dam (2 Blocks High, and 3 bo oms/tops):

(Volume Total) (Void Height) (Void Width) (Void Length)

W_{PD} $H_{Precast}$ $W_{Precast}$ $L_{Precast}$

$H_{Precast}$ $3H_c$ $W_{Precast}$ $2H_c$ $L_{Precast}$ $2H_c$ γ_c

W_{PD} 55.60 kip

Precast Dead Load Per Foot: W_{PDF} $\frac{W_{PD}}{L_{Precast}}$ — W_{PDF} 6949.86 plf

Loca on About Toe: X_{PD} $\frac{1}{2} W_{Precast}$ — X_{PD} 4.00 ft

Granite Cap:

W_G $T_{Granite}$ $W_{Granite}$ γ_G — W_G 221.10 plf

Loca on About Toe: X_G $\frac{1}{2} W_{Granite}$ — X_G 1.00 ft

By inspec on rock anchors will be required to provide an adequate factor of safety against sliding, and to meet requirements for bearing pressure and loca on of the resultant within the base of the Dam.

Compute Tension Forces:

Total Rock Anchor: T_a Loca on About Toe: X_a 6ft

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Check Overturning:

For overturning the resultant force should be in the middle half of the founda on.

Eccentricity from the centerline needs to be equal or less than: $\frac{W_{Precast}}{4}$ 2.00 ft

Anchor Load: T_a 20.61 $\frac{kip}{ft}$

Check required anchor load to resist sliding and overturning.

Anchor Loca on: X_a 6.00 ft

Sum the Moments:

$M_o \quad R_{soil} \; Y_{soil} \quad R_{u1} \; X_{u1} \quad R_{u2} \; X_{u2} \quad R_{head} \; Y_{head}$ 124.9 $\frac{kip\ ft}{ft}$

$M_R \; W_{PDF} \; X_{PD} \quad W_G \; X_G \quad T_a \; X_a \quad R_{tail} \; Y_{tail}$ 165.1 $\frac{kip\ ft}{ft}$

$M_{Total} \quad M_R \quad M_o$ 40.22 kip $\frac{ft}{ft}$

Sum Ver cal Forces:

$R_v \quad W_{PDF} \quad W_G \quad R_{u1} \quad R_{u2} \quad T_a$ 20.14 $\frac{kip}{ft}$

eccentricity from Toe: $e_o \left| \frac{M_R \quad M_o}{R_v} \right|$ e_o 2.00 ft

eccentricity from Center line: $e_r \left| \frac{W_{Precast}}{2} \quad e_o \right|$ e_r 2.00 ft

OK equal or less than

$\frac{W_{Precast}}{4}$ 2.00 ft

Resultant Loca on: $x_{Ra} \quad \frac{M_{Total}}{}$ 2.00 ft

Resultant Ra o: $r_{Ra} \quad \frac{x_{Ra}}{}$ 0.25

Base in Compression (%): C_r min 3 r_{Ra} 1.0 0.75

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Check Bearing Capacity:

$\sigma_{allowable}$ 0.5 4000psi 2000.00 psi

Find the Bearing Pressure at the toe and heel:

Effec ve Width: W_e $W_{Precast}$ 2 e_r 3.99 ft

Bearing Pressure: q_{toea} $\frac{R_v}{W_{Precast}}$ 1 $\frac{6\ e_r}{W_{Precast}}$ 43.76 psi

Bearing Heel: q_{heela} $\frac{R_v}{W_{Precast}}$ 1 $\frac{6\ e_r}{W_{Precast}}$ 8.79 psi

Concrete is OK.

Check Factor of Safety Against Sliding:

N Force: N_F μR_v c_a $W_{Precast}$ N_F 23.15 $\frac{kip}{ft}$

T Force: T_F R_{soil} R_{head} R_{tail} T_F 10.7 $\frac{kip}{ft}$

FS_s $\frac{N_F}{T_F}$ 2.16 Greater than or equal to 1.7, therefore: Okay

Find the Load required per rock anchor connec on:

S_a 4ft (Rock Anchors at 4 feet oc) F_{anchor} T_a S_a F_{anchor} 82 kip

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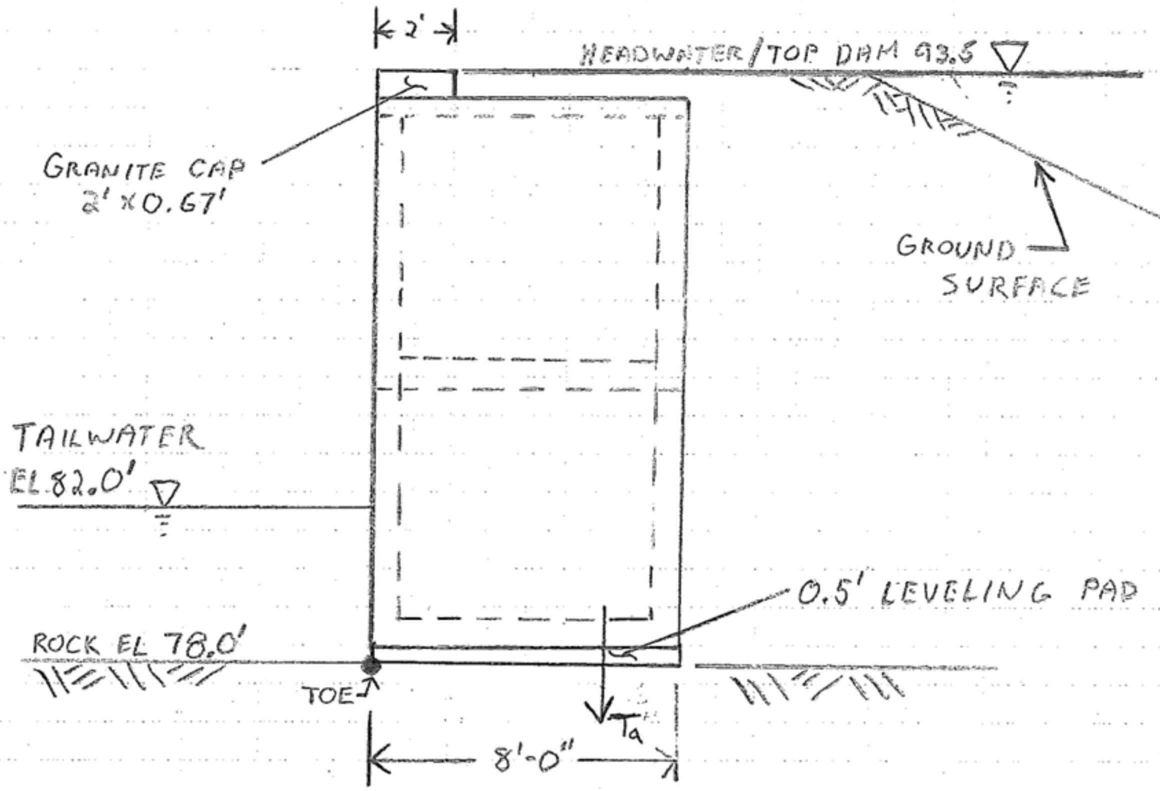
LOAD CASE 3: NORMAL POOL + EARTHQUAKE



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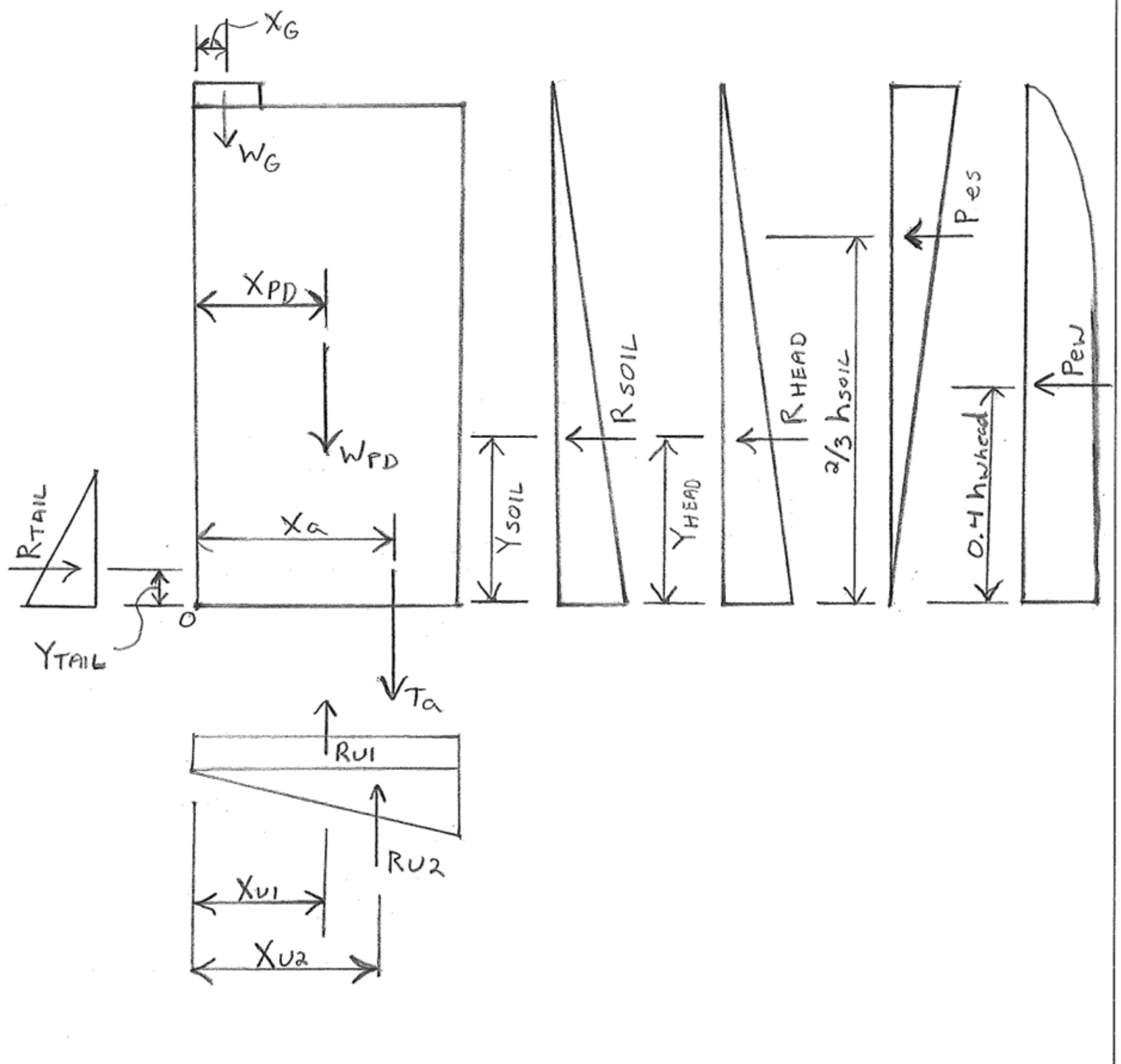
LOAD CASE 3: NORMAL POOL + EARTHQUAKE



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Stability Analysis for Load Case 3 - Normal Pool with Earthquake

Headwater Eleva on: El_{hw} 93.5ft

Tailwater Eleva on: El_{tail} 82.0ft

Compute Water Pressures and Resultant Forces:

Depth of Tailwater:	$hw_{tail}\ El_{tail}\ El_{rock}$	hw_{tail}	4.00 ft
Depth of Headwater:	$hw_{head}\ El_{hw}\ El_{rock}$	hw_{head}	15.50 ft
Depth of Soil:	$h_s\ El_{top}\ El_{rock}$	h_s	15.50 ft
Tail Water:	$P_{tail}\ \gamma_w\ hw_{tail}$	P_{tail}	249.60 psf
	$R_{tail}\ \frac{1}{2} hw_{tail}\ P_{tail}$	R_{tail}	499.20 plf
Loca on About Toe:	$Y_{tail}\ \frac{1}{3} hw_{tail}$	Y_{tail}	1.33 ft
Head Water:	$P_{head}\ \gamma_w\ hw_{head}$	P_{head}	967.20 psf
	$R_{head}\ \frac{1}{2} hw_{head}\ P_{head}$	R_{head}	7495.80 plf
	$Y_{head}\ \frac{1}{3} hw_{head}$	Y_{head}	5.17 ft

Upli : Upli pressure is calculated along the en re width of the precast units.

	$R_{u1}\ P_{tail}\ W_{Precast}$	R_{u1}	1996.80 plf
	$R_{u2}\ \frac{1}{2}\ W_{Precast}\ P_{head}\ P_{tail}$	R_{u2}	2870.40 plf
Loca on About Toe:	$X_{u1}\ \frac{1}{2} W_{Precast}$	X_{u1}	4.00 ft
	$X_{u2}\ \frac{2}{3} W_{Precast}$	X_{u2}	5.33 ft

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Compute Soil Pressures and Resultant Forces:

$$P_{soil} \quad K_a \quad \gamma_f \quad \gamma_w \quad h_s$$ P_{soil} 298.1 psf

$$R_{soil} \quad \frac{1}{2} P_{soil} \quad h_s$$ R_{soil} 2310.5 plf

Loca on About Toe: $$Y_{soil} \quad \frac{1}{3} h_s$$ Y_{soil} 5.17 ft

Compute Dead Loads:

Precast Dam (2 Blocks High, and 3 bo oms/tops):

(Volume Total) (Void Height) (Void Width) (Void Length)

$$W_{PD} \quad H_{Precast} \quad W_{Precast} \quad L_{Precast}$$

$$H_{Precast} \quad 3H_c \quad W_{Precast} \quad 2H_c \quad L_{Precast} \quad 2\ H_c \quad \gamma_c$$

W_{PD} 55.60 kip

Precast Dead Load Per Foot: $$W_{PDF} \quad \frac{W_{PD}}{L_{Precast}}$$ W_{PDF} 6949.86 plf

Loca on About Toe: $$X_{PD} \quad \frac{1}{2} W_{Precast}$$ X_{PD} 4.00 ft

Granite Cap:

$$W_G \quad T_{Granite} \quad W_{Granite} \quad \gamma_G$$ W_G 221.10 plf

Loca on About Toe: $$X_G \quad \frac{1}{2} W_{Granite}$$ X_G 1.00 ft

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Calcula on of Seismic Earth Pressure using Mononobe-Okabe Equa on

Use accelera on coefficients from Original Design

α_a 0.34 g $\quad k_h\ \alpha_a \quad k_v\ \frac{2}{3}\alpha_a$

Angle of Resultant Accelera on (from ver cal): $\Psi \quad \text{atan}\frac{k_h}{1\ k_v}$ $\quad \Psi$ 23.73 °

Earth Pressure Coefficient: K_{ae} $\dfrac{\cos\ \phi_f\ \ \Psi\ \ \theta^2}{\cos(\Psi)\ (\cos(\theta))^2\ \cos\theta\ \ \delta_f\ \ \Psi\ \ 1\ \sqrt{\dfrac{\sin\ \phi_f\ \ \delta_f\ \ \sin\ \phi_f\ \ \Psi\ \ \ \beta}{\cos\theta\ \delta_f\ \ \Psi\ \ \ \cos(\beta\ \ \ \theta)}}^{\,2}}$

$\boxed{K_{ae}\ 0.738}$

Total Force for Soil: $P_{ae}\ \frac{1}{2}\ K_{ae}\ h_s{}^2\ \gamma_f\ \ \gamma_w$ $\quad P_{ae}$ 5.55 klf

Seismic Increment: $P_{es}\ \ P_{ae}\ \ R_{soil}$ $\quad P_{es}$ 3.24 klf

Moment due to Seismic Increment: $M_{es}\ \frac{2}{3}\ P_{es}\ h_s$ $\quad M_{es}$ 33.49 $\frac{\text{kip ft}}{\text{ft}}$

Calcula on of Seismic Water Pressure using Westergards Equa on

C_e 0.051 $\frac{\text{kip}}{\text{ft}^3}$ (EM1110 2 2502)

$P_{ew}\ \frac{2}{3}\ C_e\ k_h\ hw_{head}{}^2$ $\quad P_{ew}$ 2.78 klf

M_{ew} 0.4 $hw_{head}\ P_{ew}$ $\quad M_{ew}$ 17.22 $\frac{\text{kip ft}}{\text{ft}}$

Calcula on of Seismic Internal Forces:

$P_i\ \ W_{PDF}\ \ W_G\ \ k_h$ $\quad P_i$ 2.44 klf

$M_i\ k_h\ W_{PDF}\ X_{PD}\ \ W_G\ X_G$ $\quad M_i$ 9.53 $\frac{\text{kip ft}}{\text{ft}}$

Total Earthquake:

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P_{eq} P_{es} P_{ew} P_i P_{eq} 8.46 klf

M_{eq} M_{es} M_{ew} M_i M_{eq} 60.23 $\frac{kip\ ft}{ft}$

By inspec on rock anchors will be required to provide an adequate factor of safety against sliding, and to meet requirements for bearing pressure and loca on of the resultant within the base of the Dam.

Compute Tension Forces:

Total Rock Anchor: T_a Loca on About Toe: X_a 6ft

Check Overturning:

For overturning the resultant force should be within the base.

Eccentricity from the centerline needs to be equal or less than: WPrecast 2

Anchor Load: T_a 17.6 $\frac{kip}{ft}$

Check required anchor load to resist sliding and overturning.

Anchor Loca on: X_a 6.00 ft

Sum the Moments:

M_o R_{soil} Y_{soil} R_{u1} X_{u1} R_{u2} X_{u2} R_{head} Y_{head} M_{eq} 134.2 $\frac{kip\ ft}{ft}$

M_R W_{PDF} X_{PD} W_G X_G T_a X_a R_{tail} Y_{tail} 134.3 $\frac{kip\ ft}{ft}$

M_{Total} M_R M_o 0.09 kip $\frac{ft}{ft}$

Sum Ver cal Forces:

R_v W_{PDF} W_G R_{u1} R_{u2} T_a 19.90 $\frac{kip}{ft}$

eccentricity from Toe: e_o $\left|\frac{M_R\ M_o}{R_v}\right|$ e_o 0.00 ft

eccentricity from Center line: e_r $\left|\frac{W_{Precast}}{2}\ e_o\right|$ e_r 4.00 ft

OK equal or less than $\frac{W_{Precast}}{2}$ 4.00 ft

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Resultant Loca on: $x_{Ra} \; \frac{M_{Total}}{R_V}$ 0.00 ft

Resultant Ra o: $r_{Ra} \; \frac{x_{Ra}}{W_{Precast}}$ 0.00

Base in Compression (%): C_r min 3 r_{Ra} 1.0 0.00

Resultant is within the base - OK

Check Bearing Capacity:

$\sigma_{allowable}$ 0.9 4000psi 3600.00 psi

Find the Bearing Pressure at the toe and heel:

Effec ve Width: $W_e \; W_{Precast} \; 2 \, e_r$ 0.01 ft

Bearing Pressure: $q_{toea} \; \frac{R_V}{W_{Precast}} \; 1 \; \frac{6 \, e_r}{W_{Precast}}$ 69.05 psi

Bearing Heel: $q_{heela} \; \frac{R_V}{W_{Precast}} \; 1 \; \frac{6 \, e_r}{W_{Precast}}$ 34.50 psi

Concrete is OK.

Check Factor of Safety Against Sliding:

N Force: $N_F \; \mu \, R_v \; c_a \, W_{Precast}$ N_F 23.01 $\frac{kip}{ft}$

T Force: $T_F \; R_{soil} \; R_{head} \; R_{tail} \; P_{eq}$ T_F 17.8 $\frac{kip}{ft}$

$FS_S \frac{N_F}{T_F}$ 1.30 Greater than or equal to 1.3, therefore: Okay

Find the Load required per rock anchor connec on:

S_a 4ft (Rock Anchors at 4 feet oc) $F_{anchor} \; T_a \, S_a$ F_{anchor} 70 kip

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Rock Anchor Design:

Design Rock Anchor. Size bar and bond lengths based on *PTI Recommenda ons for Prestressed Rock and Soil Anchors*.

Load/   in Rock Anchor (see stability analysis):

Load Case 1: T_{LC1} 15.47klf

Load Case 2: T_{LC2} 20.61klf

Load Case 3: T_{LC3} 17.6klf T_{max} max T_{LC1} T_{LC2} T_{LC3} 20.61 klf

Rock Anchor Spacing: S_p 4ft

Total Load per Anchor: T_D T_{max} S_p 82.44 kip (Design Load)

T_{TL} T_D 1.33 109.65 kip (Test Load)

Bar Size:

Per PTI Recommenda ons (5th Ed.), anchors shall not be stressed to more than 80% of GUTS.

Use Williams 2-inch-diamete Hollow Bar.

For 2-inch-diameter Rock Bolt:

Minimum Ul mate Strength: T_u 188kip

Minimum Yield Strength: T_y 152kip

T_{all} 0.6 T_y T_{all} 91.20 kip Greater than T_D, therefore OK.

Max Test Load: T_{mtl} .8T_u 150.4 kip Greater than T_{TL}, therefore OK.

Use Williams 2-inch-diameter Hollow Bar

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Bond Length:

Diameter of grout: D_g 3.5in

Bond Strength of Rock: σ_{urock} 250psi

Allowable Bond Strength of Rock: $\sigma_{arock} \quad \frac{\sigma_{urock}}{2.5}$ σ_{arock} 100.00 psi

Bond Strength: $BS \quad D_g \, \pi \, \sigma_{arock}$ BS 13.2 klf

Bond Length: $L_{bond} \quad \frac{T_D}{BS}$ L_{bond} 6.25 ft

Use a minimum of 20 foot embedment to match original design.

Check for single anchor and group failure related to the engaged rock mass based on bond length. Assume a 45 degree cone origina ng the base of the bond length and conserva vely assume only the mass of the rock directly above the cone.

Load/ Rock Anchor (see stability analysis): T_{max} 20.61 klf

Rock Anchor Spacing: S_p 4ft

Total Load per Rock Anchor: $T_D \quad T_{max} \, S_p$ 82.44 kip

Unit Weight of Rock: γ'_r 140pcf 62.4pcf 77.60 pcf

Bond Length: L_{cone} 20ft

Radius of Rock Cone = $R_r \quad L_{cone} \tan(45 \text{ deg})$ 20.00 ft *Maximum radius with no overlap*

Volume = $V_r \quad \pi \, R_r^{2} \, \frac{L_{cone}}{3}$ 8378 ft^3

Weight = $W_r \quad V_r \, \gamma'_r$ 650 kip

Factor of Safety: $FS_C \quad \frac{W_r}{T_D}$ 7.89

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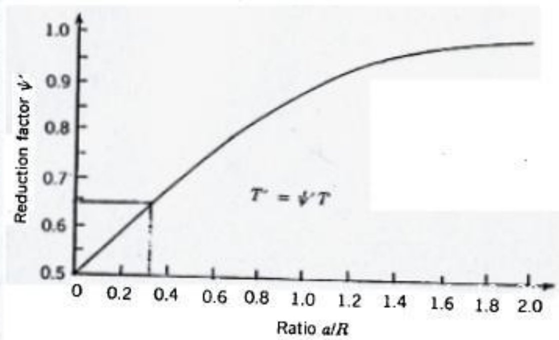
Calculate group failure since the cones will overlap. $a \; S_p$

$\frac{a}{R_r} \quad 0.20$

$\psi \quad 0.59$

STABILITY OF A MASS OF GROUND 217



a/R	ψ'	a/R	ψ'
0	0.50	1.1	0.91
0.1	0.55	1.2	0.93
0.2	0.59	1.3	0.95
0.3	0.64	1.4	0.97
0.4	0.68	1.5	0.98
0.5	0.72	1.6	0.98
0.6	0.76	1.7	0.99
0.7	0.80	1.8	0.99
0.8	0.85	1.9	0.99
0.9	0.87	2.0	1.0
1.0	0.89		

Fig. 5-9 Adjusted uplift anchor capacity calculated from the volume of cone influence as function of the ratio *a/R*.

Group Weight: $W_{Group} \; \psi \, W_r \quad 383.56 \text{ kip}$

Factor of Safety on Group: $FS_{Group} \; \frac{W_{Group}}{T_D} \quad 4.65$

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Precast Modular Design:

Check Bolted Connec on between Units so Dam performs as unit:

Design Bolts between top and bo om unit:

Based on Previous Stability Analysis assume worst case is Load Case 2: 100 Year Flood.

Headwater Eleva on: El_{hw} 97.7ft

Tailwater Eleva on: El_{tail} 88.9ft

Compute Water Pressures and Resultant Forces on top block:

Bo om of Top Block: El_{Tblock} 85.67ft

Depth of Tailwater: $hw_{tail}\ El_{tail}\ El_{Tblock}$ hw_{tail} 3.23 ft

Depth of Headwater: $hw_{head}\ El_{hw}\ El_{Tblock}$ hw_{head} 12.03 ft

Depth of Soil: $h_s\ El_{top}\ El_{Tblock}$ h_s 7.83 ft

Tail Water: $P_{tail}\ \gamma_w\ hw_{tail}$ P_{tail} 201.55 psf

$R_{tail}\ \frac{1}{2} hw_{tail}\ P_{tail}$ R_{tail} 325.51 plf

Loca on About Toe: $Y_{tail}\ \frac{1}{3} hw_{tail}$ Y_{tail} 1.08 ft

Head Water: $P_{head}\ \gamma_w\ hw_{head}$ P_{head} 750.67 psf

$R_{head}\ \frac{1}{2}\ hw_{head}\ P_{head}$ R_{head} 4515.29 plf

Loca on About Toe: $Y_{head}\ \frac{1}{3} hw_{head}$ Y_{head} 4.01 ft

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Upli : Upli pressure is calculated along the en re width of the precast units.

$R_{u1} \quad P_{tail} \quad W_{Precast}$ $\qquad R_{u1}$ 1612.42 plf

$R_{u2} \quad \frac{1}{2} W_{Precast} \quad P_{head} \quad P_{tail}$ $\qquad R_{u2}$ 2196.48 plf

Loca on About Toe: $X_{u1} \quad \frac{1}{2} W_{Precast}$ $\qquad X_{u1}$ 4.00 ft

$X_{u2} \quad \frac{2}{3} W_{Precast}$ $\qquad X_{u2}$ 5.33 ft

Compute Soil Pressures and Resultant Forces:

$P_{soil} \quad K_a \quad \gamma_f \quad \gamma_w \quad h_s$ $\qquad P_{soil}$ 150.6 psf

$R_{soil} \quad \frac{1}{2} P_{soil} \quad h_s$ $\qquad R_{soil}$ 589.6 plf

Loca on About Toe: $Y_{soil} \quad \frac{1}{3} h_s$ $\qquad Y_{soil}$ 2.61 ft

Compute Dead Loads:

Precast Dam (2 Blocks High, and 3 bo oms/tops):

$$W_{PD} \quad \frac{\overset{\text{(Volume Total)}}{H_{Precast}\ W_{Precast}\ L_{Precast}} \quad H_{Precast} \quad \overset{\text{(Void Height)}}{3H_c} \quad \overset{\text{(Void Width)}}{W_{Precast}\ 2H_c} \quad \overset{\text{(Void Length)}}{L_{Precast}\ 2\,H_c}}{2} \quad \gamma_c$$

W_{PD} 27.80 kip (one block)

Precast Dead Load Per Foot: $W_{PDF} \quad \frac{W_{PD}}{L_{Precast}}$ $\qquad W_{PDF}$ 3474.93 plf

Loca on About Toe: $X_{PD} \quad \frac{1}{2} W_{Precast}$ $\qquad X_{PD}$ 4.00 ft

Granite Cap:

$W_G \quad T_{Granite} \quad W_{Granite} \quad \gamma_G$ $\qquad W_G$ 221.10 plf

Loca on About Toe: $X_G \quad \frac{1}{2} W_{Granite}$ $\qquad X_G$ 1.00 ft

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Compute Tension Forces on bolts between bo om and top precast units:

Total bolt Anchor: T_a Four bolts, one at each corner 9 inches from edge of concrete.

Check moment capacity of bolt:

For moment capacity, assume tension of two bolts located at upstream side of the units rota ng around downstream edge of precast units: Load factors for analysis are from ASCE Strength Design For Reinforced Hydraulic Structures.

Load Factors:

Load Case	1 - Unusual	2 - Unusual	3 - Extreme
Load Factor	1.3*1.7*(DL+LL)	1.3*1.7*(DL+LL)	1.1*(DL+LL)+1.25*E

LF 1.3 1.7 LF 2.21 LF_E 1.25

Bolt Tension Load: T_a 7.04 $\frac{kip}{ft}$ *Check required bolt load to resist moment*

Anchor Loca on: X_b 7.33ft Two bolts, at upstream side of the units:

Sum the Moments:

M_o LF R_{soil} Y_{soil} R_{u1} X_{u1} R_{u2} X_{u2} R_{head} Y_{head} 83.6 $\frac{kip\ ft}{ft}$

M_R T_a X_b LF W_{PDF} X_{PD} W_G X_G R_{tail} Y_{tail} 83.6 $\frac{kip\ ft}{ft}$

M_{Total} M_R M_o 0.03 kip $\frac{ft}{ft}$

Find the Load required per bolt:

T_{block} T_a $W_{Precast}$ 56.32 kip

N_{bolt} 2

T_{bolt} $\frac{T_{block}}{N_{bolt}}$

T_{bolt} 28.16 kip

Use 1" Diameter A490 Bolt.
Per AISC Table 7-2 Available Tensile Strength
$\varphi\ r_n$ = 66.6 kips

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Check shear capacity of bolts, (Conserva vely assume no fric on or shear capacity of concrete key):

$V_F \quad R_{soil} \quad R_{head} \quad R_{tail} \quad LF$

$V_F \quad 10.6 \frac{kip}{ft}$

Find the Load required per bolt:

$V_{block} \quad V_F \quad W_{Precast} \quad 84.50\ kip$

$N_{bolt} \quad 4$

$V_{bolt} \quad \frac{V_{block}}{N_{bolt}}$

$V_{bolt} \quad 21.12\ kip$

Use 1" Diameter A490 Bolt.
Per AISC Table 7-1 Available Shear
Strength $\varphi_v r_n$ = 35.3 kips

Check combined Tension and shear:

Nominal Bolt Area: $A_{bolt} \quad 0.785in^2 \quad F_{nt}$

113ksi

$F_{nv} \quad 60ksi$

$f_v \quad \frac{V_{bolt}}{A_{bolt}} \quad 26.91\ ksi$

$\phi \quad 0.75$

$F_{nnt} \quad 1.3F_{nt} \quad \frac{F_{nt}}{\phi F_{nv}} f_v$

$F_{nnt} \quad 79.32\ ksi$

Less than F_{nt} therefore OK

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Design Bolts between horizontal unit:

Based on Previous Stability Analysis look at seismic Internal Forces:

Weight of Each Precast Unit:

Precast Dam (2 Blocks High, and 3 bo oms/tops):

(Volume Total) (Void Height) (Void Width) (Void Length)

$$W_{PD} \quad \frac{H_{Precast} \; W_{Precast} \; L_{Precast} \qquad H_{Precast} \; 3H_c \quad W_{Precast} \; 2H_c \quad L_{Precast} \; 2\,H_c}{2} \; \gamma_c$$

W_{PD} 27.80 kip (one block)

Precast Dead Load Per Foot: $W_{PDF} \; \frac{W_{PD}}{L_{Precast}}$ W_{PDF} 3474.93 plf

Calcula on of Seismic Internal Forces:

$P_i \; W_{PDF} \, k_h$ P_i 1.18 klf

Total Shear required:

$V_s \quad P_i \, W_{Precast} \; LF_E$ V_s 11.81 kip

N_{bolt} 4

$V_{bolt} \; \frac{V_s}{N_{bolt}}$ V_{bolt} 2.95 kip

Use 1" Diameter A490 Bolt to match other bolts. Per AISC Table 7-1 Available Shear Strength

$\varphi_v \, r_n = 35.3$ kips

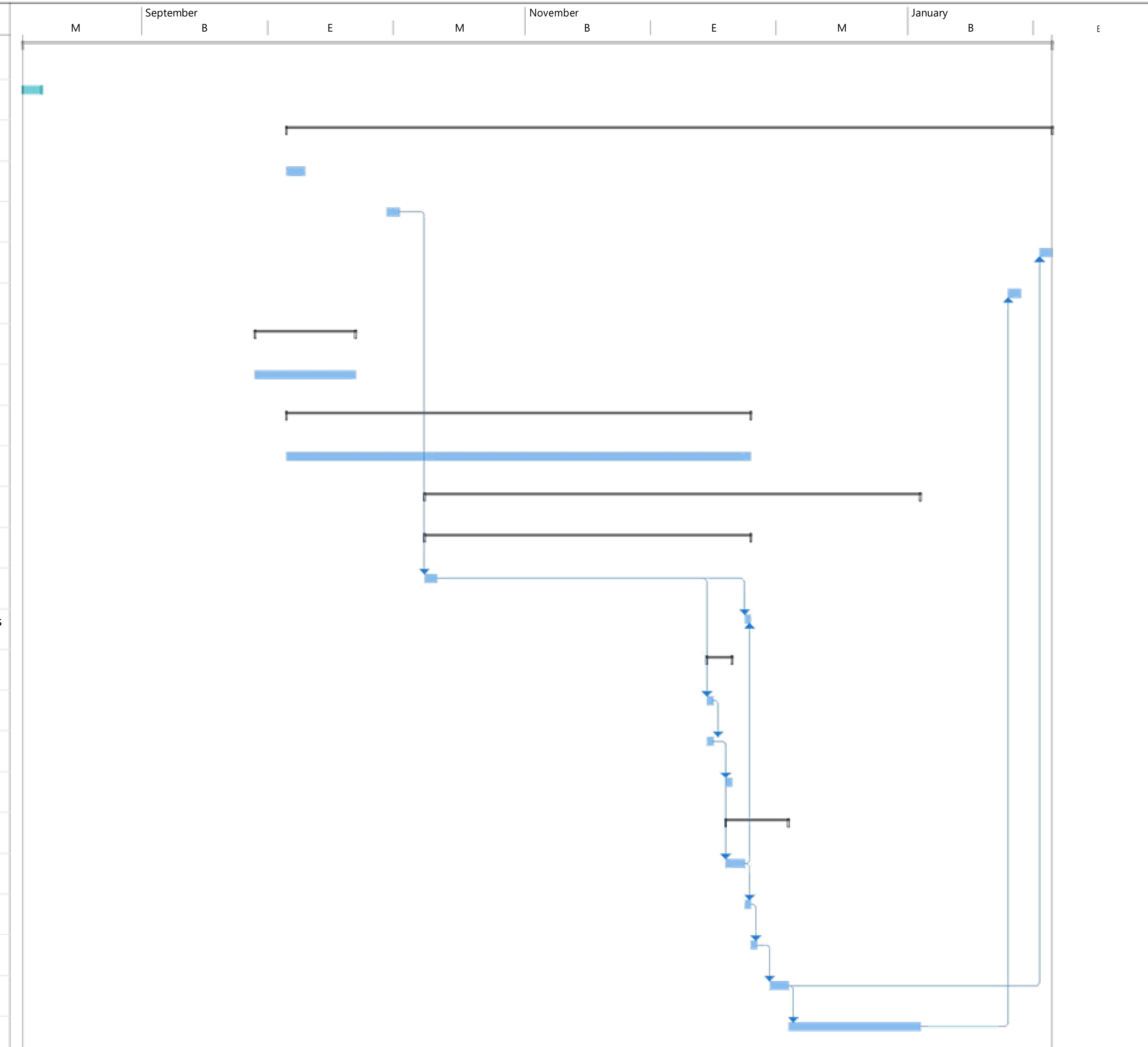
6.2: Schedule

Gantt Chart Schedule - Anchor Bolt and Dam Footing Start Dates Fixed to Photodocumentation Log dates

Gantt Chart Schedule - Anchor Bolt and Dam Footing Start Dates Free to Move

ALLENDALE DAM - FRENCH CONSTRUCTION SCHEDULE (BASED ON PHOTODOCUMENTATION REPORT)

ID	Task Mode	Task Name	Duration	Start	Finish	Predecessors
0		**Reconstruction of Allendale Dam - FRENCH Construction Schedule**	**118 days**	**Mon 8/13/01**	**Wed 1/23/02**	
1		Tree Cutting and Clearing	3 days	Mon 8/13/01	Wed 8/15/01	
2		**Surface Water Control and Diversion**	**88 days**	**Mon 9/24/01**	**Wed 1/23/02**	
3		Upstream Cofferdam Installation	3 days	Mon 9/24/01	Wed 9/26/01	
4		Downstream Cofferdam Installation	2 days	Wed 10/10/01	Thu 10/11/01	
5		Upstream Cofferdam Removal	2 days	Tue 1/22/02	Wed 1/23/02	23FS+28 days
6		Downstream Cofferdam Removal	2 days	Thu 1/17/02	Fri 1/18/02	24
7		**Concrete Wingwall**	**12 days**	**Wed 9/19/01**	**Thu 10/4/01**	
8		Concrete Wingwall	12 days	Wed 9/19/01	Thu 10/4/01	
9		**Soil and Sediment Excavation**	**54 days**	**Mon 9/24/01**	**Thu 12/6/01**	
10		Soil and Sediment Excavation	54 days	Mon 9/24/01	Thu 12/6/01	
11		**Dam Construction**	**57 days**	**Tue 10/16/01**	**Wed 1/2/02**	
12		**Rock Anchor Bolts**	**38 days**	**Tue 10/16/01**	**Thu 12/6/01**	
13		Install Rock Anchors	2 days	Tue 10/16/01	Wed 10/17/01	4
14		Test Rock Anchors	1 day	Thu 12/6/01	Thu 12/6/01	20,13FS+28 days
15		**Concrete Working Pad**	**2 days**	**Fri 11/30/01**	**Mon 12/3/01**	
16		Form	1 day	Fri 11/30/01	Fri 11/30/01	13
17		Concrete Pour	1 day	Fri 11/30/01	Fri 11/30/01	16FS-1 day
18		Strip Pour 1	1 day	Mon 12/3/01	Mon 12/3/01	17
19		**Precast Dam (12 Units per day: 26 Total)**	**8 days**	**Mon 12/3/01**	**Wed 12/12/01**	
20		Install Precast Units with Granite Capstone attached to Precast	3 days	Mon 12/3/01	Wed 12/5/01	17
21		Form Closure pours	1 day	Thu 12/6/01	Thu 12/6/01	20
22		Concrete Closure Pour	1 day	Fri 12/7/01	Fri 12/7/01	21
23		Strip Closure Pour	3 days	Mon 12/10/01	Wed 12/12/01	22
24		Downstream Riprap	15 days	Thu 12/13/01	Wed 1/2/02	23

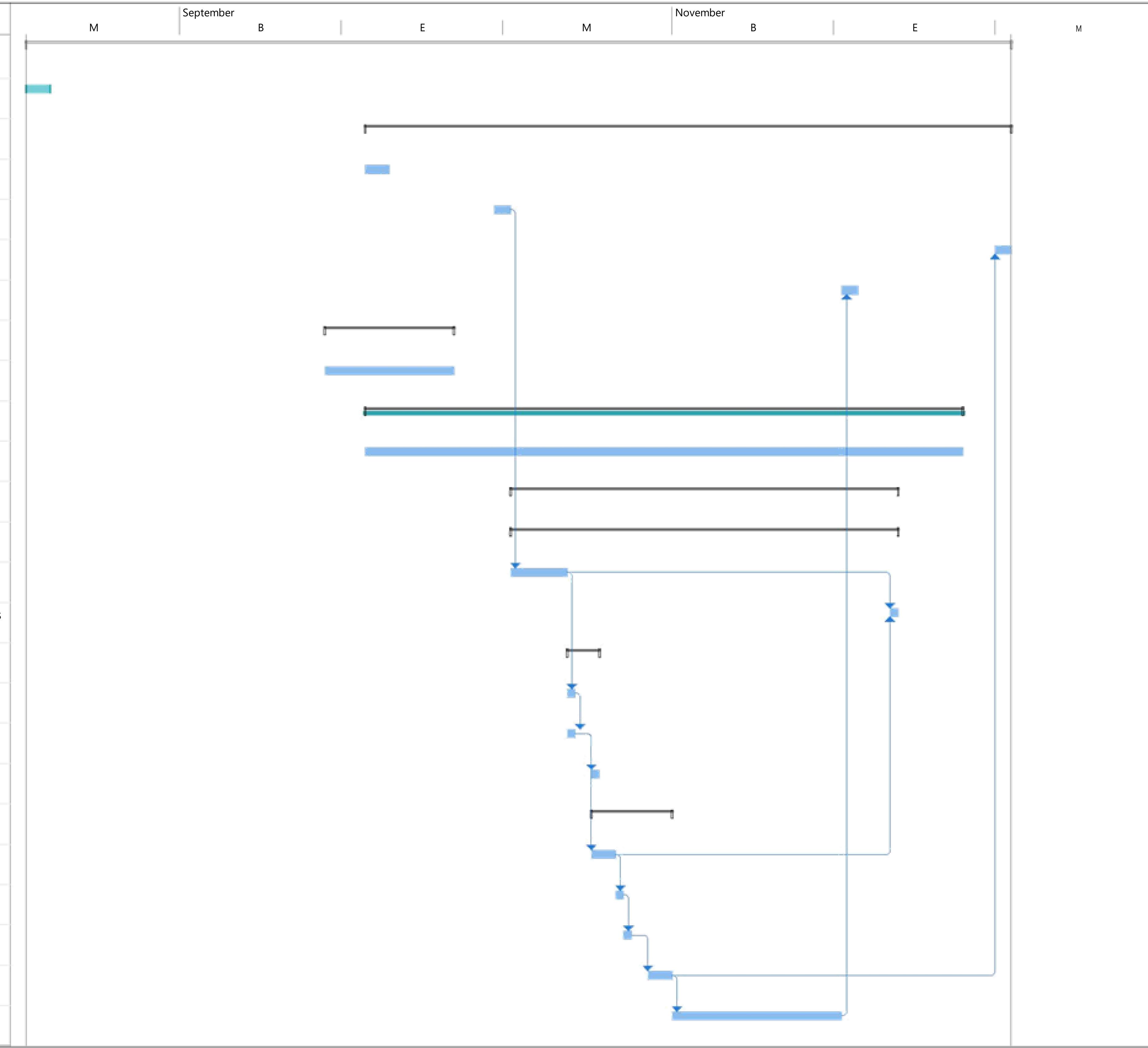


Project: Reconstruction of Allen
Date: Thu 12/8/16

Task | Summary | Inactive Milestone | Duration-only | Start-only | External Milestone | Manual Progress
Split | Project Summary | Inactive Summary | Manual Summary Rollup | Finish-only | Deadline
Milestone | Inactive Task | Manual Task | Manual Summary | External Tasks | Progress

ALLENDALE DAM - FRENCH CONSTRUCTION SCHEDULE (DATES FREE TO MOVE)

ID	Task Mode	Task Name	Duration	Start	Finish	Predecessors
0		**Reconstruction of Allendale Dam - FRENCH Construction Schedule**	**88 days**	**Mon 8/13/01**	**Wed 12/12/01**	
1		Tree Cutting and Clearing	3 days	Mon 8/13/01	Wed 8/15/01	
2		**Surface Water Control and Diversion**	**58 days**	**Mon 9/24/01**	**Wed 12/12/01**	
3		Upstream Cofferdam Installation	3 days	Mon 9/24/01	Wed 9/26/01	
4		Downstream Cofferdam Installation	2 days	Wed 10/10/01	Thu 10/11/01	
5		Upstream Cofferdam Removal	2 days	Tue 12/11/01	Wed 12/12/01	23FS+28 days
6		Downstream Cofferdam Removal	2 days	Thu 11/22/01	Fri 11/23/01	24
7		**Concrete Wingwall**	**12 days**	**Wed 9/19/01**	**Thu 10/4/01**	
8		Concrete Wingwall	12 days	Wed 9/19/01	Thu 10/4/01	
9		**Soil and Sediment Excavation**	**54 days**	**Mon 9/24/01**	**Thu 12/6/01**	
10		Soil and Sediment Excavation	54 days	Mon 9/24/01	Thu 12/6/01	
11		**Dam Construction**	**34 days**	**Fri 10/12/01**	**Wed 11/28/01**	
12		**Rock Anchor Bolts**	**34 days**	**Fri 10/12/01**	**Wed 11/28/01**	
13		Install Rock Anchors	5 days	Fri 10/12/01	Thu 10/18/01	4
14		Test Rock Anchors	1 day	Wed 11/28/01	Wed 11/28/01	20,13FS+28 days
15		**Concrete Working Pad**	**2 days**	**Fri 10/19/01**	**Mon 10/22/01**	
16		Form	1 day	Fri 10/19/01	Fri 10/19/01	13
17		Concrete Pour	1 day	Fri 10/19/01	Fri 10/19/01	16FS-1 day
18		Strip Pour 1	1 day	Mon 10/22/01	Mon 10/22/01	17
19		**Precast Dam (12 Units per day: 26 Total)**	**8 days**	**Mon 10/22/01**	**Wed 10/31/01**	
20		Install Precast Units with Granite Capstone attached to Precast	3 days	Mon 10/22/01	Wed 10/24/01	17
21		Form Closure pours	1 day	Thu 10/25/01	Thu 10/25/01	20
22		Concrete Closure Pour	1 day	Fri 10/26/01	Fri 10/26/01	21
23		Strip Closure Pour	3 days	Mon 10/29/01	Wed 10/31/01	22
24		Downstream Riprap	15 days	Thu 11/1/01	Wed 11/21/01	23



Project: Reconstruction of Allen
Date: Thu 12/8/16

Task	Summary	Inactive Milestone	Duration-only	Start-only	External Milestone
Split	Project Summary	Inactive Summary	Manual Summary Rollup	Finish-only	Deadline
Milestone	Inactive Task	Manual Task	Manual Summary	External Tasks	Progress
					Manual Progress

6.3: Cost Estimates

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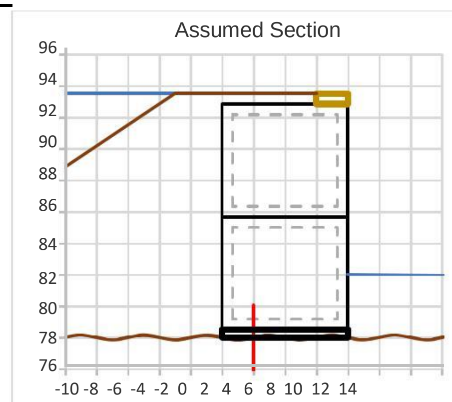
1.B Precast Modular Dam (To Match Allendale)



Assumptions

Length		105	LF
Precast Unit Columns		14	#
Precast Unit Rows		2	#
Number of Units		28	#
PC Concrete Volume per Unit		8	CY/Unit
Total PC Conc Volume		217	CY
PC Conc Vol / Unit Length		2.1	CY/LF
Units Shipped per Truck		1	#
Shipments per Day per Truck		6	#/day
Rock Anchors Per Column		2	#/Column
Assumed Site Prep Time		24	Crew Hours
Crew Cost Per Hour		$681	per hour
Abutment Concrete Volume		9	CY
Abutment Concrete Unit Cost		$1,200	per CY
Mud Mat Concrete Volume		15	CY
Mud Mat Concrete Unit Cost		$500	per CY
Site Prep Subtotal		$34,419	
Production Startup Cost		$10,000	Allowance
PC Concrete Volume		217	CY
CIP Concrete Production Cost		$500	per CY
Connection Hardware Cost		$1,000	per Unit
Unit Production Subtotal (installed)		$146,466.39	
Precast Unit Installation Time		24	Crew Hours
Precast Unit Installation Cost Per Hour		$1,101	per hour
Precast Unit Installation Subtotal		$26,415	
Number of Rock Anchors		28	#
Rock Anchors Total Unit Cost		$2,000	EA
Rock Anchors Subtotal (installed)		$56,000	
Subtotal		$263,300	
Project Overhead	20%	$52,660	
Total Price		**$315,961**	
Equivalent Unit Price		$1,456	per CY

Site Prep Crew Buildup

#	Description	Unit	Rate	Cost/Hr.
*********EQUIPMENT********				
1	Walk Behind Roller 20HP	HR	22.856	$22.86
1	Excavator, crawler, 1.5CY	HR	148.206	$148.21
1	Loader 2CY 130HP	HR	65.493	$65.49
1	Truck 3x 16TN, 12CY, 400HP	HR	99.504	$99.50
1	Pickup Crew 4x4 1 Ton Diesel	HR	24.208	$24.21
*********LABOR********				
2	Laborer-General	HR	56.55	$113.10
1	Op Eng- Foreman	HR	76.23	$76.23
1	Op Eng 2- Loader <6Y	HR	74.15	$74.15
1	End Dump Driver	HR	57.30	$57.30

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Subject	French Impoundment - Allendale Dam Costs					

Installation Crew Buildup

#	Description	Unit	Rate	Cost/Hr.
*********EQUIPMENT********				
1	Crane, SP, 4x4 w/ TB, 40TN	HR	102.494	$102.49
1	Excavator, crawler, 1.5CY	HR	148.206	$148.21
1	Loader 2CY 130HP	HR	65.493	$65.49
1	Truck tri-x 16TN, 12CY, 400HP	HR	99.504	$99.50
1	Pickup Crew 4x4 1 Ton Diesel	HR	24.208	$24.21
2	Tractor 6x4, 380 HP	HR	88.090	$176.18
2	Trailer, platform, tri-x 50TN	HR	23.920	$47.84
*********LABOR********				
3	Laborer-General	HR	56.55	$169.65
1	Op Eng- Foreman	HR	76.23	$76.23
1	Op Eng-Crane Operator	HR	76.23	$76.23
2	Flatbed Delivery Driver	HR	57.30	$114.60

Appendix B. Independent Engineer's Report

Knight Piésold
CONSULTING

Knight Piésold and Co.

1999 Broadway, Suite 600
Denver, Colorado 80202-5706
USA
Telephone: (303) 629-8788
Facsimile: (303) 629-8789
E-mail:
denver@knightpiesold.com

KP Project No.: DV103-00591/01
KP Doc. No.: DV-16-1014

December 5, 2016

French Development Enterprises, LLC
3 Survey Circle
North Billerica, MA 01862

Attn: Mr. William (Bill) French Sr.

Subject: Next Generation Hydro Project
North Billerica, MA

Dear Mr. French:

The following is the consultant letter reviewing and offering opinion on the French Development Enterprises, LLC (FDE) Project Plan and Modular System.

SCOPE OF WORK

The consultant will perform Task 4, Subtask 4.7 of the US Department of Energy (USDOE) SOPO Plan:

4. The consultant will conduct a (dry towel: Test in accordance with the Test Plan under the FDE Project Plan.
5. Verify that the structure meets targeted specification for water leakage.
6. Document the results of the test and compile a Prototype Test Report in accordance with subtask 4.14 of the USDOE SOPO Plan.
7. Produce an opinion letter on Utilization of the Modular Precast Concrete Technology in construction of new and rehabilitation of the existing Hydro and Water Control structures.

SITE VISIT

The consultant conducted a site visit on November 3, 2016 to the FDE test facility in North Billerica, MA. The weather was cloudy with periodic rain showers during the day.

Precast Concrete Module Test Arrangement

FDE had constructed a reinforced concrete test tank slab and wall. The test tank was used to test six 8 ft by 8 ft by 8 ft concrete precast modules. The modules were cast and transported to the test site and, previously assembled and stacked in the test tank prior to the site visit. The precast concrete modules were stacked two high and three across. The condition of the precast concrete modules was excellent without signs of transportation or assembly damage. See Photo 1 attached.

2.2 Temporary Bulkheads for Test Arrangement

Temporary Bulkheads were constructed on the left and right ends of the modules. The bulkheads filled the gap at the ends of the assembled modules and provided a water tight connection between the modules and the tank walls. The bulkheads were constructed of steel angles with plywood planking overladen with a geomembrane. The bulkheads were connected by bolts through the steel angles to the tank concrete walls and floor, and the adjacent precast concrete module wall. Sealant was used to seal between the precast concrete walls, the bulkhead and the reinforced concrete tank walls. See Photo 2 attached.

2.3 Module Waterstops

Rectangular hydrophilic waterstops were used at the base of the three precast modules and the tank floor. Also, rectangular hydrophilic waterstops were applied along the vertical wall surfaces between three precast concrete modules. Once modules were aligned, they were bolted one to another. The water stops were engaged by weight of the modules at the tank slab level, and through the mechanical bolting force on the sides of the precast concrete modules. The lower precast concrete modules were bolted to the floor slab after crane placement of the three lower precast concrete modules. The three upper precast concrete modules had similar bottom and top waterstop details. In addition to the waterstops, FDE applied sealant to the module wall surfaces. The combination of waterstops and sealants provided a satisfactory seal between modules.

FDE also constructed a small upstream geomembrane at the upstream floor interface with the modules. This membrane combined with the hydrophilic water stop, and sealants provided a satisfactory base seal.

3.0 CONSULTANT TESTING

3.1 Dry Towel Test & Substitution Test

The consultant was unable to perform a dry towel test. It was raining on the day of the site visit and during the time when the consultant was at the project site. The exterior concrete surfaces were damp.

The consultant had agreed that the tank should be filled to the pre- agreed 12 ft level with water and this was performed by FDE prior to the consultant’s visit. See Photo 3 attached. Water levels were marked and a manometer measured the water level in the tank. The original tank water level has not changed since original filling for four weeks. The tank test successfully demonstrates the hydraulic integrity of the assembled six modules, and is deemed satisfactory.

3.2 Visual Inspection

The consultant performed a detailed visual inspection of the downstream modular surfaces, and interior modular block surfaces. The upstream modular watered surface could only be viewed through the tank water. He also inspected the downstream surfaces of the bulkheads and their connection to the module walls and tank walls and floor. The hydraulic integrity was found to be water tights with no visible signs of leakage. No visible leakage meets the target specification for leakage.

4.0 CONCLUSION

4.1 General

The FDE prototype precast module has been demonstrated by a test tank test that it meets the desired structural and hydraulic integrity. No visible leakage was observed by the consultant during the November 3, 2016 site visit. The consultant has also monitored the water level in the test tank for a period of four weeks and observed that there has been no water level change.

4.2 Opinion

FDE has demonstrated with its prototype precast concrete module in an 8 ft by 8 ft by 8 ft module size, that a module stack of two modules 16 ft high and three modules 24 ft wide can act as a satisfactory dam. The tank test 12 ft high water level configuration for the 16ft high modular dam was that of a non-overflow structure with a 4 ft free board. The initial prototype module size was selected for ease of truck transport. See photo 4 attached. The six modules were manufactured off-site and transported to the test tank location. The modules were off-loaded and later marshalled by crane into final position. The stacked two high by three wide modular structure, was completed in less than one eight (8) hour shift.

It is the opinion of the consultant that the FDE prototype precast concrete module has wide potential for applications, and with adaptation of its design to actual project sites, can be used not only as a non-overflow structure, but also as an overflow (spillway), outlet and intake hydraulic structure. Further adaptation of design can be expected based on actual application for a temporary cofferdam, repair of existing dams and hydraulic structures, and new dam and hydraulic structures. The modularization concept has the potential of wide application and can be customized to any low head dam site. It offers the advantage of high quality control of precast concrete, increased design standardization, and reduces on-site construction time. It also has the flexibility to be combined with other traditional construction and concreting means and methods.

Thank-you for the opportunity to review the FDE prototype modular test and the FDE precast concrete modular system.

Sincerely,
Knight Piésold and Co.

N G Bishop

Norman Bishop, P.E., M.B.A.
Senior Executive Project Engineer

Enclosure: Photos 1 - 4
cc: Stuart Flett (Knight Piésold); Lenny Lozinsky (FDE)

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Photos

Knight Piésold
CONSULTING

Photo 1
Six Modules (2 high and 3 Wide) in Test Tank (Note Water on Upstream Side), No Leakage

Photo 2
Temporary Bulkhead between the Tank Wall (right) and Module (left)

Photo 3
Test Tank Filled to 12 ft. Water Level. Note Tank Wall (Right), Bulkhead (black) and Modules (left)

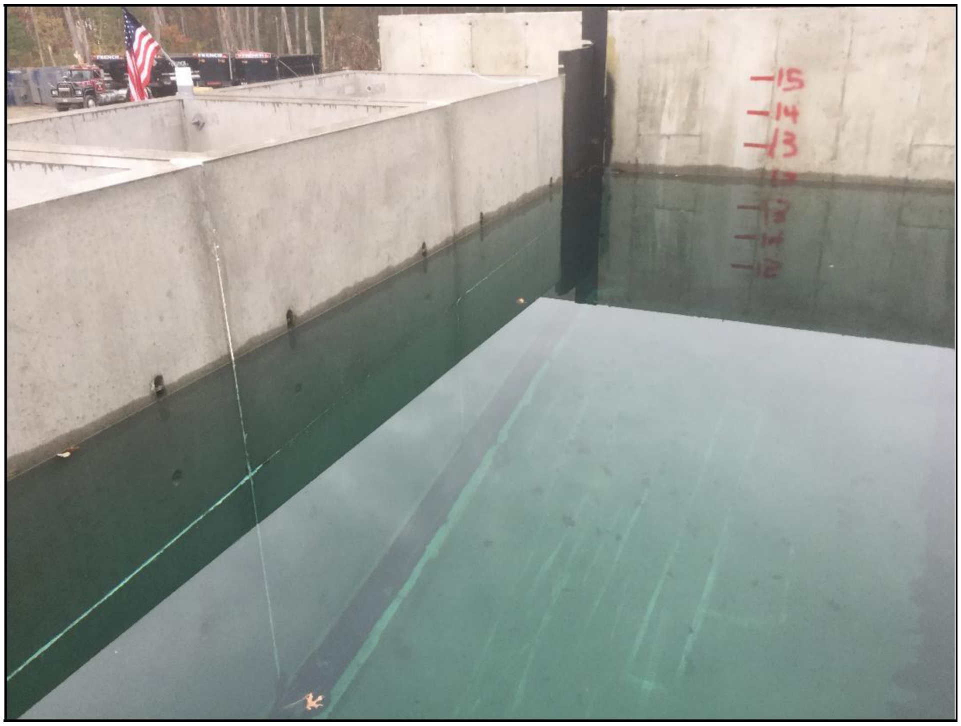


Photo 4
Precast Module Delivered by Truck to Test Site (Mobile Crane Behind Truck)

Appendix C. Field Observation Reports from Permeability Testing

FIELD OBSERVATION REPORT

Project :	Next Generation Hydro	**Date:**	Wednesday, Oct 12, 2016
Location:	North Billerica, MA	**Report No.**	013
Client :	French Development Enterprises	**Page:**	1 of 2
Contractor:	W.L. French Excavation	**GEI Proj. No.**	151669-0

Time of Arrival: 10:00 a.m. **Departure:** 3:30 p.m. **Weather:** 65's F, Sunny

Persons Contacted, Company
Bill French Sr.
Lenny Lozinsky
Scott
Donny

GEI Representatives
A. Sanna

Purpose of Site Visit: Observe Filling of Tank behind the Dam with water.

Observations:

1. 10:48 a.m. – Water turned on at hydrant. Tank starting to fill.
2. 10:50 a.m. – Floor of tank behind the Dam is covered with water.
3. 10:54 a.m. – Water level at valve, depth of approximately 3 inches.
4. 10:58 a.m. – Water at a depth of approximately 6 inches. There is no sign of water downstream of the modular dam.
5. 11:18 a.m. – Water at a depth of approximately 1 foot. There is no sign of water downstream of the modular dam. (Rate of filling approximately 30 minutes per foot).
6. 11:30 a.m. – Off site for approximately 2.5 hours.
7. 2:40 p.m. – Water level at joint between bottom and top precast modules, a depth of approximately 7.5 feet. The bulkheads are leaking a small amount of water. There is no sign of water leaking at the precast module day.
8. 3:14 p.m. – Water turned off at the hydrant. Water at a depth of approximately 9 feet. Team decided to keep water over night to let hydrophilic water stops swell overnight and see if the bulkheads stop leaking.

GEI Consultants

FIELD OBSERVATION REPORT

Project :	Next Generation Hydro	**Date:**	Wednesday, Oct 12, 2016
Location:	North Billerica, MA	**Report No.**	013
Client :	French Development Enterprises	**Page:**	2 of 2
Contractor:	W.L. French Excavation	**GEI Proj. No.**	151669-0

Photo 1 – Pre filling

Photo 2 – Water @ 1 foot

Photo 3 – Water @ 1 foot, No sign of water downstream.

GEI Consultants

FIELD OBSERVATION REPORT

Project :	Next Generation Hydro	**Date:**	Wednesday, Oct 12, 2016
Location:	North Billerica, MA	**Report No.**	013
Client :	French Development Enterprises	**Page:**	3 of 2
Contractor:	W.L. French Excavation	**GEI Proj. No.**	151669-0

Photo 4 – Leaking at Bulkhead (Left side looking upstream).

By:	Andrew Sanna	**Reviewed By:**	Varoujan Hagopian

GEI Consultants

FIELD OBSERVATION REPORT

Project :	Next Generation Hydro	**Date:**	Friday, Oct 21, 2016
Location:	North Billerica, MA	**Report No.**	015
Client :	French Development Enterprises	**Page:**	1 of 2
Contractor:	W.L. French Excavation	**GEI Proj. No.**	151669-0

Time of Arrival: 11:00 a.m. **Departure:** 3:00 p.m. **Weather:** 75's F, Sunny

Persons Contacted, Company
Celeste Fay
Lenny Lozinsky
Scott
Donny

GEI Representatives
A. Sanna

Purpose of Site Visit: Observe filling of tank behind the Dam with water.

Observations:

1. 9:53 a.m. – Received phone call from Lenny that the tank is being filled with water. Lenny informed me that the depth of water was at 2.5 feet and that the bulkheads are seeping.
2. 11:00 a.m. – Onsite.
3. 11:15 a.m. – Water at a depth of approximately 5.5 feet. There is a drip of water every 3 to 4 seconds leaking at the bulkheads. There is no sign of water downstream of the modular dam.
4. 12:05 p.m. – Water at a depth of approximately 7 feet. There is a drip of water every 3 to 4 seconds leaking at the bulkheads. There is no sign of water downstream of the modular dam.
5. 2:45 p.m. – Water turned off at the hydrant. Water at a depth of 144 3/8 inches taken at the manometer tube located on the left wall of the tank as you are looking upstream at the dam. There is a drip of water every 3 to 4 seconds leaking at the bulkheads. There is no sign of water downstream of the modular dam.
6. 3:00 p.m. – Off site.

GEI Consultants

FIELD OBSERVATION REPORT

Project :	Next Generation Hydro	**Date:**	Friday, Oct 21, 2016
Location:	North Billerica, MA	**Report No.**	015
Client :	French Development Enterprises	**Page:**	2 of 2
Contractor:	W.L. French Excavation	**GEI Proj. No.**	151669-0

Photo 1 – Right Bulkhead looking upstream.

Photo 2 – Left Bulkhead looking upstream.

Water in both photos at a depth of 12 feet. There is a drip of water every 3 to 4 seconds leaking at the bulkheads.

GEI Consultants

FIELD OBSERVATION REPORT

Project :	Next Generation Hydro	**Date:**	Friday, Oct 21, 2016
Location:	North Billerica, MA	**Report No.**	015
Client :	French Development Enterprises	**Page:**	3 of 2
Contractor:	W.L. French Excavation	**GEI Proj. No.**	151669-0

Photo 3 – Water @ 144 3/8 inches.

FIELD OBSERVATION REPORT

Project :	Next Generation Hydro	**Date:**	Friday, Oct 21, 2016
Location:	North Billerica, MA	**Report No.**	015
Client :	French Development Enterprises	**Page:**	4 of 2
Contractor:	W.L. French Excavation	**GEI Proj. No.**	151669-0

Photo 4 – Water at a depth of 144 3/8 inches taken at the manometer tube located on the left wall of the tank as you are looking upstream at the dam. There is a drip of water every 3 to 4 seconds leaking at the bulkheads. There is no sign of water downstream of the modular dam.

By:	Andrew Sanna	**Reviewed By:**	Varoujan Hagopian

GEI Consultants

Appendix D. Best Practices Guide

Best Practices for implementation of precast concrete technology to dams, diversion structures, and powerhouses

Disclosure

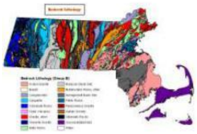
The purpose of this document is to summarize key recommendations to implement the French Dam via a high-level construction sequence. Please note – this is simply a summary document and is not meant to supplant engineer's recommendations. Dam design and construction is a site-specific process and each project requires unique design and construction considerations.

Outline of Steps

- Geotechnical investigation
- Foundation design – CIP foundation or precast alternative design structures
- Foundation preparation
- Rock bolting
- Precast manufacture and transport
- Precast assembly & linkage
- Finishing

#1: Geotechnical Investigation

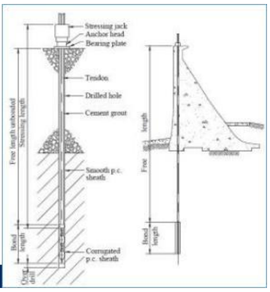
8. Reconnaissance and mapping studies
9. Borings/subsurface investigations
10. Special excavations
11. Measurements & testing
    - Shear strengths of intact portions, the sliding friction strengths of discontinuities, and the shear strength at each interface with a different material (including the strength at the interface of concrete and the material exposed on the completed excavated surface).
    - Permeability of each material
    - Deformation modulus of the foundation (ratio of applied stress to elastic strain plus inelastic strain)



Objective: Complete all geotechnical studies and create complete geological model of site

#2: Foundation Design

- Develop load requirements
  - Load cases developed (construction, head gates open/closed, tailwater & headwater levels, etc.)
  - Stability analysis
    - Horizontal (headwater, tailwater, ice, wind, earthquake, etc)
    - Vertical forces (dead weight of structure, equipment, earth, water and uplift, etc.)
- Design slab thickness and rock bolt array
- Precast alternatives may be specified
- Specify rock anchors



#3: Foundation Preparation

2.3 Site preparation for slab
- Road access
- Tree and vegetation removal
- Cofferdam/dewatering
- Excavation
  - Foundation should be unshattered, unweathered material to provide full resistance to sliding/shearing forces

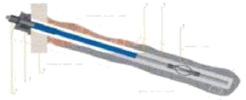
2.4 Place Foundation Slab (CIP or Precast Alternative)
- Reinforced steel placement
- GPS utilized to determine precise locations of rock bolt locations
- PVC Pipes mark Rock Bolt locations
- Pour concrete

Above: Cofferdam on Ohio River
Below: Reinforced Concrete Base

Objective: 100% Design complete

#4: Rock Bolt Foundation

2.4 **Layout** – confirm precise locations of each rock anchor
2.5 **Drilling** – drill, clean, and verify depth. Provide temporary plugs
2.6 **Anchor placement** – verify depth and inspect anchor, insert
2.7 **Grout mixing & placement** – verify grout mix and volume, mix, pump until clean grout emerges from hole, record levels and allow to cure
2.8 **Load testing (pull-out testing)** – apply loads with hydraulic jack, measure deflection with certified drop indicator



Objective: Rock-bolted foundation slab

Completed Rock-bolted Foundation Slab (4.6 MW Powerhouse)

#5: Precast Manufacture & Transport

3.2 Approvals of structural and design drawings, as well as the shipping schedule are the first steps in the precast production cycle.
3.3 Production must take place at an approved National Precast Concrete Association (NPCA) plant.
3.4 Molds and tooling are required for accurately producing products. These molds are set-up in the manufacturing plant with all the required design components including: the structural reinforcing cage; lifters for stripping away from the molds and setting the products onsite; hole openings; embedded inserts; connection dowels; and weld connection plates. Mold should be QC approved prior to pouring.
3.5 Rebar placement and cast-in components placement
3.6 Pre-pour inspection prior to casting
3.7 Pouring – approved mix design, perform required QC tests
3.8 After proper curing, the finished product is removed from the mold, post-pour QA/QC checks are performed and the product is moved to the storage yard until it is ready to ship.
3.9 Transportation can commence when the job site is ready to unload and set the products in the field. Coordination is critical so the timing and delivery order of pieces is orchestrated properly for a smooth site install.

Objective: Complete Precast Modules & Transport to Site

#6: Precast Assembly & Linkage

3.3 Products arrive at the job site and are offloaded and set in place with the crane. The site crane must be sized according to handle all of the sections.
3.4 Precast units are set in place to build out the project.
3.5 Field connections to tie the units to the foundation base or slab and each other will vary site to site based on design requirement. Some examples of these are:
- Exposed rebar dowel connections with closure poured done onsite;
- Shear key or tongue and groove sections;
- Thread connections or rock bolt ties;
- Weld connections with cast in embedded plates.

Objective: Assemble all modules into complete structure

#7: Finishing

4.2 Waterstop Selection
- Metallic waterstops are typically used in large hydraulic structures where strength is paramount
- Nonmetallic waterstops (Embedded) are best used when flexibility is of higher priority than strength
  - Butyl rubber, neoprene, styrene butadiene rubber, and other materials which may be expandable or formed

4.3 Grout
- Grouting applied to joints, voids, water cutoff at foundation

4.4 Finishing
- Precast concrete can apply attractive finishes that are aesthetically pleasing and/or blend into existing environment

Above: Embedded Rubber Gasket **Below:** Finish on precast bridge culvert

Objective: Commission Structure

Patents

4.3 US9103084 – “Intelligent hydroelectric dam with power storage” (New Dam Construction)

4.4 US8414223 – “Intelligent hydroelectric dam with power storage” (Powerhouses, Retrofit existing Dams)

4.5 Canada: 2830913 – Patent Application, Allowed (Dams, Powerhouses)

Appendix E. Meeting Minutes (Lessons Learned Meeting)

Minutes **November 10, 2016** **11 AM** **GEI Offices**

Type of meeting	Review Prototype Construction and discuss Lessons Learned
Facilitator	Lenny Lozinsky, FDE
Note taker	Peter Drown, Cleantech Analytics LLC
Attendees	Bill French, FDE Lenny Lozinsky, FDE Varoujan Hagopian, GEI Mike Walker, GEI Andrew Sanna, GEI Norm Bishoip, Knight Piesold

Agenda topics
[Time Allotted] **Lessons Learned** **[Presenter]**

Discussion

- Bill thanked everybody for work completed to date
- Several glitches included: waterstop application, bulkhead leakage, levelness of slab, recess in precast
- Project Management & Design Stage – who is responsible for what – this was a good lesson learned
  - VH: Prepared design specification which goes to entity who will execute the project; that entity than sends document to precaster, they then agree with it or develop shop drawings and resubmit for approval
  - This was NOT done in this case; it was an integrated design process where all contractors were cautious of roles and responsibilities
  - In future projects – this will not be a problem (Kelly from Oldcastle agrees)

- Precast Stage
  - Some issues with the bottom units not being leveled, this cause a gap to vary in the ceiling groove
  - That is a detail that needs to be paid attention to in full-scale
  - Straightness of walls was within expected tolerance
  - Issue with handling when stripping the first time – but this was quickly resolved
  - Lesson learned: tolerances are critical and need to be double checked during precast stage

- Testing Stage
  - Adeka compound was unable to work at lower elevation
  - Lesson learned: modules have to go directly to floor, cannot have any voids in it, bottom expandable rubber has to have more exposure to the slab and go around the entire perimeter of the dam (upstream and downstream)
  - Too large of a void in the back of modules (ask Norm for specific location)
  - Lesson learned: May not need a shelf going forward because it can become problematic with the filling elements and can create a channel for water to flow in all areas
  - Lesson learned: the hard sealing selected just takes too long to react and fully swell/ need either a new product or a custom product developed with the manufacturer (P201 is probably not the right product due to no adhesive capacity) – may just use multiple rubber gaskets
  - Lesson learned: Hydrophilic waterstops likely take 3 days to fully expand; in the meantime

water would be traveling down channel where waterstop is located and may be going to other problem areas

        - Hydrophilic waterstop may not be the best material for lower modules (VH recommends SikaFlex, Norm agrees it is a good product – only challenge is if the surface isn't clean but there is a solvent that can be applied on the surface)
        - Material needs to be capable of sealing immediately when the lower module is placed
        - Norm has put seals in in areas where there are 4-6" tolerances, it can be done
            - Lesson learned: Use grout to take care of undulation of slab – in test stage it is very well leveled, but in real life this is highly unlikely
            - Can put grout tubes through bottom then can set module (Kelly) inject grout to fill any anular space/need to grout the bolts
            - However grout will make this a permanent installation
        - Make lower foundation that is much shorter (2', e.g.) that can then accept larger modules for dams that need to be taken apart at later date
            - Norm: we use a wedge jack to take out precast grouted joints (it breaks the joint) – disagrees that 2' module at the bottom will work
    - May want to make test apparatus and test various seals prior to commercial application
    - Can we cast in gasket?
        - Yes in horizontal, challenging in vertical – more discussion warranted for this
    - Norm has membrane products to consider: closed-cell neoprene. These membranes become more watertight as they are compressed/ Norm has put in in lots of hydraulic structures
    - Lessons learned: keyways were too tight to slide top modules into side module, we need to cut a small angle there to assist in guiding the modules together via the keyway
        - Or explore option with two males and two females with waterstop in between so it is totally protected
    - Overall Lessons learned: everything is site-specific
    - Lessons learned: create manholes on horizontal level on areas where we want access
    - Side bolts are all accessible and verifiable
        - Part 12 Safety inspection work will need to see critical bolts as part of inspection
    - Need more detail on the ends of the modules to receive the bulkheads (cannot be a bolted connection such as what we have on abutments)
        - In real world could have many conditions (soil, rock, etc.)
        - Our system does not include how the interface would mate with the abutment
        - To make it a system, will need to define that
        - Abutment will have training walls then main dam/ cantilever weld/ rock-bolted weld/ etc. abutment will likely have concrete face
    - Mike Walker: In Cost estimate, the abutments will be essentially the same with precast as CIP
    - Norm Bishop: disagrees – need to define the abutment
        - Suggestion: go take pictures of abutments present at existing dams then sketch out how we would consider doing them (develop 2-3 alternatives)
        - Suggestion: use sheet pile around the abutment then tie into sheetpiling wall
    - Lessons learned: need to address abutments in future design
    - Lessons learned: need to determine drainage through structure
    - Norm summarized: The modular concept is a really good concept, team did a great job pulling this together/ all comments today are related to a "system" how we make this into a fully integrated "System" that can be sold

- Task 5/6 (Commercialization)
    - People stopped building dams out of granite blocks because when earthquakes happened they failed; we are getting around this with modular dam by providing tensile connections laterally and vertically
    - Would be good to talk with dam inspectors and ask what they would look at during a Part 12

inspection
    - Reach out to FERC/ get their thoughts on units/risk factors
        - May not necessarily need/want to do first project at a FERC-regulated structure
        - Norm will put Bill in contact with FERC inspector lead (Action Item)
            - Should attempt to get initial feedback
            - Does not put in writing/but will say; "this is what I would require for a structure like this in practice"
            - Bill is very conservative – has standing board of consultants
            - Bill's staff could perform analysis and write up comments/and he could sign a letter back to us
    - Norm: need to show some concepts of overflow spillways and modules for equipment to make this into system
        - MW: GEI can take the existing 3D drawings of powerhouses that we created and flesh out with spillways, etc.
        - NB: Travel to real dams, take pictures, make renderings as if we were to retrofit that dam with a rendering
            - If we had flow duration curve, head duration curve, then power output

Conclusions

Action items	**Person responsible**	**Deadline**
• Norm to introduce us to Bill [Last Name] from FERC to get initial feedback • Need to develop 3D AutoCAD model of turbine at the Allendale Dam (flow duration curve, head duration curve)		

Discussion
Task 5 Report – GEI will have completed by • Include Allendale Redesign with precast • Include Cost Estimating tool for full-scale • Include Norm's report Commercialization

Appendix F. Budget Period 1 Report

Continuation Application
DE-EE0007244.0000

French Development Enterprises, LLC
The French Modular Impoundment

A. PROJECT OBJECTIVES

The first goal of this project is to Design, Manufacture and Test a Prototype Precast Modular Impoundment to demonstrate and de-risk the technology, accelerating the technology from TRL 3 through TRL 6. In parallel, this project seeks to design a full-scale impoundment using this technology for an actual U.S. site, using site-specific parameters, and compare the resulting engineering and cost estimates with traditional dam construction methods. Key Project Objectives include:

(5) Design, manufacture and test a prototype Precast Modular Impoundment consisting of several modules with interlocking elements at Alden Test Facility
(6) Develop full engineering and cost/schedule reductions for baseline comparison using actual U.S. new hydropower site
(7) Complete dam safety evaluation and insurance consultation feedback
(8) Demonstrate scalability of proposed concept in heads of 10-50 feet

B. BUDGET PERIOD 1 TASKS

According to the SOPO, work was completed for Tasks 1, 2, and 5 during Budget Period 1. This included completion of the required deliverables for Budget Period 1. Full results are discussed in the following sections under the relevant subtask:

Task 1: U.S. Reference Site Baseline Criteria Development, (M1-M4)
Task Summary: This Task will select a site representative of typical U.S. low-head resource, and collect the required criteria that will serve as parameters for both the Prototype and for the Full-Scale Baseline Comparison (Task 5.) The outcome of this Task will include a working pre-FEED document with the full set of criteria including: Head, Flow, Dam Size, Foundation, Seismic/Geotechnical Feasibility, and other criteria that will allow the engineering team to conduct FEED (for both Prototype and Full-Scale Comparison) using both Precast Modular Impoundment construction and legacy construction methods. This Task will result in a pre-FEED document that mirrors an abbreviated dam feasibility study, and includes the required information for GEI and Old Castle to design the Prototype will also commencing the FEED on the full-scale unit for Baseline Comparison in Task 5.
Subtask 1.1: Site Selection & Baseline Criteria Established (M1-M4)
Subtask Summary: Site will be chosen to perform FEED for a Prototype and a Full-Scale Comparison. FDE has mentioned this idea to several developers as part of the preparation for this FOA – several sites have emerged as potential candidates, and one will be selected based off below criteria. Baseline Criteria

will be collected from site and included in pre-FEED document to inform FEED process for both prototype and full-scale comparison. Pre-FEED Document will be drafted with established criteria and parameters necessary to kick-off FEED process for full-scale unit (Task 5), and complete any additional prototype requirements to allow for representative comparison. The developed design criteria will include parameters such as hydraulic head, probably maximum flood (PMF) flow, seismic/ geotechnical conditions, hydrostatic pressures, temperature, ice, silt and debris conditions. This site-specific information is needed to provide a design basis for the dam concept evaluated in subsequent tasks.

- **Milestone 1.1:** Pre-FEED Requirements Document based off selected U.S. site drafted and submitted for review by end of Month 4.

Actual Work Completed in Budget Period 1:
Milestone 1.1 was completed in Budget Period 1, concluding Subtask 1.1. From the start of the project in December 2015, our team worked to identify an appropriate site to develop a design, cost and schedule comparison estimate for precast vs. cast-in-place methods. First, our team developed a matrix of the criteria necessary for collecting data and narrowing down appropriate candidates. The criteria evaluated are below:

- Year constructed
- Location (City, State)
- Design Availability
- Construction Schedule Availability
- Detailed Cost Availability
- Spillway Dimensions (height, length)
- Dam Dimensions (structural height, head, length)
- Primary Purpose
- New/Rehab
- Type/Material
- Project Cost (USD)
- Engineering Group
- Constructor
- Owner

Our team then identified a list of 16 dams and collected data where available from public and private sources, including the National Inventory of Dams and GEI's personal records of past projects. Most projects were eliminated outright due to the height or complexity of the dam, due to the DOE requirement that we demonstrate scalability in heads from 10-50 feet. Ultimately the decision was made to use the Allendale Dam in Rhode Island, for several factors. Allendale Dam is a relatively simple dam constructed with about 10 feet of head. The dam was constructed by standard means and methods – no specialty equipment was needed. Our preliminary cost comparison between this Dam and a comparable precast system revealed modest advantages to using precast system due to various factors. The primary reason this dam was chosen was GEI's familiarity with the dam and availability of needed information. To properly develop the Task 5 report in Budget Period 2 it is essential that we have a project with sufficient detailed information to complete the design using the French Modular Impoundment and properly compare costs.
Milestone 1.1 Pre-FEED Requirements Report is included in Appendix A.

Task 2: Prototype Design (M2-M6)
Task Summary: This will consist of a FEED process to design the Precast Modular Impoundment Prototype for delivery and testing at Alden. The Prototype will be approximately representative of the site chosen from Task 1 above, and include elements that require testing or would benefit significantly from demonstration, such as those described in detail in the subtasks below. This process will run concurrent to the site selection process, because certain elements (such as the Interlocking Features) are site-independent or are constrained by the physical test environment at Alden. Once Deliverable 1 (pre-FEED document) is completed, these additional details will be incorporated into the final 2 months of Prototype Design.
Subtask 2.1: Site-Independent Design Elements (M2-M4)

Subtask Summary: This Subtask will allow engineering team to commence designs on elements that are not site-specific, mentioned below. These are elements that will change only slightly but not significantly after results from Deliverable 1 (pre-FEED study.) Specific Design Elements to be evaluated include (1) Interlocking Elements (keyway configurations), (2) dimension constraints (constrained by Alden Test Facility to be 26 feet,) and (3) material standards and specifications. This Subtask will also determine which elements require physical inclusion in the Prototype and which will obtain more value through CFD modeling.

Subtask 2.2: Complete Prototype FEED (M4-M6)

Subtask Summary: This Subtask will provide the design basis for the Prototype for manufacture by Old Castle, and provide initial data for scale-up for the Full-scale Reference Design included in Task 5. Important design criteria include (1) Dimensions (length, width, height) of dam, (2) Structural Integrity (loads analysis) (3) Material choice and required properties, (4) Permeability between Segments, (5) Incorporation of Site-Independent Design Elements obtained through Subtask 2.1, (6) Manufacturability of components and (7) Completion on schedule to begin manufacturing in M6. Collaboration between FDE, Alden and Old Castle is critical throughout the design process, so risks can be identified and mitigated early.

- **Deliverable 2.2:** Provide final FEED (design-basis document) of Prototype for delivery to Old Castle to manufacture by end of Month 6. Full Cost Estimate will be provided by Old Castle and included in document.

Actual Work Completed in BP1:

In BP1, our team completed Deliverable 2.2 by developing a complete design of the Prototype for construction and testing in BP2. The final Deliverable can be found in Appendix B. This process was a team-based, iterative process which took place from December 2015 – April 2016 and included a series of in-person meetings, working sessions, and bi-weekly calls amongst the team members to arrive at a final design. The prototype design included structural analysis, geometric suitability for pre-cast fabrication, hydraulic sealant evaluation and application to actual hydropower development. After a site visit to the Alden Labs, it was decided that a new concrete test tank was best suited to test the Prototype (as opposed to the originally proposed Flood Wall Facility,) and the design was completed for this tank and quotations received from local fabricators. In addition, a prototype Test Plan was developed. The test plan details the objectives, roles and responsibilities, test facility and testing procedures.

The Prototype Design Report and Test Plan is located in Appendix B.

Task 5: Reference Site FEED and Baseline Comparison (M4-M18)

Subtask 5.1 will take place during Budget Period 1 of this project. Subtask 5.2 will take place during Budget Period 2, and is subject to the conditions of the Go/No-Go Decision. Activities under Subtask 5.1 will also serve as "Bridge Activities" to fill time during Go/No-Go Decision Process.

Task Summary: This Task will create a FEED for a French Modular Impoundment at an actual site in the U.S., including Design, Cost Estimate, Schedule, Risks, etc., and compare with an alternative configuration of the site using conventional dam construction methods (cast-in-place, RCC, gravity, etc.) This Task will be conducted under supervision of Structural Dam Engineer and will quantify several critical elements to determine project objectives, including: (1) ability of technology to be scalable, (2) structural loads analysis and durability, (3) ability to interface with riverbanks and foundation (4) seismic stability (5) cost & schedule reductions compared with conventional construction. The Structural Dam Engineer will compare the new method of construction to baseline conventional construction methods and identify critical risks to address in full-scale implementation.

Subtask 5.1: Preliminary FEED (M4-M8)

Subtask Summary: This Subtask will begin combining the criteria developed under Task 1 with the data produced under Task 2. Preliminary design of a French Modular Impoundment for an actual hydropower site (selected in Task 1) will be initiated. Subsequent to the Go/No-Go Decision, a Full-Scale FEED will be completed in Budget Period 2.

Actual Work Completed in BP1:

See work completed under Subtask 1.1 above. The Preliminary FEED document was completed and is available in Appendix A for the Allendale Dam site. Additional effort was expended under this task to further develop the French Modular Impoundment technology for specific dams in the U.S. and internationally, to identify the key challenges and design considerations for the first commercial project. For example, in BP1 it was discovered that the application of FMI technology to powerhouses is a critical consideration for design and construction of new hydropower – it is essentially part of the dam and cannot be ignored when developing the modular units. Several relationships were established with engineering and consulting firms to develop new powerhouse design concepts, and we are currently underway with designing this in parallel with the non-powerhouse structures.

Task 6: Project Management & Commercialization (M1 – M18)

Subtask 6.1 will take place during Budget Period 1 of this project. Subtasks 6.2-6.4 will take place during Budget Period 2, and are subject to the conditions of the Go/No-Go Decision. Activities under Subtask 6.1 will also serve as "Bridge Activities" to fill time during Go/No-Go Decision Process.

Task Summary: Project Management will serve to coordinate all scope, schedule and budget-related aspects of the project and ensure deliverables, milestones, and miscellaneous objectives of the grant are met on time. Project Management will be responsible for providing the DOE interface, and coordinating status updates and monthly reporting requirements. This Task will also manage all commercialization activity in Subtasks 6.1-6.2, including travel, conference participation, marketing, patent protection, etc. Full details are provided in the Project Management Plan.

Subtask 6.1: Preliminary Marketing & Commercialization (M1-M8)

Subtask Summary: This Subtask will initiate activities relating to commercializing this product and bringing to market. This includes identification of all necessary activities to further protect and expand patent portfolio, identification of and travel to meet with key vendors and material suppliers, internal financial controls and modeling project costs and financing requirements, Investor Prospectus, market research, business plans, etc. This Subtask will also identify key opportunities to participate in hydropower conferences to present results of the Project (NHA, Hydrovision, etc.)

Actual Work Completed in BP1:

BP1 included significant effort under Task 6 to commercialize and market the French Modular Impoundment. FDE developed relationships with several key commercial partners, including one of the largest hydropower turbine manufacturers, several notable developers in the U.S. and Canada, precast companies, and vendors of waterstops, gates, fish screens, etc. These relationships are critical in developing a commercial project, as FDE can be a "one-stop-shop" for standardized civil packages for new hydropower projects. FDE has developed several renderings of hydropower systems including complete civil works to demonstrate modular construction. In addition, FDE engaged in technical market research on the state of U.S. dams and levees and will be producing a technical paper regarding the challenges faced by this infrastructure in response to climate change. FDE also developed a website (www.fdepower.com) and various marketing materials (brochures, etc.) to present key aspects of this technology. FDE presented progress on the technology to HDR Engineering and Kleinschmidt Engineering on April 11, 2016. FDE also engaged the U.S. Army Corps of Engineers to obtain a 3D CAD Model of their existing Folsom Dam auxiliary spillway to demonstrate the concept of modular construction. Finally, FDE is developing a cost model for precast dam construction to allow for comparison with conventional construction methods.

Appendix F1-F5. Dam Selection Report
Milestone 1.1 Pre-FEED Requirements Document

Dam Selection Report
DE-EE0007244
French Development Enterprises, LLC
The French Modular Impoundment

A. BACKGROUND

This document comprises Milestone 1.1, pre-FEED Requirements Document (referred to herein as “Dam Selection Report,”) for project DE-EE0007244, the French Modular Impoundment. The Report is responsive to the following Task description contained in the Statement of Project Objectives:

Task 1: U.S. Reference Site Baseline Criteria Development, (M1-M4)
Task Summary: This Task will select a site representative of typical U.S. low-head resource, and collect the required criteria that will serve as parameters for both the Prototype and for the Full-Scale Baseline Comparison (Task 5.) The outcome of this Task will include a working pre-FEED document with the full set of criteria including: Head, Flow, Dam Size, Foundation, Seismic/Geotechnical Feasibility, and other criteria that will allow the engineering team to conduct FEED (for both Prototype and Full-Scale Comparison) using both Precast Modular Impoundment construction and legacy construction methods. This Task will result in a pre-FEED document that mirrors an abbreviated dam feasibility study, and includes the required information for GEI and Old Castle to design the Prototype will also commencing the FEED on the full-scale unit for Baseline Comparison in Task 5.
Subtask 1.1: Site Selection & Baseline Criteria Established (M1-M4)
Subtask Summary: Site will be chosen to perform FEED for a Prototype and a Full-Scale Comparison. FDE has mentioned this idea to several developers as part of the preparation for this FOA – several sites have emerged as potential candidates, and one will be selected based off below criteria. Baseline Criteria will be collected from site and included in pre-FEED document to inform FEED process for both prototype and full-scale comparison. Pre-FEED Document will be drafted with established criteria and parameters necessary to kick-off FEED process for full-scale unit (Task 5), and complete any additional prototype requirements to allow for representative comparison. The developed design criteria will include parameters such as hydraulic head, probably maximum flood (PMF) flow, seismic/ geotechnical conditions, hydrostatic pressures, temperature, ice, silt and debris conditions. This site-specific information is needed to provide a design basis for the dam concept evaluated in subsequent tasks.

- **Milestone 1.1:** Pre-FEED Requirements Document based of selected U.S. site drafted and submitted for review by end of Month 4.

B. DAM SELECTION PROCESS DESCRIPTION

From the start of the project in December 2015, our team has worked to identify an appropriate site to develop a design, cost and schedule comparison estimate for precast vs. cast-in-place methods. This has included significant industry outreach, working with our engineering partners, and market research into a large number of dams and water control structures to narrow down an appropriate sample size of dams. Our primary objective was to find a dam that would best translate from existing cast-in-place, RCC, ore other conventional construction methods to our precast technology.

First, our team developed a spreadsheet matrix of the criteria necessary for collecting data and narrowing down appropriate candidates:

- Year constructed
- Location (City, State)
- Design Availability
- Construction Schedule Availability
- Detailed Cost Availability
- Spillway Dimensions (height, length)
- Dam Dimensions (structural height, head, length)
- Primary Purpose
- New/Rehab
- Type/Material
- Project Cost (USD)
- Engineering Group
- Constructor
- Owner

Next, our team conducted significant market research of public sources (including National Inventory of Dams database and websites of dam construction contractors,) to identify plausible candidates to populate this spreadsheet. Our engineering partners, including GEI Consultants and Hydro Consulting Specialists also provided candidates of recent projects in which they have participated. This effort led to a list of the following 16 candidates, including a completely new design:

Dam Name	**Year constructed**	**City**	**State**
Labyrinth Dam	N/A	GEI Project	N/A
Otis Reservoir	2012	Otis	MA
Hickory Log Dam	2008	Canton	GA
Genesee No2 Dam	2007	Kittridge	CO
Hunting Run Dam	2002	Fredericksburg	VA
Folsom Dam	2012	Folsom	CA
Buckhorn Reservoir	1999	Wilson	NC
Deep Creek	2010	Yadkinville	NC
Franklin Dam	2006	Franklin	KY
Pine Brook	2006	Boulder	CO
Randleman Lake	2003	Randleman	NC
Tie Hack	1997	Buffalo	WY
Worumbo Hydro Station	1989	Lisbon Falls	ME
Moody Street Dam		Waltham	MA
Allendale Dam (GEI)	2002	Providence	RI
New Design	2016	N/A	N/A

Next, we gathered the criteria referenced above for each of the dams listed, where available. The result instantly allowed us to eliminate many of the structures for various reasons. Dams that were larger than the definition of "Scalability" in the DOE SOPO – between 10-50 feet of head – were ruled out, as were designs that were overly complex, too site-specific, or lacking the data required to complete the full re-design which will occur in Budget Period 2. After discussing as a team several times and reviewing all available data, our team decided to proceed with the Allendale Dam in Rhode Island, for the following reasons:

- Appropriateness to use the dam's parameters for the Prototype Test – the Allendale Dam has similar hydraulic head to the proposed prototype test.

- Appropriateness to use the dam for a Full-Scale Baseline Comparison – reconstruction of the dam was with traditional methods, cast in place concrete, and standard means and methods. No specialty equipment was needed during construction. The reconstructed dam was a very simple design that was low cost to construct. Comparison of the traditional construction methods to Precast Modular Impoundment construction will be easier to justify while making the benefits simple to visualize on larger more complex projects.
- Familiarity with the dam and the availability of needed information – GEI performed the design of the dam reconstruction. In addition, the EPA-funded project allowed for significantly greater data availability – for example, we have two 600+ page source documents for pre- and post-construction.
- Vigilant check on design – the dam design was approved by the Army Corps of Engineers
- Foundation – the concrete footings of the dam were secured to the bedrock with rock anchors embedded 20 feet into rock and spaced 5 feet on-center, similar to the Precast Modular Impoundment bedrock attachment approach.
- Schedule – preliminary discussions indicate duration of project execution will be shorter with the Precast Modular Impoundment construction verses the legacy construction, due to the speed in mobilization and needed area to execute the work. There was urgency to reconstruct the Allendale Dam, as the existing structure was badly deteriorated and repairs/replacement had to be done to limit the risk of a breach.
- Cost Savings – preliminary cost comparison between the legacy construction and the Precast Modular Impoundment construction revealed modest advantages to the precast system.

C. ALLENDALE DAM

The original Allendale Dam was a timber cribbing and earth embankment dam that was constructed in 1865. The dam was naturally breached in November 1991. The Army Corps authorized the reconstruction of the dam, which was completed in 2002. The reconstruction of Allendale Dam was a design-build project performed as part of a Superfund cleanup of contamination in and around the Woonasquatucket River in North Providence, Rhode Island. The cleanup has been under the jurisdiction of the Environmental Protection Agency with technical consultation and oversight provided by U.S. Army Corps of Engineers (USACE), New England District (NED).

The existing timber cribbing and earth embankment dam breached in 1991, allowing potential release of contaminants in sediments upstream of the dam. The NED prepared a repair scheme for a new dam prior to the site's designation as a Superfund site. GEI modified the NED repair scheme to minimize excavation of contaminated soils by constructing a rock-bolt-anchored concrete retaining wall dam immediately downstream of the existing dam. This fast-track design project proceeded with limited subsurface information in order to meet deadlines imposed by the EPA. The design was tailored to allow field modifications to match changed conditions found during construction. The final design included the rock-bolt-anchored concrete dam; modifications of the low level outlet control structure; a graded filter drain at dam toe; riprap scour protection; stream diversion; cofferdams; fish ladder considerations; and abutment wall stabilization. GEI also provided construction observation and rock bolt testing. Contract drawings were prepared in MicroStation, and contract specifications were developed from USACE guide specifications and SpecsIntact. Final documents delivered to the EPA were converted to AUTOCAD and Word files. Data on the new Allendale Dam is provided in Table 1.

EPA Description of Reconstructed Allendale Dam

The new Allendale Dam consists of a 105 ft concrete spillway with a concrete-set rip-rap spill pad and crushed gravel toe drain system. A new mechanically operated 60" x 48" sluice gate and a stop log system was installed to provide the means to regulate water levels in the Allendale Pond. The old gate-housing structure and existing stone wall abutment along both shores were preserved and reinforced to the extent possible. Based on a design modification, the new dam sits on four to ten feet of dense undisturbed till with rock anchor bolts installed into the bedrock. During construction, the Woonasquatucket River was diverted from the work area and controlled using cofferdams upstream and downstream of the Allendale dam location. Water was also pumped from a temporary stump and treated to remove suspended solids before being discharged downstream. All removal of sediment and debris was conducted as dry excavation. As planned, the construction of the Allendale Dam was largely completed by Spring 2002 with additional repair work on the existing gate structure performed in the Fall 2004.

Reconstructed Allendale Dam Data

	Feature	**Metric**
Spillway and Dam	Dam Length	234 feet
	Concrete Spillway	106 feet
	Structural Height	19 feet
	Hydraulic Height	12 feet
	Maximum Discharge	578 cubic feet per second
	Spillway Elevation	93.5 feet
	Top of West Abutment	99.95 feet
	Top of East Abutment (Gate Structure)	100.4 feet
Reservoir	Drainage Area	40 square miles
	Maximum Storage	68 acre feet
	Normal Storage	43 acre-feet
	Normal Surface Area	13 acres
100 yr Design Flood	West side of pond upstream of Dam, Elevation	96 feet
	West side of pond downstream of Dam, Elevation	94 feet
	East side of pond upstream of Dam, Elevation	95 feet
	East side of pond downstream of Dam, Elevation	93 feet

The total Cost and Design and Construction Sequence is important to consider for Budget Period 2, to compare the cost and schedule reductions for precast vs. cast-in-place. Preliminary cost comparison for precast vs. cast-in-place revealed modest (8%) cost reduction for precast. A more detailed cost comparison will be conducted in BP2. However, a significant cost advantage of precast remains in its ability to reduce construction duration, by transferring work that is typically conducted in the field to the precast yard, where modules can be assembled under controlled conditions. This schedule advantage will demonstrate itself as projects become more large and complex. However, we will demonstrate this during BP 2 for the Allendale Dam as well. The Allendale Dam followed the following construction sequence, and construction progress photos are available in the EPA reports:

1. Cutting & clearing trees (08/13/01)
2. Forming the concrete extension of the gate structure wingwall (09/24/01)
3. Placing cofferdams for water control and diversion (09/26/01)

4. Removing debris (10/11/01)
5. Rock coring and rock bolt testing (11/13/01)
6. Forming and pouring concrete footing (12/01/01)
7. Forming and pouring concrete wall (12/20/01)
8. Rip-rap placement (01/17/02)

Table 8 - Construction Cost Data[3]

Item	Cost
Engineering Cost	$157,245
Delineation and Removal of Soil & Sediment	$1,034,000
Off-site transportation and disposal of waste	$238,100
Oversight Cost with USACE	$200,000
Dam Restoration/Rebuild	$828,400

Precast materials cost: $70,900
Cast-in-place materials cost: $76,960
Explanation of cost comparison is in Appendix B.

Figure 18 - Allendale Dam Construction #1

[3] GEI is currently working to obtain and validate more detailed cost data, especially for dam restoration/rebuild category as that is the primary construction category.

Figure 19 - Allendale Dam Construction #2

Figure 20 - Allendale Dam Completed

D. Plan for Budget Period 2

The FDE Team plans to proceed forward with the plan and schedule as described in SOPO for Task 5, in performing full-scale design and cost/schedule comparison of the Allendale Dam using precast vs. cast-in-place (Milestone 5.2.2). Detailed designs of the Allendale Dam were retained by GEI and are publically available in EPA reports, and several are located in Appendix A. In Budget Period 2, these designs will be used to transform the design to precast concrete modules and develop a new cost and schedule estimate.

APPENDIX F1. Allendale Dam Design

Figure 1 – Sections, Finish Grading

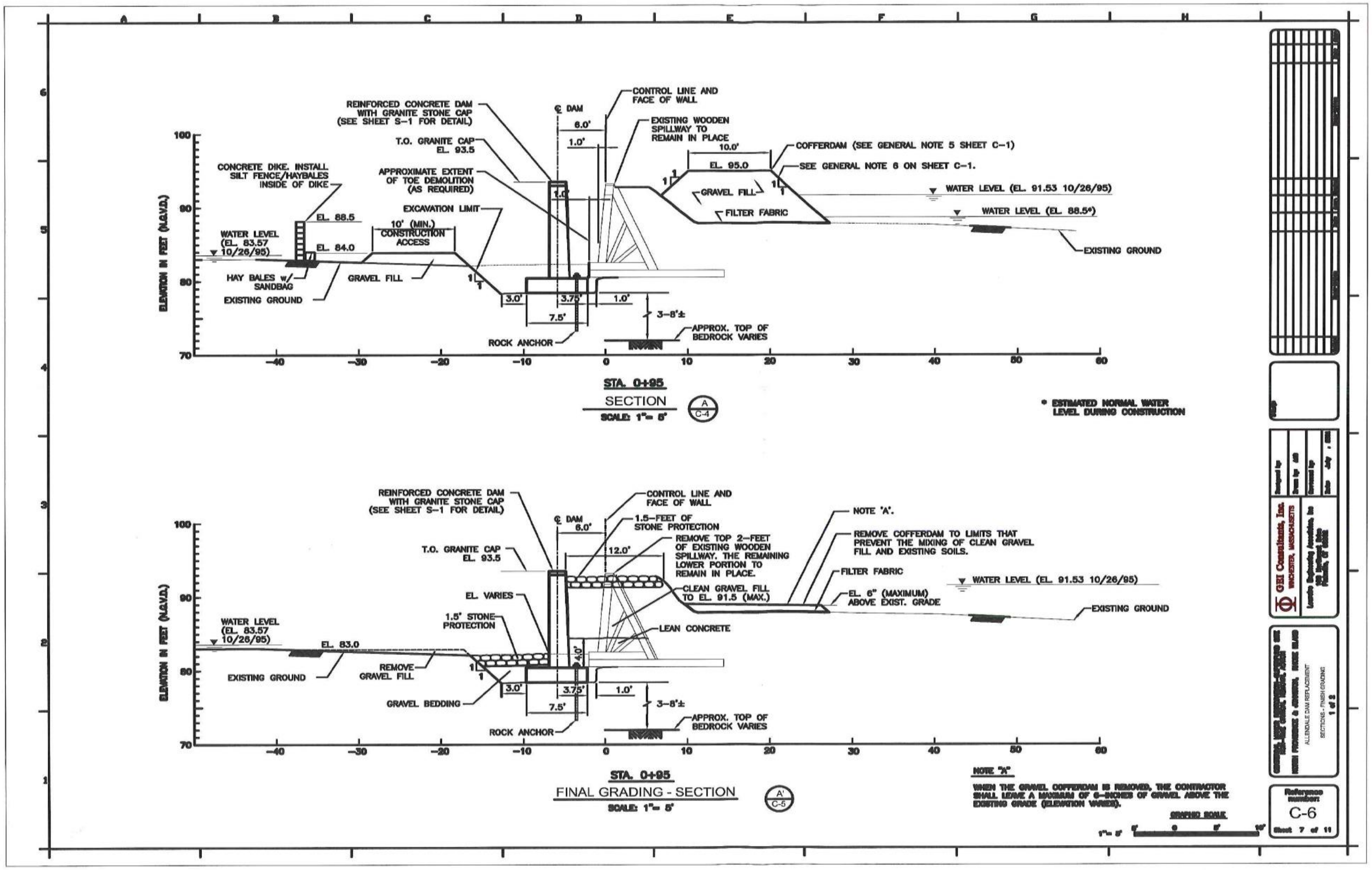
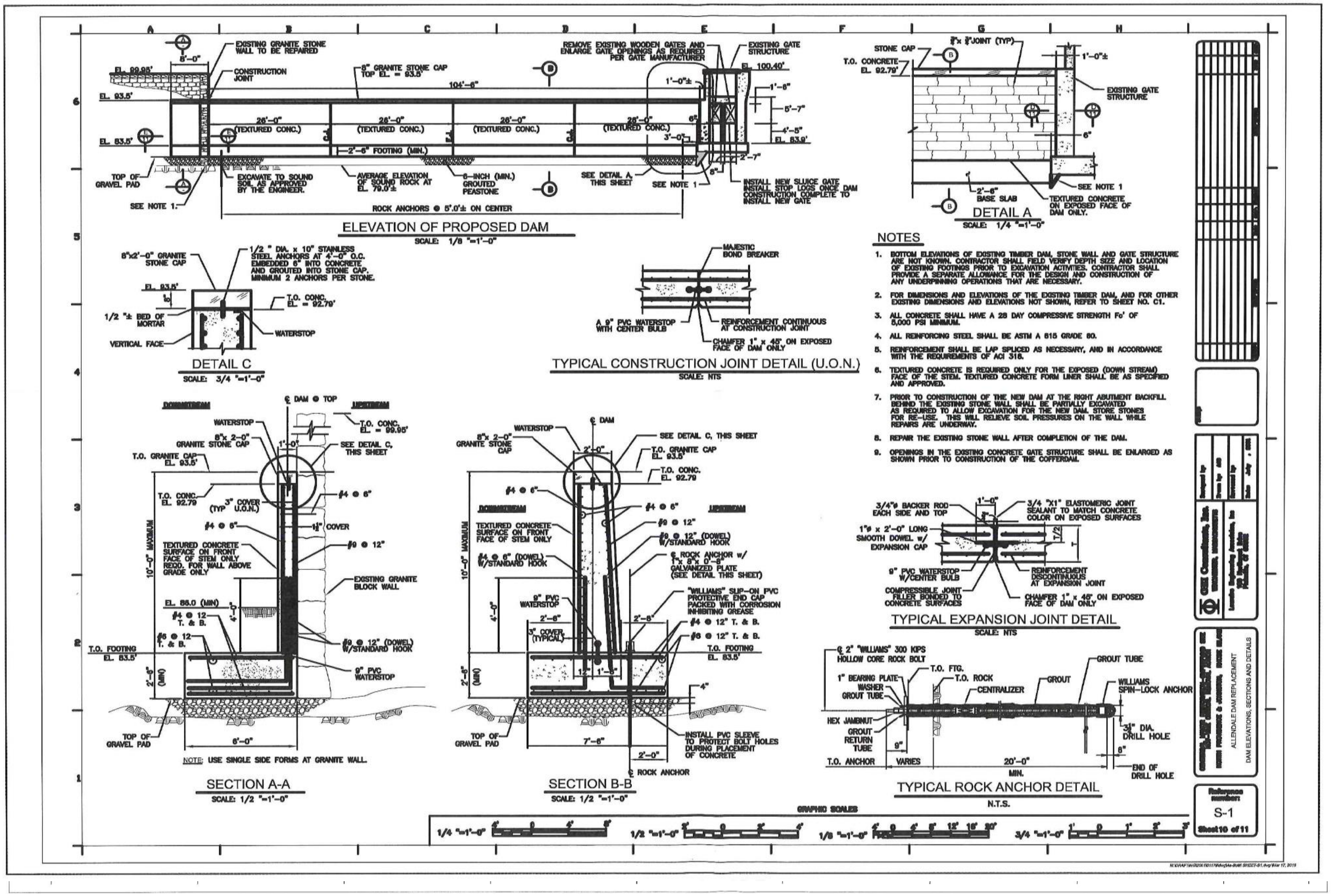


Figure 2 – Dam Elevations, Sections and Details
Figure 3 – Structural Sections and Concrete Repair Details

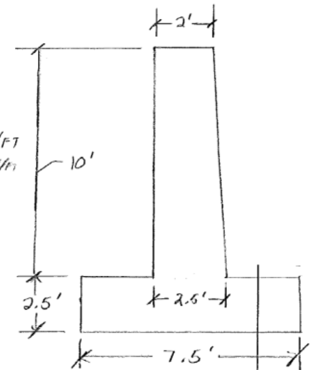


Appendix F2 Precast vs. Cast-in-place Comparison

GEI Consultants	Client	FRENCH DEVELOPMENT			Page	1
	Project	NEXT GENERATION HYDRO			Pg. Rev.	
	By	A. SANNA	Chk.	M. FLYNN	App.	
	Date	3/18/2016	Date	3/18/16	Date	
Project No.	1516690	Document No.				
Subject	PRELIMINARY COST COMPARISON					

USE RSMEANS TO COMPARE ORIGINAL CAST-IN-PLACE DAM TO PRECAST MODULAR DAM:

CAST-IN-PLACE DAM:
- 2.5 FT THICK FOOTING
- ROCK ANCHORS @ 5 FT ON-CENTER
- WALL ~ 104 FT LONG, 10 FT HIGH



QTY:
- CONCRETE FOOTING = 2.5' x 7.5' = 18.75 CF/FT
  CONCRETE WALL = (2' x 10') + (.5' x 10')/2 22.5 CF/FT
  TOTAL = 41.25 CF/FT OR 1.53 CY/FT

- REBAR
  91 LF #4 = 60.8 lb/FT
  17 LF #9 = 57.8 lb/FT
  16 LF #6 = 24.0 lb/FT
  ~ 142 lbs/FT

COST RSMEANS 2014:
- BASE FOOTING COST ON
  RS MEANS A10 FOUNDATIONS / A1010 110 STRIP FOOTINGS
  96" x 24" FOOTING = $161.5 PER LF USE $190 PER LF
- BASE WALL COST ON
  RS MEANS A2020 BASEMENT WALLS
  10' PUMPED (.493 CY/LF + 23.99 lbs/LF) = $232.5 PER LF
  OUR WALL HAS .34 CY/LF MORE CONCRETE + ~75 lb/LF MORE REBAR
  ASSUME 3x MATERIAL COST + 2 x INSTALL COST
  USE $550/LF

- TOTAL = $740/LF x 104 LF = $76,960

PRECAST MODULAR UNITS:
ASSUME PRECAST UNITS ARE 8' x 8' x 5' (13 UNITS x 2 TALL)

USING RSMEANS 33 36 13.13 PRECAST CONCRETE UTILITY SEPTIC TANK 2000 GAL SIZE = $2,550 EA.

TOTAL COST = $2,550 EA x 26 = $66,300

GEI Consultants	Client	FRENCH DEVELOPMENT	Page	2
	Project	NEXT GENERATION HYDRO	Pg. Rev.	
	By	A. SANNA	Chk. M. FLYNN	App.
	Date	3-18-2016	Date 3/18/16	Date
Project No. 1516690		Document No.		
Subject	PRELIMINARY COST COMPARISON			

PRECAST MODULAR UNITS CONT.:

- CONCRETE LEVELING PAD - ASSUME 104' x 10' = 1,040 SF USING RSMEANS 32 06 10.10 SIDEWALKS, DRIVEWAYS, AND PATIOS; CONCRETE 3000 PSI 4" THICK = $4.39/SF

TOTAL = 1,040 SF x $4.39 = $4,600

TOTAL PRECAST MODULAR UNITS = $70,900

ASSUME ALL OTHER COSTS ARE EQUIVALENT

CAST-IN-PLACE ~ $76,960

PRECAST MODULAR ~ $70,900 8% LESS

Appendix F(3) Prototype Design and Test Plan
Deliverable 2.2 Final Prototype Design Document

Prototype Design and Test Plan
DE-EE0007244.0000
French Development Enterprises
The French Modular Impoundment

E. PROTOTYPE BACKGROUND

This document comprises Deliverable 2.2, final FEED (design-basis document) of Prototype (referred to herein as "Prototype Report,") for project DE-EE0007244, the French Modular Impoundment. The Report is responsive to the following Task description contained in the Statement of Project Objectives:

Task 2: Prototype Design (M2-M6)
Task Summary: This will consist of a FEED process to design the Precast Modular Impoundment Prototype for delivery and testing at Alden. The Prototype will be approximately representative of the site chosen from Task 1 above, and include elements that require testing or would benefit significantly from demonstration, such as those described in detail in the subtasks below. This process will run concurrent to the site selection process, because certain elements (such as the Interlocking Features) are site-independent or are constrained by the physical test environment at Alden. Once Deliverable 1 (pre-FEED document) is completed, these additional details will be incorporated into the final 2 months of Prototype Design.
Subtask 2.1: Site-Independent Design Elements (M2-M4)
Subtask Summary: This Subtask will allow engineering team to commence designs on elements that are not site-specific, mentioned below. These are elements that will change only slightly but not significantly after results from Deliverable 1 (pre-FEED study.) Specific Design Elements to be evaluated include (1) Interlocking Elements (keyway configurations), (2) dimension constraints (constrained by Alden Test Facility to be 26 feet,) and (3) material standards and specifications. This Subtask will also determine which elements require physical inclusion in the Prototype and which will obtain more value through CFD modeling.
Subtask 2.2: Complete Prototype FEED (M4-M6)
Subtask Summary: This Subtask will provide the design basis for the Prototype for manufacture by Old Castle, and provide initial data for scale-up for the Full-scale Reference Design included in Task 5. Important design criteria include (1) Dimensions (length, width, height) of dam, (2) Structural Integrity (loads analysis) (3) Material choice and required properties, (4) Permeability between Segments, (5) Incorporation of Site-Independent Design Elements obtained through Subtask 2.1, (6) Manufacturability of components and (7) Completion on schedule to begin manufacturing in M6. Collaboration between FDE, Alden and Old Castle is critical throughout the design process, so risks can be identified and mitigated early.
Deliverable 2.2: Provide final FEED (design-basis document) of Prototype for delivery to Old Castle to manufacture by end of Month 6. Full Cost Estimate will be provided by Old Castle and included in document.

Go/No-Go Criteria:
Successful completion of final FEED of prototype showing:

- Structural analysis, including uplift and seepage calculations

- Details regarding scalability of design to full-scale modular impoundment
- Ability to meet a zero leakage design target between the segments
- Full cost estimates

F. DESIGN PROCESS DESCRIPTION

From December 2015 through April 2016, our team has developed iterations of the Prototype to arrive at a final design which we believe is acceptable for manufacture and testing and will meet the Go/No-Go Criteria described above. After our initial kickoff meeting on December 24, 2015, we assigned the Engineering Group with responsibility for this task to consist of FDE, Old Castle Precast, GEI Consultants, Maine Rock Drilling and Blasting, and Hydro Consulting Specialists. This group conducted bi-weekly calls to iterate on the design. Several critical factors were considered and discussed including:

- **General Geometry –** final design consists of 6, 8x8 foot hollow modules with access ladder installed (2 rows of 3 modules.)
- **Manufacturability** – units must be sized to fit existing forms Old Castle has available
- **Shipping** – units must fit on flatbed trailer for shipment. Weight and size restrictions limit the size of the modules
- **Permeability** – extensive discussion of various sealing and waterstop options for the prototype resulted in a final selection of Sikaflex-1a
- **Applicability of test results** – the modules should yield applicable test results for scalability in heads from 10-50 feet, and provide information which is commercially-valuable for making case that precast construction is preferable to cast-in-place
- **Compliance with Go/No-Go requirements**, which include:
    - Structural analysis, including uplift and seepage calculations
    - Details regarding scalability of design to full-scale modular impoundment
    - Ability to meet a zero leakage design target between the segments
    - Full cost estimates

Initially, we planned to conduct the Test Plan on the Prototype at the Alden Laboratories Flood Wall Facility. A site visit to this facility was conducted during our Engineering Kick-Off meeting January 26. Although the site was suitable for conducting the test, we subsequently decided it would be more prudent to have a dedicated test site located in North Billerica, MA which would allow potential customers to observe the prototype.

G. FINAL DESIGN

The design was a collaborative effort between FDE, GEI, Old Castle and HCS. Since the design included so many considerations (described in Section B), it was an iterative process which led to the final design.

The final design consists of 6, 8 ft by 8 ft hollow modules with access ladder installed (2 rows of 3 modules) as shown in Figure 21. Efforts to reduce the material needs and system weight were made. Ultimately, a typical unit wall thickness of 8 in was specified by GEI as the thinnest wall which will have sufficient structural capacity for this application. Each unit consists of four vertical walls and a floor, all cast as a single piece. As modules are subsequently stacked on top of each other, the top one forms the ceiling of the bottom one. To enclose the system, the highest level of units will each have an 8 ft wide by 8 ft long slab, 8 inches thick secured to its top (not shown). This further reduces unit weight as well as material quantities and shipping costs.

Each module will have an access ladder installed to facilitate inspection and maintenance (if needed) of a unit. To meet OSHA requirements, the ladders will alternate sides such that there is not a clear drop through multiple units.

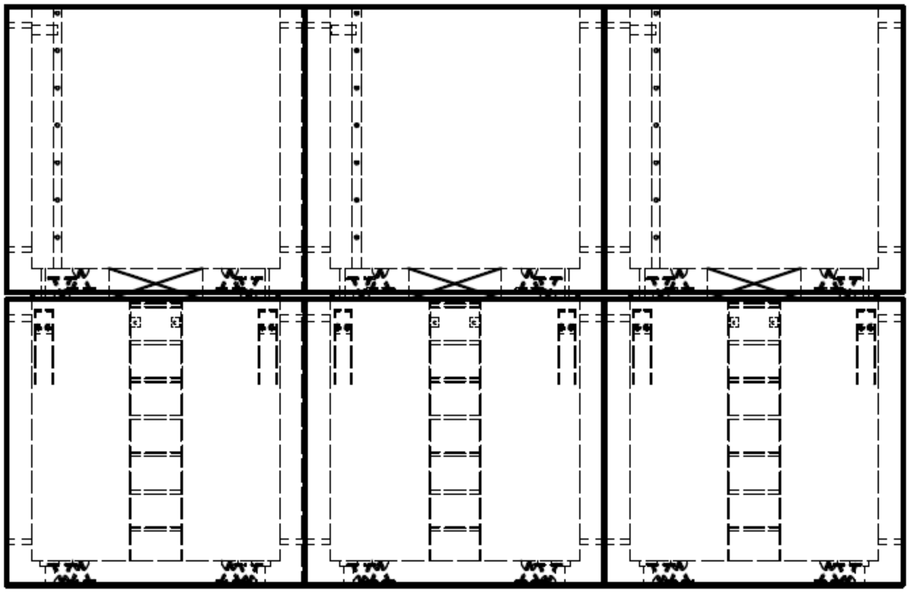


Figure 21. Final Geometry Layout

Tying of the units together was ultimately accomplished through a series of steel linkages imbedded within the pre-cast modules. For each module, will be a total of 20 connection points with 4 located along each wall of the unit as well as the floor. Steel gusset plates will be anchored near the top of the unit walls to allow the floor of the next unit up to tie in.

Approximately 2 inch diameter holes are located within the sides of the unit to allow for lateral bolting of the sections where the gusset plate is not required. See Figure 22.

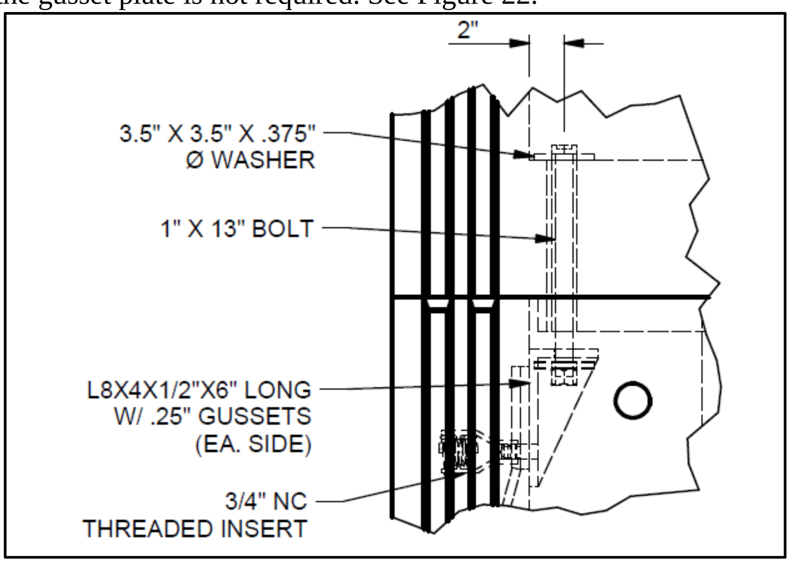


Figure 22. Detail Angle (OCI)

Units which will form the bottom layer of the dam will have an opening in the floor of the unit with a diameter of 3 inches allowing the installation of a rock anchor into the underlying slab. See Figure 23.

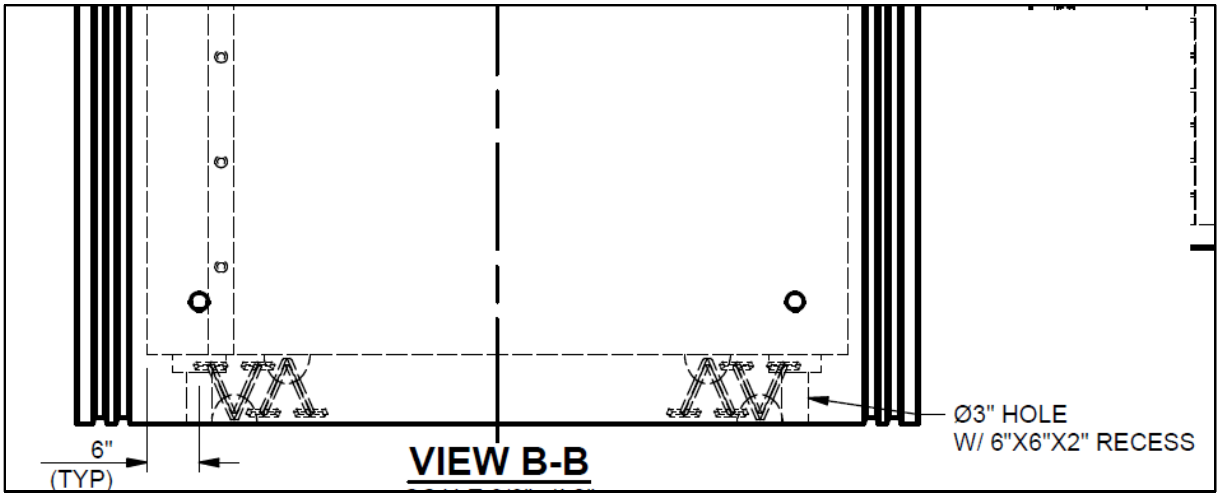


Figure 23. Bottom Module Anchoring (OCI)

The concrete units have been designed with a series of dual grooves to allow for redundancy in the sealant system. Ultimately, Akeda KM-3030 waterstop was identified as the best solution for sealing between the units. Akeda KM-3030 has been previously installed within dam construction at expansion joints and is proven in this industry. It is a modified natural rubber (vulcanized) product that allows forma controlled expansion when exposed to moisture. From its dry to set condition, the Akeda KM-3030 will expand by approximately 3 times its volume. The product is rated for up to 50 ft of head and is suitable for the FDE system as it scales to different project heads.

The waterstop will be installed within the pre-cast grooves in each unit. As seen in Figure 24, there will be redundancy in the water stop system to ensure that no leakage is present.

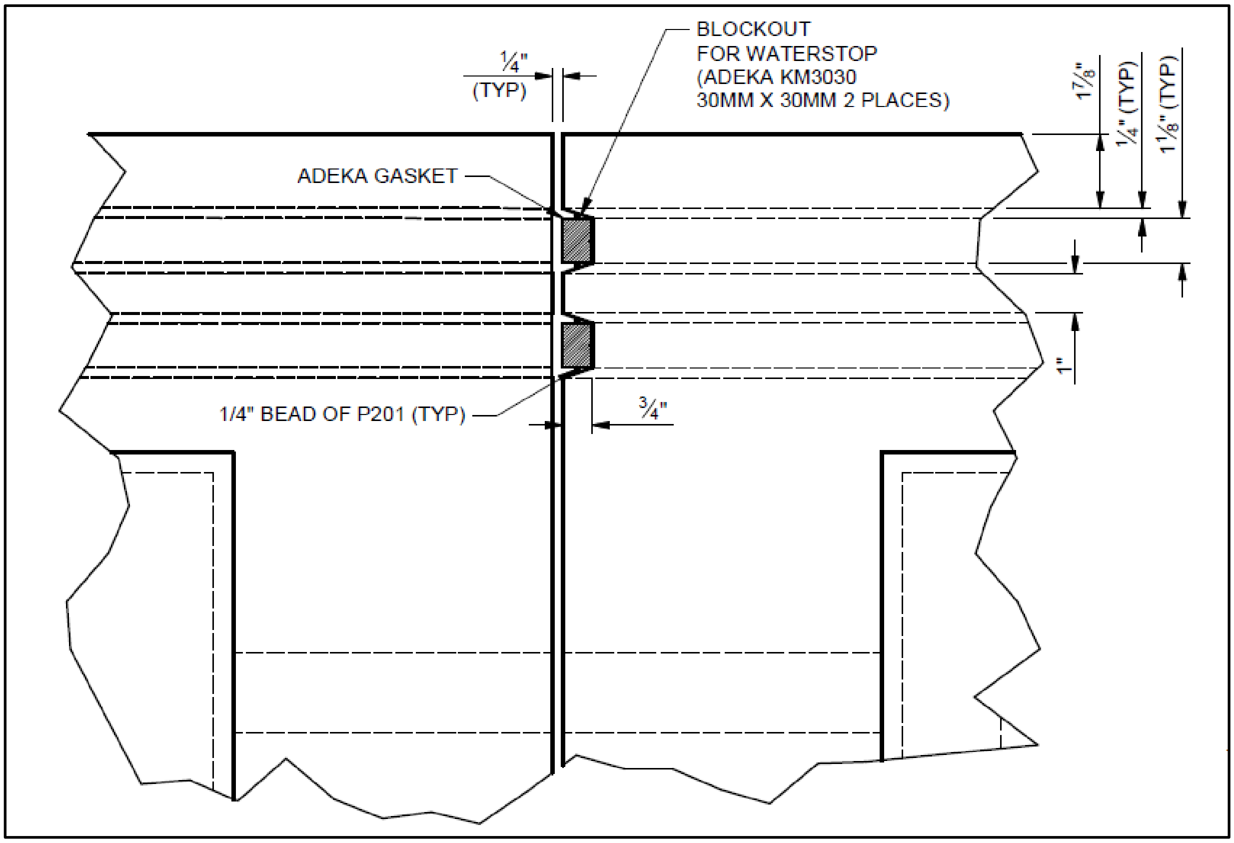
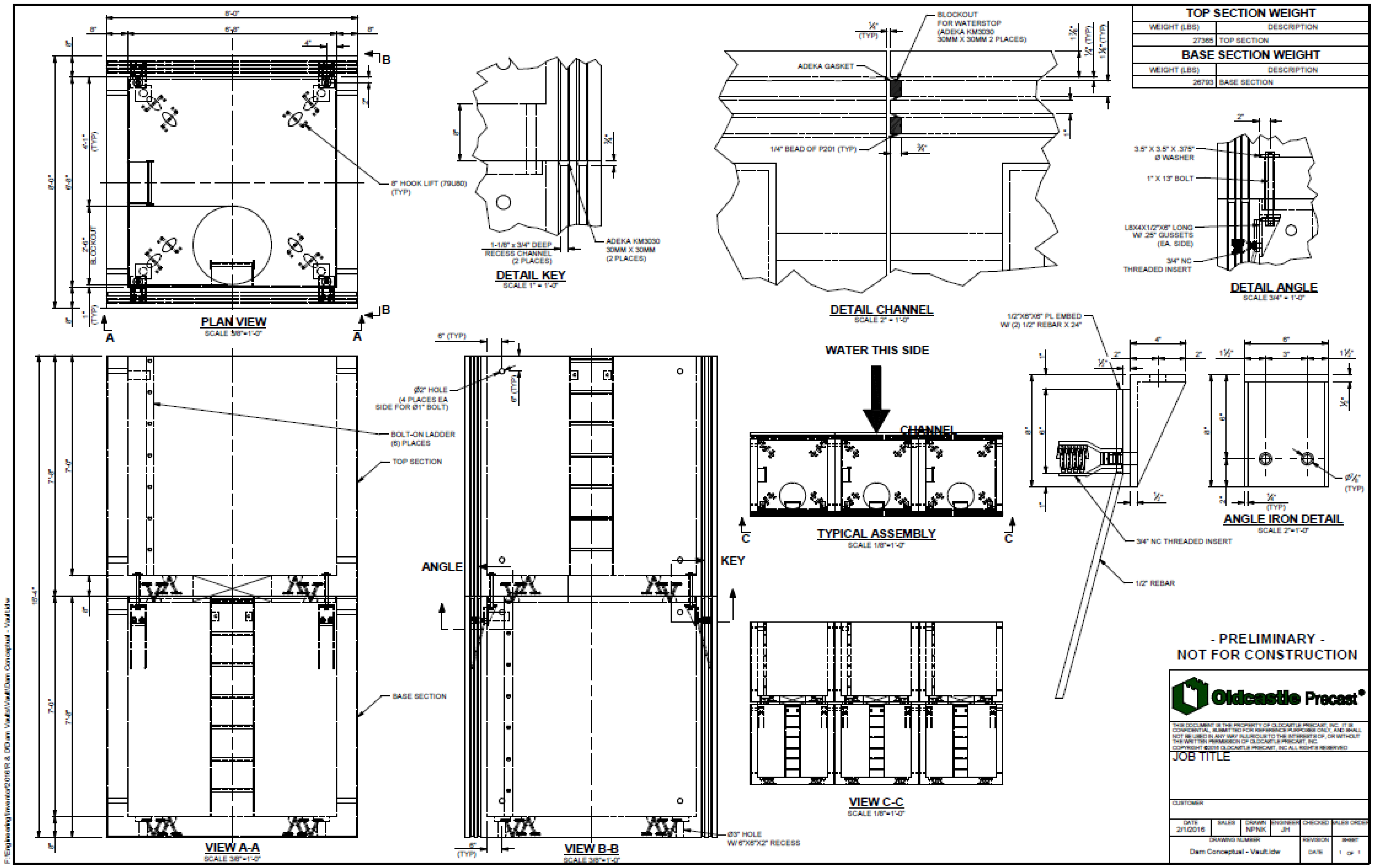


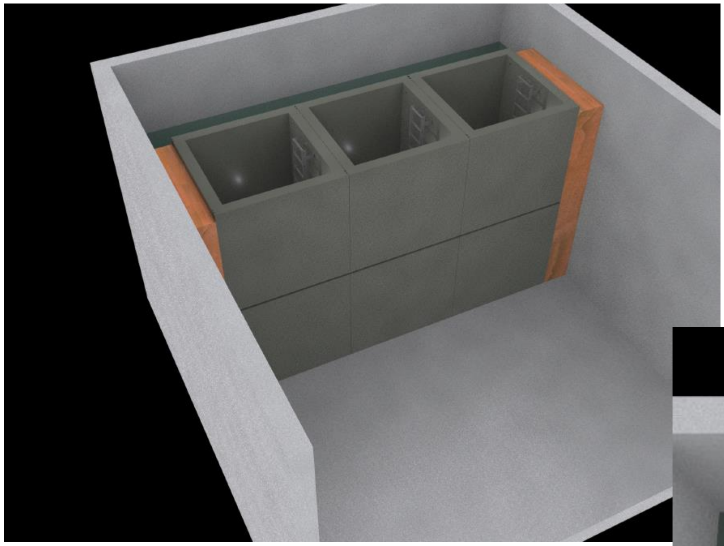
Figure 24. Waterstop Detail

The final weight of the system was estimated to be about 27,000 lbs. The allowable trucking weight varies from state to state with a Federal maximum of 80,000 lbs. As such, transportation of the units will typically be limited to 1-3 units per truck. However, Old Castle has over 50 concrete plants within the United States providing a unique flexibility to reduce transportation distance.

Prototype Design V4



Prototype in Test Tank Renderings



H. **COST –** The total cost to manufacture the 6 modules is $63,000 (see Old Castle quote in Appendix

I. TEST PLAN

Introduction

The French Modular Impoundment (FMI) is a new, rapidly deployable, cost effective hydroelectric dam and hydropower system that is being developed by French Development Enterprises (FDE). The system combines pre-cast concrete civil works with a linkage and sealing system all of which have been proven successful in other industries; however, their combination into a dam/hydropower system is new. In December 2015, the US Department of Energy (DOE) awarded FDE a grant which includes funding to complete a physical test of the FMI. This test plan details the objectives, roles and responsibilities, test facility and testing procedures. The Test Plan is developed according to requirements in the DOE Statement of Project Objectives ("SOPO") for Task 4 – Prototype Testing.

Objectives

Physical testing of the FMI has two primary goals:

1. Measure the performance of several sealing mechanisms under up to 12 feet of water pressure thus determining the preferred water stop mechanism.
2. Measure and observe the installation and removal process of each test modules including timing and identification of issues requiring design modification for rapid installation/removal.

Roles and Responsibilities

The design and testing team includes several organizations. The following briefly identifies each team member's role and responsibilities.

FDE – French Development Enterprises is involved in all aspects of the physical test and provides ultimate guidance and decision making on the project. FDE is responsible for the manufacturing of the test facility including providing construction oversite to any contractors. FDE will approve all test plans, designs, and procedures prior to execution.

GEI – GEI Engineering, Inc is responsible for the structural design of both the test structure and the test modules. The design of the test structures includes finalization of the geometry, structural and foundation design and review of hydraulic capacities to develop full loadings on module. Design of any imbedded linkage pieces or other considerations for anchoring the modules into the test facility including abutments will be completed by GEI. GEI is also working with OCI to develop the final geometry and structural design of the individual modules.

OCI – Old Castle, Inc is working with GEI to develop the test modules as discussed above. OCI's focus is on providing design input on the pre-cast concrete structures, linkage and sealing systems as this is OCI's expertise.

HDR – HDR will finalize the testing plan, provide oversite during the testing set up, document testing, record water surface elevations, estimate leakage and prepare a test report.

Description of Test Facility

Six FMI modules will be installed within a concrete tank. A plan view of the modules is shown in Figure 25 and a front elevation in Figure 26. Based on the design provided by OCI, each module will be 7'8" in height. The total height of the system will be 15'4". Each block has a width and height of 8 ft for a total system length of 24 ft. The modules will be fabricated with a linkage system that will be utilized to secure the individual components to each other and the test tank.

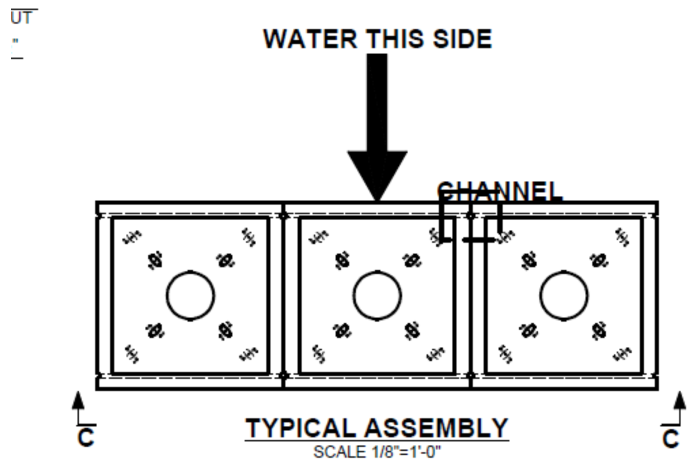


Figure 25. Plan View of Modules (OCI)

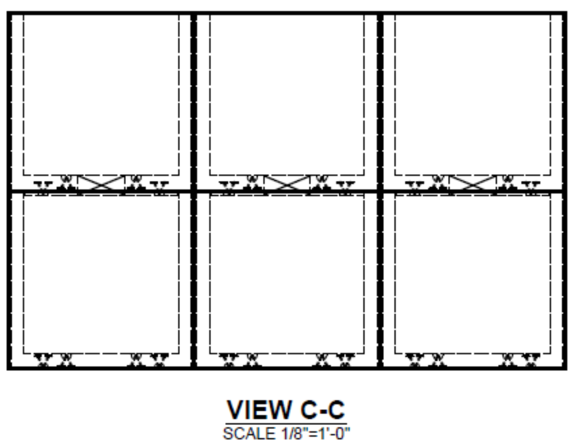
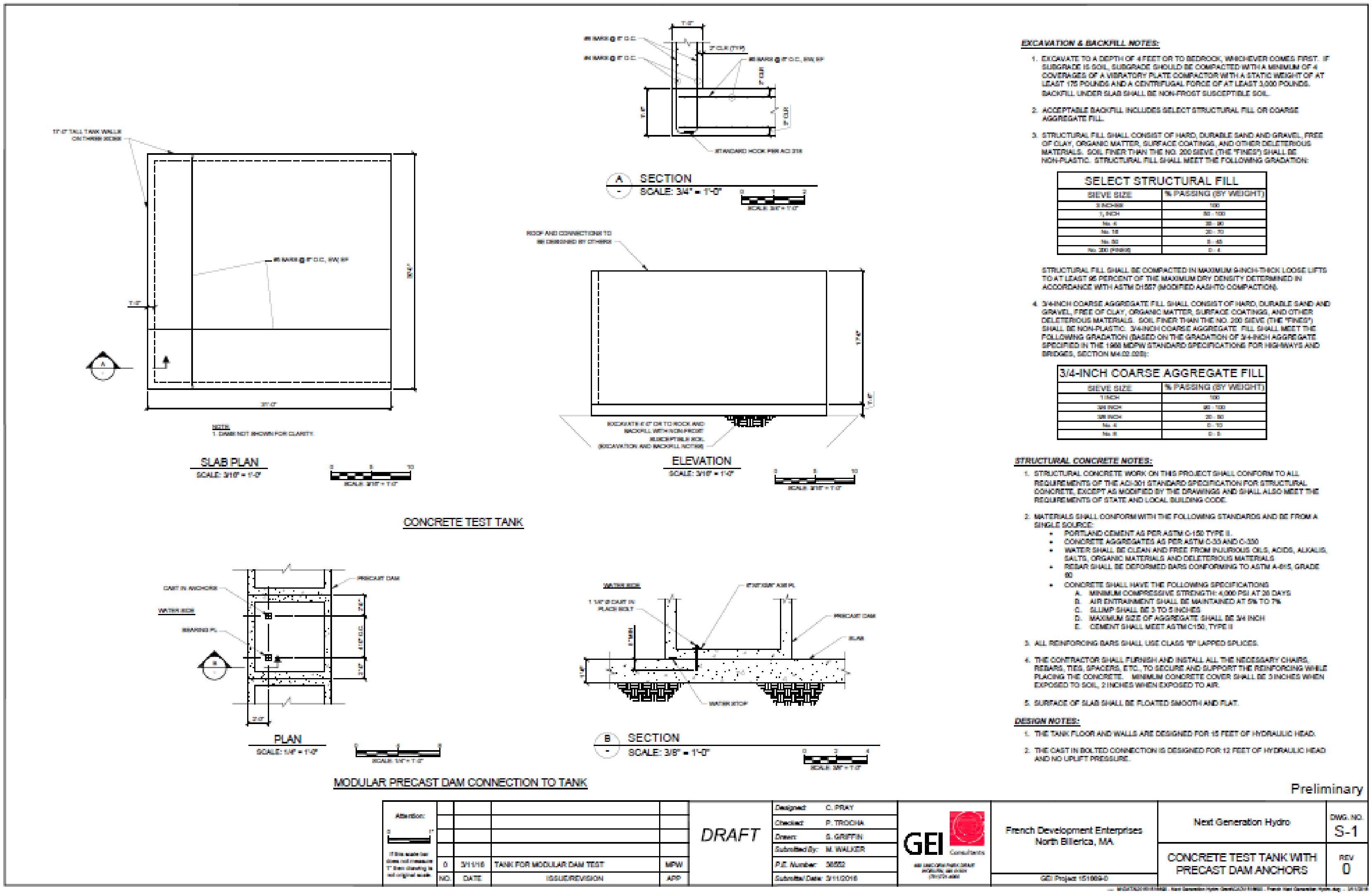


Figure 26. Front Elevation of Modules (OCI)

The test tank will have a hydraulic width of 28 ft and deth of 30 ft. The total width of modules is 24 ft leaving 2 ft on either side of the tank. These areas will be filled and represent abutments for the end modules to tie into. The area upstream of the module installation will be filled with water. The tank geometry has been specified to provide approximatley 15 ft of water distane from the module to the back wall of the tank allowing full water pressures to develop. The total height of the test walls will be 17 ft. The primary test will be conducted with 12 ft of head; however, time permitting additional tests up to 16 ft of head can be completed. See Figure 27, Figure 28 and Figure 29.

Test Tank Design



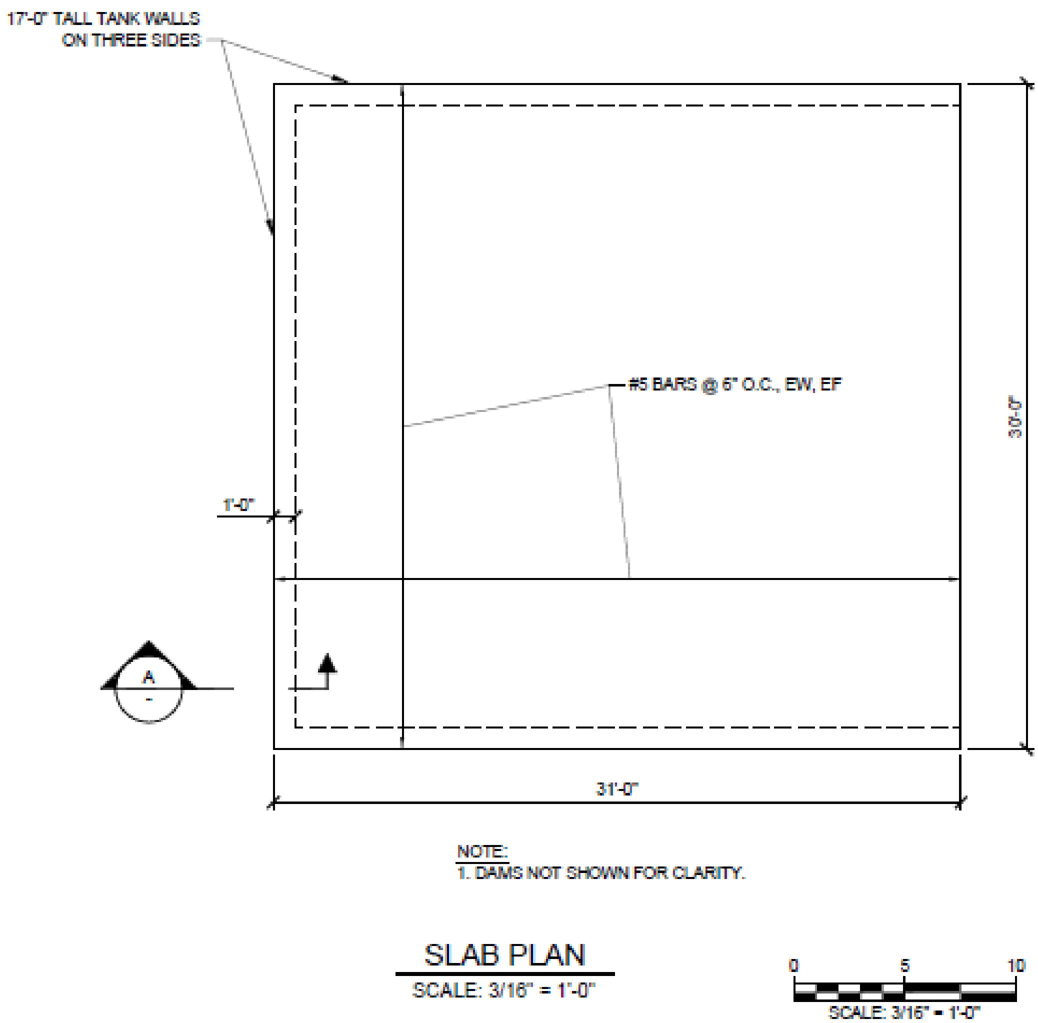


Figure 27. Plan View Test Tank (GEI)

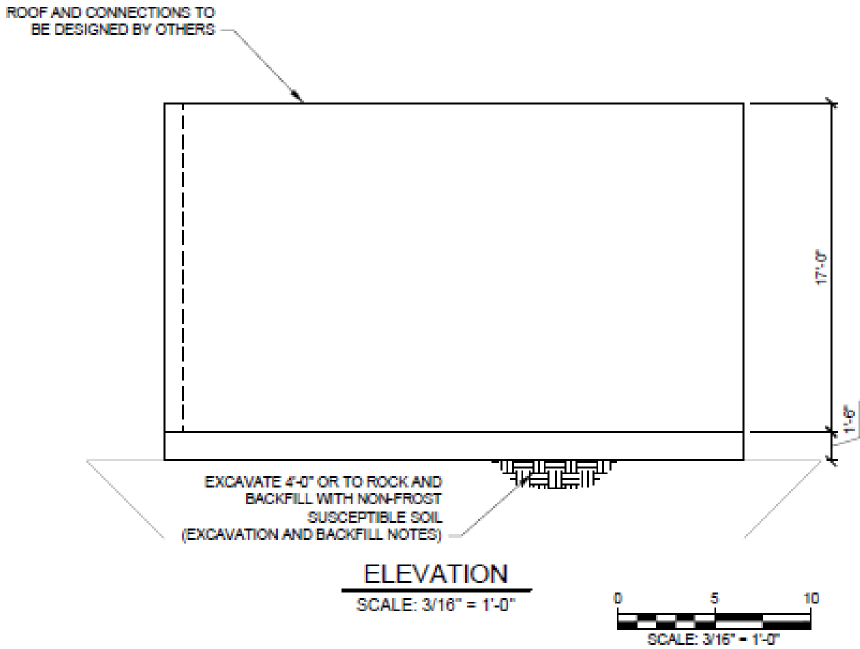


Figure 28. Elevation View of Test Tank (GEI)

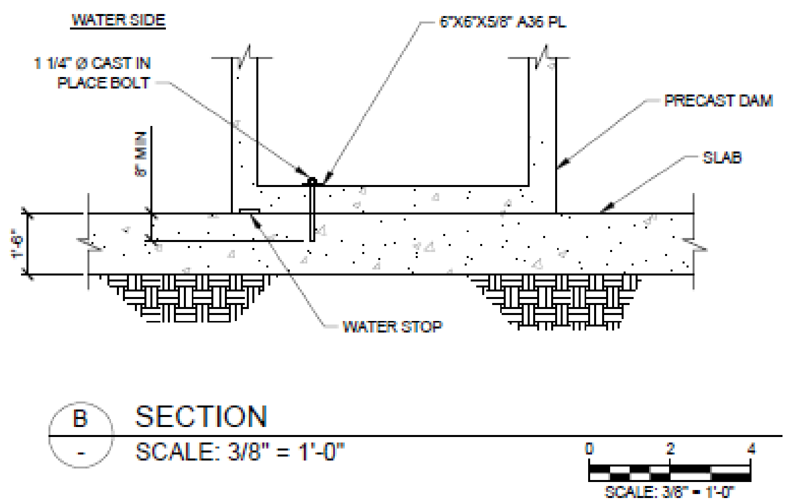


Figure 29. Section Through Module/Tank Connection (GEI)

Test Procedures

The test will consist primarily of three phases; installation, pressurization and removal.

1. Installation and Removal

The Installation and removal phases are intended to a) identify any challenges or issues during installation/removal which require design or procedural modifications and b) to measure the time required for installation and removal of the components. When evaluating the cost savings of the FMI over conventional cast-in-place concrete a significant portion of the cost savings comes from the reduction in construction schedule. Thus far, the reduction has been estimated based on other civil structures (bridges, highways, etc); however, this test will provide real data to refine these estimates. This test will also provide a key opportunity to identify any improvements which will further reduce installation/removal time.

The concrete modules will be delivered to the test location and unloaded adjacent to the test tank. All hardware and tooling will be located on-site in an orderly manner. This will include:

- Sealant material
- Hose
- Pumps
- Wrenches/Hammers
- Tag lines
- Stopwatch

A crane will be located on-site and will be utilized to move the components from the staging area into (and out of) the test tank. FDE will provide a minimum of 2 laborers to facilitate the picking and maneuvering of the concrete components. Representatives from OCI and GEI will be located on-site during the testing to facilitate any issues with the installation/removal of the units and linkage systems. When all equipment and components are on-site the installation will commence. Upon commencement of the test, HDR will start a timing mechanism. The timing mechanism shall be undisturbed until all 6 concrete units are in place, linkages set and sealant installed. The units will be installed in the order shown in Figure 6.

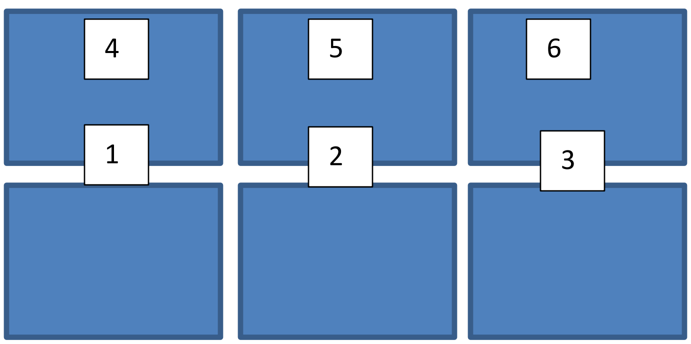


Figure 30. Unit Installation Sequence

If significant issu) can be re-set if all units and equipment are returned to the staging area and a new test started. Throughout the installation process, HDR will be responsible for documenting the progress and results. Upon finalization of the pressurization test, the system will be dismantled and removed. Similar to installation, HDR will be responsible for timing the removal process. Timing will start when the tank is drained of all water but before any linkages are dismantled or sealant removed. HDR will also document the removal process.

2. Pressurization

The seals between each of the concrete components will be tested by filling the test tank with water and identifying any leakage on the downstream face of the system. After the installation process is completed, and the linkages and seals are in place, the tank will be filled with water to a depth of 12 ft. A staff gage will be installed on the interior wall of the test tank with el 0.00 ft representing the floor of the tank. The seals take several hours to activate with the water and expand to their full diameter. Therefore, the tank will likely leak until the seals are fully activated and continuous water flow into the tank will be required to maintain the water volume. After the seals are activated and the water surface elevation is stable, the system shall be maintained for a minimum of 24 hours. During this period, the water surface elevation will be monitored regularly and a reading off of the staff gage recorded. After 24 hours, HDR shall inspect the face of the dam to identify if any leakage is present. Following the initial inspection, fluorescent dye shall be added to the tank near the upstream face of the dam. After dye has been added to the tank, the face shall be monitored for leakage for the subsequent 30 minute period. If any dye is observed on the downstream face of the structure, it shall be assumed that leakage is present. If leakage is present, FDE, GEI, and OCI shall collaborate on modifications which can be made to the sealant system to eliminate the leakage. Upon completion of the modifications, the procedure detailed above shall be completed again. If leakage is not present, the test shall be repeated under higher heads until the allotted testing resources expended.

Documentation Plan

The entire testing process will be documented to produce materials appropriate for displaying on the FDE website, for DOE reporting purposes, and for general marketing purposes. The final results from test will be included in a Prototype Test Results Report, which will include all data required under the DOE SOPO pertaining to: (1) Assembly/Disassembly Procedures and (2) Permeability Evaluation. Documentation is primary responsibility of HDR with support from the FDE Management Team and supporting consultants. The Test Results Report will be presented to the Department of Energy and the Massachusetts Clean Energy Center, both sponsors of the project, and included in the Final Report for the project.

APPENDIX F4. Structural Analysis, including uplift and seepage calculations

GEI Consultants

Consulting Engineers and Scientists

February 15, 2016
Project 1516690

Mr. William French
French Development Enterprises, LLC
3 Survey Circle
North Billerica, MA 01862

Dear Mr. French:

Re: Pressures on Precast Dam
Next Generation Hydro-Electric Dams
The French Modular Impoundment

We have performed a stability anaylsis on the prototype "French Modular Impoundment" that will be tested with 12 feet of head at a Test Facility to verify the technology can be be installed, with no seepage between adjoining precast units.

The prototype consisting of six precast concrete units with interlocking elements approximately 24-feet in length with (3 upper and 3 lower) units. Our stability analysis assumed that each precast unit was 8 feet wide, 8 feet long, and 6 feet tall (12 feet tall double stacked). Each precast unit was assumed to have 6-inch-thick walls, and one 6-inch-thick floor or roof.

Based on our anaylsis, Old Castle will need to design the precst units to the pressure diagrams in Fig. 1. The prototype precast units will need to be anchored to the 18-inch-thick concrete slab of the Test Facility.

Our anaylsis has assumed that a waterstop placed on the upstream side of the precast units between the concrete slab of the Test Facility and the precast is 100 percent effective and no uplift pressure is developed.

If you have any questions or require additional information please call Mike at (781) 721-4057.

Sincerely,

GEI CONSULTANTS, INC.

Varoujan Hagopian, P.E.
Senior Consultant, Project Manager

Michael P. Walker, P.E.
Vice President

Enclosure

VYH:jj

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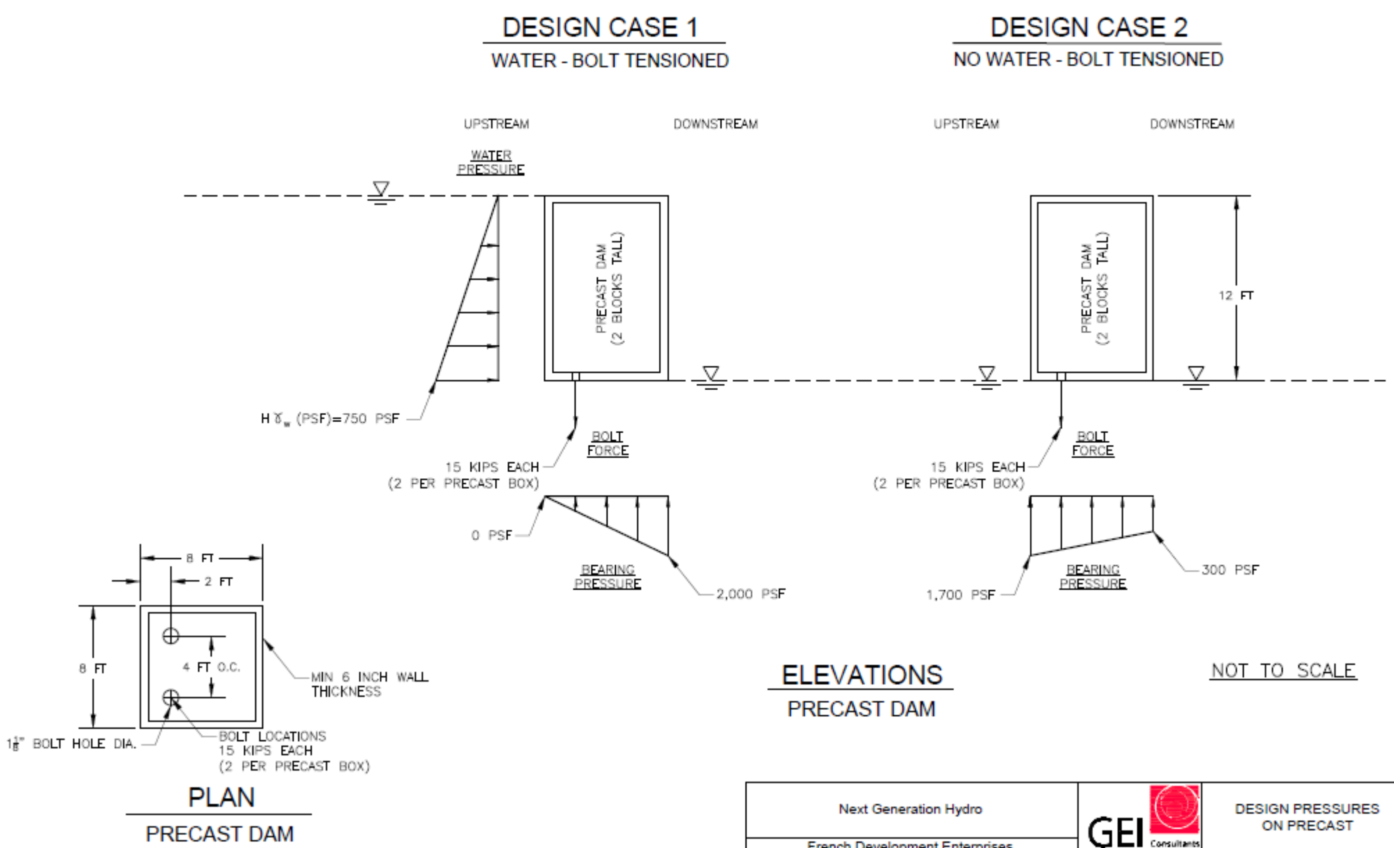
GEI Consultants, Inc.
400 Unicorn Park Drive, Woburn, MA 01801
781.721.4000 fax: 781.721.4073



Next Generation Hydro	GEI Consultants	DESIGN PRESSURES ON PRECAST
French Development Enterprises North Billerica, Massachusetts	Project 1516690	February 2016 Fig. 1

M:\DATA\2015\1516690 - Next Generation Hydro Grant\Calculations\For Construction\Pressures on Precast

Next Generation Hydro: Precast Concrete Dam

Design Precast Concrete Dam 8ft wide x 24ft long x 12 ft tall:

Calculations for width of one precast unit (8FT wide).

Givens and Dimensions

Precast Width: $P_w := 8ft$

Precast Length: $P_L := 8ft$

Precast Height (2 units): $P_H := 12ft$

Precast Wall Thickness: $H_c := 0.5ft$

Concrete Slab Thickness: $C_{st} := 1.5ft$

Height of Water: $H_w := 12ft$

Unit Weight of Concrete: $\gamma_c := 150pcf$

Unit Weight of Gravel: $\gamma_G := 120pcf$

Unit Weight of Water: $\gamma_w := 62.4pcf$

Slab Width beyond precast upstream (u), downstream (d): $S_{wd} := 0ft$ $S_{wu} := 2ft$

Dead Loads:

Precast Dam (2 Blocks High):

$$D_{PD} := \left[\left(\left(P_H \cdot P_w \cdot P_L\right)\right) - \left[\left(P_H - 2H_c\right) \cdot \left(P_w - 2H_c\right)\left(P_L - 2 \cdot H_c\right)\right]\right] \cdot \gamma_c = 34.35 \cdot kip$$

Per Foot: $D_{PDF} := \dfrac{D_{PD}}{P_L} = 4.29 \cdot klf$

Location About Toe: $x_{PD} := \dfrac{P_w}{2} = 4.00\,ft$

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Subject	Dam Stability					

18 inch Concrete Slab:

$D_{Slab} := (P_w + S_{wd} + S_{wu})(P_L)\cdot C_{st}\cdot \gamma_c$ $\qquad D_{Slab} = 18.00\cdot kip$

Per Foot: $D_{SlabF} := \frac{D_{Slab}}{P_L} = 2.25\cdot klf$

Location About Toe: $x_{Slab} := \frac{(P_w + S_{wd} + S_{wu})}{2} - S_{wd} = 5.00\,ft$

Total Dead Load: $D_{total} := D_{PD} + D_{Slab} = 52.35\cdot kip$

Total Dead Load per foot: $D_{TPF} := \frac{D_{total}}{P_L} = 6.54\cdot \frac{kip}{ft}$

Water weight on top of Slab upstream of precast dam:

$D_{water} := H_w\cdot S_{wu}\cdot \gamma_w = 1.50\cdot \frac{kip}{ft}$ $\qquad x_{water} := P_w + \left(\frac{S_{wu}}{2}\right) = 9.00\,ft$

Water Pressure (Lateral)

Water Pressure Upstream: $W_u := 0.5\cdot \gamma_w\cdot H_w^{\,2} = 4.5\cdot \frac{kip}{ft}$

Location About Toe: $y_{wu} := \frac{1}{3}\cdot H_w = 4.00\cdot ft$

Water Pressure (Uplift)

Uplift: $U_W := \frac{1}{2}(H_w\cdot P_w\cdot \gamma_w) = 3.0\cdot \frac{kip}{ft}$

Assume watertight seal at upstream edge of precast dam, therefore No Uplift: $U_W := 0$

Location About Toe: $x_{uw} := \frac{2}{3}\cdot P_w = 5.33\cdot ft$

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Sliding:

Friction Factor = 0.7 *NAVFAC DM7.2 Concrete to Rock*

Normal Force: $N_{Force} := 0.7\left(D_{TPF} + D_{water} - U_W\right)$ $\qquad N_{Force} = 5.63 \cdot \frac{kip}{ft}$

Lateral Force: $T_{Force} := W_u$ $\qquad T_{Force} = 4.5 \cdot \frac{kip}{ft}$

$$FS_s := \frac{N_{Force}}{T_{Force}} = 1.25$$

Rotation

Sum the Moments:

$$M_o := D_{PDF} \cdot x_{PD} + D_{SlabF} \cdot x_{Slab} + D_{water} \cdot x_{water} - W_u \cdot y_{wu} - U_W \cdot x_{uw} = 23.93 \cdot \frac{kip \cdot ft}{ft}$$

Sum Vertical Forces:

$$F_v := D_{PDF} + D_{SlabF} + D_{water} - U_W = 8.04 \cdot \frac{kip}{ft}$$

Resultant Location: $x_R := \frac{M_o}{F_v} = 2.98\,ft$

Resultant Ratio: $r_R := \frac{x_R}{P_w} = 0.37$

Base in Compression (%): $C_r := \min\left(3 \cdot r_R, 1.0\right) = 1.00$

Eccentricity: $e_b := \frac{P_w}{2} - x_R = 1.02\,ft$

Effective Width: $W_e := P_w - 2 \cdot e_b = 5.95\,ft$

Bearing Pressure: $q_{toe} := \frac{F_v}{P_w} \cdot \left(1 + \frac{6 \cdot e_b}{P_w}\right) = 0.89 \cdot tsf$

Bearing Heel: $q_{heel} := \frac{F_v}{P_w} \cdot \left(1 - \frac{6 \cdot e_b}{P_w}\right) = 0.12 \cdot tsf$

Design Conection Slab to take water weight upstream of precast.

Design Conection between precast and slab to keep base in compression.

Clamping Force/Anchor is required because the previous analysis assumes a connection to the slab. Re-analyze with tiedown loads.

Rotation with Tiedown

Anchor Load: $\boxed{P_a := 3.7\frac{kip}{ft}}$

Check required anchor load to resist sliding and overturning.

Anchor Location: $x_a := 6ft$

Sum the Moments:

$$M_{oa} := D_{PDF} \cdot x_{PD} - W_u \cdot y_{wu} - U_W \cdot x_{uw} + P_a \cdot x_a = 21.4 \cdot \frac{kip \cdot ft}{ft}$$

Sum Vertical Forces:

$$F_{va} := D_{PDF} - U_W + P_a = 7.99 \cdot \frac{kip}{ft}$$

Resultant Location:

$$x_{Ra} := \frac{M_{oa}}{F_{va}} = 2.68\,ft$$

Resultant Ratio:

$$r_{Ra} := \frac{x_{Ra}}{P_w} = 0.33$$

Base in Compression (%):

$$C_r := \min\left(3 \cdot r_{Ra}, 1.0\right) = 1.00$$

100% of the Base in Compression. Resultant is in the middle third - OK

Sliding:

Friction Factor = 0.7

N Force: $N_{Forcea} := 0.7\left(D_{PDF} + P_a - U_W\right)$ $N_{Forcea} = 5.60 \cdot \frac{kip}{ft}$

T Force: $T_{Forcea} := W_u$ $T_{Forcea} = 4.5 \cdot \frac{kip}{ft}$

$$FS_{sa} := \frac{N_{Forcea}}{T_{Forcea}} = 1.25$$ Okay

Find the Load required per bolted connection:

$N_{bolts} := 2$ (assume 2 bolts per precast box) $F_{bolt} := \frac{P_a \cdot P_w}{N_{bolts}}$ $\boxed{F_{bolt} = 15 \cdot kip}$

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Subject	Dam Stability					

Sliding:

Friction Factor = 0.7

N Force: $N_{Forcea} := 0.7\left(D_{PDF} + P_a - U_W\right)$ $N_{Forcea} = 5.60 \cdot \frac{kip}{ft}$

T Force: $T_{Forcea} := 0$ $T_{Forcea} = 0.0 \cdot \frac{kip}{ft}$

$FS_{sa} := \frac{N_{Forcea}}{T_{Forcea}} = \blacksquare$ Okay

Find the Load required per bolted connection:

$N_{bolts} := 2$ (assume 2 bolts per precast box) $F_{bolt} := \frac{P_a \cdot P_w}{N_{bolts}}$ $\boxed{F_{bolt} = 15 \cdot kip}$

Find the Bearing Pressure at the toe and heel:

Eccentricity: $e_{ba} := \frac{P_w}{2} - x_{Ra} = -0.93\ ft$

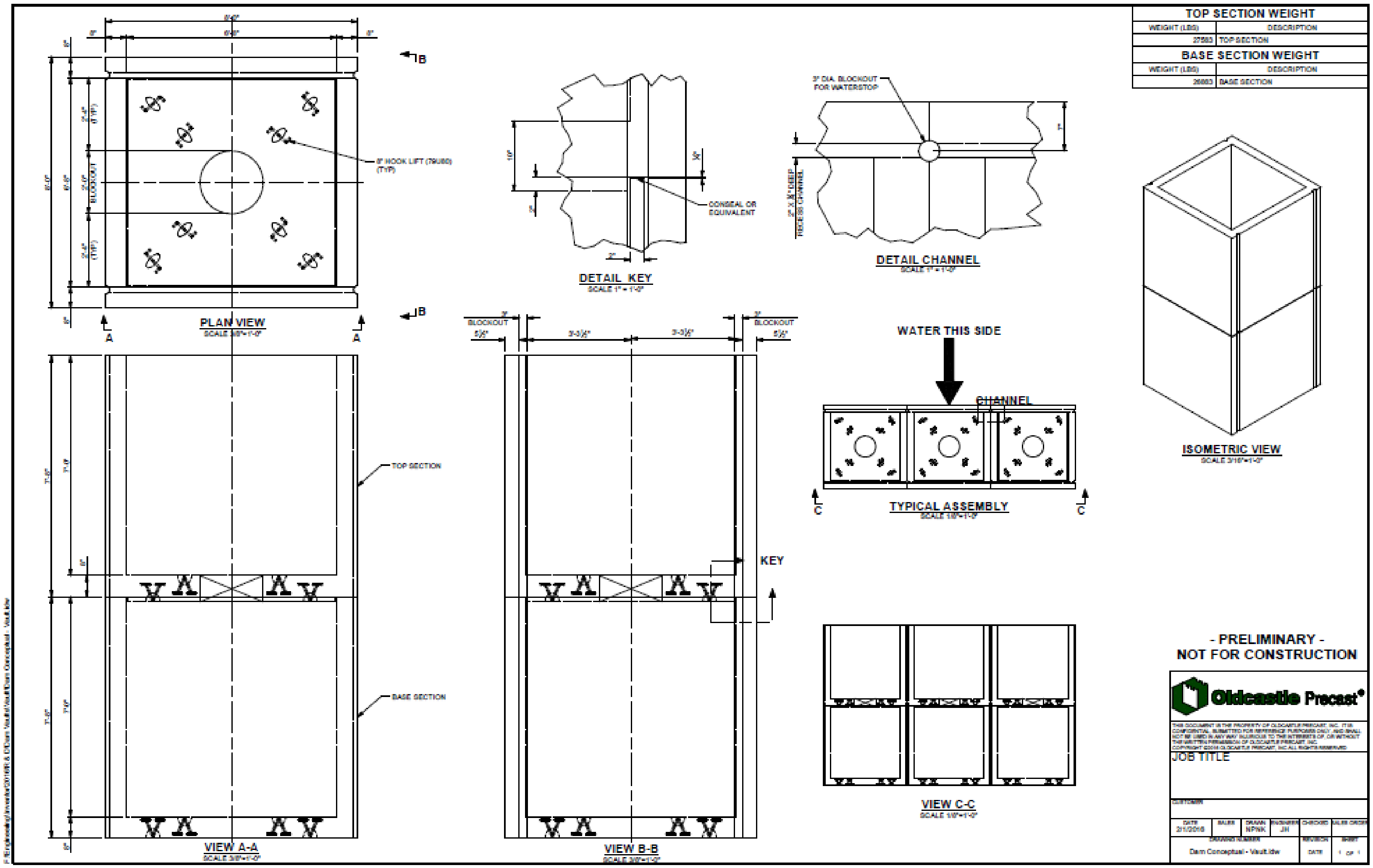
Effective Width: $W_e := P_w - 2 \cdot e_{ba} = 9.85\ ft$

Bearing Pressure: $q_{toea} := \frac{F_{va}}{P_w} \cdot \left(1 + \frac{6 \cdot e_{ba}}{P_w}\right) = 0.15 \cdot tsf$

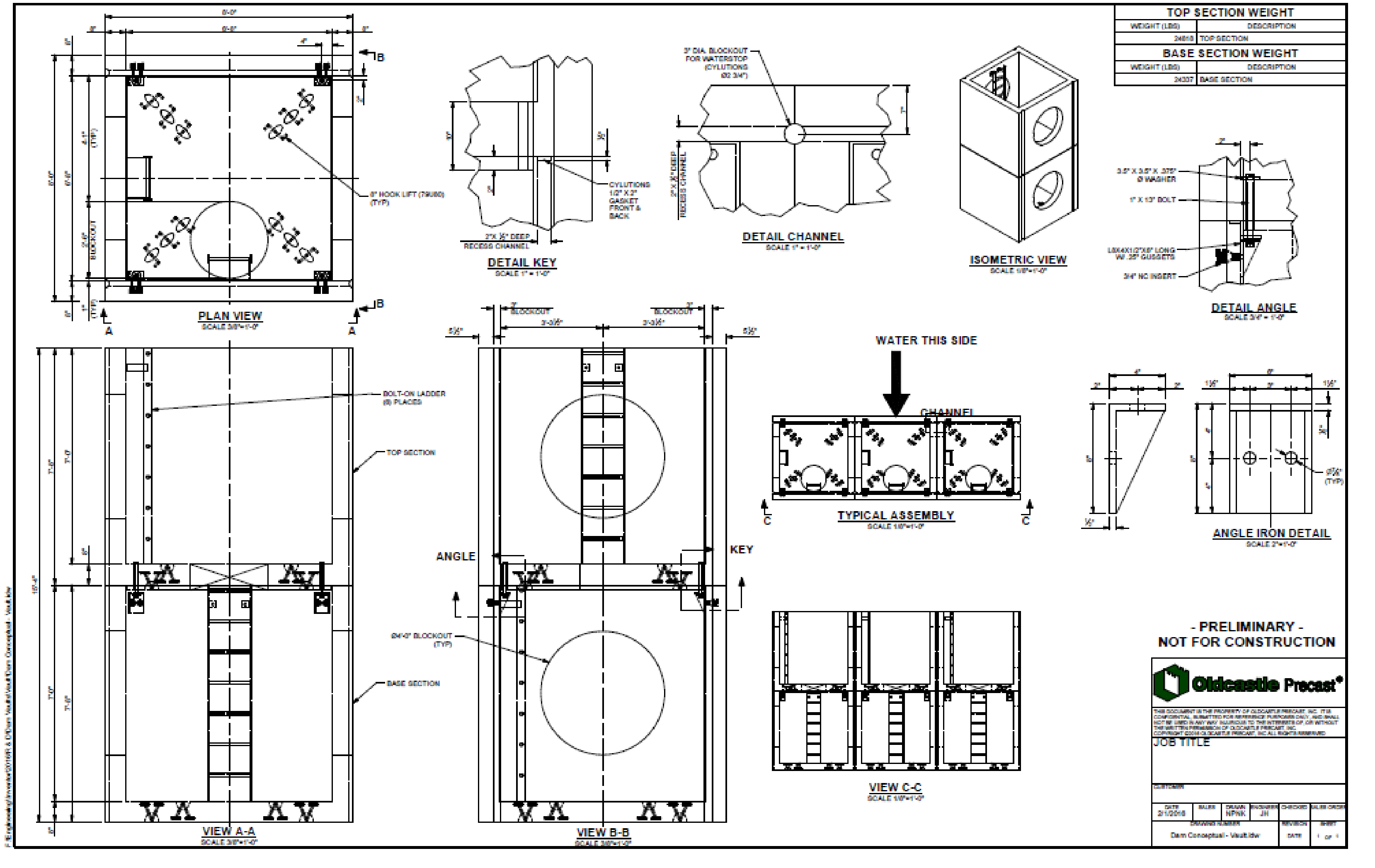
Bearing Heel: $q_{heela} := \frac{F_{va}}{P_w} \cdot \left(1 - \frac{6 \cdot e_{ba}}{P_w}\right) = 0.85 \cdot tsf$

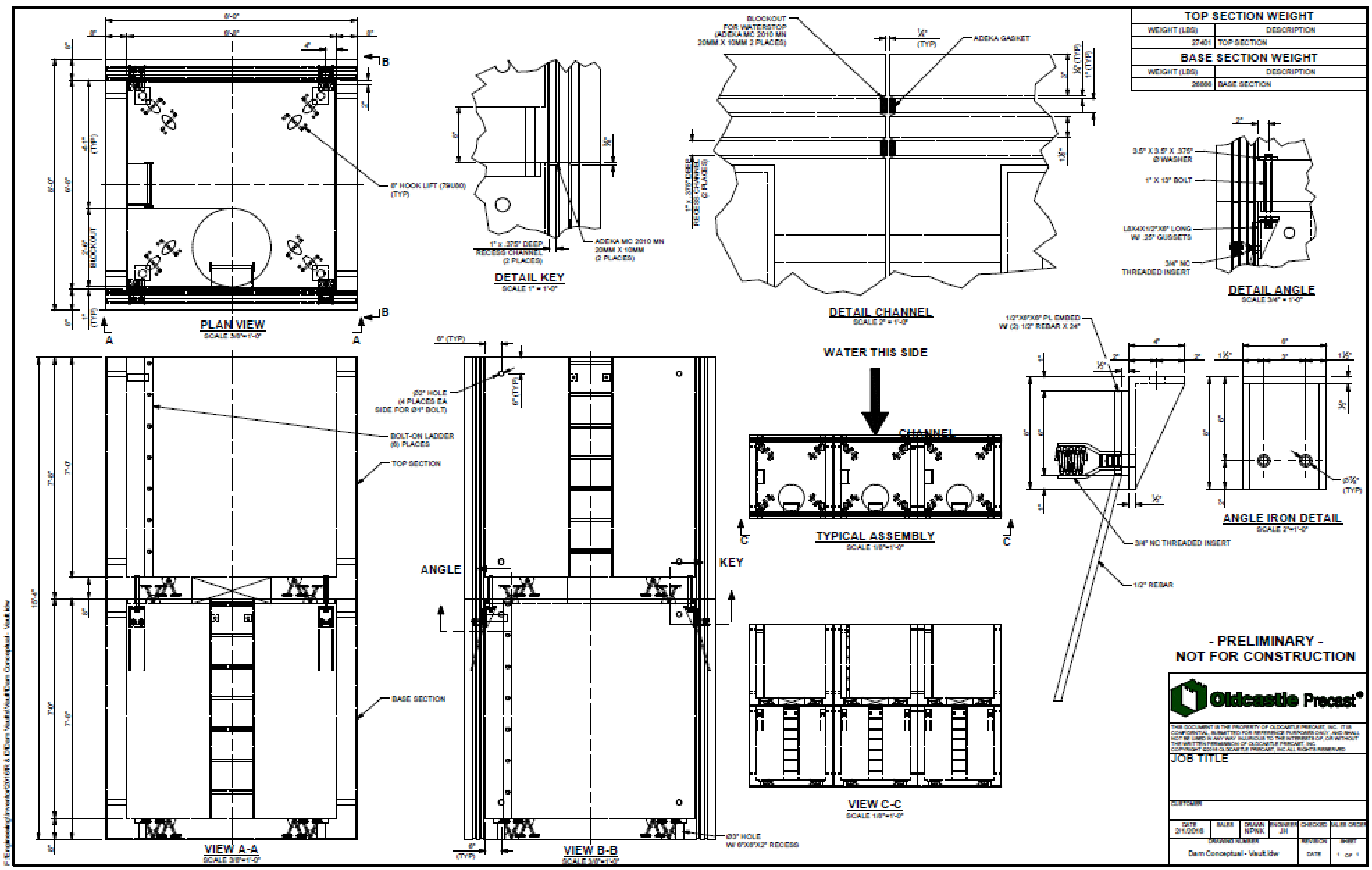
APPENDIX F5. DESIGN ITERATIONS (Chronological Order)

Design #1



Design 2
Design 2

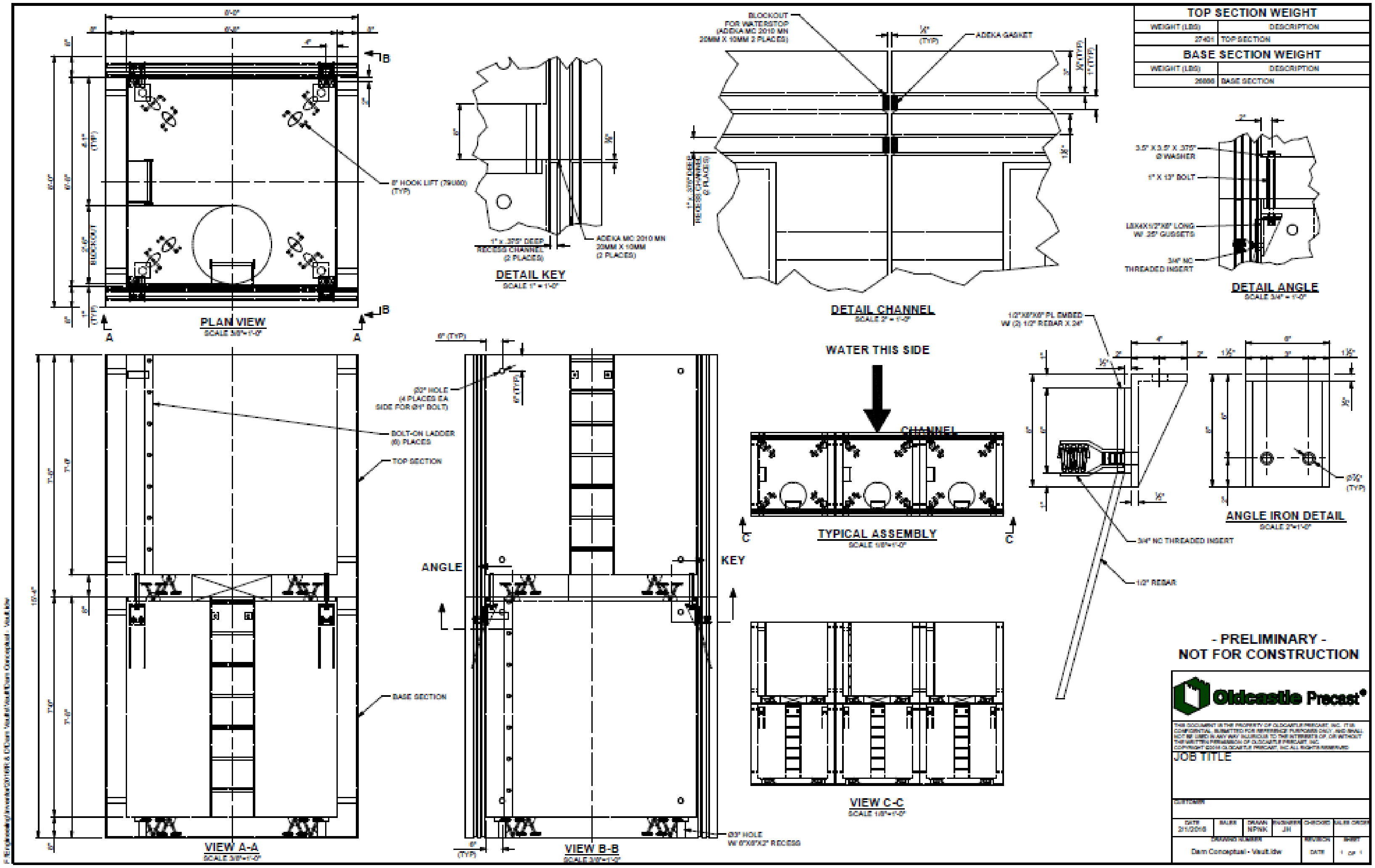




TOP SECTION WEIGHT
BASE SECTION WEIGHT
PLAN VIEW
DETAIL KEY
DETAIL CHANNEL
DETAIL ANGLE
ADEKA GASKET
WATER THIS SIDE
CHANNEL
TYPICAL ASSEMBLY
ANGLE
KEY
ANGLE IRON DETAIL
3/4" NC THREADED INSERT
1/2" REBAR
BOLT-ON LADDER (6) PLACES
TOP SECTION
BASE SECTION
VIEW A-A
VIEW B-B
VIEW C-C
- PRELIMINARY -
NOT FOR CONSTRUCTION
Oldcastle Precast
JOB TITLE

Design 3

Design 4


TOP SECTION WEIGHT
BASE SECTION WEIGHT
PLAN VIEW
DETAIL KEY
DETAIL CHANNEL
DETAIL ANGLE
WATER THIS SIDE
CHANNEL
TYPICAL ASSEMBLY
ANGLE IRON DETAIL
ANGLE
KEY
VIEW A-A
VIEW B-B
VIEW C-C
BOLT-ON LADDER
TOP SECTION
BASE SECTION
- PRELIMINARY -
NOT FOR CONSTRUCTION
Oldcastle Precast
JOB TITLE