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Reducing Seismic Dam Safety Risks through Use of Cellular Structural Systems for Foundation Strengthening

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ABSTRACT

Seismic loadings for many dam structures have increased significantly over the past several decades. These increased loadings have caused concern for existing dam foundations and embankment fills that may be susceptible to liquefaction or strain softening. This paper presents two examples of how cellular structural systems were used to construct a positive shear key and foundation to resolve the seismic stability problem without compromising dam safety during construction. A description of the final design is provided along with general site conditions and lessons learned. In both cases a unique solution was used that reduced long term seismic risks but also managed risks during construction. This method provides the positive ability to visually confirm design assumptions and provide a cost effective solution without restricting reservoir operation.

Introduction

Seismic loadings have increased significantly over the past several decades. As a result, many structures now have a seismic vulnerability under these increased loadings. A particular concern is the vulnerability of existing dam foundations and embankment fills that may be susceptible to liquefaction or strain softening. For aging structures that were never designed for high seismic loading, determining a cost-effective design solution can be a complex and daunting challenge. A well-accepted solution is excavation and replacement of problematic soils with structural backfill such a concrete.

Dam owners are under increased pressure to maintain reservoir operations at current levels due to economic impacts. Without lowering the reservoir, seismic rehabilitation of existing dam structures that include excavation at the dam toe may significantly increase dam safety risks. As a result, in situ treatment methods have been used but designers have found it difficult to confirm adequate densification and structural integrity, leaving uncertainty following construction. An owner can invest significantly in remediation yet may be unsure if the problem is fixed.

Two case histories are presented in this paper to illustrate how the cellular structural wall construction technique was used to reduce stability risks during construction as well as reducing long-term seismic dam safety risks. These cases provide two different examples of using cellular structural systems in place of open excavation and in situ treatment methods. In situ treatment methods have been used more frequently over the past several decades to provide densification of embankment and foundation soils for seismic remediation. In situ methods such as stone

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columns, jet grouting and soil mixing among others, have been shown to provide good results for seismic remediation. However, these methods can be difficult to confirm the level of densification provided in some cases. The following sections provide an overview of the design methodology for cellular structural systems as a means for seismic strengthening of existing dam foundations.

Seismic Risk and Risk Reduction of Existing Dams

Many dams were designed without consideration of significant seismic ground motions. Dynamic motions can cause weakening of materials in embankments and foundations which can lead to deformations during earthquakes or after shaking has ceased. One such example is the Lower San Fernando Dam that experienced massive deformation of the dam embankment as a result of the San Fernando Earthquake in 1971 (magnitude M6.7). As a result the dam lost all but about a meter of freeboard in the central section of the dam due to the large upstream deformations. This type of failure scenario is now considered for new and existing dams to reduce the risk of dam failure. Dynamic loading of soft materials may cause strain softening and loose granular materials may experience liquefaction as the result of the earthquake loadings. Even a thin continuous layer of these materials can lead to large deformations and potential failure. Dam embankments that experience large deformations and massive cracking may overtop or could fail due to internal erosion through these large cracks.

Downstream Foundation Strengthening with Toe Buttress Method

The costs and impacts associated with reducing reservoir levels even partially, let alone draining a reservoir completely for a year or more to accommodate modification of a structure have been shown to be prohibitive in many cases. Seismic remediation alternatives have been sought that minimize reservoir level impacts. Unique solutions have been developed to minimize these impacts. But these modification alternatives can result in significant costs and time for construction, balanced against the loss of revenue and local/regional economic impacts. For some reservoirs, economic losses can be on the order of hundreds of millions of dollars per year or greater.

Our challenge in the industry is to accomplish seismic remediation of existing dams while minimizing impact to reservoir operations. This has led to the development of the concept of the downstream "*excavate and replace*" method which strengthens the seismically vulnerable foundation and adds a buttress in the form of a downstream overlay of the existing dam. This method provides the benefit of minimizing restrictions on the reservoir level and impacts due to construction. The design concept is that a foundation shear key is created at the downstream toe and overlain by a buttress that is flexible in fitting into the often-restrictive geometric and site constraints. Additionally, buttresses can provide an added line of defense with often previously missing filter protection. The following two examples were successfully completed following this design methodology.

Mormon Island Auxiliary Dam Seismic Risk Reduction

Mormon Island Auxiliary Dam (MIAD) is part of the Folsom Project located near Sacramento California. MIAD was completed in 1953 and is located southeast of the Folsom Main Dam and is one of twelve structures that contain Folsom Lake. MIAD is a zoned earthfill embankment dam consisting of a central core, two transition zones and an outer shell on the upstream and downstream sides of the core. The crest length is about 1500 m. The embankment has a maximum height above streambed of 32 m and a maximum structural height of 53 m. Foundation conditions vary along the length of the dam. The entire core is founded directly on weathered amphibolite schist (bedrock) but a portion of the shells are founded on Quaternary alluvial deposits and historic placer mine dredged tailings. The dredged portion of this deposit is about 275 m wide, and is a mixture of sand, gravel and silt with some cobbles and boulders.

During the late 1980's it was determined that these dredge tailings were very loose and potentially liquefiable [Hynes, et al 1990]. As a result, foundation soils under the upstream shell were densified in 1991 by using dynamic compaction. The downstream foundation was densified in 1994 by constructing bottom-feed-stone-columns in a 2.7 m triangular pattern over a 60 m by 275 m area. Unfortunately, the remediation methods were later determined to not meet seismic risk guidelines. A test section using the in situ method of jet grouting was attempted at the site, but verification testing revealed significant uncertainty and concern about inadequate treatment of the foundation especially in the relatively fine grained layer just above the bedrock.

Final Design

The selected final design consisted of a key-block on the bedrock foundation with an overlay buttress. The overlay buttress includes a filter system which protects the dam from internal erosion through large cracks that may develop during a major earthquake. The selected key-block construction method uses the *"excavate and replace"* method with a uniquely applied structural cell wall system to minimize dam safety risks during construction [Harris and Scott, 2009]. Figure 1 shows a cross-section of the design including key-block and overlay features.

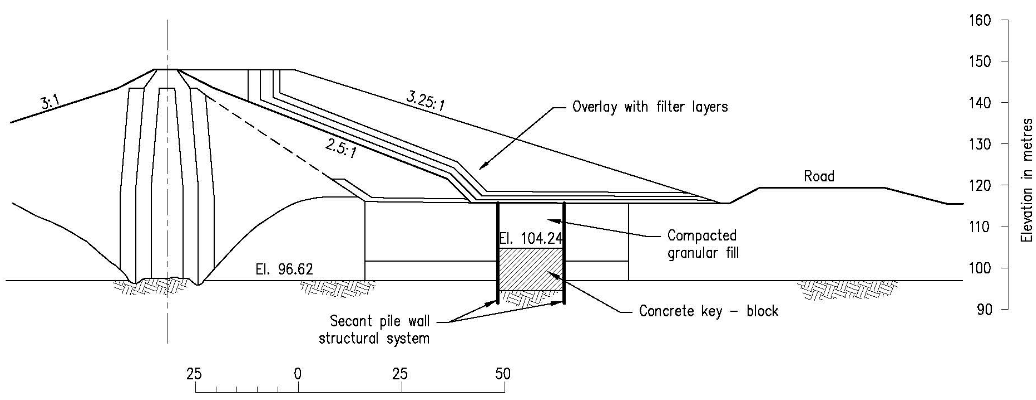

Figure 1. Typical cross-section of key-block design using structural wall system

The selected final design of the key-block was 17 m wide and 275 m long. The key-block was constructed of low strength concrete with a high slump, which allowed the material to be pumped into place and onsite aggregates to be used, thus minimizing costs. The preferred alternative specified that the key-block be constructed with no more than two cells open at a time, with no one cell longer than 45 m, and a minimum clear spacing between open cells of 76 m. This arrangement minimized dam safety risk during construction. Other advantages for this design are summarized in the lessons learned section below [Harris and Romansky, 2013].

Contracting and Construction

The key-block design requirements were included in bid documents for solicitation to contractors. This included criteria to control water pressures below the base of the excavated bedrock surface during mapping, key-block concrete placement and initial curing to ensure that a good bond is created at the bedrock/concrete interface. The contract was separated into two schedules: Schedule I included the demonstration section (17 m square) and Schedule II included the remaining portion of the key-block. The project was awarded to Shimmick Construction Co., Inc. of Oakland, California in June of 2011. Shimmick's proposed method for construction was to construct a secant-pile wall system around the perimeter of each cell with every other pile containing a structural steel member. The wall was internally braced with four to six levels of bracing, depending on the depth to bedrock and quality of the bedrock. A total of about 1,000 secant piles were constructed for this project with a designed embedment into rock of 5.5 m allowing for 2.4 m of upper bedrock removal for foundation preparation [Harris et. al., 2013]. Photos of key-block construction including support system and bedrock mapping are shown in Figures 2 and 3, respectively.

Figure 2. Key-block construction of single cell using structural wall system

Figure 3. Key-block foundation excavated and cleaned bedrock surface being mapped

Hume Dam Seismic Risk Reduction

Hume Dam is critical irrigation water supply infrastructure located on the River Murray in Australia. The main dam comprises a gated concrete gravity spillway adjoining an earthfill embankment dam with a central concrete corewall. The downstream portion of the main embankment at the junction with the spillway is retained by a concrete gravity retaining wall which is referred to as the Southern Training Wall (STW). The STW forms the left tailrace wall of the spillway stilling basin. The STW height varies from approximately 50 m near the crest of the dam to 18 m at the downstream end. The STW was founded on slightly weathered to fresh granitic gneiss rock.

In the mid to late 1980s, modifications were made to improve the stability of the STW which included the installation of sub-vertical post tensioned anchors through the full height of the STW and into the rock foundation. Additional modifications were made to the STW between 1995 and 2001 to accommodate the increased load due to a stabilising berm constructed on the downstream slope of the embankment. The modifications included the installation of post tensioned horizontal anchors that extend from the top of the STW to a concrete deadman wall located approximately 80 m into the embankment berm from the STW. A reinforced earth wall (REW) was installed to retain the berm along the crest of the STW.

The main embankment has experienced ongoing settlement of the original embankment fill materials due to the additional load from the stabilising berm fill and this has caused deflection of the horizontal post tensioned anchors. Concern was raised about overstressing and potential loss of structural capacity of the anchors due to the additional tension forces induced within the bars by catenary action. The other main issue with the STW was the stability of the wall under extreme earthquake loading conditions. The 1995-2001 STW strengthening works were designed to cope with earthquake loading events, but recent geotechnical investigations indicated that the extent of the potentially liquefiable soils within the wall backfill materials was much greater than previously thought which could overstress the wall and cause it to fail.

Final Design and Construction

Several alternatives for seismic stabilization were considered but ultimately a mass concrete gravity buttress using a structural cell wall foundation system was selected for implementation. An innovative solution was developed to maintain the stability of the existing STW structure while the deep excavations at the toe were carried out as shown in Figure 4. This comprised five individual cells formed by overlapping secant pile walls, with each cell having internal plan dimensions of approximately 10 m by 8 m. The cells were excavated into competent bedrock and backfilled with mass concrete in a carefully selected alternating sequence to minimise deflections of the existing structure. Three dimensional (3D) finite element analyses were carried out to confirm that the deflections of the STW structure during the excavation of the cells would not be sufficient to overstress the anchors.

The secant pile wall cells were supported by a reinforced concrete capping beam at the surface and the piles socketed into the rock by at least 3m to provide support at the base of the excavation. This retention system did not require installation of internal supports such as bracing or ground anchors during the excavation of the cells, which had the advantages of reducing the time that the excavations were left open, and simplifying the excavation and backfilling works in the cells. The secant pile walls were designed to become an integral component of the final buttress works [Foster et. al., 2013]. This process allowed careful cleaning and inspection of the bedrock foundation surface to a very high level of quality assurance. Concreting was also carefully controlled. Once the concrete foundation cells were complete, the remainder of the concrete buttress was constructed and the problematic anchors were de-stressed.

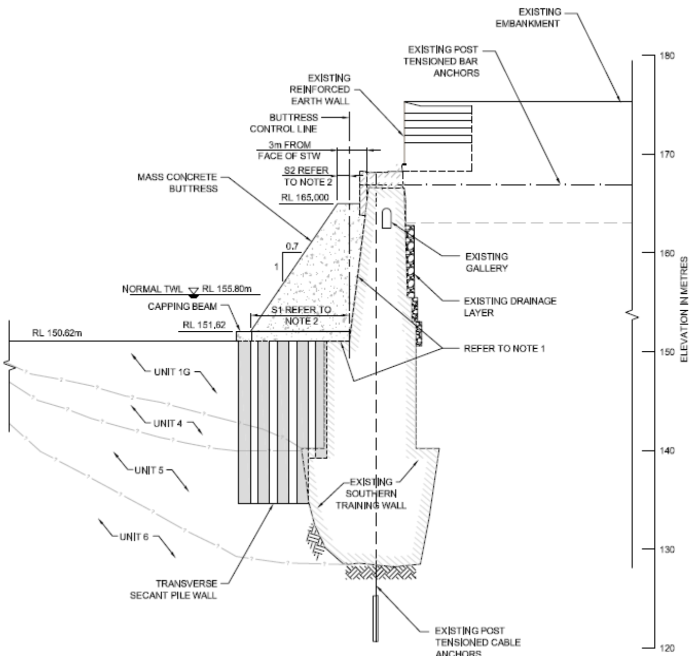

Figure 4: Cross section of STW showing the mass concrete buttress and cellular foundation

A 3-dimensional drawing of the cellular foundation system is shown in Figure 5 to help visualize the design the concrete gravity buttress.

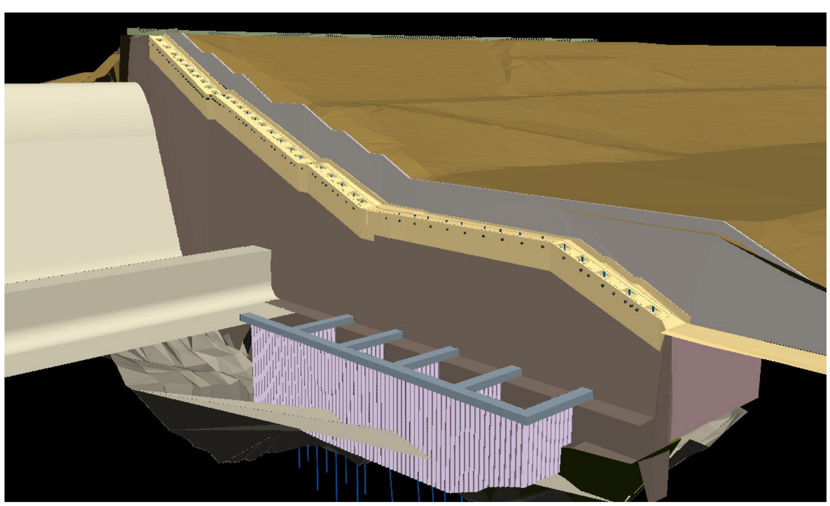

Figure 5: STW secant pile wall cell design and capping beam details

Lessons Learned

The two cases briefly presented above provided a reliable structural solution for foundation strengthening for each dam site that allowed the reservoir to maintain full operations. Knowledge learned from past projects for seismic strengthening of similar structures was used to develop a unique solution that cost-effectively managed the construction risk. The lessons learned from these two case studies and other past projects are discussed below.

Downstream "Excavate and Replace" Methodology

Construction using the downstream *"excavate and replace"* method has had precedent for seismic upgrade for dams throughout the world. The authors have been involved with several of these precedent projects. For example, the Bureau of Reclamation implemented a similar downstream foundation shear key and buttress at Rye Patch Dam in Nevada, as did ECNZ at Matahina Dam on the central North Island in New Zealand, both about twenty years ago. This was accomplished more recently by Watercare at Cosseys Dam near Auckland. However, this approach required significant reservoir restriction and dewatering to manage the excavation construction risk. *"Excavate and replace"* has the distinct advantage of physically removing any weak zones within the foundation and replacing this material with high strength backfill.

Downstream Foundation Strengthening using In Situ Treatment

In situ treatment methods have become popular in the last few decades because they can be implemented without major reservoir restriction and avoids the use of large dewatered excavations. These methods attempt to increase the shear strength of soft/loose foundation zones/layers by vibration, physical replacement, or injection of cementitious grout and mixing with the soil. These methods have been shown to increase density and strength of materials, and are effective in granular materials up to about 30 percent fines. However, they can have problems with foundations that include organics, clays, and soils with cobbles and boulders.

In situ treatment by construction of a large number of closely spaced columns through soft/loose layers of soils provides a means for increasing shear resistance through these layers. The essence of the design is based on transferring load that is concentrated in a weak layer through stronger materials and into a stronger portion of the foundation. One difficulty is assuring sufficient densification or mixing of the weak soil so that the load can be transferred into the harder layer in the foundation. Some in situ methods are not well suited to accomplish this objective and can actually create a weakness at the interface.

In situ treatment methods require conservative design assumptions of shear resistance. This is due to the uncertainty in the level of densification and treatment across the treated area. No in situ treatment should be constructed without confirmation drilling and testing of treated materials to validate design assumptions. Without comprehensive confirmation of achieving design strengths in situ treatment may not be sufficient to adequately reduce the seismic risk. Several projects have been completed where confirmation drilling raised uncertainties in the results leading to concern about their performance in an earthquake, as was the case at MIAD. There are of course many cases that have had in situ treatment where quality control and verification testing has provided compelling evidence of achieving the design intent.

Use of Cellular Structural Systems for Foundation Strengthening

Cellular structural systems can provide an attractive alternative to large open excavations or in situ treatment of problematic soils. The use of cellular structural systems for foundation strengthening for seismic remediation has the most important advantage of high reliability and quality assurance which allows more aggressive design assumptions and more efficient use of materials.

Conclusions

The use of cellular structural systems for foundation strengthening as part of seismic remediation has significant advantages over large open excavations and in situ treatment in the right conditions. This method may be cost effective if a given site has foundation materials that are difficult to treat or confirmation of design assumptions is challenging. This method also provides the positive ability to visually confirm design assumptions which decreases the uncertainty of assumed design strength. This potentially minimizes the treatment area required by making more efficient use of constructed materials and reduces residual risk of the constructed remediation method. This method can provide a cost effective solution without additional risk to dam safety during construction and minimizes restrictions on reservoir operation thus minimizing potential economic impacts.

Acknowledgments

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