Advocating publicly for the positive role of dams in society is not easy especially since negative headlines tend to get much more attention. Recent headlines such as, “Dams increase the risk of malaria,” “Dams harm coastal areas far downstream,” and “Tear down deadbeat dams” are catchy but present a limited and sometimes erroneous perspective. While dams have and will continue to serve many valuable purposes, Quenching the Thirst – Sustainable Water Supply and Climate Change, written by USSD member George Annandale, presents a clear message to help dam industry professionals understand and advocate for one of the many positive roles of dams to “satisfy the water supply needs of current generations of man, without compromising the ability of future generations to do the same.”

ICOLD Advocates — As a member of USSD you are automatically a member of a world-wide advocacy group, the International Commission on Large Dams. ICOLD is a non-governmental organization dedicated to the sharing of professional information and knowledge of the design, construction, maintenance, and impact of large dams. ICOLD benefits include free access to online resources and Bulletins, and discounted conference registration where you can collaborate with international thought leaders in our profession. Contact USSD to obtain your ICOLD access code.

Strengthening USSD’s connection to ICOLD took a big step last year when Mike Rogers, USSD past president, was elected ICOLD President, and again this year when the USSD Board formalized the role of ICOLD Liaison. ICOLD Liaison is a position filled by the immediate USSD Past President for a two-year term and is currently held by Dean Durkee. Some of the duties of the ICOLD Liaison are to coordinate and strengthen USSD representation on ICOLD technical committees, and to collect and disseminate significant information from ICOLD technical committees.

INCA Advocates — As a member of USSD, you are also a member of the ICOLD National Committee of the Americas (INCA), our regional advocacy club. In early November, INCA held the 2nd International Symposium on Dam Safety in the Americas, in Hernandarias, Paraguay. The USSD delegation for INCA was led by John Woldhope, USSD past president, and featured presentations by USSD member Enrique Matheu, PhD, Acting Associate Director, Stakeholder Engagement Division, Cybersecurity and Infrastructure Security Agency, Department of Homeland Security; and James Demby, P.E., Senior Technical and Policy Advisor, National Dam Safety Program, Federal Emergency Management Agency. As a member of INCA, USSD was asked to share its experience and best practices in dam safety, incident management, and emergency preparedness with special focus on emergency action planning. The perspectives of the DHS’s Cybersecurity and Infrastructure Security Agency — as the designated Sector-Specific Agency for the Dams Sector in the U.S.; and the Federal Emergency Management Agency — as the lead agency for the National Dam Safety Program in the U.S. — provided a valuable contribution to this important international engineering and technological event.

Levee Coalition Advocates — USSD is represented on the United States Levee Safety Coalition by Elena Sossenkina, USSD Board member, Levee Committee Chairperson, Technical Program Chair for the 2020 conference, and ICOLD Levee Committee Vice-Chair. In her leadership role on the U.S. Levee Safety Coalition, Elena has been instrumental in advocating for the National Levee Safety Program with the support of USSD Executive Director Sharon Powers, and the USSD Board of Directors. The coalition recently announced that the Senate Appropriations Committee passed its FY20 Energy & Water Development appropriations bill which included $15 million for the National Levee Safety Program’s levee inventory, triple the amount of funding this program has ever received.

Kim de Rubertis, Advocate — When I think of advocates, I think fondly of Kim de Rubertis. Our professional community was saddened when we learned Kim recently passed away in Wenatchee, Washington. In 2016 Kim was added to the list of the “rock stars” of our industry when he was awarded USSD’s Lifetime Achievement Award in recognition for his dedication, achievement and significant contributions to the dam engineering profession. In addition to his outstanding professional achievements, Kim was also a strong and tireless advocate for women in engineering and for educating future generations of dam engineers. Kim was a friend and mentor to many; he always had a smile on his face, and ready with a warm greeting. You will be missed, my friend, but your legacy will live on.

I encourage all USSD members to also be a strong and persistent advocate for our industry. There are many ways to get involved and make a difference beginning with membership on one of our many technical committees. I will continue to work closely with our Executive Director and the diverse and talented Board to continue to grow our organization and advance our mission so USSD remains a vital resource to you.

Denise Bunte-Bisnett
President, USSD
Local Connections, **Global Ideas**

Our clients face tough decisions with limited resources. That’s why we support leading water associations—like USSD—to help make great things possible for our industry.
Join us in the Mile High City for the 2020 USSD Annual Conference and Exhibition. Be a part of this premier technical event for dam and levee professionals. Learn from industry experts, share experiences, connect with colleagues, build new relationships, and collaborate with other world-class professionals dedicated to advancing the role of dam and levee systems in society.

Technical Program

An outstanding technical program is in the works, under the leadership of technical program chair Elena Sossenkina, HDR. The program begins Monday morning with the annual Legacy Lecture, where Glenn Tarbox and Kenneth Hansen will share their knowledge and wisdom on the advancement of the engineering practice for concrete dams. All attendees are encouraged to participate in USSD committee meetings on Monday and Tuesday afternoon. Concurrent technical sessions will feature five tracks with more than 100 presentations.

New in 2020: Interactive Presentations

This new format brings a number of changes to the previous poster session. Four sessions will be offered within the conference track sand will include the ability for in-depth discussions and one-on-one interactions. Each of the four sessions will focus on two or three general topics.

Exhibition — Sold Out!

Seventy-eight companies and agencies will display their products and services. Continental breakfasts, lunches, breaks and receptions will take place in the exhibit hall.

Conference Wrap Party

You won’t want to miss the Wrap Party at the History Colorado Center in downtown Denver. Learn about Colorado’s beer history, connection to the Apollo 11 Lunar Mission, life at Mesa Verde 800 years ago, the 1930s Dust Bowl, and today’s Rocky Mountains. Explore four floors of interactive displays, videos and exhibits. The party is included in full registration fee.

Workshops

Several workshops, organized by USSD Technical Committees, will be held during the conference.

- Probabilistic Flood Hazard Analysis
- Tailings Dam Safety Management and Engineer of Record
- Communication during the “Golden Hour” – Risk and Crisis Communication Strategies for Dam Safety
- Earthquake Shaking and Ground Failure Hazards for Dams, including Automated Real-time Inspection Prioritization
- Evaluation Principles for the Monitoring of Dams and Their Foundations

Field Tour

Morning and afternoon tours will be offered to the Bureau of Reclamation laboratories in Lakewood, for an additional fee of $40.

5K FUNds Run/Walk

The 6th annual FUNds Run/Walk to support the USSD Scholarship Fund will be held on Wednesday, April 22, beginning at 6:30 a.m. Participate in person or virtually for $40 ($50 after March 27). Contribute a minimum of $350 to the USSD Scholarship Fund and your logo/name will be displayed as a Partner in Education on the 5K poster.

Venue and HotelReservations

Conference activities will take place at the Hyatt Regency Denver at Colorado Convention Center. All sessions and the exhibition will take place in the hotel. The hotel is centrally located in the heart of Denver’s restaurant and entertainment district. Visit the conference website to make your hotel reservations.

More Information

http://www.ussdams.org/2020-ussd-conference/
email: 2020conference@ussdams.org
Dam Monitoring

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Mitigating PFM\S Using Construction Sequencing and Dam Safety Monitoring

Background

Five years after breaking ground, the Red Rock Hydroelectric Project (Project) is nearing completion. Located at the existing U.S. Army Corps of Engineers (USACE) Red Rock Dam on the Des Moines River near Pella, Iowa, the design of the Project began in 2011 and construction commenced in August 2014.

At peak generation, the Project will generate up to 55 MW with an average annual energy output of 178 gigawatt-hours. The Project is being developed by Western Minnesota Municipal Power Agency (WMMPA), a joint-action agency of municipalities that own their own electric systems, which is represented in the Project by the Missouri River Energy Services (MRES). Ames Construction is the General Contractor. Stantec is the Owner’s Engineer and Designer of Record.

The existing Red Rock Dam is a composite earthfill and concrete gravity dam with an overall length of 6,260 feet and a height of 110 feet. The crest elevation is approximately EL 797 (all elevations are in feet). Marion County Highway T15 traverses the length of the dam along the crest. An aerial photo of the dam prior to the start of construction of the new hydroelectric project is shown in Figure 1.

The Project is located immediately adjacent to the spillway and within the concrete gravity monolith section of the existing Red Rock Dam. The fill material present upstream and downstream of the concrete gravity monoliths comprises the “envelopment section” and consists of compacted pervious fill with 2H:1V slopes. The main embankment is a homogeneous impervious fill with downstream chimney and blanket drains. The impervious fill extends 20 feet into the envelopment section past the end of the dam’s gravity section. Additional 20-foot-wide and 40-foot-long wrap-around sections of impervious fill were placed directly against the upstream and downstream faces of the concrete monoliths to limit seepage around the end of the dam’s gravity section.

Figure 1. Aerial photo of Red Rock Dam prior to Hydroelectric Project Construction (photo provided by USACE).
A reinforced concrete intake structure founded on drilled shafts and two 21-foot-internal diameter penstocks will convey the maximum licensed flow of 10,235 cfs from the intake to the powerhouse. The penstocks will penetrate the existing dam’s concrete gravity section adjacent to the spillway and will be steel-lined downstream of the concrete monoliths. A 69-foot-high cantilevered retaining wall constructed using 130-foot-tall T-shaped concrete diaphragm wall elements socketed into rock, will retain the existing embankment dam along the new intake approach channel. A second diaphragm wall along the axis of the embankment dam forms a seepage cutoff during construction and permanent conditions. The powerhouse will house two vertical Kaplan turbine-generator units.

The Project also includes a tailrace channel bridge, multiple retaining walls, access roads, and a 69-kV substation and transmission line.

As illustrated in Figure 2 and Figure 3, significant modifications to the dam are required to construct the Project.

The construction of the permanent works required upstream and downstream cofferdams and several excavations up to 70 feet deep into the existing dam and its foundation, plus penetrations through the existing concrete gravity monolith. Recognizing the intrusive nature of this work, a Potential Failure Mode Analysis (PFMA) conducted with representatives from the USACE Rock Island District, FERC, and the Project’s Independent External Peer Review (IEPR) panel identified several potential failure modes (PFMs) for the conditions during construction. The PFMs considered to have the greatest significance were related to potential upstream diaphragm wall deformations inducing cracking in the embankment, which would shorten the seepage path, increase exit gradients and possibly increase the potential for internal erosion.

To address the PFMs, the temporary and permanent earth support systems were designed to minimize deformations, construction sequencing was established for compliance with the design basis, and a robust surveillance and monitoring program was developed. The construction sequence drawings are coupled with the Temporary Construction Surveillance and Monitoring Program (TCSMP) for the existing, temporary, and permanent structures within the Project to collectively mitigate and monitor the development of the key PFMs.

**Upstream Construction Sequence**

The entirety of the upstream works is being constructed within or on the sloping faces of the upstream side of the existing dam’s envelopment and embankment sections, both above and below reservoir levels. Prior to the start of construction, there were no flat or gently sloping areas available from which heavy equipment could operate, nor vehicular access to the area of the intake structure.

Two of the primary drivers dictating upstream construction sequence were the steep 2H:1V sloping fills within the envelopment section and the requirement to have no impact on the USACE flood control operations. The latter constraint meant that the cofferdam or working area could not extend beyond the existing spillway training wall without encroaching on the unimpeded operation of the spillway. The former constraint led to the design of a large reinforced concrete diaphragm wall to form the left approach wall of the intake channel. Excavating the upstream slope to lay back the existing ground at a safe grade was not geometrically feasible due to the existing slopes, the highway along the crest, and the diaphragm cutoff wall through the crest that was constructed to mitigate against the potential for through-going cracking of the dam’s embankment, if excess deformations were to occur during construction.
The upstream diaphragm wall temporarily retains the embankment dam along the landside of the intake and upstream penstock excavations and permanently retains the embankment along the new approach channel upstream of the intake structure, and thus is an important element of the Project’s temporary and permanent works. The design of the upstream diaphragm wall was optimized along the length of the wall to account for the critical temporary or permanent loading conditions in place at each section. It was important to limit deformations of the wall to minimize the potential for cracking of the existing embankment supported by the wall. The construction condition imposed the greatest demand on the diaphragm wall along the side of the intake and penstock excavations. In the permanent condition, the wall will be backfilled with concrete and earthfill, respectively, so there will be little to no load carried by the wall in its permeant condition in these areas. Conversely, the load against the wall upstream of the intake is essentially balanced in the construction condition, but the permanent condition is most critical, as the wall will have a 69-foot cantilever height after excavation of the new intake approach channel.

The upstream construction sequence was developed to allow construction on the steep upstream slopes and was incorporated into the design of the upstream diaphragm wall, represented in Figure 4 through Figure 7. Figure 4 shows the construction of the secant pile wall used for intake excavation and drilled shafts that support the intake structure. Figure 5 shows the completed intake excavation with bracing installed. Figure 6 shows the intake structure under construction with one of the upstream penstock excavations complete. The current construction stage is represented by Figure 7 which shows the second and final penstock excavation. Future construction stages in the upstream work area relate to excavation of the approach channel. Construction sequence drawings were developed as part of the Project’s design to describe the staged construction to reduce potential impacts to the dam’s integrity and maintain dam safety throughout construction.

**Monitoring Program**

The construction of the project presented unique challenges for the surveillance and monitoring of the existing dam throughout all stages of construction, particularly for the upstream works. Based on the PFM’s that were identified, instrumentation was designed to monitor excavations and their supports, cofferdams, and existing structures during construction, which constituted the Instrumentation Program, a portion of the TCSMP.

Development of the Instrumentation Program accounted for the construction sequencing for the Project. The
construction sequence resulted in regularly changing accessibility to installed instruments, so automated instrumentation (ADAS) was used to monitor the most critical structures. The automated readings were supplemented by manual or remote (e.g., survey) monitoring to validate recorded values. Threshold values and action levels were developed for each instrument to aid in immediate field-verification of instrumentation readings and/or to assist in determining if readings are approaching a level that would cause concern regarding the stability of a structure or project component.

A total of 125 individual instruments (including 16 existing) are used to monitor existing dam features and temporary construction works. All existing USACE instrumentation within the project boundary was incorporated into the Instrumentation Program, including survey points on the stilling basin wall and the embankment crest. An additional 109 temporary individual instruments were specified in the program. This included piezometers, inclinometers, survey points, push-in pressure cells, flow meters, strand load sensors and Rossum sand testers.

The instrumentation works together to provide a means to monitor the Project throughout construction. Clusters of instrument types were developed around key project components to more effectively monitor the performance of the systems. Instrumentation used to monitor the Project's upstream works, and a key example of the instrument clusters, includes inclinometers, piezometers, and survey monuments and prisms. Locations of each instrument type are shown in Figure 8. The most heavily monitored portion of the upstream works is the upstream diaphragm wall, which contains multiple piezometers, inclinometers, survey points, and push-in pressure cells. The construction sequencing of the upstream exposes different portions of the diaphragm wall at different phases in the project. The estimated deformations of the upstream exposes different portions of the diaphragm wall during excavation for the intake, penstocks, and approach channel were determined using Plaxis and ABAQUS finite element models. The assessed deformation values helped to determine the critical monitoring values for inclinometers and survey points, while the locations were designated based on the critical analysis sections (i.e., an inclinometer or survey point would be located in the same area as an analysis section). These locations were chosen to compare calculated displacements for each sequence to those measured during construction. Piezometer monitoring values were developed based on the phreatic levels that were used as the design basis for the excavation support systems.

The normal minimum instrument reading frequency during construction is shown in Table 1. Manual readings (non-ADAS) were increased to weekly readings during excavation in front of the upstream diaphragm wall or within a braced structure. Instrumentation data is reviewed regularly, particularly during critical stages in the construction sequence.

<table>
<thead>
<tr>
<th>Instrument</th>
<th>Reading Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vibrating Wire Piezometers (ADAS)</td>
<td>Every 4 hours</td>
</tr>
<tr>
<td>Open Standpipe Piezometers</td>
<td>Every two months</td>
</tr>
<tr>
<td>In-Place Inclinometers (ADAS)</td>
<td>Every 4 hours</td>
</tr>
<tr>
<td>Manual Inclinometers</td>
<td>Monthly</td>
</tr>
<tr>
<td>Push-in Pressure Cells (ADAS)</td>
<td>Every 4 hours</td>
</tr>
<tr>
<td>New Survey Monuments</td>
<td>Monthly</td>
</tr>
<tr>
<td>Existing Survey Monuments and</td>
<td></td>
</tr>
<tr>
<td>Surface Reference Monuments</td>
<td></td>
</tr>
</tbody>
</table>

Table 1. Normal reading frequencies during construction.

Findings

Anticipated construction sequencing played a large role in determining threshold values and action levels for developing the Instrumentation Program. With the Red Rock Hydroelectric Project under construction since 2014, the effectiveness of the construction sequence and monitoring program has been tested and provided a few key takeaways.

The Instrumentation Program has a variety of instrument
types, including survey points which make up over half of the instrumentation within the Program. These points consisted of monuments, prisms, and targets for new points, in addition to existing survey monuments. The different types of survey points were found to have varying degrees of noise or variability from reading to reading, particularly for the existing instrumentation and wall targets. Discussions with the surveyor helped to increase reading precision for these instruments and reduce noise. Obtaining lengthy baseline readings prior to construction disturbances in a given area was also beneficial to understanding trends during construction.

In addition to gathering instrumentation data, tracking construction activities played into understanding readings. Due to the confined project area, accessibility was limited, forcing surveyors to occasionally sight survey points from a different location or at a different time of day, which created some of the variability in readings mentioned above. These types of changes are communicated with field personnel in case the survey point is obstructed or shot from a different location, or if weather may have had an influence on the readings. Additionally, if there was a significant change in the reading, the surveyor would conduct a confirmation shot in the field to confirm the reading, or obtain an additional survey reading the next day if the weather was having a significant effect.

Survey readings for points near the reservoir are plotted with corresponding water level elevations. Comparing readings over time with changes in water elevations identified correlations in behavior of the instrumentation points. Correlating instrument behavior with the upstream water level has also proved helpful for the piezometers and inclinometers along the diaphragm wall. In addition, seasonal trends in measured deformation of the diaphragm wall have been observed, including horizontal deformation changes on the order of ±0.25 inches between summer and winter.

Monitoring of the key project excavation supports involved groups of varying instrumentation types. Grouping these instruments types increased confidence in instrumentation readings and provided redundancy. For instance, along the upstream diaphragm wall, variations in reported in-place inclinometer displacement were compared to nearby survey points and manual inclinometer readings to confirm the results. Similarly, vibrating wire piezometer readings are verified by comparing with manual readings, expected levels based on reservoir elevation, and piezometric readings from the push-in pressure cells.

## Conclusion

As a mitigation measure to address potential dam safety issues associated with the construction of the Red Rock Hydroelectric Project, construction sequence drawings were developed to maintain the integrity of the existing dam throughout construction and to address construction challenges. In addition to providing the construction contractor well defined parameters regarding the allowable staging of the work, the construction sequence drawings also provided an effective means to evaluate the interaction between various project features and the associated loading cases that will occur during construction, which informed the development of the Instrumentation Program and mitigation of PFMs. While the use of construction sequence drawings does impose some limitations on the construction contractor, it has been beneficial to maintaining dam safety throughout construction.

Dam safety monitoring during the various construction stages, including large excavations into and a penetration through the existing dam, required a unique and flexible approach to evaluate the performance of excavation support systems, other temporary works, and the existing dam. By combining different instrumentation, including survey points, inclinometers, and piezometers to monitor critical permanent and temporary structures, data can be cross-checked to increase confidence in readings, and deformations can be evaluated in multiple ways to determine potential causes. This approach also provides redundancy in case of instrument malfunction or construction interference. Threshold values and action levels for each instrument are based on analyses of the critical loading conditions for the various stages of construction.

The combination of instrumentation along with the individually developed threshold values and action levels allows for a thorough monitoring program to evaluate the stability and safety of the temporary works and the existing dam during construction. The resulting Instrumentation Program provides robust monitoring of critical temporary and permanent project features and allows for ongoing evaluation of the safe performance of the dam.
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2D AND 3D VISUALIZATION OF DAM SAFETY DATA
Introduction

Nonlinear deformation analyses (NDAs) are commonly used to assess the expected deformations of embankment dams and levees that are subjected to earthquake loading. Knowledge of the local geology along with site explorations (borings, lab tests, etc.) are used to assess input properties for NDAs. Seismic analyses of the embankments are then performed using several different motions (consistent with the seismic hazard) on NDA models with uniform properties (called uniform models) intended to produce reasonable system-level responses. These uniform models do not explicitly take into account spatial variability of the soil but rather use "representative" percentile properties to indirectly account for the spatial variability.

The selection of representative percentile properties for use in uniform NDA models is usually guided by past practice and engineering judgment, recognizing that selected values depend on the purpose of the analyses; e.g., a representative property could be intended to provide an unbiased (best) estimate versus a conservative estimate of a specific type of damage (e.g., crest settlement, racking of an embedded structure, deformation of foundation drains). For example, one common practice for embankment dams has been to use 33rd percentile Standard Penetration Test (SPT) or cone penetration test (CPT) penetration resistances for determining the properties of individual liquefiable strata (e.g., Perlea and Beaty 2010), which is generally consistent with studies showing that the use of mean or median properties for a liquefiable stratum can significantly underestimate earthquake-induced excess pore pressures or deformations (e.g., Popescu et al. 1997, Boulanger and Montgomery 2016). Insights on the selection of representative properties in spatially variable deposits can also be obtained from the numerous studies for other types of problems and structures, wherein the selection has been shown to depend on the nature of the structure, the scales of any deformation mechanism, the scales of fluctuation in soil properties, and the desired degree of conservatism (e.g., Baecher and Christian 2003, Fenton and Griffiths 2005).

This paper examines the selection of representative properties for use in NDAs of embankment dams of different sizes founded on a liquefiable alluvial stratum. The selection of representative properties for different geologic and site investigation scenarios are first discussed within the framework of a hypothetical segment of an event tree for a risk analysis. The effect of embankment size on selection of representative properties for an alluvial foundation layer is then examined for the event tree scenario where soil variability is assessed using site investigation data from adjacent sections. NDAs are performed for uniform models with uniform SPT \((N_1)_{60cs}\) values assigned to the alluvial layer and "stochastic" models with unconditioned, spatially correlated, Gaussian random fields of SPT \((N_1)_{60cs}\) values assigned to the alluvial layer. The NDA results are used to determine the representative percentiles of the stochastic \((N_1)_{60cs}\) values that, when used in a uniform model, produce different measures of embankment deformation (e.g., crest settlement, shell displacements) equal to those from the stochastic (random field) models. The representative percentiles are shown to vary significantly with the size of the embankment relative to the scales of fluctuation used to describe the soil’s spatial variability. The implications of these results for practice are discussed.
Representative Properties for Seismic Evaluation of Embankments

Development of Geologic Models

One of the initial steps for analysis of seismic deformations of an embankment is to evaluate the soil stratigraphy along the entire alignment and develop a geologic model based on the geologic formational history of the site. Confidence in the interpreted geologic model can vary, depending on the complexity of the site, the extent and quality of the site investigations, and whether the interpretation is preliminary or final. Allowance should be made, as indicated by the first branch in the event tree segment shown in Figure 1, for the possibility that the interpreted geologic model may be significantly inaccurate due to insufficient understanding of the formational processes or because a significant geologic feature was missed in the site investigation, especially in cases where site investigation data are sparse. An inaccuracy in the geologic model would be considered significant if it could affect seismic deformations enough to influence the computed risk or final decisions. Hypothetical (alternative) geologic models can be developed that are consistent with the site geology and available data, such as including a looser buried channel or connecting a looser continuous layer, followed by an evaluation of whether such features are large/extensive enough to impact the structure being evaluated. Explicitly allowing for the possibility that the interpreted geologic model may be inaccurate and that alternative geologic models may be applicable provides the basis for evaluating the potential benefits of performing more intensive site investigations to increase confidence in the interpreted geologic model.

Representing Spatial Variability

The geologic models (interpreted or hypothetical) can then be used to generate the analysis cross-sections for the NDAs (second branch in Figure 1). The analysis cross-sections need to represent all possible reaches (i.e., segments of the embankment length) over which significant deformations could develop largely independent of the adjacent reaches. The minimum reach length therefore depends on the scale of the potential deformation mechanisms, which depends on the size of the embankment.

This study compares deformations obtained from stochastic models and uniform models to assess the selection of uniform properties in NDAs of embankment dams so that the selection of properties accounts for the effects of spatial variability in a given stratum. The distribution of representative properties were selected to produce a distribution of embankment deformations that were expected to approximate the distribution obtained with stochastic models. This approach might use a simple three-point distribution for the representative properties, thereby requiring fewer NDA simulations compared to the use of stochastic models. The remainder of this paper examines the selection of representative values for this approach for cases where the stochastic properties would otherwise be represented with unconditional random fields.

Prior Studies on Selecting Representative Percentiles for Liquefiable Deposits

NDAs with stochastic realizations have been used to study liquefaction effects for level ground (Popescu et al. 1997, 2005), gently sloped ground (Montgomery and Boulanger 2017), and embankment dams (Boulanger and Montgomery 2016). Popescu et al. (1997, 2005) performed 2D and 3D analyses of level ground and suggested that the 20th percentile was generally conservative for obtaining estimates of maximum excess pore pressure ratio ($\text{\textit{\textit{u}}}_\text{\textit{\textit{m}}}$). Montgomery and Boulanger (2017) completed 2D NDAs with gently sloped ground and concluded that representative percentiles for predicting the expected value of lateral spreading displacements generally ranged from the 30th to 70th percentile. Boulanger and Montgomery (2016) completed 2D NDAs of a 45-m-high embankment on an alluvial foundation and concluded that representative percentiles for predicting expected values for the crest settlement or shell displacement generally ranged from the 33rd to 50th percentile. Boulanger and Montgomery (2016) concluded that representative percentiles for the liquefiable layer decreased as the thickness of the layer increased, the relative density of the soil decreased, the overburden stress increased, the ground slope increased, and the shaking intensity decreased (or magnitude of deformation decreased). A wider range of representative percentiles was required to approximate the distribution of ground displacements obtained with the stochastic models for the lateral spreading problem than for the 45-m-high embankment problem.
NDA Embankment Model

Model Configuration

Four different size embankments (see Figure 2) were analyzed using the 2D finite difference program FLAC 8.0 (Itasca 2016). Each analysis model had four material groups: the bedrock layer, the alluvial layer, the center clay core, and the embankment shells. All embankment slopes were 2.5:1 (H:V) except for the lower portion of the downstream slope which was 3.5:1. All models had a 6-m-wide crest, a 15-m-thick bedrock layer, and a 12-m-thick alluvium layer. Models were 400 m wide to ensure that the lateral boundaries did not significantly affect the deformation results. Soil elements (or zones) were generally about 0.25 m high in all models to ensure that all frequency components of interest can be appropriately transmitted.

Each embankment was incrementally built in horizontal layers to simulate the construction process and provide realistic initial stress conditions. Once the embankment construction is complete, the reservoir water level was raised in five steps. The final reservoir level was at 75 percent of the embankment height. The vertical and horizontal stresses, coefficient of earth pressure at rest (Ko), initial static shear stress ratio ($\alpha$) and other factors were checked to ensure that the initial static stress and seepage conditions were reasonable and within expected ranges (Boulanger and Beaty 2016).

Material Properties and Model Calibration

The constitutive models and parameters for each group are summarized in Table 1.

The bedrock was modeled as an elastic material. The clay core was modeled as a Mohr-Coulomb material with undrained shear strengths based on initial static consolidation stresses computed using the procedures in Duncan and Wright (2005) as applied to NDA models by Montgomery et al. (2014).

The shells and alluvium groups were modeled using PM4Sand version 3.1 (Boulanger and Ziotopoulou 2018) with properties based on a uniform SPT ($N_1$)60cs = 35 for the shells and SPT ($N_1$)60cs values for the alluvium group input as uniform values or as Gaussian random fields as described in the next section. The relative density (Dr) and shear modulus coefficient (G_s) were set based on the correlations used in Boulanger and Ziotopoulou (2018) and the contraction rate parameter (hpo) was calibrated based on single-element direct simple shear simulations to match the cyclic resistance ratio (CRR) based on the SPT based liquefaction triggering correlation from Boulanger and Idriss (2012). The remaining input parameters were kept at the default values.

Stochastic Realizations of the Alluvial Group

The alluvial group is the only material that has properties that are represented by stochastic realizations of ($N_1$)60cs. Uniform models have properties that are represented by a single ($N_1$)60cs value and stochastic models have properties that are represented by spatially correlated, Gaussian random fields of ($N_1$)60cs. The ($N_1$)60cs values were truncated so that there were no negative values, with the truncation affecting less than 0.5 percent of the elements.

A set of seven stochastic realizations of ($N_1$)60cs were generated based on spatially correlated Gaussian random fields with a mean ($N_1$)60cs of 15 and a coefficient of variation (COV) of 40 percent (Figure 3). Scales of fluctuation ($\theta$) are used to control the spatial structure of the Gaussian random fields and are defined as a measure of distance within which points are significantly correlated (Vanmarcke 2010). The anisotropic scales of fluctuation

Table 1. Embankment model group constitutive models and parameters.

<table>
<thead>
<tr>
<th>Group</th>
<th>Constitutive Model</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bedrock</td>
<td>Linear Elastic</td>
<td>Vr=0.3&lt;br&gt;G=1800 MPa&lt;br&gt; $\rho_s$=2.2 Mg/m³&lt;br&gt; $V_s$=900 m/s&lt;br&gt;k=5.0E-6 cm/s</td>
</tr>
<tr>
<td>Core</td>
<td>Anisotropically Consolidated Mohr-Coulomb (Duncan and Wright 2005 and Montgomery et al. 2014)</td>
<td>$d_s$=33 kPa&lt;br&gt;$\phi_0$=14°&lt;br&gt;$d_0$=0.03&lt;br&gt;$\psi_0$=5°&lt;br&gt;G=43 kPa at $p_t$=101.3 kPa&lt;br&gt;$\rho_s$=0.0 Mg/m³&lt;br&gt;k=5.0E-5 cm/s</td>
</tr>
<tr>
<td>Shells</td>
<td>PM4Sand version 3.1 (Boulanger and Ziotopoulou 2018)</td>
<td>Uniform SPT ($N_1$)60cs=35&lt;br&gt;Dr=87%&lt;br&gt;G_s=2.5&lt;br&gt;$\theta=3.2$&lt;br&gt;$\rho_2$=1.0 Mg/m³&lt;br&gt;k=5.0E-6 cm/s</td>
</tr>
<tr>
<td>Alluvium</td>
<td>PM4Sand version 3.1 (Boulanger and Ziotopoulou 2018)</td>
<td>Uniform SPT ($N_1$)60cs=variable&lt;br&gt;Dr=variable&lt;br&gt;G_s=variable&lt;br&gt;$\theta=variable$&lt;br&gt;$\rho_2=2.0$ Mg/m³&lt;br&gt;k=5.0E-6 cm/s</td>
</tr>
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are assigned with a value in the horizontal direction ($\theta_x$) of 20 m and a value in the vertical direction ($\theta_y$) of 1 m. The ratio of these scales of fluctuation is consistent with Phoon and Kulhawy (1999) who observed that they are typically at least an order of magnitude different. NDA results are later discussed in terms of the normalized scale of fluctuation in the horizontal direction ($NSFx=\theta_x/B$) where B is the base length of the embankment in the cross-sectional direction.

**Input Motions**

All embankment models (uniform and stochastic) were subjected to three input motions, each scaled to three peak ground accelerations (PGAs) between 0.2 g and 0.8 g. The input motions, obtained from the NGA-West2 database (Ancheta et al. 2014), are the Mudurnu station fault normal (FN) motion from the 1999 Duzce earthquake ($M=7.1$), the TCU075 station east-west recording from the 1999 Chi-Chi earthquake ($M=7.6$) and the TAPS pump station number 10-047 recording from the 2002 Denali earthquake ($M=7.9$). These motions (see Figure 4) were chosen to represent a variety of spectral shapes and ground motion characteristics so that the findings can be applicable for a wide variety of ground motion loading scenarios.

Outcrop motions were applied as a shear stress time series to the compliant base of the embankment models (Mejia and Dawson 2006). Free field conditions were applied at the lateral edges of the models, with the outer column of elements on each edge of the alluvium replaced with an equivalent elastic material to maintain confinement on the PM14Sand elements. All materials were damped using Rayleigh damping of 0.5 percent at a frequency of 3 Hz to provide a minimum level of damping in the small strain range for the nonlinear materials and a nominal damping for the linear elastic bedrock material.

**Model Results**

**Deformation Analyses**

Displacements for the uniform and stochastic embankment models are compared herein using the vertical crest settlement, horizontal displacements of the embankment toes and horizontal displacement of a point directly below the crest at the height of the top of the free field alluvium obtained at the end of shaking. The embankment “stretch,” defined as the sum of the outward horizontal displacements at the embankment toes, and embankment “translation,” defined as the horizontal displacement of a point directly below the crest and at a height even with the top of the free field alluvium. The displacements obtained from stochastic models were compared to the displacements obtained from uniform models to obtain representative percentiles for crest settlement ($P_{set}$), embankment stretch ($P_{str}$) and embankment translation ($P_{trans}$). Representative percentiles for this study are the cumulative percentile of the $(N1)_{60cs}$ data that when input as a uniform property, produces the same displacement as a stochastic model.

Normalized crest settlements and the corresponding $P_{set}$ values for the 45 m and the 10 m high embankments are shown in Figures 5a and 5b, respectively. Uniform models had $(N1)_{60cs}$ values of 7.5, 10, 12.5, 15, 17.5 and 20 which when compared to the cumulative distribution of $(N1)_{60cs}$ correspond to the 10th, 20th, 34th, 50th, 66th and 80th percentiles. The normalized crest settlements, shown in the upper plots, increase with increasing PGA and decreasing embankment size. A representative percentile for each stochastic realization is linearly interpolated from the uniform model results as shown on the lower plots of Figure 5. Values that fall below the 10th percentile are plotted at the 5th percentile and values that fall above the 80th percentile are plotted at the 90th percentile because these values are not well defined by the limited number of realizations and the ranges covered by the uniform models. The normalized crest settlement and corresponding representative percentiles for the 10-m-high embankment have greater dispersion than for the 45-m-high embankment.

![Figure 3. Cumulative distributions of $(N1)_{60cs}$ for seven unconditioned, spatially correlated, Gaussian random field realizations for the alluvium group.](image)

![Figure 4. Acceleration time series and normalized spectra for input motions (after Boulanger and Montgomery 2016).](image)
Effect of Embankment Scale

Large embankment models can have deformation mechanisms that engage a larger volume of soil than smaller embankment models. This is illustrated in Figures 6 and 7 showing results for a 45 m- and 10-m-high embankment with the same stochastic realization for the alluvium (Figures 6a and 7a) and same input motion. The deformation mechanisms are visible in the contours of maximum shear strain (Figures 6b and 7b), showing that the larger embankment causes a much larger soil volume to develop large strains. Therefore, for the 45-m-high embankment dam, the deformation behavior is dependent on the properties of a much larger soil volume than for the 10-m-high embankment, which results in greater averaging of soil resistances and less dispersion in predicted deformations as shown previously in Figure 5.

Small embankment models can have deformation mechanisms that engage a smaller volume of soil than large embankment models and can vary significantly from one realization to another. This is illustrated in Figures 7 and 8 for a 10-m-high embankment with different stochastic realizations for the alluvium subjected to the same input motion. The stochastic realization in Figure 7a, which has no significantly stronger zones along the base of the embankment, developed a relatively large crest settlement (P_set = 23%) and relatively large downstream toe movement (resulting in P_str = 18%), but a relatively small average downstream translation (resulting in P_trans > 80%). The stochastic realization in Figure 8a which has a stronger shallow zone beneath the downstream shell, developed a relatively small crest settlement (P_set > 80%) and relatively small embankment stretch (P_str > 80%), but a relatively large average downstream translation due to a deeper deformation mechanism (P_trans = 37%). These results show that deformations of smaller embankments can be controlled by localized zones of stronger or weaker soil, which can produce a larger dispersion in representative percentiles and a greater variation in representative percentiles between different displacement measures.

The effect of embankment size on representative percentiles is illustrated in Figure 9 showing representative percentiles based on crest settlement (Figure 9a) and embankment stretch (Figure 9b) versus NSFx subjected to the TAPS PS10-047 motion at a PGA of 0.6 g. Since the horizontal scale of fluctuation for these realizations is a constant (20 m), the NSFx only changes based on the embankment base width (B). The 45 m embankment (smallest NSFx) produced the smallest ranges in representative percentiles whereas increasing the NSFx (decreasing the embankment
size) increases the dispersion (range) of representative percentiles for the TAPS PS10-047 motion at a PGA of 0.6 g. The median representative percentiles range from the 37th to 60th percentile for all modeled embankments subjected to the TAPS PS10-047 motion at a PGA of 0.6 g. The effect of input motion on representative percentiles is illustrated in Figure 10 showing representative percentiles based on crest settlement (Figure 10a) and embankment stretch (Figure 10b) versus NSFx for all embankment models and input motions. The 45-m-high embankment (smallest NSFx) produces the smallest ranges of representative percentiles. These ranges in representative percentiles are generally consistent with the results from the 45 m embankment from Boulanger and Montgomery (2016). For the 5 m and 10-m-high embankments (largest NSFx), several representative percentiles fall outside the 10th and 80th percentiles. The ranges in these representative percentiles (Figure 10) are larger than obtained for individual motions and PGAs (e.g., Figure 9), indicating that uncertainty in ground motion characteristics can contribute to uncertainty in representative percentiles. The median representative percentiles range the 41st-58th percentile with no obvious dependency on NSFx.

The dispersion or range in representative percentiles tend to be greater for embankment stretch (Figures 9b and 10b) than for crest settlement (Figure 9a and 10a). This trend suggests that horizontal movement of the embankment toes may be more difficult to predict than crest settlements. This trend is attributed to the fact that localized deformation at an embankment toe can develop in a smaller volume of soil, which results in less averaging of soil behaviors. The P_set, P_str or P_trans values were only loosely correlated, such that high values for P_set sometimes coincide with smaller values for P_str or P_trans. Uniform models can be used to approximate the expected range of different embankment displacement measures, but cannot capture the complexity of the deformation mechanisms that develop in spatially variable deposits.

The Student T and the \(\chi^2\) distributions were used to assess whether additional realizations would change the distributions of representative percentiles for both single motions and for all motions (Johnson and Bhattacharyya 2011). For the smaller NSFx values, the distribution of representative percentiles is within a tighter distribution and therefore, additional realizations would not significantly change this distribution. For the larger NSFx values, additional cases would improve confidence in the distributions, but the implication of the results would not change. The representative percentiles for the largest NSFx already include cases that fall below the 10th percentiles and therefore, the choice of representative percentiles for a risk or deterministic (conservative) evaluation would need to include a branch/case controlled by these lower percentiles.

**Conclusions**

This paper examined the selection of representative properties for use in NDAs of embankment dams of different sizes founded on a spatially variable deposit of liquefiable alluvium. NDAs were performed for models with uniform and stochastic alluvial layer properties, and the results used to determine the representative percentiles from the stochastic (N1)60cs values that, when used in a uniform model, produced equal embankment displacements.

For the largest embankment (45 m high), the deformation mechanisms were large compared to the scale over which soil properties varied (\(\theta_x = 20\) m) and thus there was greater averaging of soil behaviors. The representative percentiles for these models ranged from the 40th to 70th percentile for the set of conditions and cases examined.

For the smaller embankments (e.g., 5- or 10-m-high), the deformation mechanisms were small compared to the scales over which soil properties varied (also \(\theta_x = 20\) m) and thus displacements were more strongly affected by local variations in properties. The representative percentiles had much greater dispersion (or ranges) than for the 45-m-high embankment, and often had values smaller than the 20th percentile for the set of conditions and cases examined.
The selection of representative percentiles for use in a risk analysis will depend on the extent and location of site explorations, the geometry of the structure and deformation mechanism, the variability of soil properties and the variability of input motions among other factors. Further studies building on the results presented herein are needed to provide improved guidance on the selection of representative properties for use in deterministic or probabilistic NDA studies.

Acknowledgements

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COMMITTEE CORNER

Young Professionals
Emily Schwartz, Chair (schwartze@bv.com)

The Young Professionals (YP) Committee has enjoyed another productive year full of change and growth. First are the changes to leadership: in April, our fearless leader Brandan Vavrek transitioned from Committee Chair to YP Board Advisor, with Vice Chair Emily Schwartz stepping up to the Chair position. Board Advisors Tina McMartin and Melinda Dirdal remain in their roles, and Aimee Corn continues as Vice Chair. A committee-wide vote was held this summer which resulted in the election of Yulia Zakrevskaya to the second Vice Chair position. Finally, former scholarship winner Ali Reza Firoozfar stepped into the burgeoning role of Student Outreach Subcommittee Chair. We extend a warm welcome to the newest members of our YP leadership team.

At our annual conference in Chicago over 130 attendees registered as either YPs or students, continuing the steady increase in participation and overall conference growth. Rachael Bisnett was honored with the first Exceptional YP of the Year Award. As the chair of both the 2019 Conference Planning Committee and the Embankment Dams Committee, on top of her fulltime role as a Senior Associate at Stantec, Rachael has provided tremendous contributions to the industry and organization. Thank you and congratulations Rachael! This new award was created to annually recognize a YP who exemplifies the four organization initiatives — Advocate, Educate, Collaborate, and Cultivate — through their personal and professional contributions to the dams and levees industry. If you know of a YP deserving of such recognition, please consider submitting a nomination via the USSD website.

During the Chicago committee meeting, ongoing initiatives aimed to increase student participation and conference attendance were unified into the YP Committee’s first official subcommittee on Student Outreach. Efforts are still ongoing to formalize the processes by which this subcommittee will achieve its goals. Subcommittee Chair Ali Reza Firoozfar is currently collaborating with the USSD Scholarship initiative and the Executive Committee as well as looking to engage students and faculty nationwide. If you are associated with a university with strong ties to our industry and would like to get involved as the subcommittee develops its strategy, please reach out to Ali.

In addition to the YP Committee’s flagship event, the YP Networking Social, the committee hosted the second annual Mentoring Luncheon. The event engaged YPs with a diverse group of accomplished USSD members, including several Board and ex-Board Members in an exclusive, business-casual environment. If you’re interested in being a mentor for next year’s Luncheon, please reach out to Aimee Corn.

Finally, this year, the YP Committee teamed with the Committee on Public Safety, Security and Emergency Management for Dams (COPSSEM), to host an Emergency Response Communication Workshop. This workshop was the first of an ongoing series aimed at collaborating with technical committees to simultaneously increase YP involvement in workshops and bolster technical committees’ abilities to host workshops. The workshop was well received, and another workshop is currently under development in cooperation with COPSSEM dedicated to emergency response communication during the “Golden Hour” immediately preceding an event. Please consider signing up for this workshop when you register for the conference.

We look forward to continued success in 2020, but this depends on having a solid base of support. The YP Committee is always seeking volunteers, regardless of age. Please contact me if you are interested in having a greater role within the committee.

Advocacy, Communication and Public Awareness
Keith Ferguson, Chair (keith.ferguson@hdrinc.com)

The USSD Board of Directors recently approved two position statements prepared by the Advocacy Subcommittee:

- Responsibility for Dam and Levee Safety in the U.S.
- Science, Technology, Engineering and Mathematics (STEM) Education: Creating Engineers to Meet Tomorrow’s Infrastructure Challenges

USSD Position Statements are intended to be used by members to represent a consensus position when discussing these issues in various professional, communication and education settings. Each position statement includes a summary of the USSD position, background information, and rationale for the USSD position. Position statements are peer-reviewed by expert individuals or agencies prior to approval by the USSD Board. Four position statements are now posted on the USSD website at www.ussdams.org/about/position-statements/. A statement on Dam Decommissioning is nearing completion and will be submitted for Board approval soon.

Advocacy Subcommittee YP Vice Chair Karl Tingwald is developing an initiative that will communicate legislative issues of interest to the membership in a timely manner, and allow a simple process for members to provide feedback to their Congressional representatives.

The Communication Subcommittee will continue to provide articles for Dams and Levees as publication of the Bulletin transfers to Naylor Association Solutions, a leader in association publishing with extensive knowledge of the dam and levee industry. The Bulletin will move to quarterly publication in 2020.
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2019 Lower Missouri River Flood Event

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All photos courtesy of U.S. Army Corps of Engineers.

Introduction

The Missouri river is the longest river in North America at approximately 2,341 miles in length. The Missouri River basin (Figure 1) consists of a watershed of more than 500,000 square miles that includes parts of 10 states and two Canadian provinces. Based on information with the U.S. National Levee Database (https://levees.sec.usace.army.mil/#/), within the Missouri River basin, there are over 800 levee systems totaling over 2,500 miles in length. Within these levee systems, 425,000 people live and work behind the levee and $70 billion in U.S. dollars in property value that rely on the levee to reduce flood risk. Many of these levee systems were constructed in the late 1950s through the early 1960s and have been providing flood risk reduction benefits to communities, commercial businesses, industry, and agriculture for over 50 years.

Two major flood events occurred along the lower portion of the Missouri River basin in 2019. The first major flood event occurred during March 2019 and the second major flood event during May 2019, continuing through summer 2019. This article summarizes the flood events that occurred, including the performance of levee systems along the river and recovery efforts under way.

March 2019 Flood Event and Levee Performance

Multiple factors occurred in the Missouri River basin starting in the fall of 2018 through spring 2019 that led to an extreme and devastating flood. Wet weather in late 2018 led to high soil moisture content throughout the basin. Winter of 2019 was marked by extraordinarily cold temperatures, causing significant frost depths and thick river ice. At the beginning of March 2019, there were 1 to 3 inches of snow water equivalent across the lower portion of the basin including western Iowa, southern South Dakota, and eastern Nebraska. During the week of March 11, temperatures drastically warmed, with daily highs reaching 40°F to 60°F and 1 to 4 inches of widespread rainfall occurred throughout the lower Basin. The water control reservoirs in the upper Basin had capacity to capture snowmelt; however, the flood event was caused by rainfall over watersheds and tributaries below those reservoirs. The rain on snow melt created flash flooding with little warning.

During the week of March 11, 45 river gages recorded new records along the lower river basin as shown in Figure 2. In many areas along this stretch of the Missouri River, gages exceeded records by almost three feet. The lower Missouri River stage increased from a moderate flood stage to a major flood stage in approximately 12 hours. The river at the Nebraska City gage was above the previous record set in 2011 for almost four days. This indicates the extremely high volume of water that ran off the frozen ground and into the tributaries and the ultimately into the Missouri River.

These record setting water surface elevations resulted in numerous levee systems overtopping. In many locations, overtopping of levees occurred over multiple days and over many miles of levees. The levee systems were not designed for long duration overtopping and overtopping resulted in levee breaches. Flood waters from overtopping ponded behind (i.e., landside of the levee) the levee systems, and then overtopped back into the Missouri River at the downstream end of the levee systems, causing more levee breaches. At two locations, levees breached through the tributary tieback levees. Levee systems that overtopped are estimated to have incipient overtopping at an annual chance of exceedance (ACE) of 1 in 500 or greater (i.e., 1 in 50).

Figure 3 outlines which levee systems overtopped within the U.S. Army Corps of Engineers Omaha District and the location of over 40 levee breaches (caused by overtopping). There were approximately 301 square miles of land flooded behind these levee systems. Some of the impacts of the flooding include U.S. Interstate Highway I-29 closures.
(shown in Figure 4), community evacuations and flooding, and flooding of thousands of acres of farmland. In addition to breaches to the levee systems, damages to levee systems as shown in Figure 5 included levee crest erosion, seepage berm erosion, loss of critical section, and levee ramp damage. Figure 6 provides examples of levee overtopping and levee breach due to overtopping in northern Missouri along the river.

The USACE Kansas City District, located just downstream of the USACE Omaha District along the lower Missouri River basin, experienced flood levels not seen since 1993. The Kansas City District also faced the challenge of sudden and large increases in river levels that were not well predicted. Based on the March 20 National Weather Service (NWS) river forecast, current river gage readings, and the last freeboard readings taken on the levee that evening, the Missouri River Levee System (MRLS) 500-R levee was not predicted to overtop. However, it was fully overtopping.
by the morning of March 21. On the evening of March 20, the Missouri River gage at St. Joseph was at 29.9 feet and forecasted to crest the next day at 30.5 feet. In 1993 the St. Joseph gage crested at 32.07 and the MRLS 471-460-R levee, which is on the right bank of the river at the gage location, overtopped and subsequently breached, flooding the town of Elwood, Kansas.

On March 22, 2019, the St. Joseph gage crested at 32.07 feet (USGS published data) but the 471-460-R levee did not overtop due to the large pro-active sandbagging effort of the community. Figure 7 shows the extent of levees overtopped in the Kansas City District. Several other levee systems in the Kansas City District were fully loaded. Many levee owners and operators were actively flood fighting, with the support of USACE, and were able to prevent overtopping. Levees that did not overtop still were monitored very closely and many had issues with underseepage, erosion and pipe collapses in close vicinity to the levees. Figure 8 shows damages caused by a collapsed pipe.

May 2019 - June 2019 Flood Event and Levee Performance

Record-setting precipitation persisted across much of the lower basin into the spring and summer months, causing another major flood event in the lower Missouri River basin. This record late spring rainfall on top of an already high Missouri River resulted in devastating flooding eastward of Kansas City, Missouri. Multiple rainfall events in May and June occurred in the central part of the country, inundating areas with already saturated ground. The period from April 23 to May 22 was the wettest observed period based on 131 years of rainfall recordings at Kansas City International Airport. Much of Kansas and western Missouri saw 10 to 20 inches of rain in that 30-day period, which is three to five times normal.

Much of the additional levee damage occurred in the USACE Kansas City District area while flood repair actions within the USACE Omaha District were negatively impacted by higher river levels. During this event 51 levees in the USACE Kansas City District were overtopped and 26 of these levees subsequently breached (all due to overtopping). MRLS 246-L overtopped and breached from the Missouri River and the Chariton River. Figure 9 shows the levee systems that were breached during the summer event. Again, USACE offered on-the-ground assistance to levee owners and operators even as resources became limited due to high reservoir elevations. Approximately 655 miles of levees in the Kansas City District from Kansas City to St. Louis were damaged by flooding. Damages also included flooding on the landside of levees from seepage and interior rainfall that could not drain out because of the remaining high river levels.
Additionally, due to ongoing flooding on the Missouri River, the USACE Kansas City District continued to store local runoff in USACE reservoirs in the Kansas and Osage River Basins, both tributaries to the Missouri River. A total of 9 million acre-feet, out of an available 11 million acre-feet, of flood storage was occupied. Between the end of May and mid-June, eight USACE dams set record pool levels. A total of nine USACE dams have now set new record pool elevations and two additional USACE dams were near record levels. These dams were all under 24-hour surveillance by USACE personal when levels rose near or above previous records. Some were loaded at record levels for weeks and all performed as designed without significant issues. Figure 10 shows aerial imagery of Tuttle Creek Reservoir near Manhattan, Kansas from 2018 with the lake at normal pool and from 2019 with the lake 60 feet above normal pool.

Post Flood Event Recovery

While many levees were overtopped and breached, community evacuations and flood-fight efforts helped to reduce consequences from this extreme event. One of the biggest successes during the flood event was the presence of USACE staff in the field helping levee owners and operators monitor levees and provide technical assistance and best practices for flood-fighting. Additionally, USACE teams were successful in assessing flood damages through aerial photos and helicopter surveillance where “boots on the ground” assessments were not feasible. Access to certain areas of the levee system is still very challenging even months after the peak flood levels.

More than 80 levee systems within the USACE portfolio were overtopped and breached during the event. Many of the systems experienced multiple breaches, resulting in hundreds of breaches in total. Recovery efforts will continue for months and years to come. There has been progress made in setting initial breach closures on levee systems within the Omaha District as shown in Figure 11 and 12 below. In Figure 11, the levee height is approximately 7 feet, breach width is nearly 260 feet, and breach depth is roughly 25 feet. In Figure 12, the levee height is approximately 11 feet, breach width is nearly 1,150 feet, and breach depth is roughly 65 feet.

The full impacts from the 2019 flood events will likely increase, as damage assessments are incomplete for many systems due to flood inundation. Preliminary estimates indicate that levee repair costs alone could reach $1 billion. These levees have provided flood risk reduction benefits to communities, industry, commercial businesses, and agriculture for over 50 years. Restoring the levees to continue to provide flood risk reduction benefits is a top priority.

Figure 9. Overtopped and breached levees within the Kansas City District, June 2019.

Figure 10. Aerial imagery of Tuttle Creek Lake in 2019 versus 2018.

Figure 11. Initial levee breach closure in Omaha District on March 24, 2019.

Figure 12. Initial Levee System L611-614 levee breach closure on June 12.

The full impacts from the 2019 flood events will likely increase, as damage assessments are incomplete for many systems due to flood inundation. Preliminary estimates indicate that levee repair costs alone could reach $1 billion. These levees have provided flood risk reduction benefits to communities, industry, commercial businesses, and agriculture for over 50 years. Restoring the levees to continue to provide flood risk reduction benefits is a top priority.
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Over a two-month period beginning in early April, rainfall totals of 20 to 30 inches were observed across southeastern Kansas and northeastern Oklahoma. As a result, many of the reservoirs were filled to flood-control capacity, and nearly a dozen reservoirs utilized induced surcharge capacity. The reservoirs most significantly impacted included John Redmond, Toronto, Fall River, Fort Gibson, Oologah, Kaw, Keystone, and Eufaula. A total of 11 record pools were set at USACE owned and operated Dams within the Tulsa District (SWT) boundaries during the 2019 flood event.

Keystone Lake, located upstream of Tulsa, Oklahoma, experienced a total inflow volume of 8.3 million acre-feet during the month of May, which is a record event since the completion of the dam in 1964. In comparison, the total flood-control storage volume in the reservoir is 1.2 million acre-feet. The inflow into the reservoir exceeded 200,000 cfs for 11 consecutive days, with a peak inflow of 317,000 cfs. This flood event fully exhausted the induced surcharge capacity of Keystone Dam and set a new pool of record for Keystone Dam.

A maximum release of 275,000 cfs was made from Keystone Dam, just shy of the maximum release of 307,000 cfs made in October 1986. When flows on the Arkansas River below Keystone Dam reach approximately 100,000 cfs, the river begins to load portions of the Tulsa-West Tulsa Levee System which stretches approximately 20 miles from Sand Springs, Oklahoma, to Tulsa, and reduces flood risk for approximately 10,000 people who live or work behind the structure. Due as a reminder that districts should consider the potential for dam safety engineers to be requested to support other emergency response efforts or to provide inspection and flood fighting capabilities at levee systems.

Forecasting and water management resources operated 24/7 for several weeks. The leaders from neighboring districts quickly recognized the need for support within Tulsa District and volunteered their resources, which SWT Engineering and Construction leadership quickly accepted. These volunteers from Fort Worth (SWF) and Little Rock (SWL) districts provided water management and dam and levee safety support throughout the flood fight.

Throughout the flood event, one deviation from normal operations was requested and approved by the Southwestern District (SWD) following the process outlined in the Water Control Manual. Per the Water Control Manual, once Keystone Reservoir exceeds the surcharge pool, the outflow is normally increased to pass the inflow. As Keystone Reservoir filled its surcharge pool, the load on the Tulsa-West Tulsa levees of 275,000 cfs led to multiple locations of concern with seepage, sand boils, and levee stability. Due to the risk of levee breach, a deviation was approved to increase the top of surcharge pool by 0.5 feet and allow releases from Keystone Dam to remain at 275,000 cfs instead of increasing to over 300,000 cfs and putting additional load on the System. The peak pool reached at Keystone Reservoir was 757.2 feet, or 0.2 feet above its surcharge pool elevation.

Tulsa District’s Chief of the
Hydrology and Hydraulics Engineering Section at SWT was stationed in the City of Tulsa’s Emergency Operations Center (EOC) during the flood to explain USACE operations to local officials and serve as a liaison between the Tulsa District and the City and County of Tulsa. Coordination between USACE and local officials, including the City of Tulsa and Tulsa Area Emergency Management Agency, was essential. By communicating and explaining the release plan with as much advance notice as possible, the City of Tulsa and local agencies were able to make preparations to infrastructure and to provide warning to citizens who live and work within the affected areas. USACE liaisons were also located in the Kansas EOC, Oklahoma City EOC, and Muskogee EOC. USACE shared hydraulic models with the City of Tulsa and the city updated the models and generated inundation maps for specific release scenarios using recent terrain data available from development along the Arkansas River within the vicinity of the city limits. These maps were shared with emergency officials and the public to assist with planning and evacuation efforts. For cities or communities with modeling and mapping capabilities, USACE should consider partnering with the city to generate event specific inundation maps particularly when the City has updated terrain data from recent building and site development.

Downstream, combined flows along the Arkansas, Verdigris, and Grand Rivers resulted in a peak flow of 635,000 cfs at Muskogee, Oklahoma, and set the flood of record at Van Buren, Arkansas, one of the key regulating gauges for the Arkansas River system. While discharges from the Keystone Reservoir led to flows of 275,000 cfs through the Tulsa area and flows further downstream through Muskogee, Oklahoma, reached 635,000 cfs, initial estimates showed the unregulated flow through the city of Tulsa would have been on the order of 375,000 cfs and flows through Muskogee, Oklahoma, would have approached nearly 1,000,000 cfs. This shows that despite the widespread rain and inflows, the dams within the Arkansas River Basin worked as a system and performed as intended to help store flood waters and minimize downstream flooding.

SWT is developing ideas and considering bi-annual workshops with stakeholders, local EM, county and city engineers, and other local leaders. The intent is to discuss and train these local leaders on reservoir operations, impacts/inundation from releases, channel capacity constraints, and the overall watershed system of reservoir operations.

USBR International Seminars

The Bureau of Reclamation’s International Affairs Office announces two International Seminars and Workshops to be held in 2020.

Safety Evaluation of Existing Dams International Technical Seminar and Study Tour, June 8-18, 2020. This seminar will provide professional personnel with a comprehensive guide to establishing or enhancing a visual inspection/evaluation program and increase the technical capabilities of those responsible for safety evaluations. The seminar is designed for managers, administrators, engineers, and geologists responsible for the design, construction, operation, maintenance, and safety of dams.

International Reservoir Sedimentation and Sustainability Workshop, August 24-31, 2020. The primary goal of this workshop is to provide participants with important information about reservoir sediment management for long-term sustainability. Participants at this workshop will gain an understanding about reservoir sedimentation processes and rates, as well as how to analyze and develop conceptual designs of reservoir sediment management systems. The course will cover the basic principles of sediment transport and reservoir sedimentation.

More information: www.usbr.gov/international.

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ICOLD UPDATE

Oroville Named RCC Milestone Project

The Oroville Spillways Recovery Project was recently named a Milestone RCC Project by ICOLD. President Michael F. Rogers presented the award during the 8th International Symposium on RCC Dams, held November 11-12, 2019, in Kunming, China. ICOLD. The Oroville Spillways Recovery Project was one of three to receive the prestigious award, selected by an Expert Panel from nine nominations. Oroville Dam is owned and operated by the California Department of Water Resources and the Recovery Project contractor was Kiewit Corporation.

In February 2017, as a result of record-breaking precipitation, Oroville Dam’s Flood Control Outlet (FCO) Spillway suffered a catastrophic failure of the lower chute area, and significant erosion and scour was caused by the Emergency Spillway overflow. RCC was selected as the preferred method and materials to replace the eroded FCO foundation and a new apron at the Emergency Spillway.

ICOLD 88th Annual Meeting, New Delhi

The 88th ICOLD Annual Meeting & Symposium will be held April 4-10, 2020, in New Delhi, India. Hosted by the Indian National Committee, the event will provide an excellent platform for information exchange in the fields of energy and water resources management. India currently has 437 large dams under construction.

The Symposium theme is Sustainable Development of Dams and River Basins. A highlight of the Annual Meeting will be a Technical Tour which will include the famous Taj Mahal and its incredible water features. Several workshops will also take place during the meeting.

For more information, visit http://www.icold2020.org.

27th ICOLD Congress to be Held in Marseille

The 27th ICOLD Congress will be held in Marseilles, France, during 2021. The Congress will feature four Questions:

Q104 – Concrete Dams Design Innovation and Performance
Q105 – Incidents and Accidents Concerning Dams
Q106 – Surveillance, Instrumentation, Monitoring and Data Acquisition
Q107 – Dams and Climate Change

All Congress papers must be submitted through the appropriate National Committee. USSD will issue a Call for Papers for the 27th Congress next year.

ICOLD Bulletins — Free to USSD Members

ICOLD Bulletins are a key ICOLD activity. For three to five years, experts representing a number of National Committees work to produce a state-of-the-art report on a topic important to dam engineering. USSD members are also members of ICOLD, and as such, have access to all ICOLD Bulletins free of charge. Recent bulletins available for download on the ICOLD website in preprint form (files as submitted by the Technical Committee, without final page layout) include:

• Challenges and Needs for Dams in the 21st Century
• Management of Expansive Chemical Reactions in Concrete Dams & Hydroelectric Projects
• Selection of Dam Type
• Sediment Management in Reservoirs: National Regulations and Case Studies
• Tailings Dam Design - Technology Update
• Dam Surveillance - Lessons learned from Case Histories
• Asphalt Concrete Cores for Embankment Dams
• Operation of Hydraulic Structures of Dams

Contact USSD at info@ussdams.org to receive your members only access code for the ICOLD website.
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Introduction

As of 2019, more than 91,460 dams operate across the U.S. The Federal Emergency Management Agency reported a total of 15,500 high-hazard dams as of 2016 in the U.S. According to the 2019 USACE National Inventory of Dams (NID), the average age of dams in the U.S. is 57 years old, and American Society of Civil Engineers reports that by 2025, 70 percent of dams will be more than 50 years old. Seventy-four percent of high-hazard dams have an emergency action plan. The Association of State Dam Safety Officials (ASDSO) estimates that the nation’s non-federal and federal dams will require a combined total investment of $64 billion for rehabilitation. Given the limited budget for repair and maintenance, national infrastructures require a comprehensive and consistent plan in the context of the next-generation dam safety framework. Fortunately, detailed information about all the operating U.S. dams is available in the NID. A sample of graphical information for Colorado is shown in Figure 1, including all the 1,803 dams.

This paper starts with a general review on current dam safety terminology which is consistent with other sister societies and agencies such as Pacific Earthquake Engineering Research (PEER) center. Next, a more detailed review is provided for historical dam safety and risk analysis approach, followed by a big picture from all the elements who should be involved in a modern quantitative risk analysis (QRA). The paper is based on two journal review articles: Hariri-Ardebili (2018) and Hariri-Ardebili and Nuss (2018).

Figure 1. Graphical metric provided by NID for U.S. dams; sample for Colorado.

Review of Dam Safety Terminology

This section briefly discusses the differences and similarities among six widely-used concepts in safety analysis and management of infrastructures. Some of these concepts have a root in PEER Center performance based earthquake engineering (PBEE) methodology, some in potential failure mode analysis (PFMA), and others in general earthquake engineering.
Reliability

In reliability analysis, the failure probability, $P_f$, (in terms of limit state (LS) function, $G(X) = R(X) - S(X)$) can be expressed as:

$$P_f = \mathbb{P}\{R(X) \leq S(X)\} = \int_{G \leq 0} f_R(R(f_S(S))dRdS$$

where $R$ and $S$ are resistance and stressor, respectively; $X \in \mathbb{R}^M$ is a random vector of $K$ basic variables $X = X_1, X_2, \ldots, X_K$; the randomness of $R$ and $S$ is expressed by probability density functions (PDFs) $f_R$ and $f_S$. In this context, $G(X) \leq 0$ corresponds to failure, see Figure 2a.

Hazard

Seismic hazard refers to an uncertain relationship between some levels of seismic intensity measure (IM) and the frequency or probability of a particular location experiencing at least that level of excitation. Usually, a probabilistic seismic hazard analysis (PSHA) is performed to derive the hazard curve. It expresses a plot where the horizontal axis is the IM at a site and the vertical one is annual frequency of exceedance, $\lambda_{IM}$, (inverse of the return period, $T_R$), see Figure 2b. is usually determined from a Poisson probability model:

$$\lambda_{IM} = -\frac{\ln(1 - P_f)}{t}$$

where $P_f$ is the occurrence (at least one) probability during life time $t$ (usually assumed to be 100 years for dams and 50 year for the buildings). $P_f$ might be 2-5% for the rare events.

Risk

In the context of the dam safety, risk can be defined as: “Measure of the probability and severity of an adverse effect to life, health, property, or environment. In the general case, risk is estimated by the combining probability of occurrence, probability of failure given the occurrence, and the associated consequence” (ICOLD, 2005). Risk can be quantified as:

$$R = \mathbb{P}\{\text{Load Events}\} \times \mathbb{P}\{\text{Responses | Loads}\} \times C\{\text{Loads, Responses}\}$$

where $P(A|B)$ is the conditional probability that $A$ is true given that $B$ is true, and $C$ stands for the consequences.

Risk assessment is the process of deciding whether existing risks are tolerable and if not, whether alternative risk control measures are justified or will be implemented (ANCOLD, 2003). Tolerable risk means different things to different people and organizations. Some focus on economic risks to their company or organization (e.g., insurance, offshore oil and gas) while others focus on loss of life. Most of the technical codes such as USACE, U.S. Bureau of Reclamation, ANCODL, NSW and CDA use a so-called “risk curve” (either in the form of f-N or F-N chart). An example of societal risk guideline is shown in Figure 2c (where ALARP stands for “as low as reasonably practicable”).

Figure 2. Comparison of different terminologies involved in safety assessment of concrete dams; (a) Reliability, (b) Hazard, (c) Risk, (d) Fragility, (e) Vulnerability, (f) Resilience.
Fragility

Fragility is a continuous function showing the probability of exceedance of a certain LS for a specific level of ground motion IM, im, Figure 2d:

$$\text{Fragility} = \mathbb{P}[D \geq C_{LS} | IM = im]$$

where D is the demand parameter and $C_{LS}$ is the capacity associated with the given LS.

Fragility analysis is one of the main steps in PBEE and can be derived from analytical simulations, experimental data or expert opinion. The fundamentals of fragility analysis of concrete dams with a comprehensive state-of-the-art literature review can be found in Hariri-Ardebili and Saouma (2016).

Vulnerability

Vulnerability is different from fragility (Porter, 2015). The former measures loss (in terms dollars, deaths, and downtime) while the latter measures probability. A vulnerability curve expresses the loss as a function of IM. Three major types of vulnerability curves are:

- Measuring repair cost: In such a case the repair cost is normalized by the replacement cost and is called damage factor, Figure 2e.
- Measuring life safety: In such a case the number of casualties is normalized by the number of population at risk, and expressed as a function of IM parameter.
- Measuring downtime: It is measured in terms of fraction of a year during which the structure cannot be used.

Resilience

Community resilience is the ability to prepare and plan for, absorb, recover from, and more successfully adapt to actual or potential adverse events. Resilience is a normalized function indicating capability to sustain a level of functionality or performance, $Q(t)$, for a given building, dam, lifeline, or community over a period of time, $t_{LC}$ (life cycle time). $t_{LC}$ includes the structure recovery time, $t_{RE}$, and the business interruption time, $t_{BI}$ (usually negligible). $t_{RE}$ is the time necessary to restore the functionality of a critical infrastructure system (and usually is a random variable with high uncertainties). Resilience can be defined as:

$$\text{Resilience} = \int_{t_0}^{t_{RE}} \frac{Q(t)}{t_{RE}} \, dt$$

where $t_0$ is the earthquake occurrence time. Resilience and loss of resilience, the complementary part, are usually shown through a so-called “recovery function”, Figure 2f. A comprehensive report on resilience of dams and levees can be found in National Research Council (2012).  

Historical Dam Safety and Risk Analysis

A few organizations in ICOLD member countries started to practice the risk analysis in the context of dam safety programs before the mid-1990s. In its early stages, the procedure is called “risk-informed” approach because it still has elements from deterministic approaches. The more mature version which fully accounts for the uncertainties and is in the context of probabilistic methods is called “risk-based” approach. Several methods and frameworks have been introduced into the dam safety and risk analysis during the past 30 years. One may classify them as qualitative, semi-quantitative, and quantitative approaches.

The Reclamation Risk-Based Profile System (RBPS), developed in 1997, is one of the qualitative risk analysis approaches which is based on the “Failure Index” (FI) definition. The FI is multiplied by a loss of life factor to characterize the consequences associated with a failure, and is called the “Risk Index” (RI). On the other hand, the USACE-Reclamation Semi-Quantitative Risk Analysis (SQRA) approach is a way to reevaluate the incremental risk and urgency of action. This approach is mainly built on four concepts: a) failure likelihood (remote, low, moderate, high, very High), b) consequences (level 0 to level 5), c) confidence level (low, moderate, and high), and d) risk matrix. Finally, QRA attempts to quantify risks by developing event trees of sequences of events (nodes) leading to failure and estimating probabilities at these nodes for various probability load ranges. Estimating likelihood of occurrence at the event nodes is done by expert elicitation (Bowles et al., 1998). One issue with the current QRA process is the consistency of risk estimations given different people providing estimates and potential biases with the estimators.

Elements of the Next-Generation Dam Safety Framework

The authors propose the future of risk analysis should be more numerical simulations accompanied with expert elicitation. The ultimate goal is to accurately model the failure sequence of a structure in the computer using material and load inputs based on their uncertainty distributions. Through Monte Carlo simulations, the numerically derived probability of failure can be computed. Engineers have a good feel for material properties and their uncertainty distributions; however, estimating probabilities is not as intuitive. Furthermore, material properties and uncertain bounds can be determined in the laboratory. To be trustworthy, the computer simulations must be calibrated and be able to accurately compute the structural response to a given load. Fortunately for dams, we have
minimal examples of failures and let’s keep it that way!
But, this makes it difficult to calibrate computer analyses.
Therefore, the authors propose the dams industry use large scale dam models to induce failures and back-calculate this in the computer to start validating computer models. The authors believe expert elicitation is still needed to proof-test the analyses results. We should never get to the point when computer simulations are blindly accepted. Figure 3 illustrates detailed elements for performance and safety evaluation of dams.

References


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Design Trends and Guidance for Substratum Pressure Relief Wells for Dams and Levees Using Computational Methods

Jack Cadigan, Louisiana State University

Introduction

Substratum pressure remediation methods for dams and levee embankments help maintain the integrity of critical infrastructure. While various pressure remediation methods are used in dams, levees, navigational locks, and other hydraulic structures, relief wells provide readily expandable systems alongside relatively small geographic footprints. Relief wells have been used in the United States since the 1940s, and the current iterative design procedures (EM 1110-2-1914) rely on nomographs and charts which are not always fully mathematically explained (USACE 1992, Wolff 1993). As with any underseepage analysis, engineers are expected to rely on experience to form judgements about geological and geotechnical conditions during design of relief well systems. To help provide practical guidance to designers and system managers, an effort is being made towards incorporating previous design characteristics and modern computational methods to relief well system analysis.

Average Relief Well System Design

The first step towards providing practical guidance on the design and analysis of relief well systems is to assimilate available data about existing systems and trends in implemented designs. The USACE hosts an online National Levee Database (NLD, https://levees.sec.usace.army.mil/) containing information for 9844 relief wells located primarily in the Mississippi River Valley. Practical application of this data is illustrated by the penetration depth (Figure 1); none of the relief wells in the NLD are deeper than 140 feet, with an average well penetration ratio of 50 percent. Thus, it will be beneficial to focus relief well design guidance around partially penetrating well systems.

In-Situ System Analysis

Once a well system is in place, monitoring system performance over time is critical for decisionmakers. Performance degradation can arise from head losses due to the incrustation and clogging of screen, filter, and sands around the wells around the top-most section of the well screens (Houben 2006). The most common causes of well corrosion are low pH water, high dissolved oxygen content, presence of organic acids or iron sulfate, and the presence of hydrogen sulfide or other similar gases (Cedergren 1967). Example calculations from the Profit Island, LA, relief well systems indicate that the wells were designed assuming that the screen was already 75 percent clogged to account for future well degradation (USACE 1994) which also conservatively accounts for uncertainty at the time of design.

Re-arranging the equations presented in EM 1110-2-1914 in terms of a partially penetrating relief well which has a maximum substratum pressure in the plane of relief wells, a method is presented at right to quickly and simply compare inter-year performance of relief wells in terms of only observed well flow rates and observed river stage using typical blanket theory analysis variables.

$$H_{w} = \frac{N \cdot SQ_{w}}{ekD}$$

$$i = \frac{H_{w}}{z_{i}}$$

$$FOS = \frac{z_{c}}{i}$$

where the only variables which vary after a system is in place are $H$, the river stage, and $Q_{w}$, the observed well flow rates. In an actual system, any head losses modify the observed flow rates and are represented analytically through $H_{w}$. Thus, the head losses for any individual factor do not need to be estimated or assumed constant with time. The re-arranged equations require the blanket theory parameters from the original seepage analysis performed during the design stage, and it is possible to study the Factor of Safety (FOS) provided by a system of relief wells even if the blanket theory parameters change, i.e. the landside seepage exit point changing due to borrow pits or other system developments.
Numerical Modeling

Though rehabilitation data is critical information about the current state of the well system, rehabilitation information has been reported for less than one percent of all relief wells in the NLD. Regardless of whether this is due to a lack of maintenance overall, or a lack of record keeping, as the well systems continue to age there is a growing need for an ability to model the effects of well degradation on relief well performance. Incorporating relief wells into commonly used software programs allows the engineers analyzing the well systems to modify the screen properties to account for clogging and degradation. In doing so, a potential history match between relief well properties and measured flow rates can be utilized to analyze the current state of the relief wells.

To verify a methodology for modeling relief wells, a set of USACE Waterways Experiment Station physical models were recreated in the finite element program RocScience RS3. The physical models were used in the late 1930s to early 1940s to conduct extensive laboratory experiments on the effects of borrow pits and relief wells (Figure 2). Two scenarios were modeled, one featured a borrow pit and the second featured a line of fully penetrating relief wells.

Figure 2. USACE physical relief well models (Photos:USACE 1949).

Figure 3 shows a comparison between substratum pressures observed during the USACE physical models as compared to the 3-D FEM implementation. Figure 3A shows the results for a borrow pit located on the landside, while Figure 3B shows the pressure distribution for a simulation of relief wells. In the relief well simulation, the RS3 model results deviate at the well line from the physical model results. Current research is being conducted to close the gap between in-situ systems and computational models.

Conclusions

While relief wells represent readily expandable substratum pressure control systems, their design requires engineering judgement based on previous designs and experience. Incorporating previous design trends into future guidance will help provide practical guidance for designers and system managers. The pressure head for relief well systems can be back analyzed using well flow rates and river stages without the need to explicitly know values for each of the head-loss inducing factors. It is anticipated that relief well modelling will provide the ability for practicing engineers to employ transient analyses considering a variety of scenarios such as well degradation. Perhaps the greatest potential advantage of implementing numerical modelling may be the ability to analyze relief wells in transient conditions.

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Jack Cadigan is Civil Engineering PhD student at Louisiana State University. He received his undergraduate degree, also from LSU, in Petroleum Engineering. He is especially interested in projects involving complex geological conditions.

Figure 3. 3-D Finite element model results as compared to USACE (A) borrow pit model, and (B) relief well model.

Jack Cadigan is Civil Engineering PhD student at Louisiana State University. He received his undergraduate degree, also from LSU, in Petroleum Engineering. He is especially interested in projects involving complex geological conditions.
Alternative Use of Synthetic Nanoclays for Permeation Grouting in Dam and Levee Engineering

Amy Getchell, Purdue University

Motivation

Today, with the growth in population, new dams and reservoirs are in high demand; however, many of the optimal sites for construction have already been used or are restricted because of environmental protection and new dam infrastructure must be built on more difficult sites requiring ground modification and new techniques for construction. Grouting is a common method used in both instances by which open geological defects are sealed to reduce seepage and/or to strengthen the foundation (Bruce et al. 2008; Weaver 2007).

Recently, bentonite-water dispersions have been used to directly treat foundation soils below dams and levees that may be experiencing seepage problems through a technique called “permeation grouting” which reduces the hydraulic conductivity and strengthens a soil deposit without disturbing the original soil structure (Koch 2002). However, the use of bentonite-water dispersions for permeation grouting is limited to concentrations of 5% clay by weight due to their low penetrability through soils. Dispersants, such as sodium pyrophosphate (SPP), have successfully been used to control the mobility of the bentonite-water dispersions and allow permeation of greater clay quantity dispersions (Clarke 2008; Yoon and El Mohtar 2014). Yet, even with the use of SPP, bentonite dispersions may still not be able to permeate all geomaterials with small voids due to the particle size (200 – 1000 nm in diameter) (Ochoa Cornejo 2015). These shortcomings related to bentonite dispersions served as the major motivation for this research, which intends to propose the use of carefully engineered Laponite RD (herein referred to as laponite) dispersions for the use of permeation grouting in dam and levee engineering.

Why laponite-water dispersions?

Small particle size

Laponite is a nanoclay with a (2:1) layer structure similar to that of a natural hectorite with individual disc-shaped particles that are 1 nm thick and approximately 25 nm in diameter (about one order in magnitude smaller than bentonite). The smaller particle size suggests more possibilities for permeating finer soils and, unlike bentonite, laponite is synthetic and, therefore, impurities should not hinder consistency between batches (Ochoa Cornejo 2015).

Tunable rheology

Laponite-water dispersions have unique rheological behavior characterized by initial Newtonian behavior followed by the development of a yield stress resulting from the formation of a gel structure. For example, it was found that a 3% laponite dispersion in deionized water shows essentially Newtonian behavior with viscosity ~ 4.4 mPa·s immediately after mixing; transitions to non-Newtonian response after ~ 0.5 hours, with sol to gel transition at ~ 1.5 hours, and increasingly solid-like response with continued aging. Hence, a 3% laponite dispersion can be permeated into the ground without any modification. Moreover, the rheology of laponite dispersions can be carefully tailored to control initial viscosity and the period over which the behavior remains Newtonian through the use of dispersants such as SPP. By adding 3% SPP by mass of laponite to a 3% laponite dispersion, the window of Newtonian behavior can extend to almost 24 hours. In the field this would translate into increasing the time window over which a dispersion can be injected into the ground.

Effects on hydraulic conductivity

Laboratory work has shown that Ottawa sand can be permeated with small quantities of laponite (3% by mass of water, ~1% by mass of sand) and have a resulting hydraulic conductivity almost 3.5 to 4 orders of magnitude smaller than that of a clean sand specimen (El Howayek 2011). Whereas, bentonite permeated specimens require 7.5 to 12% clay to see a reduction in permeability of 4 to 5 orders of magnitude (Hwang et al. 2011), suggesting more bentonite clay is required to see the same effect on permeability as laponite.

Final thoughts and ongoing work

Laponite-water dispersions present an alternative material to traditional bentonite-based grouts for permeation grouting in dam and levee engineering. Individual laponite particles are smaller than bentonite and when mixed with water the rheology can be controlled by varying clay concentration, dispersant (such as SPP) and time. At the laboratory scale, Ottawa sand specimens have successfully been permeated with laponite dispersions containing up to 9% clay (with the assistance of SPP). After permeation and occupation of the void space, the laponite dispersion forms a gel-like structure and ultimately reduces the hydraulic conductivity of the geomaterial.

Work is currently being conducted on consolidated undrained triaxial tests on clean Ottawa sand specimens and on sand specimens permeated with different laponite dispersions (3%RD, 6.5%RD + 5%SPP, 9%RD + 9%SPP) to probe the shear response of permeated sand specimens at times at which the rheological tests indicate...
the pore fluid will exhibit distinct and known rheological behavior. Additional custom rheological tests are also being performed at different frequencies and after a shearing/disturbance event to better understand the thixotropic nature of the dispersions relevant to potential seismic events in the field.

**References**


Amy Getchell is a PhD candidate in Civil Engineering at Purdue University. She received a masters of science degree in Civil Engineering from the University of New Hampshire, and a BS in Civil Engineering from the University of Maine.

**Improving Methods to Evaluate the Effect of Strain-Softening Clays on the Stability of Dams**

**Michael Kiernan, Auburn University**

**Introduction**

Numerical analyses are becoming increasingly common tools for evaluating potential deformations of dams under static and seismic loading. The reliability of results from these analyses depends on having techniques which can accurately represent the response of the materials within both the dam and foundation. For soils, this includes being able to capture materials that may be susceptible to strain-softening, where the soil may lose strength when sheared. The characterization and modeling of strain-softening materials, including mildly to moderately sensitive clays, can be challenging and so these materials are often represented using simplified models that cannot fully capture many important aspects of the material behavior. Additional research is needed to develop modeling approaches that can accurately model the strain-softening clays while still being useable for practicing engineers.

**Motivation**

Strain-softening of sensitive clays poses a significant risk to geotechnical structures, such as slopes and dams. Strength loss in a foundation clay was blamed for the 1998 tailings dam failure at the Aznalcollar mine in Spain (Alonso et al. 2010). In 2015, the Mount Polley tailings dam breached due to failure of a moderately sensitive clay in the foundation (IEEIRP 2015). The resulting breaches from the failures each released millions of cubic meters of water and tailings (Alonso et al. 2010, IEEIRP 2015).

Seismic loading can also lead to softening of clays and subsequent failures. The Fourth Avenue landslide occurred during the 1964 Great Alaska Earthquake leading to significant damage in downtown Anchorage (Shannon and Wilson 1964). This failure has been attributed to cyclic softening of the sensitive Bootlegger Cove clay underlying the site (Shannon and Wilson 1964). Cyclic softening in clays has been recognized as a significant risk at multiple dams including Scoggins Dam where potential strain-softening clays have been identified within the foundation (Torres 2010). In order to effectively evaluate risks and design remediations for projects with strain-softening clays, engineers need tools which can accurately assess the potential for strength loss and associated deformations.

**Modeling Strain-softening**

Numerical simulations are one commonly used tool to evaluate potential deformations of critical infrastructure projects, such as dams. These simulations require a constitutive model which can accurately represent material behavior and a numerical integration approach which can accurately solve the governing equations for the problem being analyzed. Significant research has been conducted to validate both constitutive models and simulation approaches used to model potential liquefaction-induced deformations of granular soils. This has led to a number of validated constitutive models which can capture important aspects of soil response during liquefaction (e.g., Elgamal 2002, Beatty and Byrne 2011, Boulanger and Ziotopoulou 2015). The calibration procedures for these models are also fairly well-established and often can be completed using common in-situ test data, such as Standard Penetration Test (SPT) results.

Relatively few models are publicly available to evaluate potential deformations due to cyclic softening in clayey soils. The models that are available generally fall into two categories: total stress- and effective stress-based models.
Total stress-based models (e.g., Anderson and Jostad 2005, Beaty and Dickenson 2015, Kiernan and Montgomery 2018) are often easier to calibrate, but produce a simplified representation of true soil behavior. More complex effective stress-based models (e.g., Park 2011, Seidalinov and Taibet 2014) can account for coupling between the solid and fluid phases, but can be difficult to calibrate given the limited information available in practice. An additional complication with modeling of strain softening comes from the tendency for strains to localize in a single row of elements leading to results being dependent on the size of the elements used (e.g., Thakur et al. 2006). When a numerical simulation is mesh dependent, the engineer must perform additional studies to ensure that the mesh size they are using is sufficiently small to give accurate results. This requires considerable time and effort to verify, which can delay projects and reduce confidence in the results.

Research Objectives

The current research aims to address two major sources of uncertainty when modeling the effects of strain-softening clays on dams: 1) calibration of an advanced constitutive model with limited data; and 2) errors in final results due to mesh dependency of the solutions. The recently published PM4Silt constitutive model (Boulanger and Ziotopoulou 2018) has been formulated to model the cyclic response of low plasticity silts and clays. Kiernan and Montgomery (2019) slightly modified PM4Silt in an attempt to better represent sensitive clays and used the approach to successfully model the Fourth Avenue landslide. A softening-scaling technique (Pietruszczak and Mróz 1981) is currently being implemented in PM4Silt to reduce numerical errors associated with mesh size. The model will be applied to two well-documented dam case histories involving monotonic (e.g., Mount Polley tailings dam) and cyclic loading (e.g., San Fernando Dams) in order to validate the modeling approach and develop calibration procedures that can be used in practice.

Summary

Numerical analyses are increasingly being used to evaluate potential deformations of dams under static and seismic loading. These simulations require a constitutive model which can accurately represent material behavior of interest. Relatively few constitutive models are publicly available to evaluate potential deformations due to strain-softening of sensitive clays; which pose significant risk to some dams. Available options include simplified total stress models and more complex effective stress models. An additional challenge in modeling strain-softening is that solutions may be mesh dependent.

The current research aims to address two major sources of uncertainty when modeling the effects of strain-softening clays on dams: 1) calibration of an advanced constitutive model with limited data; and 2) errors in final results due to mesh dependency of the solutions. The recently published PM4Silt constitutive model has been modified to represent the response of sensitive clays and a softening-scaling technique is currently being implemented to reduce mesh dependency of the solution. The model will be applied to two well-documented case histories in order to validate the modeling approach and develop calibration procedures that can be used in practice.

References


show much deformation or strain is required to fully remold on dams or embankments that are founded on clays are:

Key questions when performing nonlinear dynamic analyses ductile. the onset of localizations and make the material more localization. Prior studies have shown that the inclusion of localizations leads to an increased strain rate within the material, and the corollary, after a given deformation what strength is maintained? The answer to these questions has significant implications when performing stability and deformation dynamic analyses of embankments and dams founded on or containing clay.

**Research Objectives and Approach**

The objectives of this research are to evaluate the effect of strain-rate on post-peak strain-softening and localization tendencies of clays and to develop procedures to account for those effects in nonlinear dynamic analyses (NDAs). This research is proposed in three phases, as discussed below.

Phase 1 is to quantify how strain rate dependence affects the post-peak strain-softening and the development of localizations within a material. Work during this phase involves the development of a strain-rate dependent version of PM4Silt (Boulanger and Ziotopoulou 2018, 2019, Oathes and Boulanger, 2019) that can account for the evolution of a strain-rate dependent material. After development of the constitutive model, a parametric analysis will allow for the quantification of the combined effects on the stress-strain behavior.

Phase 2 is to develop a numerical strategy for modeling post-peak strain-softening and localization in non-linear dynamic analyses. The numerical strategy will utilize various known case histories of clay failures to identify the steps, assumptions, and other factors that capture the observed behavior reasonably well. The aim of this phase is to connect the behavior from Phase 1 to NDAs of embankments and dams founded on clays.

Phase 3 is developing practical guidance for selecting relevant material properties for indirectly accounting for strain-rate effects in NDAs. The desired outcome is a framework which will efficiently incorporate strain-rate dependent behavior in evaluating strength loss and localization tendencies using commonly accessible geotechnical data.

**Initial results and potential implications**

Initial results from the implementation of strain-rate dependency into PM4Silt have shown that strain-rate effects help to delay the onset of localizations as compared with non-strain rate dependent models (Figure 1). The delay in the onset of localizations allows the material to continue to maintain its strength over larger deformations. The initial results show that materials in strain-rate dependent simulations can withstand as much as five times the deformation before a large loss of strength occurs when compared to the same simulations without strain-rate dependency.
The ability to accumulate larger deformations without substantial loss of strength may have important implications for the performance of dams in seismic loading events. Needing larger deformations before a severe loss of strength occurs could require larger seismic events to generate observed failures. The additional accumulation of deformation due to strain rate effects which may be a reason that failures in earth structures founded on clays are not commonly observed after earthquakes.

**Conclusion**

The incorporation of the combined effects of strain-rate dependence, strain-softening, and localizations into analyses of earthen structures in dynamic loading can lead to an increased understanding and confidence in the results of the analyses. Furthermore, this work will potentially provide a foundation for additional research, and enable informed decision making in engineering practice that is able to incorporate these important aspects of clay behavior.

**References**


*Tyler Oathes is a PhD candidate in Civil and Environmental Engineering at the University of California, Davis. He received a BS degree in Civil Engineering from Oregon State University. He is a member of the Graduate Student Advisory Committee at UC Davis.*
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Scholarship Applications and Award Nominations

USSD annually awards scholarships to students whose academic program involves dams and levees. During 2020, USSD will offer $20,000 in scholarships for up to four students, with the amounts awarded each student to be determined by the Awards Committee. Scholarship finalists will also receive funding to present their research at the 2020 Annual Meeting and Conference in Denver.

USSD members are encouraged to submit nominations for the annual USSD Awards Program.

Lifetime Achievement Award honors an individual whose lifetime of dedication and achievement has contributed significantly to the dam engineering profession.

Award of Excellence in the Constructed Project recognizes the significant contributions made to the dam community and to society through the construction and remediation of water resources projects.

Public Safety and Security of Dams Recognition is given to an individual, organization, or project for contributions to public safety and security relating to dams and levees.

Exceptional Young Professional of the Year recognizes an individual age 35 or under who, through both career and community involvement, exemplifies USSD’s mission.

Scholarship applications and award nominations are being accepted now at www.ussdams.org/about/awards/. For more information, contact USSD, info@ussdams.org.

New USSD White Paper

Dam Safety Monitoring Training for Dam Operating Personnel was prepared by the USSD Committee on Monitoring of Dams and Their Foundations, with input from the Association of State Dam Safety Officials. The lead author was Jay N. Stateler, Bureau of Reclamation, retired; co-authors were Manoshree Sundaram; Stantec; Amanda Sutter, USACE; Brett Cochran, Denver Water Board; and Emeruwa Anyanwu, Federal Energy Regulatory Commission. The White Paper may be downloaded from the USSD website at https://www.ussdams.org/resource-center/publications/white-papers/.

Member News

The Bureau of Reclamation has named Janet White as chief of Engineering and Laboratory Services Division for the Technical Service Center in Denver, Colorado.

Rod Eisenbraun has joined the Bureau of Indian Affairs in Denver, Colorado, as a Supervisory Civil Engineer. He will be leading the BIA’s Dam Safety, Security and Emergency Management Branch.

Edwin R. Friend is now a Senior Geotechnical Engineer, USACE Risk Management Center, Lakewood, Colorado.

Jim Herbert is a Senior Geologist, with HDR in Walnut Creek, California.

Roger Kitchin is a Senior Engineer in Freese and Nichols’ Water Resource Design Division in Houston, Texas. He will be working on projects related to dams and complex flood control structures for clients across southeast Texas and Louisiana. He is a native of the UK with more than 40 years of international experience.

Steven A. Samuelson has joined Schnabel Engineering as a Senior Associate engineer in the firm’s Seattle, Washington, office. Samuelson is a civil engineer experienced in structural analyses of hydraulic structures.

Del Shannon has joined Schnabel Engineering as Principal/Senior Vice President.

In Memorium


Rodney Holderbaum, 68, of Lewisberry, Pennsylvania, passed away on August 30, 2019. He joined Gannett Fleming Inc. in 1980, where he worked until his retirement in 2017. At the time of his retirement, Rod was a Vice President and National Practice Leader for Dam Engineering.

Byron C. Karzas died at the age of 93 on July 31, 2019. He retired as Vice President and Secretary from Duff & Phelps in 1987. He and his wife Diane attended many ICOLD meetings.
A Tribute to Kim de Rubertis

USSD Life Member Kim de Rubertis passed away on September 7, 2019, at the age of 81. In 1965, Kim went to work for Harza Engineering Company in Chicago. He was resident manager for several dam projects in the U.S. and worked in Iran for several years as resident manager for a dam being constructed on the Karun River. In 1976, Kim went into private practice as a consulting engineer, working on both domestic and foreign assignments. During these years, Kim also devoted time and expertise to passing on knowledge to the next generation through volunteer teaching stints at various universities, teaching classes at utility districts, federal regulatory agency offices, and power companies and through mentoring young engineers as they began their careers. Kim was particularly concerned with the role of human error in dam safety. As a commercial construction arbitrator and testifying expert, Kim enjoyed helping people settle disputes.

In 2016, USSD gave Kim the Lifetime Achievement Award, recognizing his 54 years of civil, geotechnical, and construction engineering experience and service. The same year, he was awarded the Rickey Medal by the American Society of Civil Engineers for contributing to the advancement of knowledge in the field of dam engineering and safety assessment. He is survived by his wife, Barbara, sons Corbin and Ben, and five grandchildren.

USSD Members Offer Memories and Reflections

I don’t believe the poets have created the words to convey the impact that Kim de Rubertis had on this industry. His work developing the foundations of thousands of young professionals will live on for decades. But now, Kim has passed the torch of knowledge transfer off to all of us. Let’s not let him down. Toby Brewer

We all know dedicated, capable engineers, but what set Kim apart was his humanity, generosity, and humility. He simply had a way of connecting personally with everyone he worked with. Kim was always there with a fresh perspective and his blunt but accurate observations. With calm clarity he would identify the underlying issues, educate us on the fundamental workings of a dam, and then share a box of fresh cherries. His humanity was evident in his real interest in our lives, and the generosity with which he shared technical knowledge and wisdom. We have lost a truly unique individual who mentored so many of us, inspiring us to do better. Catrin Bryan

I knew Kim especially through the two ASCE Task Committees that developed the guideline books for Monitoring Dam Performance in 2000 and 2018, and my wife and I developed a great friendship that extended to Kim’s wife Barbara and one of his two sons, Ben, who welcomed us so well when we moved to Denver in 2003. Kim has also been a great technical mentor for me over the years and I have always enjoyed so much his insights. Now that we’re in Vancouver, BC, Kim’s great welcoming home and orchard in Cashmere, Washington, were very close, and we were so glad to visit them regularly. Pierre Choquet

Kim was a mentor and friend. He had a strong compass and knew when to put you on the back or kick you in the... when necessary! He will be missed greatly. While a big loss, his legacy and lessons live on in our memories. Farewell, Kim. R. Craig Findlay

I won’t try to recount the many engineering accomplishments that my friend compiled during his career or speak of his knowledge and insights about dam foundations and dam safety. Many of you have worked with Kim or know him from his talks and lectures at USSD functions. His presentations were always insightful and delivered with a fine sense of humor. Instead I will focus on his efforts to impart some of his knowledge and experiences on dams to students and young engineers getting started in the profession. Kim volunteered his time and efforts freely toward this goal during his career. Each year he devoted a week to lecturing students at his alma mater, Colorado School of Mines. A lot of young engineers got their start on summer intern jobs thanks to Kim’s recommendations to his students. Teaching classes at utility districts, federal regulatory agency offices, and power companies and through mentoring young engineers as they began their careers. Kim was particularly concerned with the role of human error in dam safety. As a commercial construction arbitrator and testifying expert, Kim enjoyed helping people settle disputes.

In 2016, USSD gave Kim the Lifetime Achievement Award, recognizing his 54 years of civil, geotechnical, and construction engineering experience and service. The same year, he was awarded the Rickey Medal by the American Society of Civil Engineers for contributing to the advancement of knowledge in the field of dam engineering and safety assessment. He is survived by his wife, Barbara, sons Corbin and Ben, and five grandchildren.

Mike Rogers
“Remember the rule: No fear!” is the mantra Kim regularly repeated to me. I’ve been leaning into that mantra heavily since the heartbreaking news of Kim’s passing. He always knew just what to say. There aren’t words to encompass what Kim meant to me, to the YP community, to our industry as a whole. Friend, silly, mentor, advocate, champion — he was all of these and more.

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I first met Kim back in 2012 as part of an ASCE task committee — I really didn’t know what to expect and was honestly a bit nervous with all of the experts in the room. Kim included. However, that fear was unfounded as I quickly learned Kim’s rule of life, “No Fear”.

Kim was one of the most generous and unselfish individuals I’ve met and his unwavering dedication and shear enjoyment in helping, coaching, mentoring, nurturing, and encouraging us to strive to be our best is unmatched. Kim also always made sure to regularly remind me to enjoy life as well. Many people and organizations in all corners of the globe benefited from Kim’s generosity and enthusiasm, especially his efforts in championing women and minorities, and fostering inclusivity. Though his physical presence and regular reminders and gentle (or not so gentle!) nudges will truly be missed, his impact remains and continues to inspire me to continue to live Kim’s lessons and pass those lessons along.

Manoshree Sundaram
Kim took so many people under his wing and helped them grow professionally. I am so glad I got to be one of them. I don’t know how I knew who I was the first time he invited me to help facilitate a workshop he was putting together. After that experience, he was in my corner regularly encouraging me to reach for more, assuring me I could do it. Our profession has lost a valuable asset with his passing. I have lost a true mentor and friend.

Amanda Sutter
There was always room at Kim’s table, even if all the seats were taken. Kim always had room for one more person to mentor, encourage, and champion and had a passion for inspiring young engineers. I was fortunate enough to get to know Kim and will never forget his willingness to share his knowledge and ideas. Kim was a voice for our industry and organization, a passionate teacher and advocate, and a true leader. I’ll miss finding my spot at Kim’s table.

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