ICOLD

INTERNATIONAL COMMISSION ON LARGE DAMS

COMMITTEE ON COMPUTATIONAL ASPECTS OF ANALYSIS AND DESIGN OF DAMS

15TH INTERNATIONAL BENCHMARK WORKSHOP ON NUMERICAL ANALYSIS OF DAMS

9 – 11 SEPTEMBER 2019, MILAN, ITALY

Theme A - Formulation

SEISMIC ANALYSIS OF PINE FLAT CONCRETE DAM

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Record of Revisions to the Theme A Formulation

Revision number	Description	Date
0	Formulation Document	Feb. 7, 2019
1	Figure 3 – D/s slope info removed Sect. 7.3 & 8.3 no damping; Section 6 through 11- results request - updated;	March 11, 2019

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1. INTRODUCTION

The ICOLD Committee A on Computational Aspects of Analysis and Design of Dams is organizing the 15th International Benchmark Workshop on Numerical Analysis of Dams. Theme A of the workshop is related to a seismic analysis of Pine Flat Dam. The case study proposed for the workshop in Milan is a continuation of the investigations initiated by the United States Society on Dams (USSD) Concrete Dams Committee and Earthquakes Committee during the workshop organized for the 2018 USSD Annual Conference and Exhibition in Miami, Florida on May 4, 2018, titled, "*Evaluation of Numerical Models and Input Parameters in the Analysis of Concrete Dams.*"

The purpose of these workshops is to investigate uncertainties in finite element (FE) analyses of concrete dams in a focused, systematic and controlled approach with collaborative participation from the international dam industry and academia. The USSD initiative started with the workshop in Denver in 2016, when a study of Monticello Dam was conducted. Then during the USSD Conference in Anaheim in 2017 the seismic aspects of concrete dam analysis were discussed. In the following year, investigations of a simple linear model of Pine Flat Dam was offered for analyzing seismic loads. The study results are summarized in the report [7].

The Formulation Committee (Formulators) for Theme A have defined new case studies for the workshop in Milan based on the outcomes from the Miami USSD workshop. The goals of this study are to identify key uncertainties that may significantly affect numerical modeling results, determine the need for future workshop investigations, determine research needs, and develop best practices in the advanced analysis of concrete dams. The case studies are intentionally narrow in focus so that the assumptions, factors and methods having the greatest effect can be identified. Little is gained if participants perform benchmark analyses and get widely varying answers when the input parameters, boundary conditions, and implicit assumptions are also so widely varying that the key factors cannot be clearly identified.

The validity of any numerical model ultimately rests on how well the model predicts data obtained from the field (e.g., ground motion time histories from earthquakes, shaker test results, inelastic crest displacements, or mapping of cracks). Only then will there be confidence that the model can reasonably approximate real-world behavior. Ideally, a fundamentally sound computation model can be developed systematically from first principles and established empirical relations, and then independently assessed and verified for its capabilities. Unfortunately, a comprehensive framework for modeling the nonlinear behavior of concrete dams and reservoirs under realistic seismic loadings has yet to be developed.

The objective of this effort is to begin the development of such a framework by examining how complex problems might be divided into simpler sub-problems that have tractable solutions. Development of this framework is expected to benefit the profession by establishing a common conceptual basis for the advanced seismic analysis of concrete dams. Simple model geometry and inputs are selected for the case studies so as not to overtax workshop contributors (Contributors), but which, in aggregate, will produce meaningful results from the work submitted by each Contributor.

2. FORMULATION

Pine Flat Dam, located on King's River, east of Fresno, California (Figure 1), was constructed by the US Army Corps of Engineers in 1954. It consists of thirty-six 15.25 m-wide and one 12.2 m-wide monoliths. The length of the straight gravity dam is 561 m and the tallest non-overflow monolith is 122 m high (Figure 2).

Figure 1 Downstream view of Pine Flat Dam [ref. Wikipedia.org]

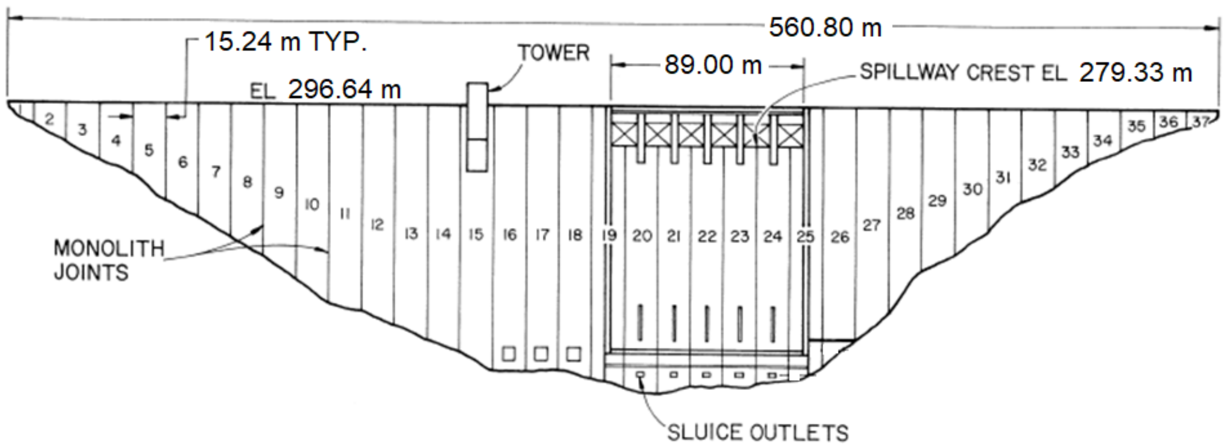


Figure 2 Downstream view of Pine Flat Dam [1]

The case for Theme A includes an analysis of the tallest non-overflow dam monolith, no. 16 (Figure 3). The case is selected because of its relatively simple geometry and because it was extensively studied in the 1970's and 1980's at the University of California at Berkeley [1 - 6] and during the 2018 USSD Workshop in Miami [7]. These past studies provide measured and calculated responses for correlation and comparison. The model geometry for Theme A will be the same as was used in the 2018 studies. The same elastic material properties for the dam, foundation, and reservoir also will be assumed. However, new seismic loads and non-linear material properties are defined for the Theme A formulation.

The Formulators have developed a list of cases for Contributors to perform comparative studies. The cases are marked as “obligatory” (to be studied by all Contributors) and “optional” (to be selected by any Contributor). The obligatory and optional cases provide all Contributors the ability to analyze cases with numerous modeling and parameter assumptions to contribute to the workshop results.

2.1. Roadmap

In the analyses defined for Theme A, the same model geometry, material properties and basic loads are intended to be used by all Contributors. To accomplish the goal of the workshop, Formulators are encouraging the Contributors to study the results of the Miami workshop (included in the report [7]) for verification purpose, conduct their analyses, and then submit their results.

The following are the formulated case studies:

- **Case A – EMVG Test Simulation**

  Simulation of the eccentric-mass vibration generator (EMVG) performed at Pine Flat Dam in 1971 is considered for Case A.

- **Case B – Foundation Analysis using Impulsive Loads**
  Investigate the effect of foundation size and analyze the efficiency of non-reflecting boundary conditions in the dynamic analysis of dams. Analyses will use the Impulsive Stress Records.

- **Case C – Dynamic Analysis using Impulsive Loads**
  Case B, which considers only the foundation, is extended to include the dam and reservoir. Analyses will use the Impulsive Stress Records.

- **Case D – Dynamic Analysis for Various Reservoir Levels**
  Investigate the effect of water levels. The dam-reservoir-foundation model defined in Case C is analyzed for various reservoir water levels and Taft Record.

- **Case E – Non-linear Dynamic Analysis**
  Investigate the effect of nonlinear behavior of the dam. The dam-reservoir-foundation model defined for Case D is extended to include nonlinear properties for concrete. Analyses will use the Taft Record and the ETAF Record.

- **Case F – Massless Foundation**
  Investigate the effect of using a massless foundation. The dam-reservoir-foundation model defined for Case D is modified to use a massless foundation. Analyses will use the Taft Record.

3. SCHEDULE

The schedule of milestones, leading up to the workshop is shown in Table 1. The schedule is intended to provide the participants a reasonable length of time to complete the analysis work and reporting, and then allow the Formulators to assemble the combined results for presentation at the workshop.

Table 1 Schedule

Milestone	Date
Model information and handouts of input data for the Benchmark available to Contributors	February 8, 2019
Deadline for results and paper submission by Contributors	May 31, 2019
Contributors submitting workshop presentations	September 1, 2019
Draft summary of the results will be provided to Contributors	August 25, 2019
Workshop presentations	September 9, 2019

4. DELIVERABLES

4.1. Analysis Results

Submission of analysis results must be done using the Analysis Results spreadsheets provided by the Formulators. Results are to be reported in International System Units (SI units), as shown in the template file to be provided by the Formulators, **(*Template)*_Theme A_Case XX_Results.xlsx**. Submission of result file should use the file format: **(Your Name)_Theme A_Case XX_Results.xlsx**.

Contributors should indicate if their name and their affiliations can be revealed during the workshop and the following workshop publications, or if their name should remain anonymous.

The Formulators will assess, evaluate and assemble the results for a general presentation at the workshop. Results will be presented in a logical order during the workshop identifying general findings and/or correlations within data sets. The summary of the results prepared by the Formulators will be available to the Contributors at least two weeks before the date of the workshop for verification and preparation of a constructive discussion during the workshop.

4.2. Paper

Contributors will prepare a summary paper documenting methods and approaches used in the study. The paper will summarize the results and will describe variations of results based on the parameters specified in the problem formulation, as well as optional parameters chosen by the Contributors for the investigation. In addition, the paper should include a discussion on what was learned from the analysis, general observations regarding the model parameters that had the largest impact to the results, and recommendations for either additional studies or research. The paper is to summarize the additional modeling recommended and analysis to be carried out based on the results. This could include how the participant would proceed with the next steps and specific issues that were encountered. The paper is limited to 12 pages. The paper should follow the format provided in the accompanying file, "(Template)**_Summary_Paper.docx**". Submission of summary paper should use the file format, **(Name)_Summary_Paper.docx**.

4.3. Workshop Presentation

During the Workshop session the Formulators will describe the case studies and present a summary of analysis results submitted by the Contributors.

The Formulators will ask Contributors to present their overall findings, interesting investigated aspects, unexpected results in the analysis, or issues to address in the future. Presentation of the case study analysis results should not be part of the Contributor's presentations. The length of presentations will be determined by the Formulators based on the number of participants.

The presentations will be followed by a discussion of all the workshop participants.

5. CASE STUDY INFORMATION

All the input data used in the formulation for Theme A, including dimensions and forces, are provided in the International System (SI) of units. For the consistency and convenience of the Contributors, the original dimensions and data defined form the Imperial Unit (US) system were converted by the Formulators. The Contributors can use any unit system in the analysis, however, all the analysis results should be submitted in the SI unit system. The units are: *kg, meters, N or MN, Pa or MPa, seconds*. A standard value for gravity of 9.80665 m/s^2 is assumed.

The general information provided in Section 5 is applicable to Cases A through F, as described in Section 6 through Section 11.

5.1. Geometry

5.1.1. Monolith

Monolith 16 is the tallest non-overflow section of Pine Flat Dam with the cross-section geometry shown in Figure 3.

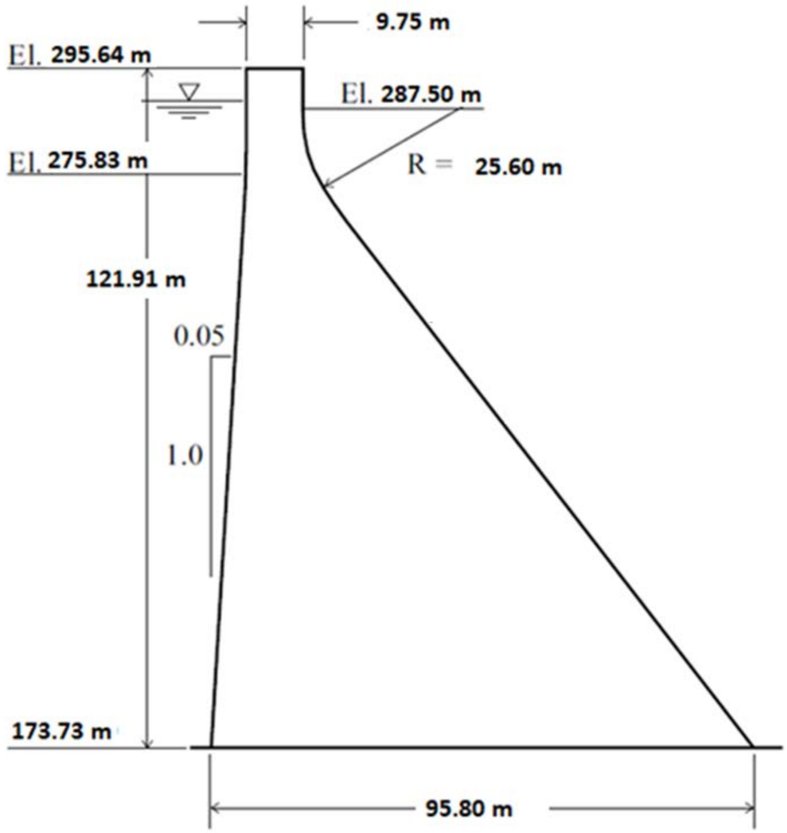


Figure 3 Cross Section Geometry of Monolith 16

5.1.2. Model

The model consists of the 15.24 m-wide dam monolith and a corresponding strip of the foundation. The origin of the axis system and key reference nodes are shown in Figure 4. The axis and reference nodes are located on the mid-width of the monolith. The axis system and nodes are to be used consistently for reporting results of the analysis cases.

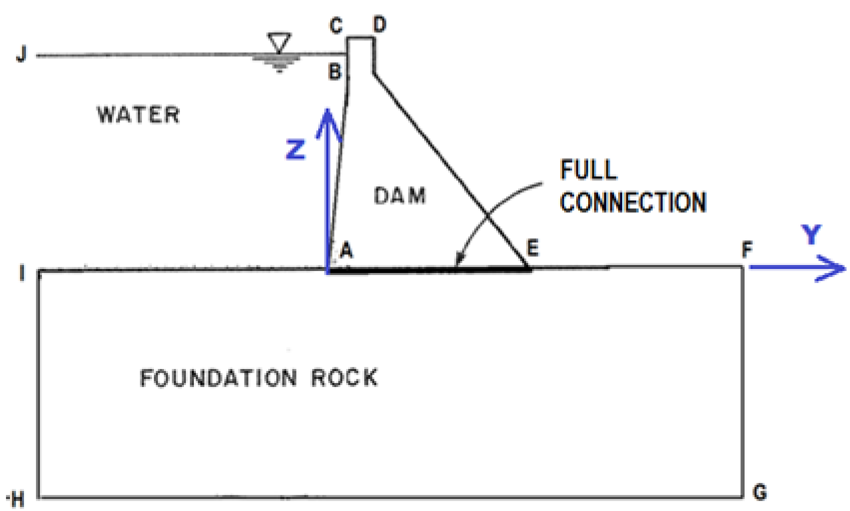


Figure 4 Model cross section

The Contributors may consider the symmetry of the model to conduct analysis. Justification for the use of a “half” model and documentation of the modeling are to be included in reporting.

5.1.3. Model Base Configuration

A “*base configuration*” of the model is defined here as:

- Dam dimensions shown in Figure 3
- Foundation dimensions in Figure 4 are:
    - Length: H-G = 700 m
    - Depth: I-H = 122 m
    - Dam heel location: I-A = 305 m
- Reservoir water level at 268.21 m, recorded at Pine Flat Dam in winter 1971.
- Elastic properties for concrete in Table 2 and for foundation in Table 3.

5.2. Material Properties

The material properties are provided for consistency and they correspond to the values used in the studies referenced in Section 13.

5.2.1. Dam

Concrete is assumed to be homogeneous and isotropic throughout the entire dam, with the properties listed in Table 2.

Table 2 Concrete Properties

Parameter	SI Units
Modulus of Elasticity	22 410 MPa
Density	2 483 kg/m^3
Poisson’s Ratio	0.20
Compressive Strength	28.0 MPa
Tensile Strength	2.0 MPa
Fracture energy	250 N/m
Compressive strain at peak load	0.0025
Tensile strain at peak load	0.00012

5.2.2. Foundation

The foundation material is assumed to be homogeneous, isotropic, and elastic with the properties listed in Table 3.

Table 3 Foundation Rock Properties

Parameter	SI Units
Modulus of Elasticity	22 410 MPa
Density	2 483 kg/m^3
Poisson Ratio	0.20
Shear Wave Velocity	1 939 m/s
Compressional Wave Velocity	3 167 m/s

5.2.3. Water

Water is considered to have a unit weight of 1 000 kg/m^3 and compression wave velocity of 1439 m/sec.

5.3. Loads

The following loads are used for various analysis cases.

5.3.1. Static Loads

Static load includes weight of concrete dam and the reservoir only (weight of foundation block should not be included).

5.3.2. Reservoir Levels

Three reservoir water levels are considered:

- Winter reservoir water level (WRWL) at El. 268.21 m
- Summer reservoir water level (SRWL) at El. 278.57 m
- Normal reservoir level (NRWL) at El. 290.00 m

5.3.3. Impulsive Stress Record

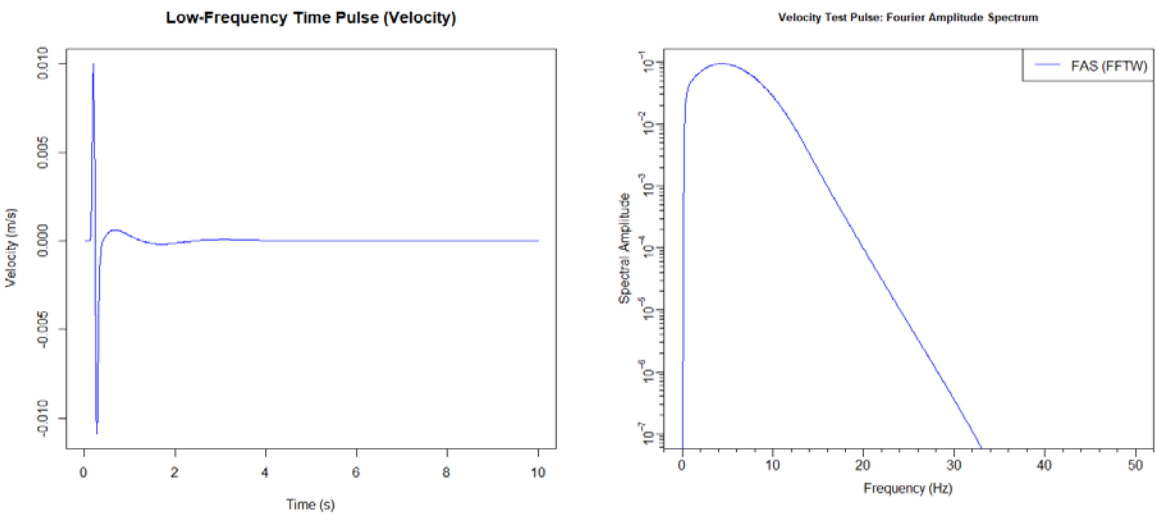
An impulsive excitation allows for easy visualization of the response of the model, and aids comparison between Contributor results. Artifacts and errors can be identified by inspection of the model output, or by direct comparison to the input. An impulsive time-history representing velocity in the horizontal (Y axis) direction at the free surface has been generated for the case study. As described at the end of this subsection, the free-surface velocity time-history is scaled to a shear stress to be applied at the base of the foundation. The impulsive time-histories are baseline corrected to minimize drifts after the excitation has been applied.

Table 4 Foundation Dimensions and Impulse Frequency Limits

Parameter	SI Units
Foundation Depth H	122.0 m
Minimum Element Size h	1.5 m
S-Wave Minimum Frequency f_0^S	4.0 Hz
S-Wave Maximum Frequency f_1^S	130 Hz

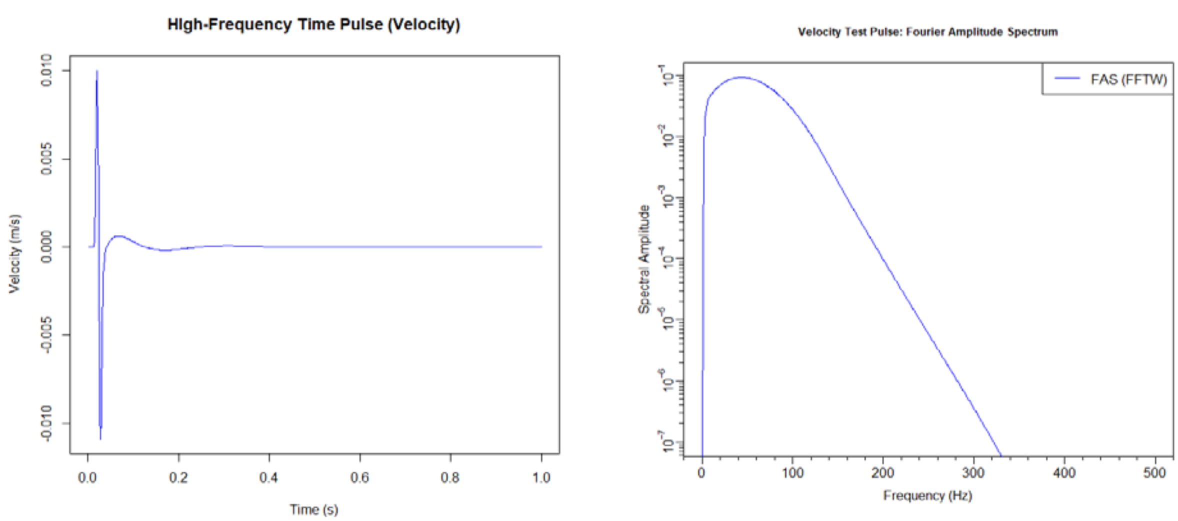
The frequency content of the Impulsive Stress Record is determined by the depth, minimum element size, and material properties of the foundation. Characteristic frequencies and bandwidth of the impulse have been selected to aid visualization and avoid inaccuracies as follows. Let H be the depth of the foundation (Section 5.1.3). Assuming the quarter-wavelength approximation, the lowest frequency f_0 for an elastic plane shear-wave (S-wave) that can resolve H is given by $f_0 = \frac{v_S}{4H}$, where v_S is the shear-wave velocity. The characteristic frequency f_N of the Impulsive Stress Record is chosen so that $N \geq 1/4$ impulse wavelengths will fit within H, which leads to the condition $f_N = 4N f_0$. We use values of $N = \frac{1}{4}$ and $N = 2.5$. To avoid inaccuracies, the highest frequency f_1 of the impulse is selected so that the wavelength is greater than 10 times the smallest spatial element size h, leading to the condition $f_1 \leq \frac{v_s}{10h}$. In practice, a corner frequency $f_{hi} < f_1$ is selected to low-pass filter the impulse so that its spectral amplitude at f_1 is substantially diminished from its peak value near the characteristic frequency f_N. Table 4 lists the resulting frequency limits, which are based on the foundation rock properties listed in Table 3.

Two versions of the Impulsive Stress Record are provided for the benchmark: (1) a low-frequency impulse such that the characteristic frequency corresponds to the quarter-wavelength approximation for the foundation ($N = \frac{1}{4}$); and, (2) a high-frequency impulse such that several wavelengths of the impulse are contained within the foundation depth ($N = 2.5$). The time increment of the low and high-frequency time histories are 0.01 s and 0.001 s, respectively, and the positive amplitude is set to 0.01 m/s. Both records are 2000 points long. The velocity time histories are shown in Figure 5 and Figure 6 (trimmed here to 10 and 1 seconds, respectively), as well as their corresponding Fourier amplitude spectrum. There is a 10-point quiet period before each impulse, where the velocity is set to zero.



δt (s)	$F_{Nyquist}$ (Hz)	F_N (Hz)	F_{lo} (Hz)	F_{hi} (Hz)	Impulse Amp. (m/s)	Impulse Dur. (s)	Zero-pad
0.01	50	4.0	0.5	8.0	0.01	20	10

Figure 5 Low-frequency impulsive time history: free-surface velocity – Upstream/Downstream direction



δt (s)	$F_{Nyquist}$ (Hz)	F_N (Hz)	F_{lo} (Hz)	F_{hi} (Hz)	Impulse Amp. (m/s)	Impulse Dur. (s)	Zero-pad
0.001	500	40	5	80	0.01	2.0	10

Figure 6 High-frequency impulsive time history: free-surface velocity – Upstream/Downstream direction

Incident plane S-waves can be represented as a stress time-history using a non-reflecting base boundary condition initially formulated by Lysmer and Kuhlemeyer [8]. The input stress time history can be obtained by scaling the free-surface velocity time-history for a medium having the same material properties as the foundation. We assume the case of a vertically-propagating plane shear wave polarized in the horizontal direction (SH-wave). Let $v_Y(t)$ be the velocity time-history representing S-wave particle motion in the Y-direction at the ground surface. The velocity time-history at the ground surface can be converted into a time-history of shear stress $\tau_{YZ}(t)$ at the base of the foundation by using $\tau_{YZ}(t) = \rho v_S v_Y(t)$, where ρ is the foundation density, and v_S is the shear-wave velocity (e.g., Joyner and Chen, [9]; Mejia and Dawson, [10]). In this benchmark, we will consider only vertically-propagating SH-waves, with the shear-stress input to the base boundary using the non-reflecting boundary condition.

The Impulsive Stress Record is to be applied in the upstream/downstream direction (Y-direction) at the base of the foundation. The velocity and corresponding shear-stress time series are provided in the accompanying file, "**Impulse_TimeSignal.xlsx**", with one tab for the low-frequency impulse, and another for the high-frequency impulse. To ensure that the excitations remain baseline-corrected, the full duration of the time series (20 seconds for the low-frequency impulse, 2 seconds for the high-frequency impulse) should be input (i.e., don't truncate the record in time).

5.3.4. Taft Record

The M 7.3 Kern County, California, earthquake occurred on July 21, 1952. Accelerations from this earthquake were recorded at the Lincoln School (tunnel), in Taft, California, and are used for this benchmark. In particular, we use the baseline-corrected S69E component (Figure 7), which has a peak horizontal acceleration of 0.18 g. The vertical record is not used.

The recorded Taft acceleration time history is assumed to represent ground motions at the free surface, i.e., the motions that would be recorded at the top of the foundation in the absence of the dam and reservoir. Because free-surface motions are appropriate for input to models with a massless foundation, this acceleration record will be used for input to Case F.

In contrast, free-surface motions typically are deconvolved for input to models using a foundation with mass. The Taft free-surface acceleration record therefore has been deconvolved to the base of the foundation. The deconvolution process assumes a vertically-propagating SH-wave in a uniform half-space having the same material properties as the foundation (Table 3). It is computed for a depth equivalent to the base of the foundation and includes the Rayleigh mass and stiffness damping specified in Section 5.4. Figure 8 shows a comparison of the free-surface and deconvolved Taft acceleration records. The deconvolved Taft acceleration record will be used for input at the base of the foundation for Cases D and E. For Contributors who prefer instead to use the stress input method for Cases D and E, an equivalent shear stress record is provided, which also should be applied at the base of the foundation. The shear stress record was computed by multiplying the free-surface Taft velocity time history (Figure 7) by the foundation density and shear-wave velocity, as described in Section 5.3.3.

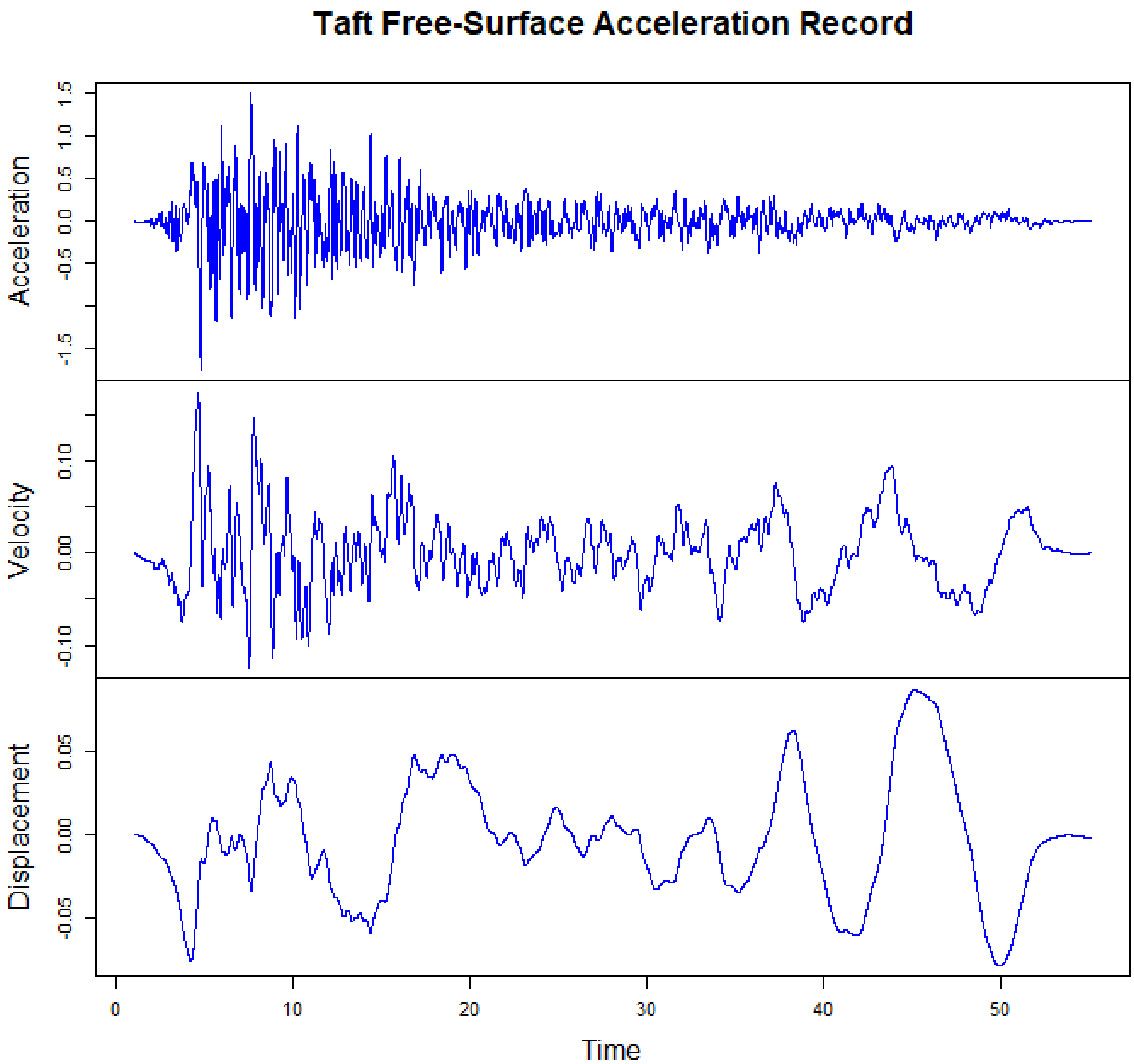


Figure 7 **Baseline corrected Taft Free-surface Acceleration Record – Upstream/Downstream Direction (SI units)**

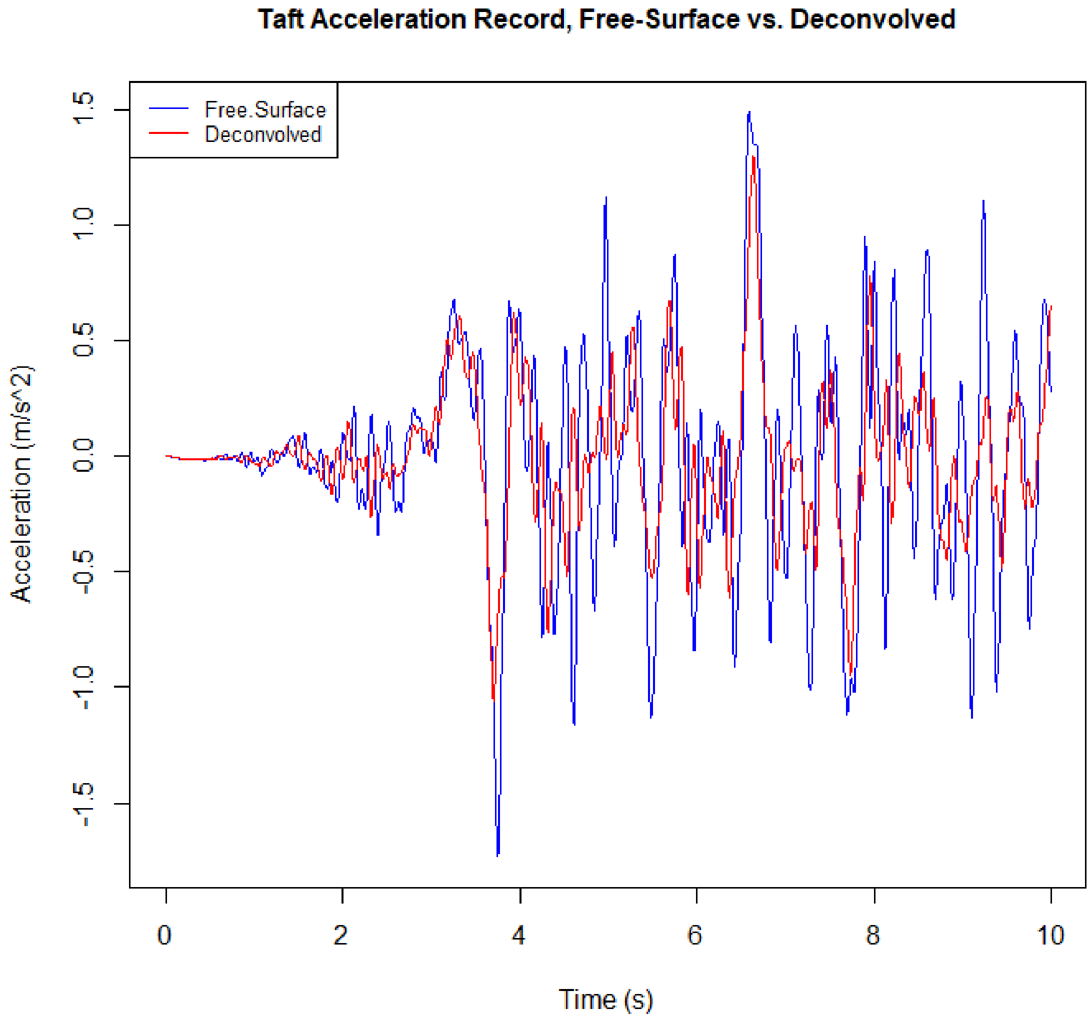


Figure 8 **Comparison of Taft Free-Surface and Deconvolved Acceleration Records for 0-10 second range – Upstream/Downstream Direction (SI units)**

The Taft Record is to be applied in the upstream/downstream direction (Y-direction). Various time histories representing the Taft Record are provided in the accompanying file "**Taft_TimeSignal.xlsx**". The free-surface acceleration time history should be used for input to Case F, while either the deconvolved acceleration time history or the shear-stress time history should be used for input to Cases D and E. The free-surface velocity and displacement time histories are provided for purposes of comparison and are not used for input in this benchmark.

5.3.5. Endurance Time Acceleration Function Record

Endurance Time Analysis (ETA) is a dynamic pushover procedure which estimates the seismic performance of the dam when subjected to a pre-designed intensifying excitation. The simulated acceleration functions are aimed to shake the dam from a low excitation level - with a response in the elastic range - to a medium excitation level - where the dam experiences some nonlinearity - and finally to a high excitation level, which causes the failure. All these response variations can be observed though a single time history analysis.

The Endurance Time Acceleration Function (ETAF) is an artificially designed intensifying acceleration time history, where the response spectra of the ETAF linearly increases with time. Ideally, the profile of the acceleration time history and response spectrum increase linearly with time. Figure 9 shows a sample ETAF and its response spectra at three different times (i.e. 5, 10, and 15 sec). As seen, the spectrum at t = 5 sec is nearly one half of the one at t = 10 sec and 1/3 of the one at t = 15 sec. In this technique, the seismic performance is determined by the duration the structure can endure the dynamic input.

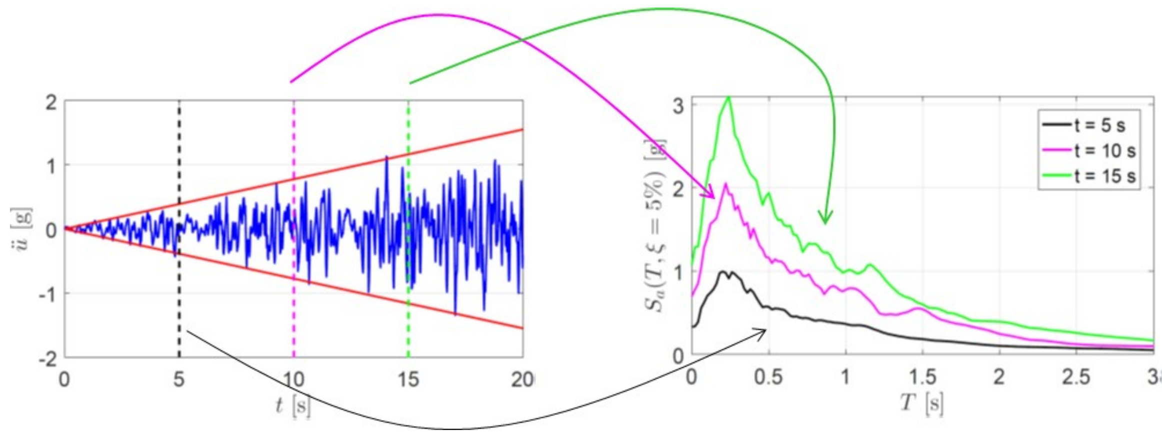


Figure 9 Sample ETAF, its acceleration profile, and time-dependent response spectra

ETA procedure is identical to a conventional acceleration time history analysis except that ETAFs are used as input for the numerical model instead of the real ground motions.

Figure 10 (left plot) shows the engineering demand parameter (EDP) – e.g. crest displacement - in terms of the time. Next, the absolute value of this time history is computed (red curve in right plot), and its cumulative absolute value is determined (black curve in right plot) versus time.

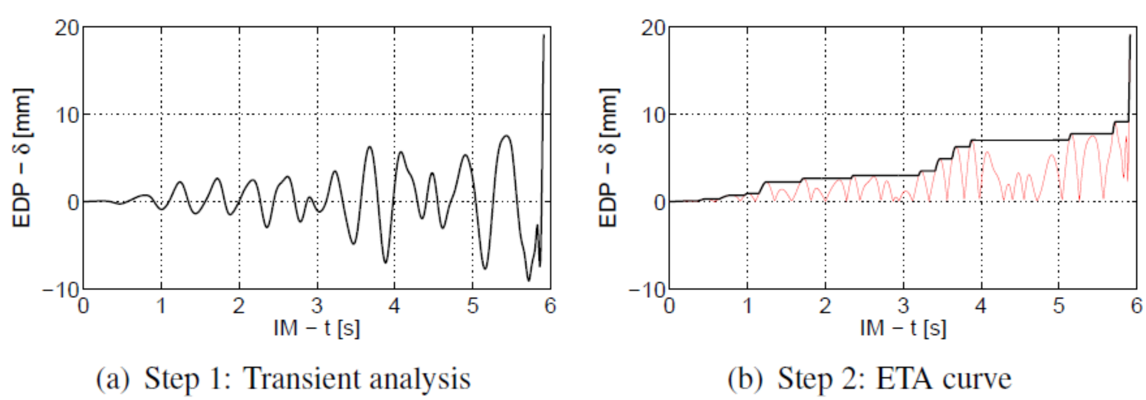


Figure 10 Step-by-step procedure to perform ETA and interpret the results

The ETAF Record is shown in Figure 11. The acceleration time history is to be applied in the upstream/downstream (Y-direction) at the base of the foundation. The acceleration, velocity, and displacement time histories are provided in the accompanying file, "**ETAF_TimeSignal.xlsx**", though only the acceleration time history should be used for input.

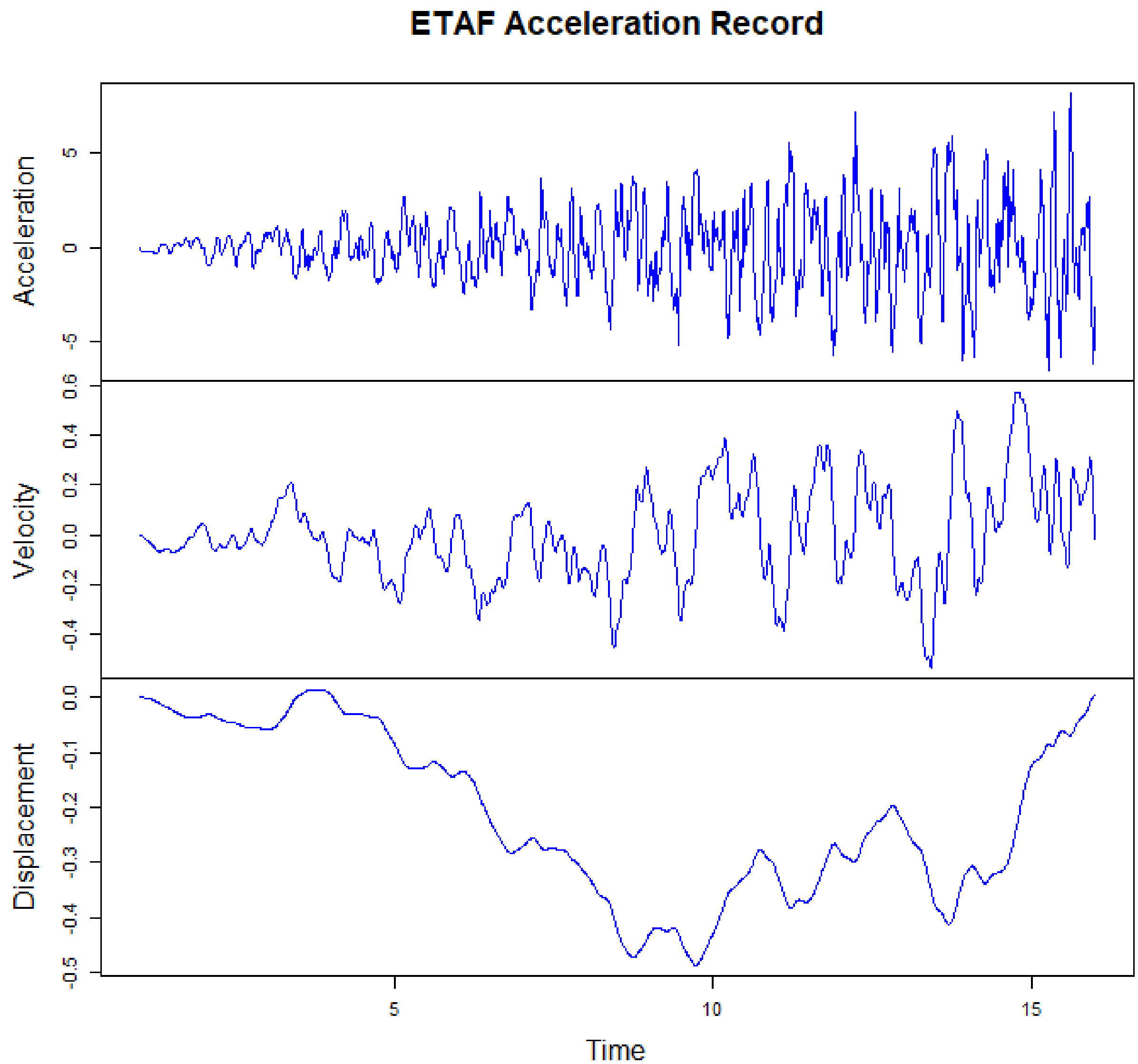


Figure 11 ETAF Record – Upstream/Downstream Direction (SI units)

5.3.6. EMVG Time History

Loading of an eccentric-mass vibration generator is simulated by a harmonic-force time history, which is to be applied at the crest of 15.24-m wide Monolith 16 as a line load in the middle of the crest width. The sine wave has an amplitude of 35.4 kN and a frequency of 3.47 Hz, corresponding to the fundamental frequency of Pine Flat dam determined during the Winter 1971 test [1]. To reduce artifacts and better emulate the mass vibration test results, the sine wave is windowed with a 25-percent Hann taper (Figure 12) rather than a boxcar.

The EMVG time history is provided in the accompanying file, “**EMVG_TimeSignal.xlsx**”.

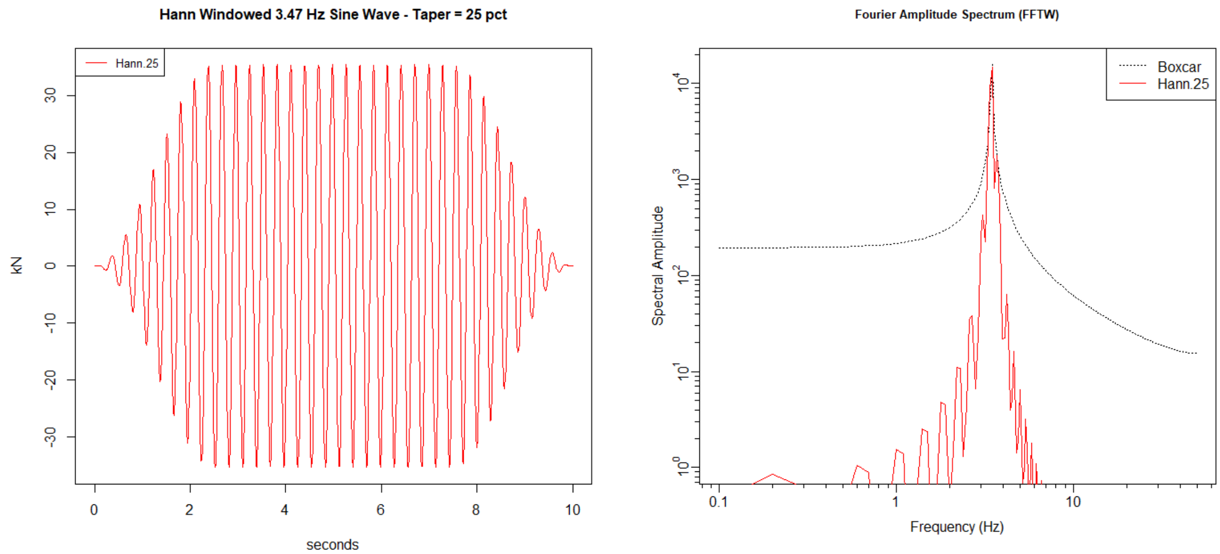


Figure 12 EMVG Time History, and corresponding Fourier amplitude spectrum

5.4. Damping

In common practice, the term viscous damping is used to describe the parameter employed to stabilize a numerical model to determine the natural mode shapes and frequencies of the structure. Based on the application of viscous damping however, the term can be characterized in several ways. Participants should use viscous damping based on their current understanding, modeling practice and method of analysis chosen, and document the usage and definition in the final submitted report.

For consideration, participants may consider using Rayleigh viscous damping (Rayleigh damping). Rayleigh damping matrix, by definition, is proportional to a linear combination of mass matrix (M) and stiffness matrix (K), through the constants of proportionality, denoted as α and β in the equation (Eq. 1):

$$C = \alpha M + \beta K \qquad \text{(Eq. 1)}$$

In reference to the values in Figure 13 for α and β, the value of Rayleigh damping can vary depending on the frequency, and matrix constants. For the purpose of the workshop analysis, a frequency between 4 and 10 Hz should be considered, depending on the combination of dam, reservoir and foundation. For this frequency range, the average Rayleigh damping equates to approximately 2 percent, using the input matrix constants as shown below. The constants of proportionality should be defined as α = 0.75 1/sec and β= 0.0005 sec.

Damping parameters

Beta = 0.000500 (stiffness)

Alpha = 0.750000 (mass)

Frequency	Viscous Damping	Stiffness Damping	Mass Damping
1.00	6.13	0.16	5.97
2.00	3.30	0.31	2.98
3.00	2.46	0.47	1.99
4.00	2.12	0.63	1.49
5.00	1.98	0.79	1.19
6.00	1.94	0.94	0.99
7.00	1.95	1.10	0.85
8.00	2.00	1.26	0.75
9.00	2.08	1.41	0.66
10.00	2.17	1.57	0.60
11.00	2.27	1.73	0.54
12.00	2.38	1.88	0.50
13.00	2.50	2.04	0.46
14.00	2.63	2.20	0.43
15.00	2.75	2.36	0.40
16.00	2.89	2.51	0.37
17.00	3.02	2.67	0.35
18.00	3.16	2.83	0.33
19.00	3.30	2.98	0.31
20.00	3.44	3.14	0.30

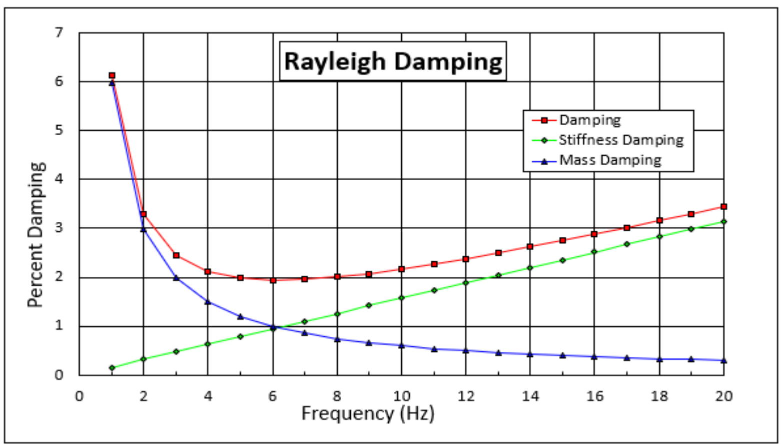


Figure 13 Damping

Depending on the method of analysis chosen by the participants and incorporation of viscous damping, the overall effects to dynamic response could vary among the analyses. Contributors are encouraged to establish a clear understanding of how damping is applied in the analyses and the parameters that are defined within the term. Behavior of the structural system should be verified with fundamental principles, to ensure that the applied damping is reasonable.

5.5. Sign Convention

For the purposes of this study and consistency among results, the sign convention for tension shall be "+", or positive, and the sign convention for compression shall be "-", or negative.

6. CASE A – EMVG TEST SIMULATION

6.1. Formulation

In Case A, a dynamic linear analysis of the dam-foundation-reservoir system for the harmonic force record exerted by an eccentric-mass vibration generator positioned at the dam crest is considered. The analysis corresponds to the tests conducted on Monolith 16 in 1971 [1].

6.2. Configuration

The "base configuration" is considered for Case A with the EMVG excitation applied at the crest of the dam.

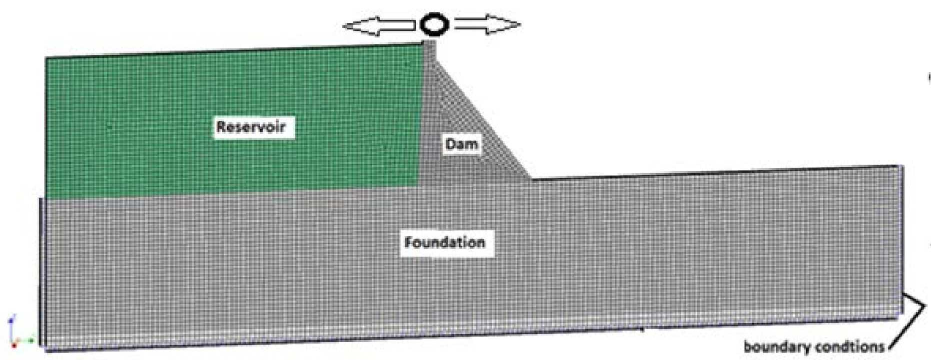


Figure 14 Model for EMVG test simulation

Boundary conditions are to be defined and justified by the participant. However, plane strain boundary conditions at the side faces of the model should be applied. At the bottom, upstream and downstream faces of the foundation, appropriate conditions for a dynamic analysis (far-field/non-reflecting) should be selected by Contributors.

6.3. Input Parameters

For Case A, elastic material properties for concrete and rock are considered. Based on the provided information, the Contributors should determine the response of Monolith 16 for the specified loads.

For analysis consistencies, please consider the following:

- Concrete elastic properties in Table 2
- Water properties are defined in Section 5.2.3
- Foundation elastic properties in Table 3
- The mass of the dam, reservoir and foundation
- Do not consider the static weight of the foundation
- The static weight of the dam and reservoir

- 2% viscous damping for the dam and foundation. Reference Section 5.4 for additional consideration.

6.4. Loads

Loads include:

- EMVG harmonic-force time history record applied at the middle of the dam crest in the upstream /downstream direction. The load should be applied by the Contributors as shown in Figure 13 and provided in the accompanying file, "**EMVG_TimeSignal.xlsx**".

6.5. Analysis

Perform linear dynamic analyses for two water levels:

- **A-1**: Natural frequencies for WRWL at El. 268.21 m
- **A-2**: Natural frequencies for SRWL at El. 278.57 m
- **A-3**: Dam - foundation - reservoir system with WRWL at El. 268.21 m
- **A-4**: Dam - foundation - reservoir system with SRWL at El. 278.57 m

6.6. Results

Present the following results in the Analysis Results spreadsheet provided.

a. The 6 first natural frequencies of the model and their normalized mode shape
b. Displacement time history of the upstream nodes at the dam crest [C] and the dam heel [A]
c. Acceleration time history of the upstream nodes at the dam crest [C] and the dam heel [A]

7. CASE B – FOUNDATION ANALYSIS USING IMPULSIVE LOADS

7.1. Formulation

The purpose of this study is to investigate the effect of foundation size and to verify the efficiency of the non-reflecting boundary conditions in the dynamic analysis of dam foundations. A foundation block is a reduced-domain model for a uniform half-space. To evaluate how well the model performs, this case explores the response to a vertically-propagating SH wave by using an impulsive signal for the free-surface velocity. The corresponding shear-stress time history is provided for input to the base of the foundation. Note: This shear stress input should reproduce the assumed free-surface velocity time history at the top of the model if it provides a good representation of the semi-infinite domain (i.e., the uniform half-space).

7.2. Configuration

For a simple 122-m deep foundation block, as shown in Figure 15, two configurations are considered with the block length of 700 m and 3700 m. Plane strain or 3-D analysis could be considered. The side boundaries should be represented by far field or non-reflecting boundary conditions, while the base boundary should be represented by a non-reflecting boundary.

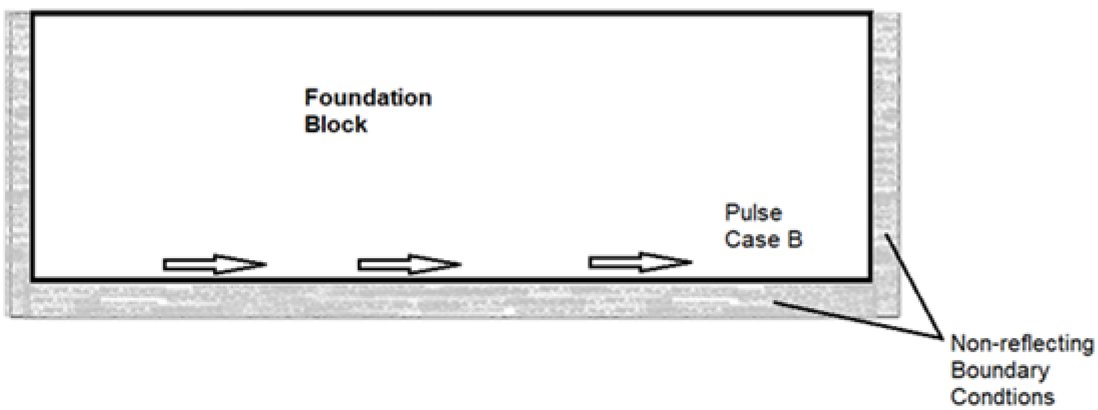


Figure 15 Configuration for the "free-field" model

7.3. Input Parameters

- A set of elastic material properties is defined in Table 3
- Mass type foundation should be considered in the analysis.
- This is a wave propagation analysis only. Static analysis due to weight of foundation material (gravity loads) should not be included in the analysis.
- A boundary condition similar to this used for Case A should be implemented for the Case B study.
- ~~2%~~ zero viscous damping should be used for the models.
- The maximum element size should be 1.5 m.

7.4. Loads

The stress input time history is applied at the base boundary of the foundation block. It represents the shear stress from a vertically-propagating plane SH wave.

The excitation is defined by the horizontal (Y axis) Impulsive Stress Records shown in Figure 5 and Figure 6. Stress records should be applied uniformly at the base of the foundation block as shown in Figure 15. The baseline corrected impulsive time histories are provided in the accompanying file, "**Impulsive_TimeSignal.xlsx**".

7.5. Analysis

Model configuration and analysis type combinations are shown in Table 5. The analysis time should be 2-second long for the high frequency impulse and 20-second long for the low frequency impulse.

Table 5 Case B Analysis

Models	Length I-F	Impulsive Signal
B-1	700 m	High Frequency
B-2	700 m	Low Frequency
B-3	3 700 m	High Frequency
B-4	3 700 m	Low Frequency

7.6. Results

Participants should report plots of velocities response histories at several locations identified in sheet Case B of the Results Excel File provided by the Formulators.

8. Case C - Dynamic Analysis using Impulsive Loads

8.1. Formulation

The analysis type for Case C is related to the one conducted for Case B-1 and B-2. Here, the dam and the reservoir are considered together with the foundation model. The purpose of the study for Case C is to investigate the effect of the dam and reservoir presence on the wave propagation in the foundation and to compare the analysis results with the free field motions studied in Case B.

8.2. Configuration

In the Case C analysis, the foundation block is 700 m long and 122 m deep. Two model configurations are considered.

- **C-1 & C-2** – Dam, reservoir and foundation - “Base configuration” as defined in Section 5.1.3
- **C-3 & C-4** – Dam and foundation (no reservoir)

8.3. Input Parameters

For analysis consistencies, please consider the following:

- Concrete elastic properties in Table 2
- Water properties are defined in Section 5.2.3
- Foundation elastic properties in Table 3
- The mass of the dam, reservoir and foundation
- Do not consider the static weight of the foundation, dam, nor reservoir
- ~~2%~~ zero viscous damping for the dam and foundation. Reference Section 5.4 for additional consideration.
- The maximum element size should be 1.5 m.

8.4. Loads

The following should be considered in applying the loads:

- The Impulsive Stress Records are applied at the base of the foundation as in Case B (do not consider gravity loads for the dam-reservoir-foundation system)
- The excitation is defined by the horizontal (Y axis) input stress time histories shown in Figure 5 and Figure 6. Stresses should be applied uniformly at the base of the block as shown in Figure 15. The baseline corrected impulsive stress and velocity time histories are provided in the accompanying file, "**Impulsive_TimeSignal.xlsx**"
- RWS at El. 268.21 m

8.5. Analysis

Model configuration and analysis type combinations are shown in Table 6. The analysis time should be 2-second long for high frequency and 20-second long for low frequency impulse.

Table 6 Case C Analysis

Models	***Model***	**Impulse Signal**
C-1	**Dam-Reservoir-Foundation**	High Frequency
C-2	**Dam-Reservoir-Foundation**	Low Frequency
C-3	**Dam-Foundation**	High Frequency
C-4	**Dam-Foundation**	Low Frequency

8.6. Results

Report plots of ground surface velocities response histories at several locations identified in sheet Case C of the Results Excel File provided by the Formulators.

9. CASE D – DYNAMIC ANALYSIS FOR VARIOUS RESERVOIR LEVELS

9.1. Formulation

In Case D, a dynamic analysis of the dam-foundation-reservoir system is performed considering the elastic material properties, the Taft earthquake record, and the reservoir water at three different elevations. The intent is to evaluate the dam response due to various reservoir levels.

9.2. Configuration

The “base configuration” of the model, as defined in Section 5.1.3 is considered for Case D with three reservoir water levels.

9.3. Input Parameters

For analysis consistencies, please consider the following:

- Concrete elastic properties in Table 2
- Water properties are defined in Section 5.2.3
- Foundation elastic material properties in Table 3
- The mass of the dam, reservoir and foundation
- Static weight of the dam and reservoir but do not consider weight of the foundation
- 2% viscous damping for the dam and foundation. Reference Section 5.4 for additional consideration.

9.4. Loads

The loads include:

- Taft Record applied in the Upstream/Downstream direction, using one of the following options:
    - Option 1: apply the Taft deconvolved acceleration time history at the base of the foundation block for three different reservoir water levels.

- Option 2: apply the Taft stress time history at the base of the foundation block for three different reservoir water levels

- Gravity loads should be applied to the dam, reservoir but not to the foundation in combination with Taft Record.

9.5. Analysis

Perform a linear dynamic analysis for the dam/reservoir/foundation system for various reservoir water levels including:

- **D-1**: WRWL at El. 268.21 m
- **D-2**: SRWL at El. 278.57 m
- **D-3**: NRWL at El. 290.00 m

9.6. Results

Report the analysis results in sheet Case D of the Results Excel File provided by the Formulators including:

a. Crest [Point C] and heel [Point A] total displacement (static and dynamic) time history in the upstream direction.
b. Hydrodynamic pressure (dynamic component only) time history at the dam heel [Point A in Figure 4].
c. Crest [Point C] and heel [Point A] acceleration time history in the upstream direction.

10. Case E – Non-linear Dynamic Analysis

10.1. Formulation

The intent of Case E is to perform a dynamic analysis with concrete non-linear material properties. The dam-foundation-reservoir system used in Case D will be analyzed except the non-linear material properties of concrete are considered.

10.2. Configuration

The “base configuration”, as defined in Section 5.1.3 and used in Case D, is considered for Case E with assumed non-linear material properties for concrete. Two different dynamic loads are applied at the base of the foundation block: the Taft Record (deconvolved acceleration or shear-stress) and the ETAF Record.

10.3. Input Parameters

For Case E, typical in engineering practice material properties for concrete (Table 2) and foundation (Table 3) are provided. The Contributors should select a non-linear material constitutive model for concrete and define all the parameters needed for the material model based on the provided information.

For analysis consistencies, please consider the following:

- All the parameters for the concrete non-linear model should be defined by Contributors based on data provided in Table 2. Any material model parameters not provided in this table, should be determined by the Contributors based on the current practice and engineering judgment.
- Water properties are defined in Section 5.2.3
- Foundation elastic properties in Table 3
- The mass of the dam, reservoir and foundation
- Do not consider the static weight of the foundation.
- The static weight of the dam and reservoir
- 2% viscous damping for the dam and foundation. Reference Section 5.4 for additional consideration.

10.4. Loads

Loads include:

- Taft Record (time history for either deconvolved acceleration or shear-stress) applied similarly to Case D at the base of the foundation block.
- ETAF horizontal acceleration time history record at the base of the foundation block
- Gravity loads should be applied to the dam, reservoir but not to the foundation in combination with the Taft Record and ETAF Record.

10.5. Analysis

Perform a non-linear dynamic analysis for the dam-reservoir-foundation system using the WRWL at El. 268.21 m. The analysis for two seismic load cases should be performed including:

- **E-1**: Taft Record combined with the static load
- **E-2**: ETAF Record combined with the static load

10.6. Results

Report the analysis results in sheet Case E of the Results Excel File provided by the Formulators including:

a. Crest [Point C] and heel [Point A] total displacement (static and dynamic) time history in the upstream direction.

b. Hydrodynamic pressure (dynamic component only) time history at the dam heel [Point A in Figure 4].

c. Crest [Point C] and heel [Point A] acceleration time history in the upstream direction.

d. Provide the extent of damage in dam body. Damage in body is summation of individual (macro) damage to the dam elements. Depending on the constitutive model and the software used, different definitions for the "damage" might be sought. The participants might adopt the most representative definition and justify it.

    i. For Taft Record, only a scalar number at the end of simulation.

    ii. For ETAF Record, a time-dependent damage curve.

e. Provide the damage index (DI) along the dam-foundation interface (along line A-E). Damage index is computed as a ratio of the damaged length to the total dam base length.

    i. For Taft Record, only a scalar DI by the end of simulation.

    ii. For ETAF Record, a time-dependent DI curve (evolution of DI).

f. Provide the time - in seconds - (only for the ETAF simulation) in which the dam is failed. In numerical simulations, the failure usually corresponds to the large deformations; however, the participants may provide their own definition of the failure as well.

11. CASE F – MASSLESS FOUNDATION

11.1. Formulation

In Case F, the model of the dam-foundation-reservoir system is similar to Case D except a massless foundation subject to the free-surface acceleration Taft record is considered.

11.2. Configuration

The "base configuration" of the model, as defined in Section 5.1.3, is considered for Case F with a massless foundation.

11.3. Input Parameters

For analysis consistencies, please consider the following:

- Concrete elastic properties in Table 2
- Water properties are defined in Section 5.2.3
- Foundation elastic properties in Table 3
- The mass of the dam, reservoir but massless foundation
- Static weight of the dam and reservoir but do not consider the weight of foundation
- 2% viscous damping for the dam and foundation. Reference Section 5.4 for additional consideration.

11.4. Loads

The loads include:

- Taft Record (free-surface acceleration time history) applied to the base of the foundation block for three different reservoir water levels.
- Gravity loads should be applied to the dam and reservoir, but not to the foundation in combination with the Taft Record.

11.5. Analysis

Perform a linear dynamic analysis for the dam/reservoir/foundation system for various reservoir water levels including:

- **F-1**: Winter reservoir level at El. 268.21 m
- **F-2**: Summer reservoir level at El. 278.57 m
- **F-3**: NRWL at El. 290.00 m

11.6. Results

Report the analysis results in sheet Case F of the Results Excel File provided by the Formulators including:

a. Crest [Point C] and heel [Point A] total displacement (static and dynamic) time history in the upstream direction.

b. Hydrodynamic pressure (dynamic component only) time history at the dam heel [Point A in Figure 4].

c. Crest [Point C] and heel [Point A] acceleration time history in the upstream direction.

12. SUMMARY OF THE REQUIRED ANALYSES

The summary of the analyses required for the Theme A is reported in Table 7.

Table 7 List of the case-studies and analysis types

Case	Analysis Type			
	1	2	3	4
A	Obligatory	Obligatory	Obligatory	Obligatory
B	Optional	Optional	Optional	Optional
C	Optional	Optional	Optional	Optional
D	Obligatory	Obligatory	Optional	
E	Obligatory	Optional		
F	Optional	Optional	Optional	

12.1. Estimation of Effort

The estimated time need to complete the analysis and submit the results to the Formulators is:

- Obligatory part is 10-15 staff days
- Optional part is 15-20 staff days

Contributors should estimate additional time for writing the paper to be published in the workshop proceedings and to prepare the workshop presentation.

13. REFERENCES

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[3] Fenves G., Chopra A. K., - Simplified Analysis for Earthquake Resistance Design of Concrete Gravity Dams, Report No. UCB/EERC-85/10, June 1986

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