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RELIABILITY BASED ANALYSIS AND DESIGN OF ANCHOR RETROFITTED CONCRETE GRAVITY DAMS

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SUMMARY

This paper presents a comparison between level of safety provided by traditional allowable stress evaluation of monolithically built concrete gravity dams and a more recent Ultimate Limit States or Reliability Based evaluation of these gravity based structures subject to seismic and non-seismic loads. Level of safety resulting from a conventional seismic stability analysis of a typical concrete gravity block subject to normal static loads and pseudo static earthquake load is compared with the level of safety determined from reliability or ultimate limit states design principles. The tallest monolith of the Pine Flat Dam is used as an example case for calculation of traditional sliding factor of safety and reliability indices. The dam monolith with and without post tensioned anchors are considered. This paper demonstrates that while some measure of safety is ensured by following the conventional factor of safety approach in the stability analysis and subsequent remedial design, reliability based analysis and design of stabilized concrete gravity dams provides a consistent level of structural reliability in the stability analysis of such structures, in particular where remedial design is required.

INTRODUCTION

Recent publication of dam safety guidelines and research around the world reflect a growing concern on the seismic safety of hydroelectric and flood control gravity dams and other similar water retaining gravity structures built prior to 1950's. Although probable maximum flood (PMF) is a critical lateral load in the sliding stability of concrete gravity dams, by and large horizontal and vertical accelerations due to maximum credible earthquake (MCE) have become the main factor in the stability evaluation of concrete gravity dam structures.

A typical medium to large size concrete gravity dam comprises various service or relief structures that include a number of expansion joints. By controlling temperature ingredients, expansion joints are mainly provided to minimize cracking of concrete mass due to heat of hydration of concrete during and immediately after construction of dams. Inclusion of expansion joints results in the creation of several rigid concrete monoliths or blocks along the length of the dam. During a strong seismic event in the upstream-downstream direction (i.e. along the centerline of river), concrete blocks tend to slide at the base

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of concrete block or rock independently between the joints, thus resulting in a potential breach in the dam structure. A potential dam breach would result in endangering the public safety and also immense economic loss. To remedy this problem, post tensioned multi strand rock anchors are typically employed to increase the stability of portions or all of a hydroelectric or flood control dam structure. This paper presents reliability based principles applied in assessing the safety of gravity dams and design of post tensioned anchors where dam stabilization is required.

MECHANICS AND EFFECTS OF POST TENSIONED ANCHORS

In most dam rehabilitation projects, anchor installation to restore dam stability is deemed the least cost and most time effective option when compared against other options such as adding substantial volume of concrete to main body of dam or constructing alternative water passages to alleviate water pressure on the dam. A rock anchor is comprised of a group of high strength strands that are bundled together and designed to the required post tensioning or stabilizing force. Since multi strand anchors are the primary component in dam stabilization, a typical anchor is examined for its mechanical properties and behavior.

In general, the beneficial effects of rock anchors can be examined by considering the installation process and composition of rock anchors, which are as follows:

1) Drill anchor hole to the designed diameter and depth;
2) Water pressure test the drilled hole by filling up the hole to the top and observing whether water level lowers down or not. A drop in water level indicates the presence of fissures and cracks in foundation rock material, which as a remedy, drilled hole is grouted and later drilled to the designed diameter and depth. The water pressure test is again conducted and if the test fails again, the grout, drill and water pressure test process is repeated until water pressure test passes;
3) Place shop fabricated anchor in drilled hole. Place and fill stage 1 grout to the designed bond length which is within the rock foundation;
4) After stage 1 grout has reached the designed compressive strength, post tension the anchor to the designed stress level (e.g. $0.80f_{su}$) and lock off at anchor head.
5) Place and fill stage 2 grout by completely filling the anchor hole with grout and cap off the anchor head.

It should be noted that the main purpose of the grouting operation is to fill the drilled hole, without any voids or cavities, with a cement based material to act as a corrosion protection layer for the post tensioned steel strands. The grout material should also possess sufficient bond strength to transfer strand skin or bond stresses to the surrounding concrete or rock material. Cost permitting, strain gauges could be installed along the length of anchors to allow for short and long term monitoring of bonded post-tensioned anchors. The presence of the designed post tensioning force or any reduction in force due to possible long term relaxation in anchors can be verified by strain gauges.

In the past, structural engineers relied upon factors of safety in the design and analysis of structures, in particular hydroelectric and gravity dam structures. Although the use of common safety factors provided some assurance of safety in a particular design situation, it did not, among other factors, provide a consistent level of structural reliability in the design. The basic concept is straightforward: factor of safety of some quantity is simply the ratio of the allowable value, called *Capacity* (C), to the calculated value, called *Demand* (D), and is generally expressed as:

[1]
$$FOS = \frac{C}{D}$$

The widely used Working Stress Design method (WSD) uses allowable and calculated stresses from applied loads in a particular structural member or component, and ensures that equation [1] is greater than a predetermined value of the safety factor, FOS. In the past, and to some extent even at present, subjective judgment by engineers was used to select a proper factor of safety in design. Invariably, this approach did not explicitly take into consideration the variability in loads and material strength.

Principles of probability theory, in the form of reliability analysis have recently been utilized to minimize the high variability in safety that is inherent in the simple Working Stress Design method. The concept of reliability based design has now been adopted by most industrialized countries in the form of Limit States Design (LSD) or Load and Resistant Factor Design (LRFD). Ongoing research is being conducted to determine accurate load and resistance factors. Recent literature (Hasofer, Lind, 1974; Allen, 1975; Foschi, 1978; Sexsmith, Fox, 1978; Sexsmith, 1979; Whitman, 1984; Foschi, Folz, Yao, 1989 and 1993; etc.), is briefly reviewed here.

In general, a structural design problem involves the interaction of several random variables which mainly includes the geometric and material properties of the structure, and can be mathematically formulated as a vector of basic random variables, in the form of:

[2] $$X = \{X_1, X_2, X_3, ..., X_n\}$$

The limiting state and performance of the structure, in terms of $\mathbf{X}$, may then be described by a *performance function* in the form of:

[3] $$G\{X\} = \{X_1, X_2, X_3, ..., X_n\}$$

To differentiate between Capacity and Demand random variables, as it is the usual case in structural design, equation [3] can be re-written as:

[4] $$G = C - D$$

Therefore, failure of structure is imminent when $G < 0$ (i.e. demand is greater than capacity), and conversely, survival of the structure is indicated by $G > 0$. Then, the boundary between failure and survival, $G = 0$, can be considered as a *limit state* surface between all the variables involved in the performance function [3]. From a structural design point of view, a limit state is defined by various states of collapse and un-serviceability that are to be avoided (Allen, 1974). *Ultimate limit states* relate to the safety and integrity of the structure, and relate to the load carrying capacity of the entire structure or part of it. Material fracture, structural instability and excessive crushing are some typical cases that are encountered in structural design. On the other hand, *serviceability limit states* correspond to those cases that relate to the level of comfort in a structure under specified loads. Common examples are floor vibration, excessive floor deflection and cracking. The work presented herein relate to sliding movement of concrete gravity blocks which ultimately could lead to dam breach and as such governed by ultimate limit states principles.

In general, an entity's reliability, by definition, is the probability that it will remain fully functional throughout its design life span. For a structural entity, reliability can be viewed as the complement of the probability of failure, P_f, which is given by the multiple integral:

[5] $$P_f = \int_{\Omega_{Failure}} f_{1,2,3,\ldots,n}(X_1, X_2, X_3, \ldots, X_n)\,dX_1\,dX_2\,dX_3 \ldots dX_n$$

where the failure domain, $\Omega_{Failure}$, is expressed as $G\{\mathbf{X}\} \le 0$, and the integrand is the joint probability density function of the intervening variables. Alternatively, equation [5] may be simply expressed in terms of the vector of basic random variables $\mathbf{X}$:

[6] $$P_f = \int_{G\{X\}<0} f_X(X)\,dX$$

On the other hand, the reliability or probability of survival is the volume integral over the safe region:

[7] $$P_f = \int_{G\{X\}>0} f_X(X)\,dX$$

Since a high number of random variables define a typical structural problem, the joint probability density function of all involved variables is difficult to model, and if indeed the function exists, then the closed form solution of the governing integral [6] or [7] becomes very complicated. Aside from approximate Monte Carlo simulation techniques, a commonly used approach to this reliability evaluation procedure is to work with the so called *reliability index*, β, which is based on approximate FORM (First Order Reliability Method) and SORM (Second Order Reliability Method) procedures. Thus, the probability of failure, P_f, may be estimated by the standard normal (or Gaussian) probability distribution function, $\Phi(.)$:

[8] $$P_f = \Phi(-\beta)$$

It should be noted that if all participating variables are normally distributed with known model parameters and the performance function [3] is linear, then the probability of failure [8] can be determined exactly. However, to generalize the analysis, by knowing the mean, $\overline{X_i}$, and standard deviation, σ_{X_i}, of each involved variable in the equation [3], and introducing reduced variate $\xi_i = X_i - \overline{X_i} \big/ \sigma_{X_i}$, for $i = 1, 2, 3, \ldots, n$, the reliability index, β, as depicted in Figure 1 for the simple case of two variables, can be established as the *minimum distance* from the origin to the limit states surface. The intersection point between this minimum distance and the limit states surface may be called the design point, ξ^*, and can be found through FORM/SORM techniques (Foschi *et al.* 1989).

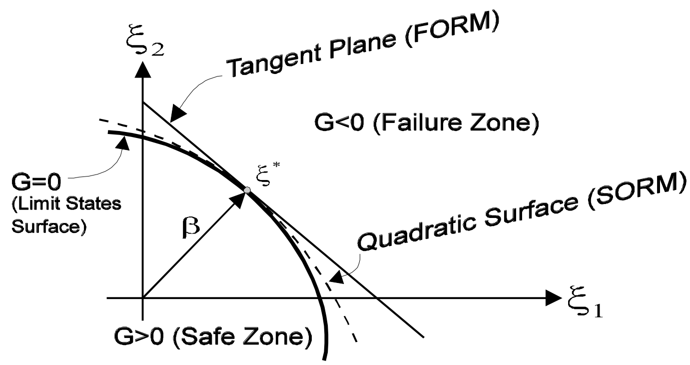

Figure 1 - Mathematical interpretation of Reliability Index.

As shown in Figure 1, FORM approximates the limit condition by fitting a tangent plane to the limit states surface at the design point, while SORM utilizes a quadratic surface to better approximate the true limit state. In addition, to refine the approximation of the probability of failure, P_f, it is required that the related variables be non-correlated. Since most structural problems involve correlated variables, a proper transformation is required to acquire uncorrelated variables and consequently estimate P_f (Der Kiureghian and Liu 1986).

A computer program RELAN (RELiability ANalysis) was developed by Foschi, Folz, Yao (1989) to carry out the above analysis and calculations. RELAN includes the Rackwitz-Fisseler algorithm for non-normal variables and transformation of correlated variables by Der Kiureghian and Liu (1986). Since the work of this paper is focused on the reliability aspects of concrete gravity dams, this program is considered to be of use to compute and establish reliability indices for not reinforced and anchor reinforced gravity dam monolith under various load cases.

The level of safety comparison between not reinforced and anchor reinforced cases is best illustrated by a case example shown herein.

GRAVITY DAM WITHOUT ANCHOR REINFORCEMENT

While concrete gravity dams are generally designed and built to suit specific geological and hydrological site conditions, or for instance, in the case of power generation, to suit specific electric power demand, the exact concrete outline and dimensions of concrete gravity dams around the world vary for each individual dam. Items such as concrete and foundation rock structural properties and weight of associated structures (bridges, gates etc.) are site specific and are required for the stability calculations. However, when conducting a static or pseudo static stability analysis, stabilizing and destabilizing loads and forces acting on gravity dams are generally similar in nature and, for illustration, only a typical non over-flow concrete gravity section is considered herein. There is ample literature on the stability analyses of not reinforced gravity dams and as such stabilizing and destabilizing loads acting on a typical gravity block are briefly reviewed herein. The rigid gravity section shown in Figure 2 is primarily subjected to the following destabilizing loads:

1. Hydrostatic pressure of upstream reservoir water and downstream tail water.
2. Silt pressure. In static conditions, submerged weight of silt is considered, while in dynamic conditions, silt is considered to be liquefied and thus would exert an equivalent liquid pressure.
3. Ice load, if any, which acts as a uniform distributed load at the upstream reservoir surface.

4. Added mass of water due to hydrodynamic pressure of upstream reservoir water.
5. Vertical uplift and horizontal earthquake inertial loads generated within the body of dam and associated attachments, such as bridges, gates, hoists and cranes, if any.
6. Vertical uplift pressure at the base of the concrete gravity monolith. The magnitude of the uplift pressure depends on the presence of drain holes at the base of the dam. Because drain holes get plugged with sediment migration over time, depending on specific circumstances, some degree of drain effectiveness is usually assumed in stability analysis.

The following loads are stabilizing loads considered in the stability analysis:

1. Self weight of concrete gravity block and associated attachments such as bridges, gates, hoists and cranes, if any.
2. Downward vertical component of hydrostatic pressure of water or silt if the upstream face of dam is inclined.
3. Downward vertical component of seismic inertial loads, if applicable in load combination under consideration.

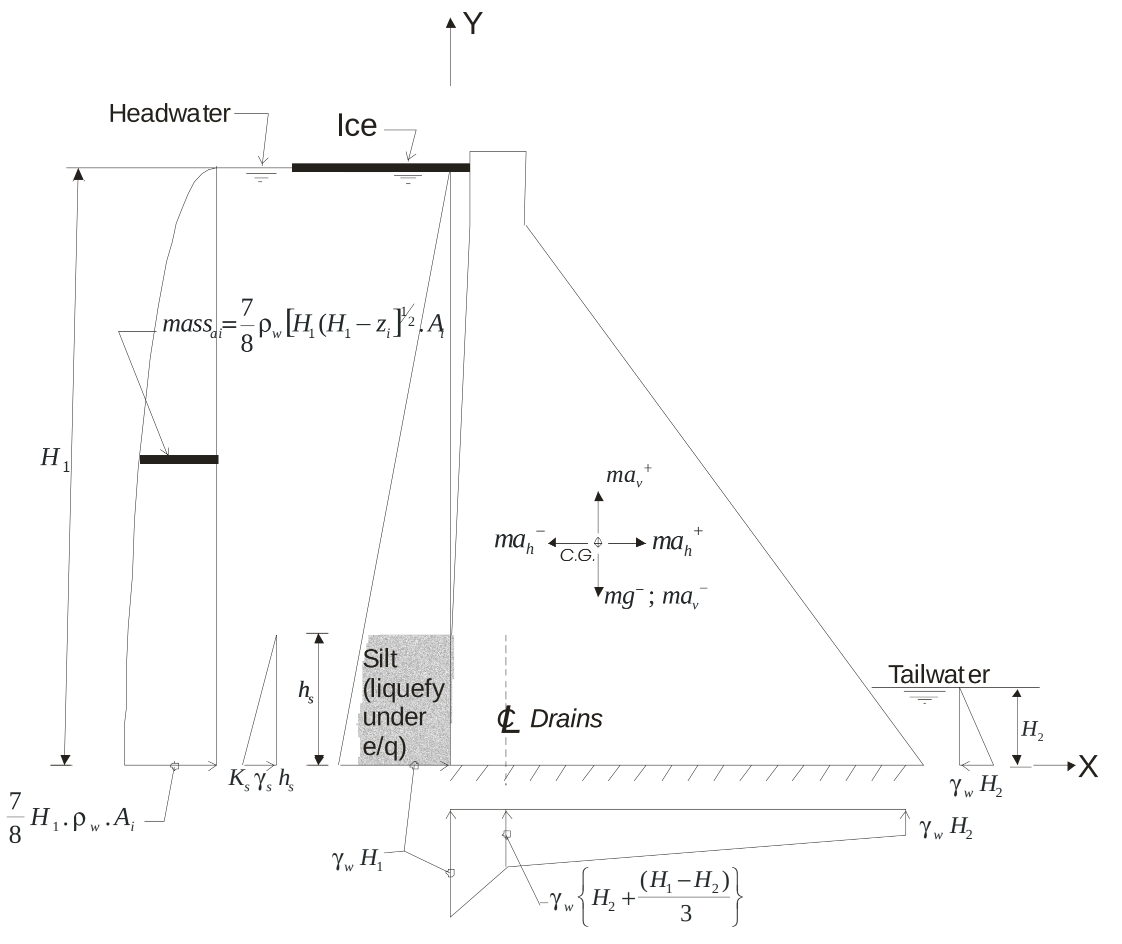

Figure 2 – Typical non-overflow concrete gravity dam monolith

It should be noted that, depending on load combination under consideration, the most unfavorable and possible combination of above loads should be considered in the stability analysis of concrete gravity dams. The beneficial effect of downward seismic inertial component, for instance, may be neglected as the opposite upward component has a destabilizing effect, which usually is considered in the critical load combination.

EXAMPLE PROBLEM

The application of reliability analysis is best illustrated by considering a typical gravity dam subject to primary loads. The tallest monolith of the 561 m long Pine Flat Dam is considered. A simplified two dimensional finite element model of this monolith is shown in Figure 3. A conventional sliding stability analysis of the same monolith was conducted for illustration purposes. It should be noted that there are relatively ample studies of the Pine Flat dam in the available literature and thus considered for this study.

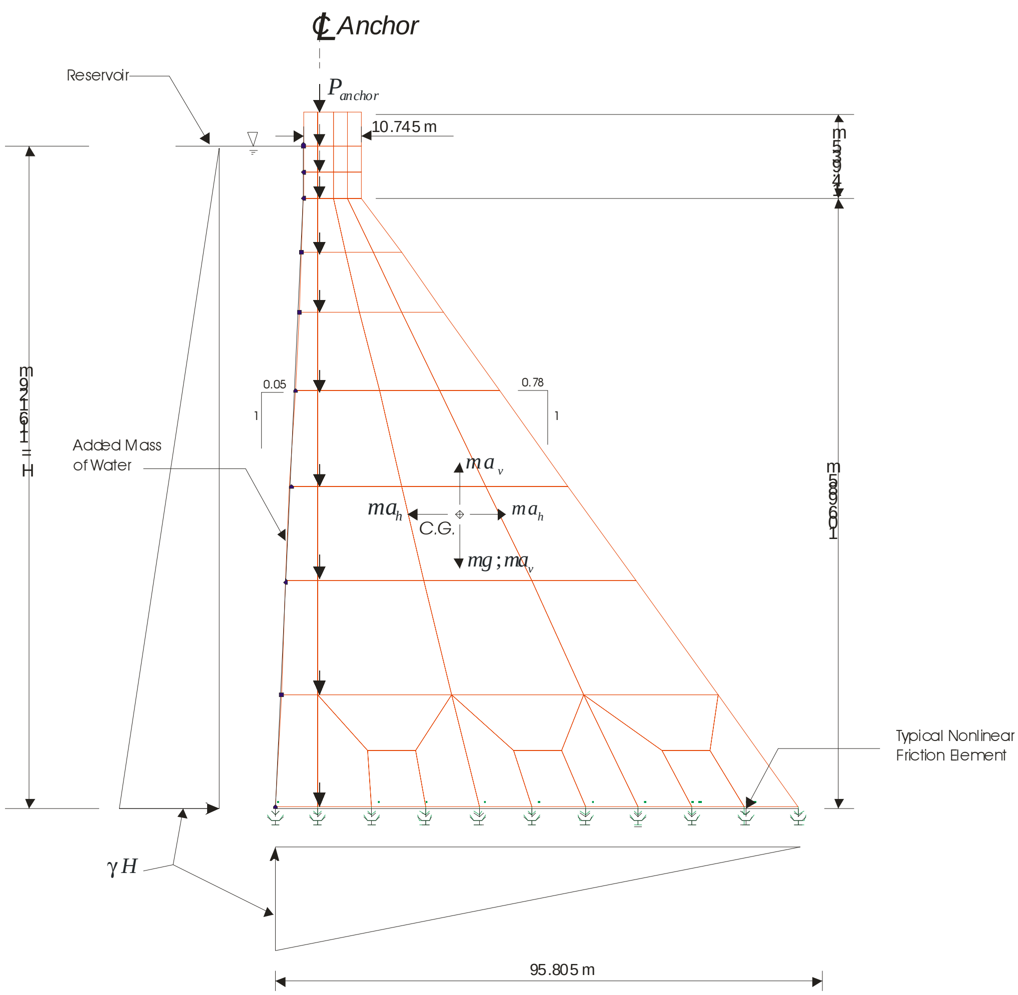

Figure 3 – Model of Pine Flat Dam

The commercial finite element package SAP2000 was used to conduct a non-linear time history analysis to obtain maximum dam crest deformations, base sliding and shear time histories due to a horizontal component of the Taft Earthquake, obtained at the Lincoln School Tunnel in the 1952 Kern County Earthquake. The earthquake record was scaled up to a peak horizontal acceleration of 0.4g. The effective uplift pressure is considered to be 40% of the total uplift due

The tallest monolith is idealized by shell elements with the base horizontal. The dam concrete is assumed to be isotropic, homogeneous and elastic with a constant modal damping of 0.10. A series of biaxial friction link elements were used to model the gap and friction behaviour between rock and concrete surfaces. The reservoir bottom and foundation rock were assumed to be rigid. Hydrodynamic forces due to earthquake were considered by considering the added mass of water (Westergaard) calculated at point *i* along the upstream face of the dam, as follows:.

[9] $$m_{ai} = \frac{7}{8} \rho_w \left[H(H - z_i) \right]^{1/2} A_i$$

where H = depth of water, z_i = height above the base of the dam and A_i = tributary surface area at point *i*. Because of reservoir height of about 116.3 m, fluid compressibility was taken into consideration by increasing the above added mass values by 30%.

Analysis Results—Sliding Factor of Safety

A traditional factor of safety analysis of the tallest monolith of the Pine Flat Dam is conducted and summary of forces acting on the horizontal sliding plane are shown in Table 1.

Table 1 – Summary of forces to estimate sliding factor of safety

Force Component	Σvert (kN)	Σhoriz (kN)
Concrete	42,153	
Water on upstream face	987	
Vertical uplift	-6,653	
Hydrostatic thrust		20,162
Hydrodynamic thrust		8,490
Horizontal earthquake		16,861
Totals	**36,487**	**45,513**

The factor of safety against sliding is thus calculated as:

[10] $$Factor of\ Safety\ against\ Sliding = \frac{\tan(\phi)\sum Vert}{\sum Horiz} = \frac{\tan(45)\times 36{,}487}{45{,}513} = 0.80$$

Based on above, stabilizing measures are required. This can be provided either by installation of vertical or inclined post tensioned rock anchors, change in concrete profile by increasing its area or improvement in foundation drainage to relive foundation uplift pressure. To meet a minimum acceptable factor of safety against sliding, a stabilizing force of 9000 kN is required.

Analysis Results—Finite Element Analysis

A nonlinear time history modal analysis of the tallest monolith of the Pine Flat Dam subject to the scaled Taft earthquake record reveals that significant upstream downstream deformations occur at the crest level. This is combined with simultaneous sliding at the concrete to rock interface. Results do indicate that assumptions made in the non-linear analysis and damping of this dam affect the response. Results are summarized in Figure 4. Note that stresses within the body of dam change significantly for both cases involving anchors and no anchors. Instantaneous tension cracks may develop, however tension cracks will close due to reversal of cyclical earthquake loading. This can further be remedied, if not eliminated, by installation of post tensioned high strength anchors where a compressive preload along the height of dam would resist opening of tension cracks. Results indicate that sliding at the concrete-rock interface is of significant importance as this interface is subjected to maximum static and dynamic loads. Post tensioned high strength anchors are commonly installed to minimize this upstream-downstream slip and thus stabilize and increase the safety of the dam during an earthquake.

As both traditional stability and finite element analyses reveal that stabilizing measures are required, a finite element analysis of the Pine Flat Dam monolith with a vertical anchor is conducted. An anchor force of 9000 kN is used since this force was estimated from the traditional stability analysis to be the minimum preload required for stability of the monolith. Thus, a vertical anchor is placed close to the upstream face of the monolith and the anchor force is distributed uniformly along the height of the monolith. The presence of high strength anchor across the concrete-rock interface provides a restraint against translation in horizontal and vertical directions.

As shown in Figure 5, results indicate that maximum average displacement at the concrete-rock drops from 10 mm to 2 mm for the dam monolith without and with vertical anchors. Energy plots show that inclusion of anchors dramatically reduces link hysteretic energy absorption at the concrete-rock interface in friction elements in the upstream downstream direction. The total input energy, which is work done on the dam monolith by force and earthquake acceleration is also reduced. This is expected as the dam monolith with the anchor reinforcement is stiffer than the dam monolith without any anchor reinforcement. Crest displacement is reduced from a peak value of 61 mm to 47 mm for the dam monolith with anchor reinforcement. Change in average base shear is relatively minimal as this value is influenced by equilibrium of forces at the concrete-rock interface.

The results of the above finite element analyses will be used in reliability analysis of the Pine Flat Dam Monolith.

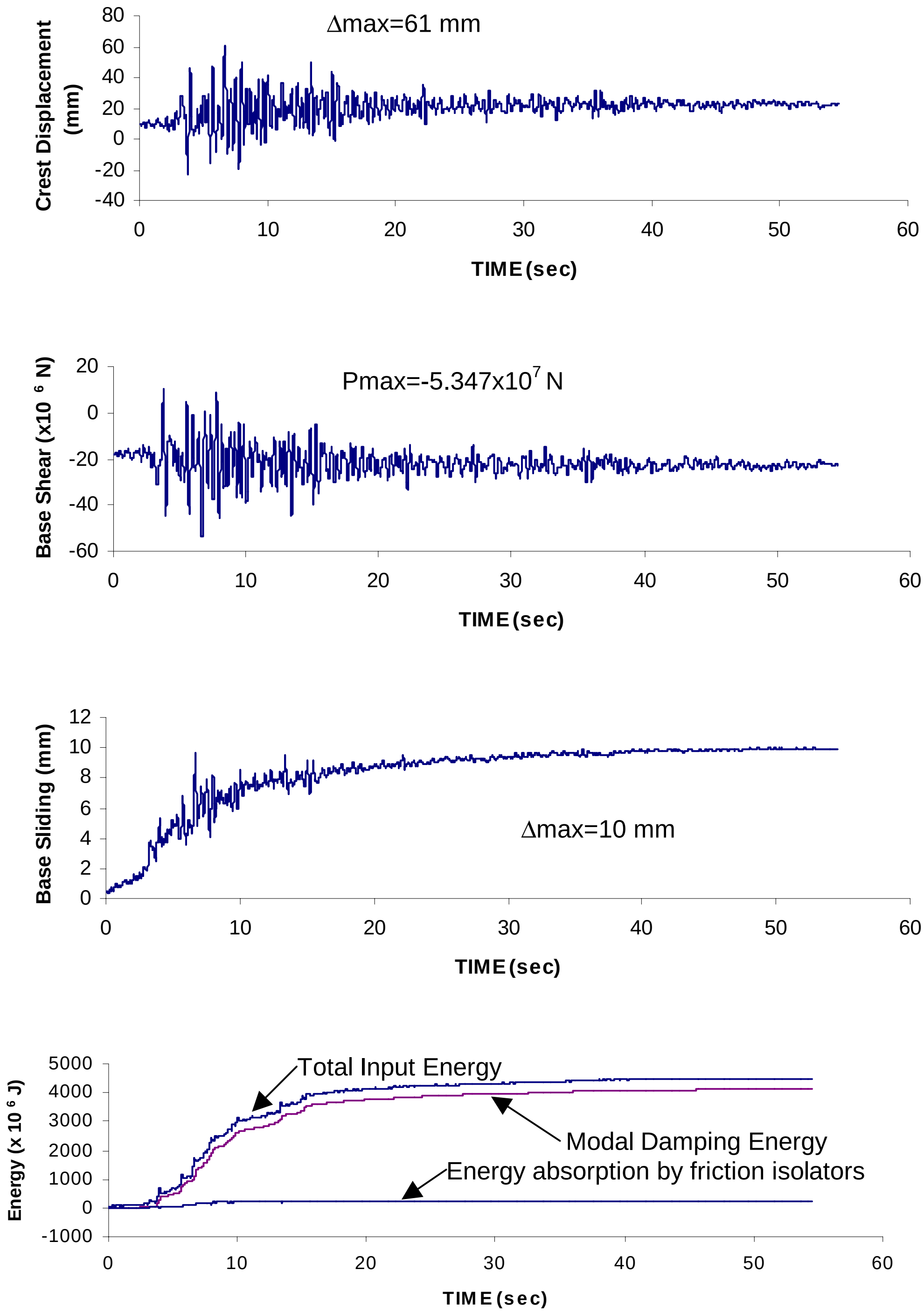

Figure 4 – Time History Response of Pine Flat Dam without Anchors

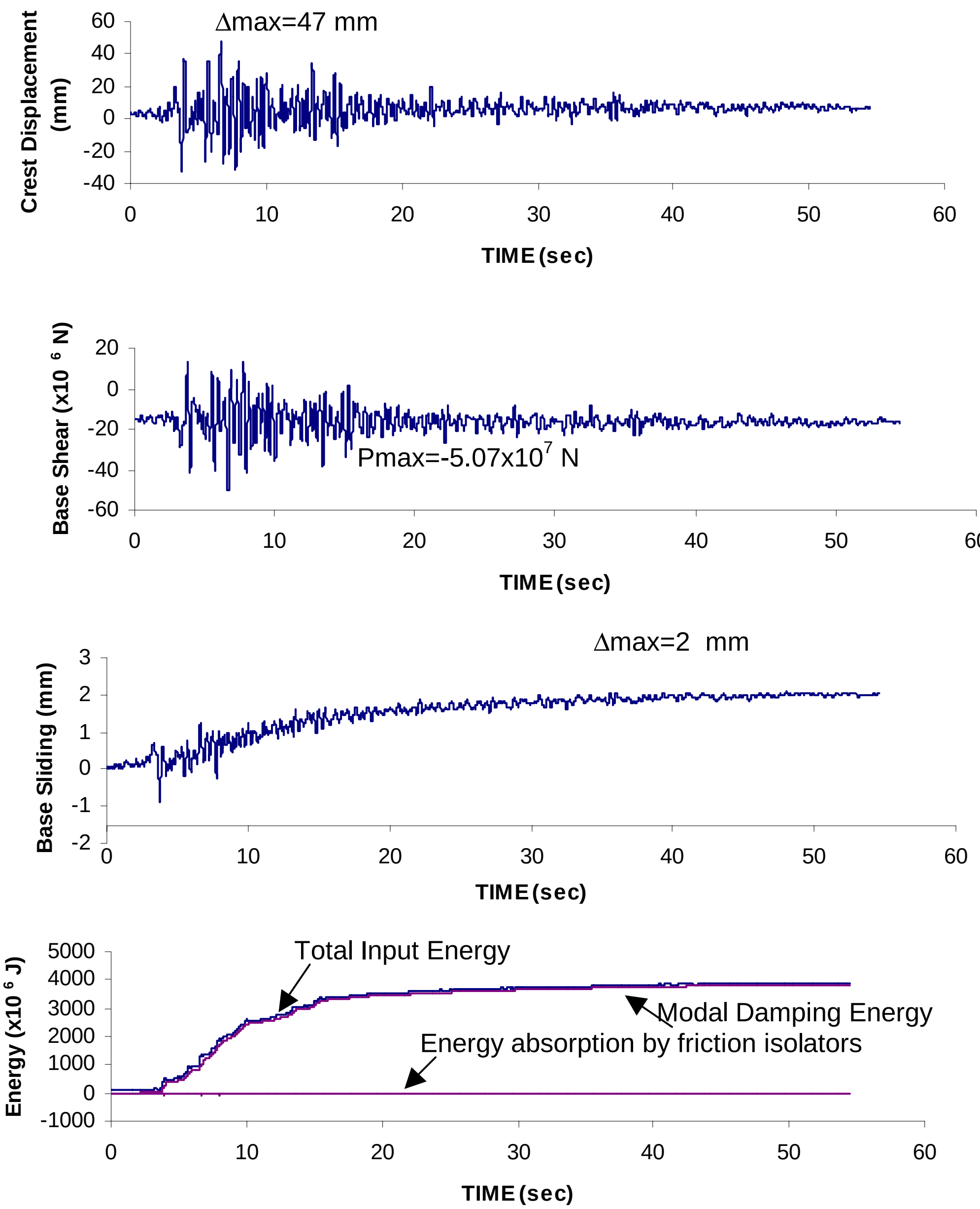

Figure 5 – Time History Response of Pine Flat Dam with a vertical Anchor

Reliability Analysis

The stochastic earthquake loading is simplified by considering maximum and critical response values obtained from the traditional factor of safety and finite element analyses of the Pine Flat Dam Monolith. A failure function is formulated by considering the free body diagram of forces acting on the dam monolith with and without anchors. A model showing the primary forces acting on the dam monolith with a vertical anchor is shown in Figure 6.

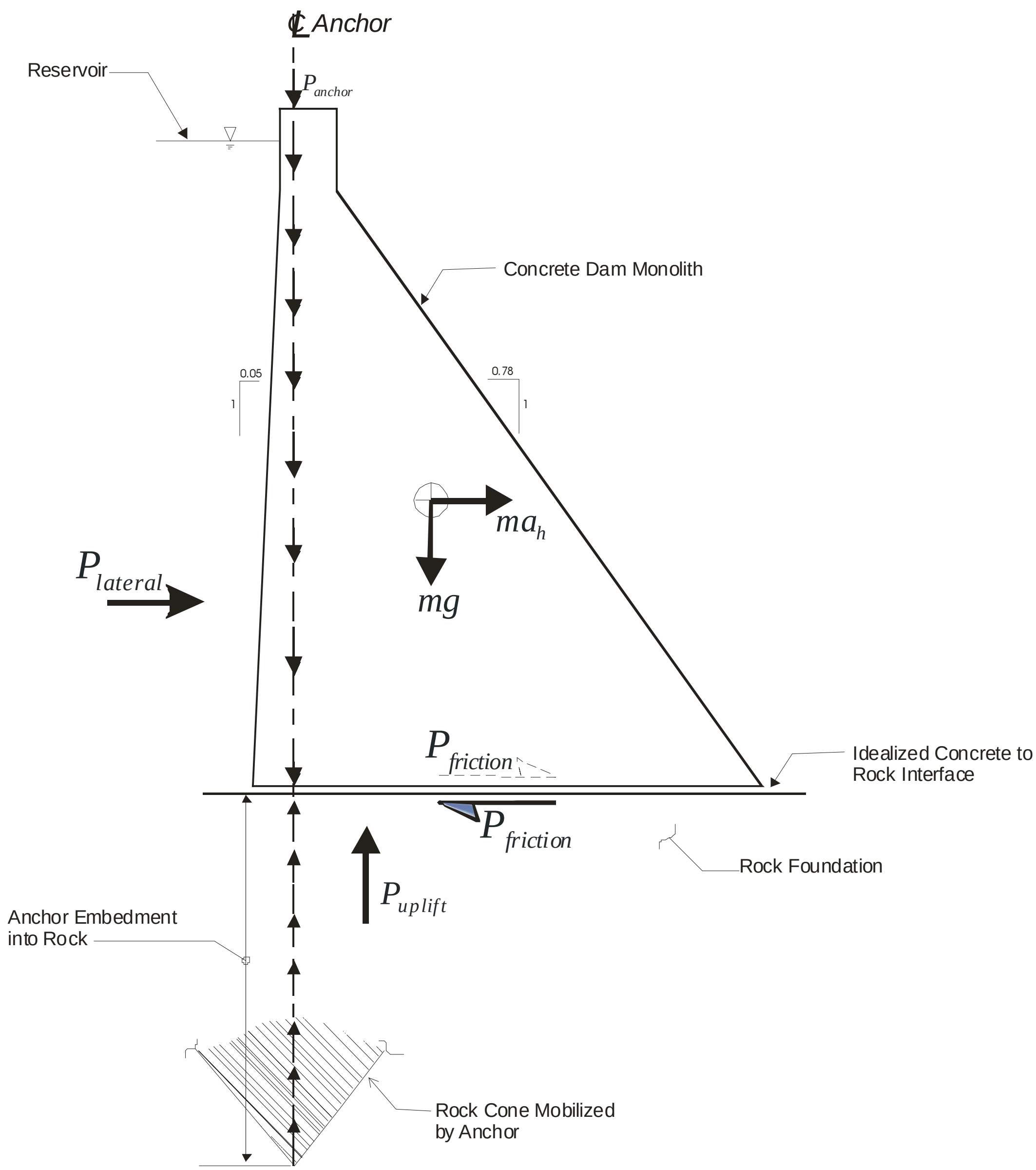

Figure 6 – Primary Forces Acting on The Pine Flat Dam Monolith with Anchor.

For the case involving the determination of factor of safety against sliding by the traditional approach, the linear failure function is written as:

[11] $$G = Capacity - Demand = \mu \sum vertical - \sum horizontal$$

where

$$\sum vertical = mg + P_{water} - P_{uplift} + P_{anchor} \; ;$$
$$\sum horizontal = P_{hydrostatic} + P_{hydrodynamic} + P_{earthquake} \; ,$$

and $\mu = \tan(\phi)$ of 1.0 assumed for this example. To consider variability in structural materials and loads, in causing horizontal and vertical loads and for use in the Relan computer program, relevant mean and standard deviations are assumed and presented in Table 2.

Table 2 – Load and material mean and standard deviation values

Item	Variability	Mean, $\overline{X_i}$	Standard Deviation, σ_{X_i}
Coefficient of friction, μ	Moderate	tan(ϕ) = 1.0	0.10
Concrete material, γ	Moderate	23.6 kN/m^3	2.36 kN/m^3
Anchor strand, f_{pu}	Low	1862 N/mm^2	46.55 N/mm^2

Note: Unit weight of water, γ_w, of 9.81 kN/ m^3 is used as a deterministic variable.

Equation 11 is solved by the Relan computer program for the dam monolith without and with the vertical anchor and safety indices are listed in Table 3. The term P_{anchor} is zero in equation [11] for the dam monolith without anchor.

For the case involving the finite element analysis of the dam monolith the failure function is written as:

[12] $$G = Capacity - Demand = \mu \sum vertical - P_{friction}$$

where $P_{friction}$ is the peak base shear response obtained from the non linear finite element analysis of the Pine Flat Dam monolith. As in the previous case, Equation 12 is solved by the Relan computer program for the dam monolith without and with the vertical anchor and safety indices are listed in Table 3. For illustration only, results for time steps yielding maximum base shear with corresponding vertical load and minimum vertical load with corresponding base shear are shown as a range of safety.

Table 3 – Summary of factors of safety and safety indices

Method	Anchor Provided	Anchor Load [kN]	Factor of Safety	Safety Index	Probability of Failure	Range of Safety
Limit Equilibrium-Static and RELAN	No	-	1.80	3.42	0.416E-03	Survival
Limit Equilibrium-Seismic and RELAN	No	-	0.80	-1.874	0.969E-00	Failure
Limit Equilibrium-Seismic and RELAN	Yes	9,000	1.0	0.00	0.500	Limit States
Finite Element and RELAN-Seismic	No	-	-	-0.651 3.569	0.742 0.179E-03	Failure Survival
Finite Element and RELAN-Seismic	Yes	9,000	-	1.354 4.804	0.878E-01 0.778E-06	Survival Survival

Design Code Approach

For reference only, the National Building Code of Canada specifies that characteristics and probability of occurrence of the design seismic ground motion are required for seismic design of structures. In this respect, the probability of exceedance of the seismic ground motion parameters is noted as 10 percent in 50 years, or 0.0021 per annum (1 in 475 years). This is often used by dam safety practitioners in estimating acceleration records and values for the Design Basis Earthquake (DBE) case. A more rigorous prbability of exceedance is applied to estimate acceleration records for the Maximum Credible Earthquake (MCE) case. The application of DBE and MCE cases are not included in the Pine Flat Dam example, although these characteristics may serve as a reference for development of limit states or reliability based procedures for analysis and design of concrete gravity or arch dams for that matter.

CONCLUSIONS AND RECOMMENDATIONS

An applied reliability based model in estimating the safety of concrete gravity dam monoliths is outlined in this paper. As an illustration, the sliding stability of a typical monolith of the Pine Flat Dam is assessed by the traditional factor of safety method and then compared against results obtained from a two dimensional finite element and reliability analyses.

It is determined that vertical and horizontal loads calculated in accordance with the traditional limit equilibrium method are conservative and thus anchor loads calculated are conservative. Although the results are influenced by assumptions made in the model and modelling of the dam-foundation and dam-reservoir interactions, results obtained from nonlinear time history analysis of a two dimensional finite element model illustrate this conservatism.

A comparison of sliding stability analysis results reveals that the traditional sliding factor of safety approach produces a factor of safety value that is about 25 % below the acceptable limit, whereas the time history dependant reliability based design approach produces a safety index that, at its lowest time step, is about 10% below the limit. In addition, anchor load provided to ensure a sliding factor of safety of 1.0 indicates safety at a limit states but a sliding factor of safety of 1.0 does not necessarily indicate safety of the dam. A small change in analysis or design assumptions may yield factor of safety less than 1.0, thus revealing ambiguous safety levels.

Assuming load, resistance and importance factors of 1.0 for the earthquake load case and for comparison purposes, the same anchor load used in the reliability based analysis yields safety of the structure. Depending on the method used in the stability analysis, this comparison reveals that relatively lower anchor loads and smaller anchor sizes may potentially be calculated. This ultimately influences the economics of dam rehabilitation and stabilization.

The use of reliability based design and finite element analysis is recommended for safety evaluation and design of concrete gravity dams. The same method can be applied in stability analysis and design of arch dams. This reliability based design also unifies the analysis and design approach employed in reinforced concrete and steel structures in hydroelectric dam facilities. In parallel with other design codes, it is further recommended that the above procedure be implemented in dam safety guidelines and design codes for use by practitioners.

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