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Evaluation of dam strength by finite element analysis

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Abstract. Current code procedures for stress and stability analysis of new and existing concrete-gravity dams are primarily based on conventional methods of analysis. Such methods can be applied in a straightforward manner but there has been evidence that they may be inaccurate or, possibly, not conservative. This paper presents finite element modeling and analysis procedures and makes recommendations for local failure criteria at the dam-rock interface aimed at predicting more accurately the behavior of dams under hydraulic and anchoring loads.

Keywords: dam; concrete; stability; failure; sliding; overturning; finite element; interface

1. Introduction

Despite developments with regard to methods of analysis, design procedures and code requirements for concrete dams are based primarily on conventional tools. Generally speaking, the practice of dam design lacks information and guidance with regard to the use of the finite element method. Agencies such as the U.S. Army Corps of Engineers, the U.S. Bureau of Reclamation, the U.S. Federal Energy Regulatory Commission (FERC) and the Canadian Electricity Association have developed design guidelines and requirements for the analysis of existing and new dams by conventional methods (U.S. Army 1995, United States 1987, 1976, Meisenheimer 1995) that have been applied to the design of thousands of dams throughout the U.S., Canada and the world in the early part of the last century. Although several studies have reported significant findings on finite element procedures (Ebeling *et al*. 1992, 1993, 1996, Curtis *et al*. 1998), no such code guidelines are available on the application of the finite element method. The conventional tools have proven satisfactory in practice but, given the underlying simplifications and assumptions, there is little clarity as to the level of conservatism in the results. Hydrologic data in recent years have pointed to major stabilization needs of existing dams (Freese and Nichols 1998, 2001). Post-tensioning anchoring stabilization has emerged as a popular, cost-effective approach (Freese and Nichols 1998, 2001, Wolfhope *et al*. 2001, Boyd *et al*. 1999). The finite element method can potentially become a practical tool for evaluating the stability of dams and may lead to engineering solutions at reduced costs, if a simple methodology is devised that can predict more accurately the behavior of dams under hydraulic and anchoring loads.

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2. Modeling and analysis of dams

Current code requirements for new or existing dams are based on several considerations with regard to *overturning stability, sliding stability and stress levels in the dam concrete and foundation rock*. A brief summary of each of these requirements is provided below.

The overturning stability is assessed by applying the vertical (ΣV) and lateral forces for each loading condition and then summing moments (ΣM) about the toe (downstream end of the base). The location of the resultant reaction along the base of the dam is calculated from the ratio $\Sigma M/\Sigma V$. In calculating the location, several loading conditions should be considered as required by the various agencies including usual, unusual and extreme load cases. A factor of safety against overturning is indirectly addressed by setting limit criteria on the location of the resultant along the base of the dam depending on the loading condition in consideration. It is based on the overall applied loads through the resultant force and the resultant eccentricity, and not on the ratio of the resisting to the overturning moments. More liberal requirements are set for loading conditions of low probability of occurrence. Theoretically, the resultant can be located anywhere from the centroid to the toe but cannot lie outside the base, because tension cannot develop at the heel of the dam. In the extreme case of the resultant located at the toe, the factor of safety against overturning is equal to one. From the practical point of view, the resultant cannot lie at the toe as that would imply an infinite stress level at that location. Actually, the resultant should lie upstream of the toe to maintain adequate contact area that will ensure base pressures are within prescribed limits. Stability and stress analysis calculations for a cracked base should consider full uplift pressure on the crack faces.

The sliding stability is evaluated by a factor of safety as a measure of the resistance of the structure against sliding, defined as the ratio of shear strength and the applied shear for the plane in question. The required factor of safety varies with the type of loading. Shear strength is normally calculated using a Mohr-Coulomb failure criterion, which assumes linear variation of shear strength with normal stress. The cohesive component of shear strength should include only that portion of the base area in contact with the foundation. Various regulatory agencies provide criteria for using either peak or residual shear strength parameters for shear-strength determination.

Stresses in the dam concrete or the foundation rock should not exceed the respective allowable material stresses appropriate for each loading condition. A stress analysis should be performed in order to determine the magnitude and distribution of stresses in the structure including the dam, the interface, and the foundation and to investigate the adequacy of the dam structure and foundation.

2.1 Conventional equilibrium and finite-element methods of analysis

The gravity or conventional equilibrium method is a simplified, approximate method that uses a cantilever beam model for the analysis. The response of a dam is actually three-dimensional but the method treats the dam as a series of two-dimensional cantilever beam sections ignoring shear-deformation effects. Other assumptions of the method are:

a. Two-dimensional behavior: out-of-plane stresses and strains are negligible.
b. Linear distribution of normal stresses on horizontal planes.
c. Rigid foundation (infinite foundation stiffness).

The finite element method is ordinarily used in the final design stages, if a more detailed analysis is required. However, no code-type design procedures have been established yet. It can account for a wide range of design parameters such as complex geometric configurations and out-of-plane asymmetry, nonlinear material behavior, dam-foundation interaction, variation in material properties, fractured zones in the foundation, or stress concentrations at discontinuities and at tension zones.

The method uses the principle of virtual work and an assumed displacement function to generate the system of nodal equilibrium equations of a discretized continuum. The accuracy of the results is, therefore, dependent upon the fineness of the discretization. A finite-element model of a dam, as compared to a beam model, will lead to an improved approximation of the actual distribution and magnitude of stresses in a section either at its base or away from it considering shear-deformation effects. A finite-element analysis can also be used to calculate the required resisting anchoring force for any desired deformation state at the dam-foundation interface. Several agencies and research studies report that conventional methods of analysis may be more conservative than necessary especially when making a determination as to the need for remedial strengthening to improve the stability of the existing dam (Ebeling *et al*. 1992, 1993, 1996, Curtis *et al*. 1998). If the conventional methods indicate the need for remedial strengthening, then refined finite-element analysis can be performed to further investigate this requirement. A refined finite element analysis should accurately model the strength and stiffness of the dam and foundation to determine the following:

a. The extent of possible cracking at the interface.
b. The base area in compression.
c. The magnitude and distribution of foundation pressures.
d. Prediction of stresses at discontinuities such as the heel and the toe.
e. Modeling of foundation-structure interaction.
f. Improved distribution of the driving and resisting forces at the dam-foundation interface and ability to model non-linear failure criteria at the interface.
g. Modeling of the progressive development of local uplift movement and local shear sliding at the dam base.

2.2 Geometric modeling in two dimensions

A representative section of an existing dam that includes a fifty-nine-foot wide non-overflow rollway section and a nine-foot wide overhanging pier was modeled as a plane section to explore finite-element modeling parameters outlined in Section 2.1 in order to develop a methodology for accurate prediction of the behavior of dams under hydraulic, silt, gravity and anchoring loads. Three-noded triangular and four-noded quadrilateral plane-stress and plane-strain elements were used. In general, element sides do not exceed 4 ft and have an aspect ratio less than about 2.5. Triangular elements appear mostly at the crest of rollway. Results of the analyses are presented in Section 3.

2.3 Hydraulic, silt, self-weight and anchoring loads

Upstream, tailwater, uplift and crest hydraulic loads and silt and dam self-weight loads were applied as equivalent nodal forces at the respective elements. Anchoring loads were specified as externally applied concentrated nodal forces at the appropriate crest or non-overflow location. Since anchors were modeled as externally applied loads and not as structural elements, their local effect (tension) in the rock mass (in cases of models that include the underlying rock) could not be accounted for in the present procedure. It should be noted that only static loads were considered in the present study.

2.4 Material properties

The following material properties, as determined from physical tests on cores obtained at various locations in the dam concrete and the foundation rock were used as input modeling parameters:

Concrete:	Plane stress:	Plane strain:
	Elastic modulus: 5.77E6 psi	Elastic modulus: 5.94E6 psi
	Poisson's ratio: 0.17	Poisson's ratio: 0.205
Rock:	Plane stress zone 3	Plane stress zone 2 (Fault zone):
	Elastic modulus: 9.0E6 psi	Elastic modulus: 0.2E6 psi
	Poisson's ratio: 0.25	Poisson's ratio: 0.2

2.5 Local failure criteria at the dam base

One of the most important considerations in predicting accurately the physical behavior of a dam under hydraulic and anchoring loads is the representation of movement that takes place at the dam base with respect to its foundation. Movement at the base can be limited to only a portion of the base, which implies local failure because a certain local strength may have been exceeded, but it does not necessarily constitute overall dam instability. Modeling of this movement is necessary in order to determine the final deformation and redistribution of stresses in a dam section and to verify that overall equilibrium of the section can be established. If the movement is found excessive in the final equilibrium position, it can be contained locally, at a portion of the base, by means of a stabilization method such as the addition of extra concrete weight at the downstream face of the dam or by using post tensioning anchors drilled through the concrete and anchored into the underlying rock (Freese and Nichols 1998, 2001, Wolfhope *et al*. 2001, Boyd *et al*. 1999). The movement is either vertical uplift or horizontal sliding starting from the heel and progressing downstream toward the toe. A finite-element model should be capable of representing with sufficient accuracy the mechanism of any uplift separation and local shear sliding at the base of the dam relative to its foundation. The condition for movement is set in two failure criteria as follows:
Criterion 1: There is no tensile strength at the interface.
Criterion 2: The shear strength at the interface is represented by a Mohr-Coulomb failure criterion.

Test cores obtained at the interface of the actual dam showed no tensile strength between the rock and the concrete, and thus verified Criterion 1. This criterion is also a design recommendation of several agencies (Meisenheimer 1995). The basic consideration is that local uplift separation of the dam base occurs when the effective interface forces or stresses at the interface become tensile. To apply this consideration, a restraint from movement in the vertical direction is removed as soon as the interface force or stress would become tensile.

For Criterion 2, local shear sliding occurs when the shear force at the interface exceeds the respective shear strength. The Mohr-Coulomb failure criterion was used to calculate the shear strength of the interface. This criterion assumes a linear relationship between shear strength and normal stress. A cohesion value of 12 psi representing the intercept of the linear relationship was used. As suggested by several agencies, the separated portion of the base was modeled to provide no cohesion contribution to shear strength. The respective angle of internal friction, which is the slope of the relationship, was assumed as 35°. Both cohesion and angle of internal friction values used in the analyses conservatively represent the residual shear strength. The same overall factor of safety of 1.3 against sliding of the entire base as was used in the conventional analysis was also used in the finite-element study. In addition, a local sliding failure criterion of 1.3 was also used in the finite-element analysis. This implies that at every nodal point at the interface a local factor of safety for sliding is established as determined from the ratio of shear strength and shear force. As will be explained in further detail in the following section, sliding failure will occur locally at a nodal point when the shear strength to shear force ratio drops to a value of 1.3 or less. The horizontal restraint is then removed at this node and a horizontal force equal to the shear force with no factor of safety is applied in opposite direction to sliding. A local shear check is more restrictive than an overall sliding check because the criterion has to be satisfied locally at every point and not in an average sense over the entire base.

2.6 Dam-foundation interface elements

Interface elements, as defined in this study, are either finite elements or beam type members inserted at the transition between the dam and the foundation. They either connect the dam to a fixed support when the model does not include the foundation rock or they connect the dam to a compressible foundation when the rock is included in the model. These elements were used to facilitate modeling of the physical behavior of the dam base through application of the failure criteria, if the dam progressively uplifts or displaces horizontally and partially under the action of hydraulic loads. The procedures for applying the failure criteria to models without the rock (concrete models) and to models with the rock (concrete-rock models) are described in the following sections.

2.6.1 Concrete models

In concrete models, interface finite elements were directly connected to pinned supports providing full horizontal and vertical restraint. These supports assume an infinitely stiff foundation with no vertical or lateral movement of the base. Modeling of any movement at the base is actually a progressive event that can be accomplished through successive iterations. If in the initial iteration the vertical reaction at the heel is tensile, the vertical and horizontal restraints at this location are removed allowing the base to move freely upwards and laterally in subsequent iterations. This procedure is continued until no support is in tension, i.e., until failure Criterion 1 is satisfied for the entire base length. At that point, a check is made to determine whether the shear strength at the first remaining nodal support away from the heel (referred to as the tip) is smaller (with a factor of safety of 1.3) than the horizontal reaction at the support. The horizontal restraint is then removed and a horizontal force equal to the shear strength at the support (with no factor of safety) is applied at the node. Since the tip is defined as the first point along the base away from the heel provided with vertical restraint, it is therefore a floating point in the iteration process to achieve a final equilibrium position. The procedure is repeated, checking one support at a time, until the horizontal nodal reaction is found to be smaller than the respective shear strength. In the iterations, redistribution of stresses at the entire base takes place and partial closing of the crack may occur, requiring repeated checks and relevant changes to be made. If the criteria for uplift and sliding are eventually satisfied, then horizontal and vertical equilibrium of the dam is achieved. At this equilibrium position, the extent over which the base was allowed to move upwards (i.e., the distance between the heel and the tip) represents the crack length. The distance between the heel and the first horizontally restrained support represents the length over which the dam base has moved horizontally. This length is always at least equal to the crack length. It should be noted that full uplift pressure corresponding to the upstream pool elevation is applied over the entire crack length.

Tension at the base is established using the following internal forces or stresses:

a. Reaction at the tip of the base.
b. Tip element nodal stress.
c. Tip element centroidal stress.

2.6.2 Concrete-rock models

In models that include the foundation rock, short, interface, vertical, dummy beam members with very large axial and shear stiffness and infinitesimal bending stiffness are used to connect the dam concrete to the foundation rock. The axial and shear stiffness of these interface elements model the vertical support and shear resistance, respectively. A small (but nonzero) bending stiffness is assigned to these elements because the interface does not possess any rotational resistance. However, these members attract slight bending moment due to the relative horizontal displacement of the two ends (concrete and rock). The length (l) of the interface members is set at a fraction, say, 1/1000, of the typical (concrete or rock) finite-element size (h). The interface members are used because they allow easy modification of the concrete model to include the rock without any change in the concrete element connectivities, readily provide tip vertical and shear forces at the interface used for failure analysis, and can be easily removed to satisfy the failure criteria without the need for any model changes at the interface.

Interface elements are only used to facilitate application of the set failure criteria, and their presence should not affect the response of the dam. A parametric study was conducted to determine stiffness values of interface members that would not influence the stress and deformation behavior of the dam both near and away from the interface. A similar model without the presence of these members was used as a basis for comparison. Results have indicated a range of stiffness values that give this model essentially the same behavior as the model without the members. However, within this range the forces attracted by the members change considerably. This sensitivity is due to numerical instability in the solution of the system of equations due to the small (rotational) stiffness on the diagonal of the system matrix. To further examine this problem a separate computer program was developed to calculate the correct nodal forces (resultants of nodal forces on concrete and rock elements on either side of the interface) for the model without the dummy members. The interface member stiffnesses were then varied to obtain comparable forces in these members. In the calculations presented below, the axial stiffness of the interface elements is set equal to $EA=Eh^2$ (E being the modulus of elasticity of concrete) and the bending stiffness to $EI=Eh^2l^2$.

An iterative procedure similar to one described above for concrete models was used to apply the uplift and sliding failure criteria at the interface of the concrete-rock models with the only difference that interface members are not connected to supports but connect corresponding dam and rock nodes. In order to satisfy the no-tension criterion, beam members are removed successively from the heel, if they are in tension. For the sliding criterion, if shear in members is greater than the shear strength, the shear degree-of-freedom is released at both member ends and the shear strength is applied as an internal-force couple at the two member ends. Again, as with concrete models, member forces, tip element nodal stress and tip element centroidal stress are used to determine failure at the interface.

2.7 Material stress failure criteria

Two material failure criteria are used to evaluate the state of stress in the concrete and the rock:

1. Principal concrete and rock stresses were directly compared to ultimate compressive and tensile strengths.
2. A Drucker-Prager failure criterion (Fardis *et al.* 1983). This criterion takes into account the increase in concrete strength under compression and the decrease in strength under combined tension and compression.

3. Finite element analysis results

3.1 Models without base separation

Two plane finite-element models of the dam section were developed, one without the rock and one with the rock foundation (Figs. 1 and 2). The models represent a single pier, 9 ft wide, and half of the adjacent rollways, each measuring 25 ft in width, for a total out-of-plane width of 59 ft. The rock foundation was modeled to a depth of 100 ft below the base. The dam base is 75 ft wide. For

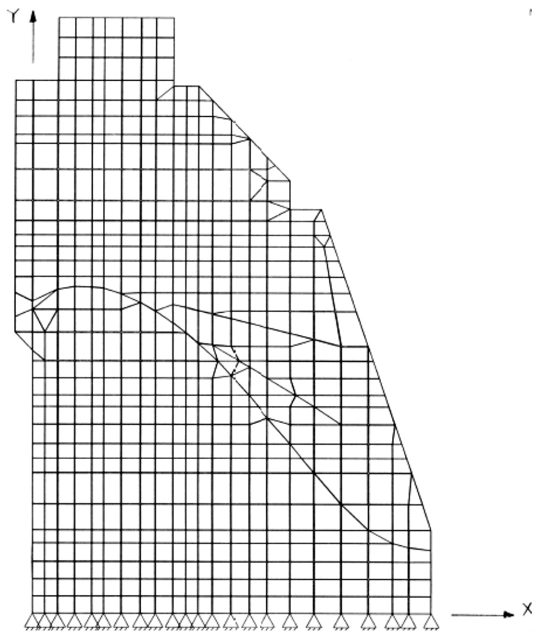

Fig. 1 Finite-element mesh and support conditions: concrete model

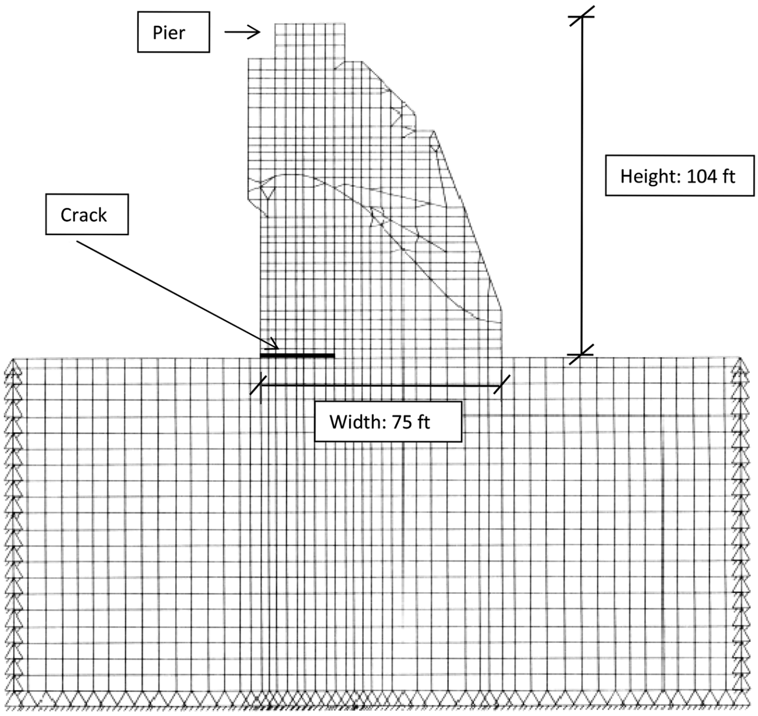

Fig. 2 Finite-element mesh and support conditions, concrete-rock model

the discretization used there are 23 nodes along the base. The dam base is at elevation of 738 ft, while the rollway crest elevation is 795 ft and the top of pier 842 ft. The applied hydraulic loads represent the Probable-Maximum-Flood (PMF) event with pool elevation of 849.4 ft and tailwater elevation 819.5 ft. The anchors are located 18 ft downstream from gate seals with 21^{o} of inclination angle with respect to the vertical. All structural models were developed and analyzed using the commercial structural analysis and design software program STAADPro (STAADPro 1996). Additionally, all results were verified using a finite element program developed specifically for this study. It should be noted that the same iterative procedure and failure criteria, as outlined in Sections 2.5 and 2.6, were implemented in both STAADPro and the finite-element program developed in this study.

3.1.1 Concrete model

Finite element analysis has shown that, for the mesh in Fig. 1, an anchoring force of 17730 kip is necessary to keep the dam base in full contact with the foundation support. This force represents the minimum anchoring force required to keep the heel in compression. Four successive mesh refinements, however, indicated an increasing anchoring force requirement to about 27710 kip with slow rate of convergence. The respective anchoring force found from the conventional analysis was 5700 kip.

In order to examine the magnitude of the anchoring force requirement estimated by the conventional equilibrium method which uses the Bernoulli beam theory that ignores the effect of shear stresses on deformation, a series of simple, cantilever-beam models with rectangular finite-element meshes were generated, resembling the dam section. These models avoided the complex geometry of the actual dam, facilitated the generation of finer meshes and were used to examine anchoring force requirements of dams with varying aspect ratio. Uniform in-plane pressure was applied to one face of the beam to simulate the upstream hydraulic load and a vertical joint force was applied at the free end of the cantilever to represent the restoring anchoring force. A length-to-depth beam ratio (L/d) of 10 was used to model a beam with predominantly flexural deformations and an L/d of 1 to model a beam with predominantly shearing deformations. For each beam, four finite-element meshes were used: 1×10, 2×20, 4×40 and 8×80 for L/d of 10, and 10×10, 20×20, 40×40 and 80×80 for L/d of 1.

The results shown in Table 1 indicate that, for the flexural beam, a coarse finite element mesh (1×10) underestimates by as much as 33% the anchoring force requirement, while for the finest mesh used (8×80) the finite element and conventional methods give practically the same results. However, the anchoring force requirement for the shear beam is shown to be excessively underestimated by the conventional method. For a finite element mesh of 80x80 the anchoring force requirement calculated by the finite element method is more than twice of that calculated by the conventional method. These results indicate that the conventional equilibrium method can predict accurately the anchoring force required for the entire dam base to be kept in compression in predominantly flexural sections, but it can seriously underestimate the force when shearing deformations are prevalent. The latter is the case of the dam sections under investigation, which have L/d of 1.4 at the pier and 0.75 at the rollway. It also appears that the discontinuity at the heel causes a stress concentration that cannot be accurately assessed with conventional beam analysis or a coarse-mesh finite element model. A finite element analysis with a very fine mesh is required to model the tensile-stress concentration at the heel and calculate correctly the anchoring force that would completely remove the tension at the heel.

Table 1 Anchoring force requirement (restoring force; kip)

	L/d = 10	
	Minimum restoring force required to keep the heel in compression	
FE Mesh	STAAD	In-House FE Program
1×10	5.00	5.00
2×20	6.67	6.66
4×40	7.27	7.26
8×80	7.43	7.43
Beam theory	7.50	

	L/d = 1	
	Minimum restoring force required to keep the heel in compression	
FE Mesh	STAAD	In-House FE Program
1×10	1.19	1.24
2×20	1.38	1.45
4×40	1.57	1.57
8×80	1.71	1.72
Beam theory	0.75	

3.1.2 Concrete-rock model

For the concrete-rock model in Fig. 2, the dam base was connected directly to the rock elements without intermediate connection with beam interface members. The rock mass was modeled to a distance of 75 ft upstream and 75 ft downstream and to a depth of 100 ft. Pinned supports restraining movement in the horizontal and vertical directions were inserted along the horizontal boundary rock line. Supports preventing lateral movement and allowing vertical movement were placed at the vertical upstream and downstream sides of the rock mass boundary. Finite element analysis results showed a drastic drop in anchoring force to 9710 kip compared to 17730 kip for a similar mesh of the concrete model. Although the rock is relatively much stiffer than the concrete (rock modulus used was 9 million psi), it is shown to act as a "cushion to reduce the severity of stress concentration at the heel and to more evenly distribute the stresses at the entire concrete-rock interface. In addition, the presence of a deformable medium at the dam base allows the interface to deflect without loss of contact with the rock along the entire base, as opposed to a straight line interface and full base contact requirement in the concrete model. Modeling the rock foundation is therefore considered essential in predicting the dam behavior, which in effect reduces the anchoring force requirement approximately by 50%. Further refinement of the mesh with this model has shown, as before with the concrete model, an increase in the anchor force requirement, though not as pronounced as before, probably because of the presence of the underlying deformable medium. This increase, as will be shown in the following section, is attributed to the severe restriction requiring full contact of the dam on its base, throughout its length. Subsequent analyses in the following section will show that mesh refinement is not necessary and a coarse mesh would suffice if this restriction is relaxed and the dam is allowed to independently move from its base, a condition closer to the actual behavior of the dam under the action of the hydraulic loads.

3.2 Models with base separation

In the following analyses, the behavior of the concrete and concrete-rock models analyzed in the previous section is further investigated by relaxing the requirement of full contact at the base and allowing for independent movement and deformation of the base to take place with respect to the foundation. This type of analysis requires application of the failure criteria in an iterative form of solution that involves successive changes in the boundary conditions, the interface members and the applied hydraulic uplift pressures at the interface. At each successive run of both STAADPro and the finite-element programs, results of the previous run were used to manually modify the respective input model to be used in subsequent runs.

3.2.1 Concrete model

Results of the analysis with base uplift allowance are presented in Fig. 3. Each point on the graphs represents a minimum required anchoring force that would just keep the tip of the base in compression for a corresponding specified crack length. The three graphs represent three different methods of calculating the required anchoring force. The *Rf*, *Rj* and *Rc* graphs were obtained by calculating the reaction force at the tip, the stress at the tip, and the centroidal stress at the tip element, respectively. Each graph was generated by removing one support at a time beginning from the heel and moving downstream, and applying an anchoring force that would just keep the tip of the base in compression. Several iterations were carried out for a single crack length until the local sliding failure criteria were implemented to satisfy equilibrium. Each point on the graph represents,

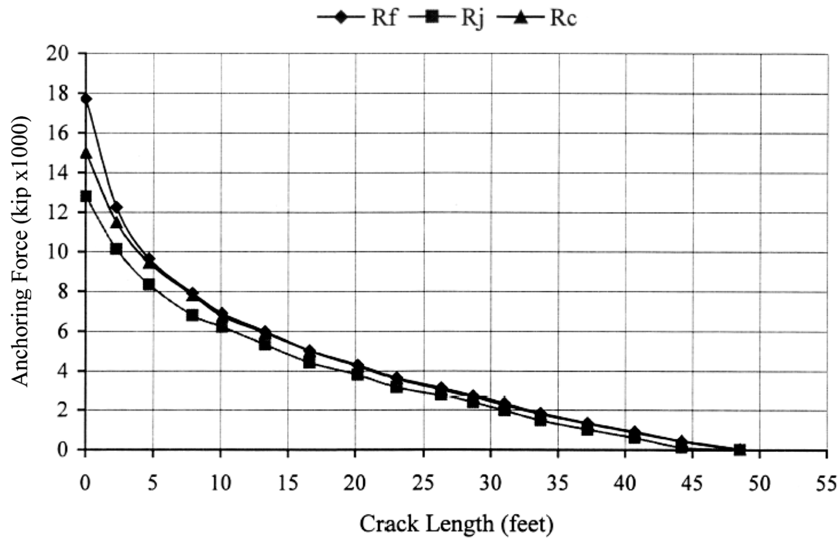

Fig. 3 Anchoring force requirement vs. crack length: concrete model

therefore, an equilibrium position uniquely defined by a set of hydraulic loads, an anchoring force and a predefined base deformation pattern.

A basic observation from the graphs is that irrespective of the method of calculating the anchoring force there is a significant drop in anchoring force requirement with increasing crack length. For the section analyzed, the anchoring force dropped from 17730 kip for no uplift allowance, to zero force for 48.5 ft of uplift representing 65% of the length of the base. A faster drop in anchoring force occurs at smaller crack lengths as indicated by the higher slope of the graphs at small crack lengths. A nearly constant slope exists beyond 20 ft of crack. This behavior is beneficial because a slightly cracked base can reduce substantially the anchoring force requirement. For the particular section, a 5% base uplift reduces the anchoring force by more than 40%. It is apparent that smaller anchoring forces are calculated with allowance for uplift because the base does not need to remain straight or completely in contact with the foundation. Furthermore, anchoring calculations are based on tip forces or stresses, which are less influenced by the discontinuity as the tip moves away from the heel. As the crack length increases, the factor of safety against sliding drops and the location of the resultant shifts towards the toe of the base because of reduction in cohesion, increase in uplift pressure and reduction in the magnitude of the anchoring force. The iterative crack base analysis will not necessarily yield equilibrium with no anchoring and may result in dam instability, if the failure criteria are not satisfied. A minimum local factor of safety against sliding of 1.11 was calculated for the case of no anchoring force. This is less than the established minimum sliding factor of safety of 1.3 and, therefore, the condition of no anchoring is not acceptable. A factor of safety of 1.3 occurs at 42 ft of crack or, 56% of the length of the base, for an anchoring force of 750 kip.

There is a substantial difference in anchoring force requirement among the three graphs for no base uplift (zero crack length). The difference of 2500-5000 kip for no crack drops to a few hundred kip beyond 20 ft of crack. The *Rf* and *Rj* graphs are in closer agreement and essentially identical beyond 5 ft of crack because they are calculated on the basis of forces and stresses at the tip. The *Rc* graph yields relatively smaller anchor force requirement because the centroidal stress on which the method is based on is inwards from the tip. Mesh refinement would bring the three graphs in closer agreement. These results show that the anchoring force requirement is highly dependent on the method of calculation used (force or stress), the location in question (tip or centroid), the size of the mesh and the deformability of the interface. Anchoring force calculations based on tip reaction give the most accurate results among the three methods because forces are, in an average sense, more accurate response quantities than stresses in finite element analysis. If no crack is allowed at the base the force method should be used because stress calculations would require an excessively fine mesh to yield accurate results. For cracked-base analysis and crack lengths greater than 10% of the base, calculation based on the stresses at the tip would also be adequate. Also, with increasing crack lengths, the mesh size becomes less critical.

3.2.2 Concrete-rock model

Results for the anchoring force requirement as a function of crack length in the concrete-rock model are presented in Fig. 4. A difference still exists among the three graphs at no crack or at crack lengths less than 5% of the base but is not as distinct as in the concrete model. There is almost perfect agreement among the three graphs beyond 10% base length uplift. The anchoring force for no uplift as calculated from interface forces in the dummy beam members is 9720 kip. No anchoring is required for a crack length of 44.2 ft or about 60% of the base. A factor of safety for sliding for that case was calculated at 1.19, which is not acceptable. A minimum required factor of safety of 1.3 is obtained at a crack length of 40 ft (see the deflected shape in Fig. 7) and a corresponding anchoring force of 200 kip. At this anchoring force there is no effective force at the tip member located 40 ft from the heel. When the sliding criteria were applied and interface members failed in shear beyond the tip, an effective compressive force developed at the tip. As more members failed in shear and compression increased at the tip member, a check was made to examine whether the compressive force was adequate to close the crack up to the next node upstream from the tip. To verify that an interface member was replaced at the node imposing restraint in the vertical direction and with releases in the horizontal and a check was made for

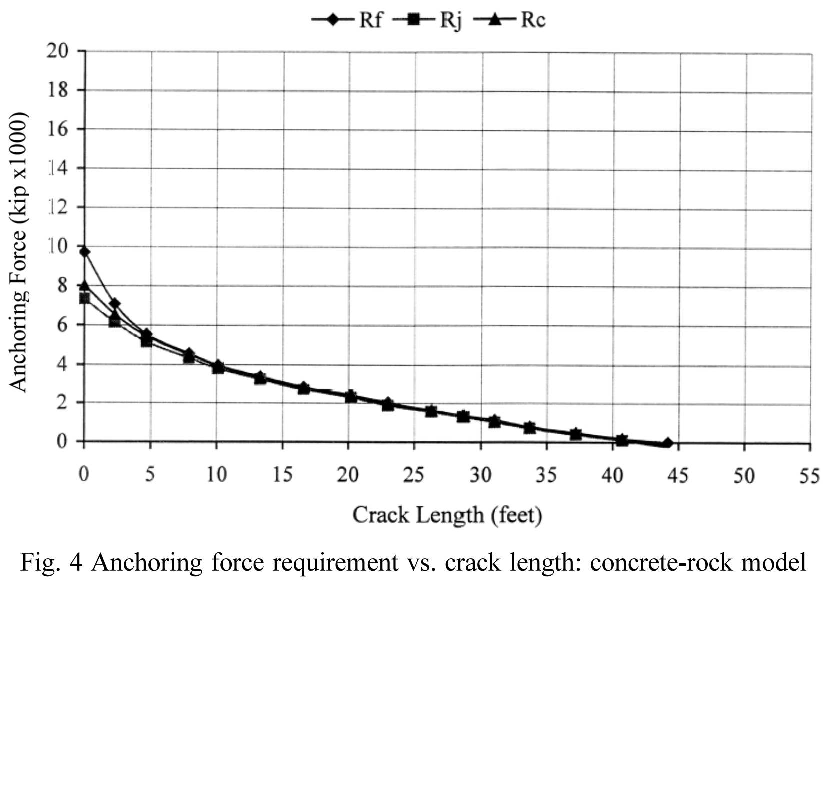

Fig. 4 Anchoring force requirement vs. crack length: concrete-rock model

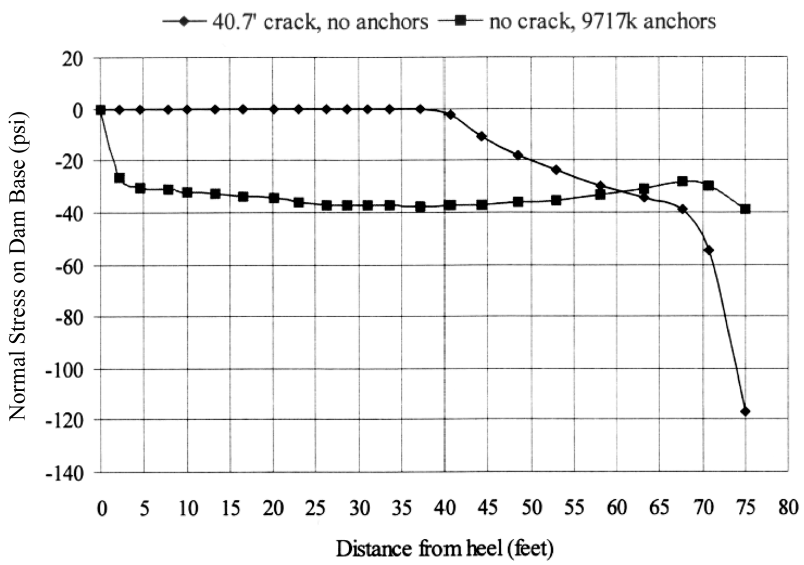

Fig. 5 Normal stress distribution on the base of the dam, with and without anchors: concrete-rock model

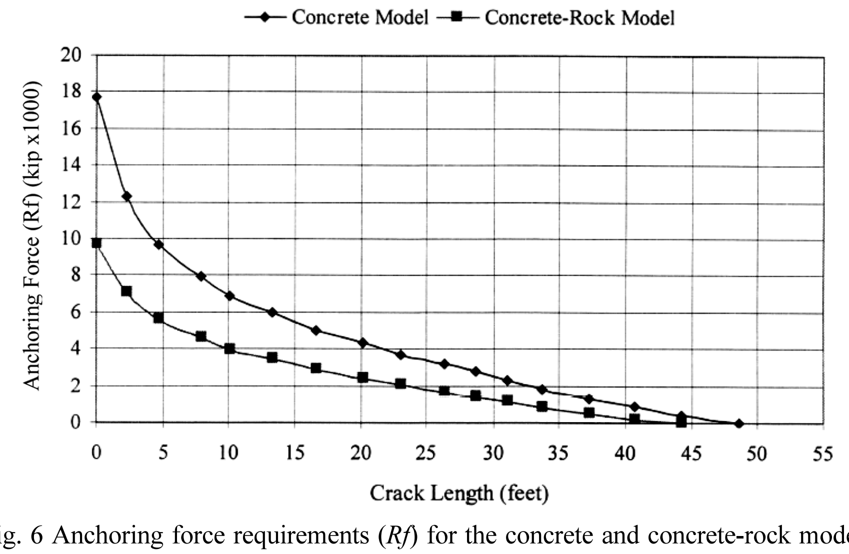

Fig. 6 Anchoring force requirements (*Rf*) for the concrete and concrete-rock models

tension in the member. For the particular case the crack length closed to 40.7 ft or 54% of the base and the factor of safety increased to 1.25. Closing of the crack increases the provided cohesion by the base and reduces the uplift pressure.

The distribution of normal effective stresses at the base of the dam is presented in Fig. 5 for the case of no uplift and 9720 kip anchor force and for 40.7 ft of uplift and no anchoring force. For the first case, there is an almost constant pressure throughout the base as opposed to the conventional-method results of linear variation of normal stress with zero at the heel and maximum compression at the toe. For the second case, normal stresses exist only at the contact length beyond 40.7 ft from the heel. The stress variation is linear with maximum compression of nearly 120 psi at the toe. Both stresses are small compared to the compressive and bearing strengths of concrete and rock.

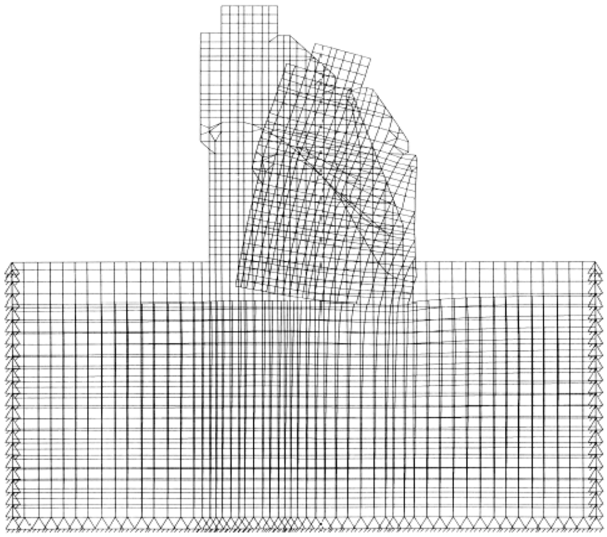

Fig. 7 Deflected shape: concrete-rock model

3.3 Comparison of results from the concrete and concrete-rock models

Fig. 6 compares the anchoring force requirements for the concrete and the concrete rock models. The anchor requirements differ substantially at small crack lengths and reduce considerably with increasing crack length allowance. The difference decreases from 8020 kip at zero crack, to 3030 kip at 7.9 ft or 10% crack and to 1630 kip at 23 ft or 30% crack. Percentwise this difference amounts to 45% irrespective of the crack length.

4. Conclusions

The paper has presented the methodology and discussed various parameters related to finite element modeling, and stability and stress analysis of new and existing dams comparing the results with the corresponding code accepted procedure termed the conventional equilibrium or gravity method. Important parameters examined include the presence/absence of the deformable dam base medium, failure criteria at the concrete dam-rock base interface as developed during the course of the study and progressive dam base sliding and uplift modeling techniques.

The conventional method is a reliable method for predicting the anchoring force requirements and stress distributions in tall and slender dams where flexural deformations are prevalent. As dam sections start to deviate from a predominantly flexural behavior and shearing deformations become more significant, the method becomes increasingly insufficient in predicting accurately the dam behavior and more reliable analytical methods must be used.

The anchoring force requirements in the conventional method call for complete contact of the concrete dam at its base thereby implying no allowance for tension to develop along the entire base. This condition imposes a severe restriction in the finite element analysis leading to increased anchor force requirements in order to retain both a straight base and a complete base contact with the foundation. The force demands are shown to be reduced considerably when the foundation rock is included in the model as the restriction for straight base is relaxed by the inclusion in the model of the rock foundation, but the complete base contact requirement is retained. In essence, the increased force demand by the finite element method in this particular analysis is not an overestimation but an actual force requirement imposed by the no-tension restriction.

Relaxing the requirement for infinitely stiff base through the inclusion in the model of the deformable foundation medium and allowing for crack opening and/or sliding to take place at the dam base through the developed failure criteria, has been shown to reduce considerably the magnitude of the required anchoring force. Both considerations have been shown to be very essential in modeling the movement that occurs at the dam base. The cracked base imposes local failure criteria to enforce local equilibrium and redistribution of forces in attempting to achieve a final equilibrium state.

The proposed finite element modeling methodology and failure criteria at the dam base represent a simple procedure yet a much more accurate method than the conventional method in predicting the dam behavior. The method can be used to assess the stability and state of stress of existing dams or to calculate the anchoring force requirements at any desired factor of safety in existing or new dams.

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