FINITE ELEMENT MODELLING AND DYNAMIC ANALYSIS OF CONCRETE GRAVITY DAM USING SAP-2000

A DISSERTATION SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE OF

MASTER OF ENGINEERING

IN

CIVIL ENGINEERING
with specialization in

STRUCTURAL ENGINEERING

SUBMITTED BY
NIDHI GUPTA
(Roll NO. 8807)

UNDER THE ESTEEMED GUIDANCE
OF
Prof. A. TRIVEDI
&
Asst. Prof. A. K. GUPTA

DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING
DELHI COLLEGE OF ENGINEERING
UNIVERSITY OF DELHI
2004-2006

Department of Civil & Environmental Engineering
Delhi College of Engineering,
Delhi-110042

CERTIFICATE

This is to certify that the project entitled **"Finite Element Modeling and Dynamic Analysis of Concrete Gravity Dam using SAP-2000"**, which is being submitted by **Nidhi Gupta,** is a bonafide record of the student's own work carried by her under our guidance and supervision in partial fulfillment of requirement for the award of the Degree of **Master of Engineering in Structural Engineering, Department of Civil and Environmental Engineering, Delhi College of Engineering, Delhi, University of Delhi.**

The matter embodied in this project has not been submitted for the award of any other degree.

Dr. A. Trivedi (Professor)
Mr. A. K. Gupta (Asst. Professor)
Department of Civil & Environmental Engineering
University of Delhi

Acknowledgements

I would like to avail this opportunity to humbly thank and appreciate the contribution and support, which various individuals have provided for the successful completion of this dissertation. My heartfelt thanks to my mentor and guide **Dr. A. Trivedi**, Professor, Department of Civil and Environmental Engineering, Delhi College of Engineering, who provided me an opportunity to work under his able guidance. His valuable guidance and motivation has helped me to complete my project successfully.

I wish to express my deep sense of gratitude to my co-guide **Mr. A. K. Gupta**, Assistant Professor, Department of Civil and Environmental Engineering, Delhi College of Engineering, for his enthusiastic support throughout the study. His keen interest and guidance were the source of inspiration for the successful completion of the project.

I also express my grateful thanks to **Prof. P.B. Sharma**, Principal, **Dr. Mrs. P R Bose**, Professor & Head, and all the Faculty Members of Civil & Environmental Engineering Department, Delhi College of Engineering, for their valuable suggestions and help, wherever needed.

Thanks is too small a word to acknowledge the contribution of my friends, **Anshu Dev, Nupur Das, Om Sehgal, and Purnima Bajpai** in my dissertation.

I would also like to extend my sincere appreciation to **Mrs. Beena Dilip** and **Ms. Kanika Bharadwaj** for being my sounding boards throughout this project and providing valuable comments and thoughtful criticism.

I am indebted to my **parents and family** for their constant encouragement, love and prayers. Lastly, I thank Almighty GOD for his countless blessings.

Nidhi Gupta

ABSTRACT

A concrete dam has been defined as a structure which is designed in such a way that its own weight resists the external forces. These dams can be constructed with ease on any dam site, where there exists a natural foundation strong enough to bear the enormous weight of the structure. Compressive strength of concrete has an important role in design of concrete gravity dams. However, the tensile stress of concrete may go above its maximum capacity especially during severe earthquakes, which cause cracks to occur at dams. The upper part of the dam, where the slope of the dam changes abruptly, especially on the downstream face, is critical and is vulnerable to tensile cracking during an earthquake. The effect will be even more pronounced owing to the fact that the fluid cavitations is found to occur in the upper part of the reservoir. The increase of the tensile stress at the upstream face of the dams causes erosion on the concrete lining of the dam surface. As a result, it causes the cross-section of the critical region to weaken thereby causing cracks, which can lead to potential instability from sliding or overturning.

No major concrete dam is known to have failed due to earthquake-induced ground motion, although several are known to have experienced damage due to strong ground motion. Most of these dams were usually designed using pseudo-static methods, ignoring the dynamic characteristics of the structure as well as the characteristics of the ground motion. Therefore, it is important to carry out detailed investigations of seismic behaviour of dam in order to assess and evaluate the safety of existing dams and improve the design guidelines for moderately high dams to be constructed in the future.

Concrete dams are very complex structures. The dynamic interactions among various components of the structure must be considered when developing the failure modes and the analysis models. In particular,

- interaction effects of the structure with its foundation and
- the impounded, surrounding, or contained water should be included in the analysis.

In the present work,

- Dynamic analysis of Concrete Gravity Dam is studied by using two dimensional finite element modeling technique in SAP 2000 software.
- Dam-Water interaction and Dam-Foundation interaction is taken into consideration in the dynamic analysis to find the response of dam during the earthquake and to visualize the behavior of dam on to the screen.

Finite element model is generated in SAP 2000software. The combined response of static and dynamic loads for two different time histories which are Design accelerogram and Koyna accelerogram is obtained.

Effect of Dam-Water interaction on response of dam during earthquake is studied by the comparison of the results of empty and full reservoir conditions. And the effect of Dam-foundation interaction on the response of dam during earthquake is studied by comparison of the results obtained from model obtained by considering foundation portion into dynamic analysis with the results obtained by without considering foundation and considering dam with fixed base.

Report EQ_81-23 "Analysis of Tallest Overflow Section of Navagam Dam"**,** which is done by IIT Roorkey in 1981, is taken as case study. All the results obtained after the dynamic analysis using SAP2000 are then compared with those in the report mentioned above.

NOMENCLATURE

α	Design horizontal seismic coefficient
σ_{11}	Normal stress in x direction
σ_{12}	Transverse shear stress in x-y direction
Υ_{12}	Transverse shear strain in x-y direction
σ_{13}	Transverse shear stress in x-z direction
Υ_{13}	Transverse shear strain in x-z direction
σ_{22}	Normal stress in y direction
σ_{33}	Normal stress in z direction
ϕ_n	Eigen vector
Δt	Time interval
C	Damping matrix
C_m	Maximum value of C_s
C_n	Modal damping matrix
C_s	Coefficient which varies with depth and shape of dam.
h	Depth of reservoir
K	Stiffness matrix
K_n	Modal stiffness matrix
m	Mass matrix
M_n	Modal mass matrix
n	Time steps
Ø	Inclination of upstream face of dam with vertical
p	Hydrodynamic pressure at depth y
p(t)	Load vector
P_n	Modal load matrix
T	Time duration
u(t)	Displacement vector
ü(t)	Acceleration vector
ů(t)	Velocity vector
W	Unit weight of water
y	Depth below surface
$Y_n(t)$	Modal response of n mode
ε_{33}	Normal strain in x direction
ε_{33}	Normal strain in y direction
ε_{33}	Normal stress in z direction
ξ_n	Modal damping ratio
ω_{Dn}	Damped natural frequency
ωn	Undamped natural frequency

CHAPTER 1

INTRODUCTION

1.1 General

A dynamic analysis determines the structural response based on the characteristic of structure and the nature of earthquake forces but dynamic or seismic analysis of dam is itself a dynamic process-in the sense that this analysis has to be constantly kept in the process of critical review and updating. It is never a one-time affair. The concept of dynamic forces as well as the response of structure like dam couple with the reservoir water and foundation rock certainly need continuous rethinking. This is because several structures have been subjected to kinds and magnitudes of such forces in their lifetime, which were never thought, would exist at the time of their design.

Gravity dams which were built before 1970, they were analyzed by traditional methods with simplified assumption and with nominal provisions for earthquake related forces. The interaction of dam-water-foundation as a system was not known. After inventions of Finite Element modeling and high-speed computer and improvement of knowledge regarding dam-water-foundation interaction, dynamic analysis is now carried out by this realistic seismic safety analysis and old dams were strengthened by this approach.

1.2 Dams and Earthquake

Although many Dams have failed throughout history, very few of these failures resulted from earthquake, and none involved major concrete dams [10]. However, the earthquake induced damage to the Koyna Dam (Asgher Vatanani Oskouei and A.A. Dumanoglu, November 2000; Chopra A.K., 1976; Chopra, A.K., and P. Chakrabarti, 1971)], near Poona, India, in 1967 and the Hsinfengkiang, Dam, near Canton, People's Republic of China, in 1962 indicated that concrete dams are not immune to earthquake damage as had commonly been presumed. During this century, increasing numbers of concrete dams have been constructed, and more are expected to be built, in seismic regions. Sooner or later these will be exposed to major earthquake in addition to the usual sources of potential damage. Because millions of people live in the floodplains downstream of these dams, it is essential that increasing attention be given to the earthquake safety of these structures.

The ability to evaluate the effects of earthquake ground motion on concrete dams is essential in order to assess the safety of existing dams, to determine the adequacy of modification planned to improve old dams, and to evaluate proposed designs for new dams to be constructed. The prediction of the performance of concrete dams during earthquake is one of the more complex and challenging problems in structural dynamics. The following factors contribute to this complexity:

- Dams and reservoirs are of complicated shapes, as dictated by the natural topography of the site.
- The response of dams may be influenced significantly by variation in the intensity and characteristics of the ground motion over the width and height of the canyon. However, for lake of appropriate instrument records, the spatial variation of the ground motion cannot be defined with confidence at this time.
- The response of a dam is influenced, generally to a significant degree, by the earthquake-induced motion of the impounded water; by the deformability of the foundation rock, which invariably is fragmented by joints and fissures; and by the interaction of the motions of the water, foundation rock, and the dam itself.

1.3 Methods of Analysis

1.3.1 General.

For a concrete dam on a rock foundation with no major known structural deficiencies, such as significant cracks in the structure, major weak joints, or adversely oriented discontinuities in the foundation, stability during an earthquake loading based on previous case histories and/or analyses should be satisfactory if all the following conditions are satisfied:

- The dam is well-constructed (quality concrete and lift joints) and is in good condition.
- Peak bedrock accelerations are 0.2g or less.

- The resultant location for a gravity dam under static conditions is within the middle 1/3 of the base and the factor of safety against sliding is acceptable for static conditions.

If these conditions are not satisfied, the methods of dynamic analysis currently being applied for various levels of studies and types of dams, in the order of complexity, are linear-elastic response spectrum, linear-elastic time-history, and non-linear time-history methods.

1.3.2 Dynamic Analysis Methods

Dynamic analysis methods are those that determine the structural response based on the dynamic characteristics of the structure and the earthquake ground motions established for the site. The dynamic method most commonly used is the linear elastic modal superposition analysis method. This method is based on the fact that, for certain forms of damping that are reasonable models for many structures, the response in each natural mode of vibration can be computed independently of the others and the modal responses can be combined to determine the total response. Either a response spectrum or an acceleration-time record can be used with the analysis technique. Time-history responses can also be calculated using step-by-step direct integration.

1.3.2.1 Response Spectrum Method

(a) Description

In this method, the maximum response in each mode of vibration is directly computed from the design response spectrum and the dynamic characteristics of the structure. These modal responses are then combined to obtain estimates (but not the exact value) of the maximum modal response. Adding the absolute values of each mode gives an upper limit for the total response, while using the square root of the sum of the squares yields a more probable total response. Either a simple beam analysis (P/A +/- Mc/I) or finite element analysis can be used to compute the stresses. Dynamic stresses are combined with the static stress to obtain the total stress. The response spectrum method does not account for stress variation with time. See U. S. Army Corps of Engineers (USACE 1999) for guidance on the use of the response spectrum method for concrete dams.

(b) Evaluation of Response

In most cases, when the dynamic response of the structure is entirely within the linear elastic range, the response spectrum analysis will suffice. However, in the case of earthquakes for which the calculated stresses for mass concrete structures may approach or exceed the tensile strength of the concrete, a time-history linear dynamic analysis provides valuable information for the approximation of the potential damage, or the expected inelastic response behavior that may occur and the potential need for a nonlinear analysis.

1.3.2.2 Time-History Method

(a) Linear-elastic Analysis

In this method, the response of the dam is calculated for the entire duration of an acceleration time-history, starting with initial static conditions and computing the response at the end of each time interval. The stresses and/or deflections of each mode are added together for each time interval to yield the total time-history response. The dynamic responses are added algebraically to the static responses. Stress histories are developed to show the maximums, the number and duration of excursions beyond the tension or compression limits, and the areal distribution of stresses in the dam at particular times during the earthquake.

(b) Evaluation of Response

For major earthquakes occurring close to some concrete dams, it is probable that the elastic capacity of the mass concrete would be exceeded, and some damage or yielding could occur. For these cases, the prediction of the actual response and estimation of the expected damage and inelastic behavior of the dam can only be judged in a linear analysis. Evaluating actual damage and failure requires a more involved nonlinear analysis. However, linear analysis can still be very valuable for a preliminary assessment of the damage and the level of post-elastic response, and can aid in the decision of whether a nonlinear analysis should be performed. As part of this evaluation, the results of a linear analysis for hydraulic structures should be examined in a systematic manner to identify the extent of over-stressed regions at any particular point in time. This will aid in the production of plots to show time-histories of stresses and other response quantities of interest, and to establish statistics on the number of stress cycles exceeding the allowable values and the corresponding excursions of these stress cycles beyond the specified limits. Minor, local damages, indicated from an elastic analysis, would have little effect on the overall integrity of the structure and can still be evaluated by proper interpretation of the results of linear analyses. An alternative evaluation technique that has been used by some agencies to evaluate a structure indicating distress using a linear elastic analysis is to assume cracking along the total width of the plane being investigated and evaluate structural stability in the post earthquake (static) condition. Stability during the earthquake would also have to be checked or assured if cracking took place prior to the conclusion of the event.

(c) Nonlinear Analysis

A complete nonlinear analysis should take into account all sources of nonlinearity that contribute significantly to the nonlinear behavior. The damage caused by earthquake shaking is normally associated with significant loss in the structural stiffness, a result of concrete cracking, yielding of steel, opening of contraction joints, slippage across the construction joints or cracking planes, and nonlinear material behavior. Additional sources of nonlinearity arise from the nonlinear response of the foundation supporting the structure, as well as the separation of the structure and the foundation at the contact surface. A complete and reliable nonlinear dynamic analysis that includes tensile cracking of concrete, yielding of reinforcements, opening of joints, and foundation/abutment displacements is becoming more practical. Having a complete and reliable nonlinear analysis for the seismic safety evaluation of dams depends on continuing developments in the following areas:

- Definition of spatially varying seismic input.
- Energy absorption factors for reservoir sides and bottom and at infinite boundaries at the reservoir and foundation extents of the models.
- Boundary identification and specification of significant nonlinear mechanisms (joint opening, tensile cracking, steel yielding, nonlinear material behavior under dynamic loads, etc.).

- Development of idealized models representing the nonlinear behavior (contact surfaces for modeling contraction joints and sliding along discontinuities, and validated constitutive models for concrete cracking)
- Development of efficient and numerical sound techniques and solution strategies for computing the nonlinear response.
- Development of criteria for acceptable performance.
- Identification of possible modes of failure.

1.3.2.3 Foundation Stability in Dynamic Analysis

(a) General

A dynamic stability analysis should generally be performed on the same foundation blocks and wedges that were identified and used in the static stability analysis. The same methods of foundation analysis used for static stability calculations are used for dynamic evaluations with the following additions, modifications, and exceptions:

- The analysis must include dynamic loads from both the dam acting on the foundation blocks and the inertial loads from the blocks themselves.
- The inertial load from the mass of the foundation block is included. If a time-history analysis is being performed, the load is equal to the mass multiplied by the acceleration at any instant of time. If a response spectrum analysis is being performed, appropriate effective peak acceleration is used.
- If the analysis indicates that the sliding resistance on the potential sliding plane (frictional resistance plus intact rock cohesion) is exceeded at some point during the earthquake loading, a strength reduction (typically reduction or elimination of cohesion) is necessary. Dynamic strengths of the intact rock and/or concrete must also be accounted for in making the analysis.
- Deformations can be computed for the foundation blocks by a Newmark-type rigid-block sliding analysis, and the dam's performance can be evaluated by comparison with acceptable-deformation criteria. This approach is especially valuable for cases where blocks can be assumed to be formed by continuous joints, faults, and shear zones. For atypical dams, commonly-used criteria may not be applicable, and there can be separate issues for appurtenant features.

- Research has shown that dynamic loading can affect water pressures in joints and shear zones. At this time, however, water pressure adjustments to account for that are not routine in practice.

(b) Seismic Stability Criteria

Seismic stability criteria for foundations depend on the method of analysis and the basic assumptions made for the geologic discontinuities. The three general cases are:

- A specified minimum factor of safety in a pseudo-static analysis at the time of critical dynamic loading from the dam, in which the rock is considered to be intact.
- A specified minimum factor in pseudo-static analysis of blocks assumed to be bounded by fully continuous joints, shears, or faults.
- Acceptable deformations calculated by time-history analysis with or without consideration of the strength of intact rock, in which sliding during excursions are evaluated below a factor of safety less than 1.0 (if any).

(c) Coupled dam and foundation analysis

Typically, the foundation stability analysis is performed separately from the dam analysis, as discussed above. A more realistic approach is to analyze the dam and foundation together in a coupled analysis. This way, the interaction between the dam and the rock wedges in question can be modeled. Because a coupled analysis can model non-linearity caused by changes in geometry, explicit coupled methods can be very effective.

1.3.2.4 Material Properties for Dynamic Analysis

The ability to analyze a dam often exceeds the ability to define material properties. This is particularly true for nonlinear analysis where many times the material models in the finite element codes have theoretical inputs that can only be estimated and not tested. Concrete and rock are not homogeneous materials. The material properties can be quite variable and possess a degree of uncertainty. This should be kept in mind when performing investigations. Sensitivity studies using variations on material properties should be performed where appropriate.

(a) Concrete Properties for Analysis

The concrete properties required for input into a linear dynamic analysis are the unit weight, Young's modulus of elasticity, damping, and Poisson's ratio. The concrete properties used should account for, as nearly as practicable, the effects of aging and existing cracks and the expected rate of loading.

(b) Concrete Strengths for Evaluation of Results

The concrete properties needed to perform dynamic analysis are the compressive, tensile, and shear strengths. The standard unconfined uniaxial compression test, excluding creep effects, is generally acceptable as a first test for the compressive strength. Usually, this test suffices, even though it does not account for the rate of loading, since compressive generally does not control the outcome of the analysis. The tensile strength of concrete generally governs because it is a small fraction of the compressive strength.

(c) Concrete Tensile Strength

Tensile strengths of the lift joints and the parent concrete are determined from tests on cores taken from test fill placements for new dams and the actual concrete in existing dams. The modulus-of-rupture test and the splitting tensile test are commonly used for the parent concrete. Direct tensile tests are preferred, but less-expensive splitting tensile tests can be used, with the results adjusted to reflect the results of a smaller number of direct tensile tests. The direct tensile test is also used to measure the strength of the lift lines. From these tests, allowable design tensile stresses can be established for both lift joints and parent concrete can be adjusted using factors to account for the high strain rate associated with dynamic loading. However, the best approach is to actually test the strength of the parent concrete and lift lines under rapid loading. Tensile and bond strengths of concrete dams are as much related to mix consistency, placement methods, and compaction procedures as they are to mix proportions. Maximum aggregate size, consistency, and mortar bedding also influence tensile strength of the joints. The effects of pre-existing cracks, lift joints, and construction joints on the tensile strength must also be accounted for, whether assessing the results of linear-elastic analysis or performing nonlinear analysis. If dynamic analyses indicate high tensile stresses, the effects of cracks and weakened joints can be modeled conservatively by assuming that cracking occurs, and reanalyzing the dam to determine whether the remaining capacity of the dam in the cracked state is sufficient.

(d) Rock Properties for Analysis

When the foundation is included in the seismic analysis, elastic moduli and Poisson's ratios for the foundation materials are required. If the foundation is not considered as massless, rock densities and damping characteristics are also required. Determining the elastic moduli for a rock foundation may be accomplished using one of several approaches that account for the effects of rock inhomogeneity and discontinuities on foundation behavior. The determination of foundation compressibility should consider both elastic and inelastic (plastic) deformations. The resulting "modulus of deformation" is lower than the elastic modulus of intact rock. The effects of the rate of loading on the foundation moduli is considered insignificant relative to the other uncertainties involved in determining rock foundation properties; rate effects are generally not measured. To account for the uncertainties, lower and upper limits for the foundation moduli should be used in the structural analysis to determine the sensitivity of the results to variation in moduli. For dynamic loading, the upper limit is generally the most realistic estimate because the inelastic "set" has already occurred. However, the shear strength used for static analysis is usually used for dynamic analysis without adjustment. The shear strength of discontinuities is generally a critical element in the stability analysis.

(e) Damping Ratio

For concrete gravity dams built on competent rock, where cracking of the concrete does not occur, the viscous damping ratio is usually assumed to be 5 percent of critical. If calculated stresses indicate that cracking would occur, a value from 7 to 10 percent of critical can be used (depending on the severity of cracking expected). Some analysis codes use hysteretic damping and Rayleigh damping, which require different values specific to the form of damping. It may also be necessary to account for radiation damping and the effects of reservoir bottom reflection (Nuss et al 2003, Ghanaat 1995).

1.3.3 Sliding Stability

1.3.3.1 Methods of Analysis

The sliding stability evaluation of hydraulic structures under earthquake loading can be made according to the traditional pseudostatic (seismic coefficient) and permanent displacement approaches (Ebeling and Morrison 1992). In the traditional approach, the sliding stability is expressed in terms of a prescribed factor of safety against sliding, whereas in the permanent displacement approach, the structure is permitted to slide along its base but the accumulated displacement during the ground shaking is limited to a specified allowable value. Some federal agencies have encouraged the use of conducting a post earthquake stability analysis to demonstrate seismic stability. Under this type of analysis, conservative assumptions are used to reduce the amount of information needed of the ground motions, and the exploration and testing required of the dam/foundation material properties. For example, a typical analysis would assume the seismic event has created a crack that extends entirely through the base of the dam. Sliding along deep seated failure surface in the foundation should also be considered or ruled out. An analysis is then conducted, assuming no cohesion and a reduced friction angle, to determine if the dam has an adequate post earthquake factor of safety or if remediation or additional analysis is warranted.

1.3.3.2 Pseudostatic (Seismic Coefficient) Method

In this method, seismic inertia forces due to weight of the structure and to hydrodynamic pressures are included as part of the driving force, and a static analysis of sliding is performed (Ebeling and Morrison 1992). Treating the system above the failure surface as a rigid block, the inertia force associated with the mass of the structure is computed as the product of the assumed earthquake acceleration (seismic coefficient x g) and the mass of the block. Similarly, the product of the earthquake acceleration and the added-mass of water that is moving with the structure, according to the theory of Westergaard (1933), produce inertia force due to the hydrodynamic pressures. The motion of the structure relative to the failure surface is resisted by the shear strength mobilized between the structure and failure surface by the friction and cohesion. When the earthquake forces are included in the sliding stability analysis, the calculated factor of safety against sliding (i.e., ratio of the resisting to driving shear forces) may become less than one. When this occurs, the sliding is assumed to occur for as long as the ground acceleration is greater than the critical value required for the driving force to exceed the resistance. However, due to oscillatory nature of the earthquake ground motion, the sliding displacement is limited and can be estimated by the permanent displacement approach described below.

1.3.3.3 Permanent Displacement Method

The factor of safety against sliding required by the pseudostatic method (e.g., >1.0-1.5 at all times) may not be attainable for larger seismic forces representative of moderate to high intensity earthquake ground motions. Where the sliding factor of safety drops below 1.0, sliding can occur during periods of time when the acceleration cycles exceed a critical acceleration period of time associated with the acceleration cycles exceeding a critical acceleration, a_c, and the relative velocity between the structure and the base is greater than zero (Chopra and Zhang 1991). Knowing the critical acceleration a_c, the permanent sliding displacements can be estimated using Newmark's rigid block model (Newmark 1965). Newmark's model provides an easy means for approximate estimation of the upper bounds for permanent sliding displacements, but is based on certain assumptions that ignore the true dynamic response behavior of the sliding. More accurate estimates of the sliding displacements can be made from the response history analysis proposed by Chopra and Zhang as referenced above.

1.3.4 Rotational Stability

Hydraulic structures subjected to large lateral forces produced by major earthquakes may tip and start rocking when the resulting driving moment becomes so large that the structure breaks contact with the foundation or cracks all the way through at an upper elevation. However, under earthquake excitation large driving moments occur for only a fraction of a second in each cycle, with intermediate unloading. Stability studies of structures (Housner 1963, Scholl 1984) infer that dynamic rotational stability for rigid structures with height to base width ratios typical of gravity dams is not a problem. In addition, the study by Chopra and Zhang (1991) indicate that a concrete dam under earthquake loading will slide downstream before tipping will occur. U. S. Army Corps of Engineers (USACE 1999) discusses each of the above references. However, if the dynamic analysis indicates that extensive cracking, block rocking, or translational displacement is expected, the dam in its post-earthquake condition should be checked for sliding stability to assess if it is capable of containing the reservoir for a sufficient period of time to allow for strengthening of the dam, if necessary, to restore the dam to acceptable factors of safety for the various credible loading conditions. The post-earthquake analysis should use residual shear strength parameters.

1.4 Scope of Work

In the present work, Navagam Dam built over the river Narmada has been taken as a case study and a dynamic analysis of the dam is carried out by using two-dimensional finite element technique. and Report EQ _81-23 "Analysis of Tallest Overflow Section of Navagam Dam", which is done by IIT Roorkee in 1981 is considered as case study.

Finite element model of concrete gravity dam is generated by using SAP 2000 software and results obtained from the dynamic analysis are compared with those in the report. The behavior of dam considering the effect of Dam-Foundation interaction and Dam-Water interaction is studied after that.

1.5 Organization of Report

This report has been divided into six chapters:

- **Chapter 1** gives a general introduction, history of performance of concrete gravity dam during earthquake and scope of work.
- **Chapter 2** elaborates the literature review carried out to study the various aspects of concrete gravity dams and their analysis.
- **Chapter 3** describes the procedure for dynamic analysis, performance criteria for response of dam during earthquake, method of analysis for dam, effect of Dam-Foundation interaction and Dam-Water interaction on response of dam and Time history solution technique. and introduction of SAP 2000 software and plane element which is used for modeling of Dam.
- **Chapter 4** defines geometry, properties of materials, and loads of dam taken as case study, also described the procedure of making model of dam in SAP 2000.
- **Chapter 5** shows the comparison of results of static and dynamic analysis, response of dam for two different time histories and how Dam-Foundation and Dam-Water interaction influence the response of dam during earthquake.
- **Chapter 6** shows conclusion and future scope of this work.

CHAPTER 2
LITERATURE REVIEW

2.1 General

Concerns about the seismic safety of dams have been growing during the recent years, partly because the population at risk in locations downstream of major dams continues to expand and also because it is increasingly evident that the seismic design concepts in use at the time most existing dams were built were inadequate. Questions about the safety of concrete dams were first brought into focus by the failure of St. Francis Dam in California in 1928, which caused extensive property damage and the loss of over 400 lives (Commission on Engineering and Technical systems, 1990).

The hazard posed by large dams has been demonstrated since 1928 by the failure of many dams of all types and in many parts of the world. However, no failure of a concrete dam has resulted from earthquake excitation; in fact the only complete collapses of concrete dams have been due to failures in the foundation rock supporting the dams (USCOLD, 1985). On the other hand two significant instances of earthquake damage to concrete dams occurred in 1960: Hsinfengkiang in China and Koyna in India (UNESCO Committee of experts, 1968). The damage was severe enough in both cases to require major repairs and strengthening, but the reservoirs were not released, so there was no flooding damage. This excellent safety record, however, is not sufficient cause for complacency about the seismic safety of concrete dams, because no such dams has yet been subjected to maximum conceivable earthquake shaking while retaining a maximum reservoir. For this reason it is essential that all existing concrete dams in seismic regions, as well as new dams planned for such regions, be checked to determine that they will perform satisfactorily during the greatest earthquake shaking to which they might be subjected (U. S. Army Corps of Engineers, 1958).

Major factors that must be considered in verifying the seismic safety of existing or proposed concrete dams include definition of the expected seismic excitation and evaluation of the response of the structure to this input. Usually, the structural response is first calculated assuming that the dam is a linear system in which the displacements are directly proportional to the input excitation (Alarcon, P. N., 1975). To establish the ultimate resisting capacity, however damage mechanisms and the resulting non-linearities must also be considered. Observational evidence provides the best indication of the true performance of a dam in the non-linear response range; hence, one important part of earthquake engineering for concrete dams is the evaluation of data from dynamic tests-real earthquake response information as well as laboratory and field test data. Finally to ensure that proposed or existing concrete dams provide adequate safety, suitable performance criteria must be established against which the predicted earthquake performance of a dam can be judged.

2.2 Earthquake input

To evaluate the expected earthquake performance of a concrete dam, it is first necessary to establish the intensity and frequency characteristics of the earthquake motions to which the dam might be subjected (Chopra, A. K., 1970).

The first factor to be considered in the response analysis is how the prescribed earthquake motions are applied to the dam. In the early days of earthquake engineering it was always assumed that the structure was supported on a rigid base and that the earthquake input was applied as specified motions of this base. For light weight relatively small structures supported on hard foundation rock, this input assumption was acceptable. However, for massive stiff concrete dam structures that are supported by very broad foundation rock surfaces, it is not reasonable to assume that the base support system is rigid. Clearly this assumption would not be consistent with the fact that an earthquake is actually a vibration wave being propagated through the earth's crustal structure (US Bureau of reclamation, 1966).

The typical strong motion earthquake record portrays three components of the acceleration history observed at a single point, usually at the ground surface in a free field location. Analytical procedures have been developed that account for the differences between the specified free field earthquake motions and the motions of the points at which the structure is supported; however, a major difficulty in the analysis of the dam response is the fact that there may be significant variations between the motions to be expected in the free field at widely spaced points on the dam-foundation interface. Many theoretical studies have been mad in the spatial variation of seismic waves propagated to highly idealized dam-canyon interfaces, but there is almost no information on the seismic motions that have actually occurred at the boundaries of a real dam (Chopra, A. K., 1970). Hence, the planning and installation of strong motion accelerograph networks at the canyon sites of actual concrete dams and also at free field sites in canyons where concrete dams might be built anew among the urgent research needs in the general area of defining earthquake input for concrete dams.

2.3 Linear response analysis

In the earliest efforts to represent the effects of earthquake on dams, the dams were considered to be rigid systems supported on a rigid foundation. Thus, the earthquake forces acting in the structure could be expressed as the product of the earthquake acceleration and the mass of the corresponding part of the structure (Lotif, V., J. .M. Roesses, and J. L. Tassoulas, 1987). Most recently the elastic properties as well as the mass of the dam materials have been incorporated into the mathematical models, and as a result the vibration properties of dams (the mode shapes and frequencies) have been recognized to have a controlling influence on earthquake response behavior.

Because of the ability of the finite element method to define mathematical models with arbitrary geometries and variations of material properties, this method is usually adopted in formulating a mathematical model of a concrete dam. In the sense the seismic response analysis of a concrete dam may be considered to be similar to any other structural dynamic analysis. However, the concrete dam analysis is greatly complicated by the fact that the structure interacts with its environment during its dynamic response-with the water retained in the reservoir and with the deformable foundation rock that supports it.

These interaction mechanisms may be included in the model in a crude way by combining finite element mesh representing a limited extent of the reservoir water and foundation rock together with the model of the concrete dam (Fenves, G., and A. K. Chopra, 1984). However, it is recognized that dynamic pressure waves will be generated by the earthquake in the reservoir water, and similarly there will be stress waves in the foundation rock. The effect of these waves acting in the actual unbounded media will be to transmit energy away from the dam. Thus, a significant energy loss mechanism is not represented by the bounded finite element models of reservoir water and of foundation rock, and the development of mathematical models that properly account for these effects has become a major objective in research on seismic behavior of concrete dams (Lotif, V., J. .M. Roesses, and J. L. Tassoulas, 1987).

Gravity dams and their environment often can be idealized as simple two-dimensional systems, and for this reason interaction response analysis procedures were first developed for gravity dams; the most powerful analytical techniques a substructure formulation and a frequency-domain response analysis procedure. In these analysis, however, the foundation rock is still modeled as a bounded finite element mesh, and the elimination of this rock boundary constraint is important current research objective.

Another problem of present concern is the evaluation of the effective stiffness of typical reservoir bottom boundaries, which are generally covered by a layer of silt having indeterminate thickness and mechanical properties (Fenves, G., and A. K. Chopra, 1984). A field measurement program to obtain this information is urgently needed, because the bottom absorption directly affects the earthquake response stresses that are calculated in the dam.

2.4 Non Linear Response Analysis

Although linear response studies have provided great insight into the earthquake performance of concrete dams, it is evident that a rigorous estimate of seismic safety of a dam can be obtained only by non linear analysis if a significant amount of earthquake damage is expected. In fact, minor local damage have little effect on the global stiffness of a dam, and in such cases it is possible to make a reasonable estimate of the expected degree of damage by proper interpretation of the results of a linear response analysis ((Fenves, G., and A. K. Chopra, 1987). But for the cases of severe damage, in which a collapse condition may be approached, the dynamic behavior is drastically changed from linear response mechanism, and the true non-linear performance must be incorporated into the analysis procedure if a valid estimate of the damage is to be made.

A non linear response analysis requires considerably greater computational effort than a linear analysis, because a much more detailed description of the performance is required to express the non linear response mechanisms. Special techniques are needed to carry out such calculations, and continued effort must be directed towards improving the efficiency of non linear analysis procedures of concrete dams (Pal, N., 1976) However, the greatest impediment to effective non linear response analysis at present is not the computational procedures but is the lack of knowledge about the non linear properties of the mass concrete typically used in dams. It is thus essential that major programs of dynamic testing be carried out to determine material properties suitable for use in non linear seismic safety evaluations.

2.5 Observational evidence

All analytical predictions of earthquake performance of concrete dams are based on numerous assumptions, each of which has a limited range of validity. Even the design of shaking table experiments employing physical models of concrete dams require the introduction of limiting assumptions, with regard to both the nature of the simulated earthquake motions to be applied and the properties of the model structure and of the foundation rock in which it is placed (Niwa, A., and R. W. Clough, 1980). For these reasons the best evidence by far about the earthquake behavior of concrete dams is that obtained from real dams that have been subjected to real earthquakes.

The number of instances when severe earthquake motions have acted on major concrete dams is quite small, and only in two cases was significant damage caused by the earthquakes: Koyna Dam in India and Hsinfengkiang Dam in China (Chopra, A. K., and P. Chakrabarti, 1973) . In neither of these cases did the damage cause release of reservoir, so no disaster resulted in the downstream region; however, it must be recognized that no typical major concrete dam has been subjected to maximum credible earthquake excitation, so there is no cause for complacency about the seismic safety of concrete dams.

Because real earthquakes provide the best test of earthquake performance, it is essential that many more existing concrete dams be provided with adequate seismic instrumentation so that quantitative evidence can be obtained to be used in verifying safety evaluation procedures for concrete dams.

However, even if the input is well established, the degree of damage to be expected from severe earthquake excitation of a concrete dam cannot be predicted with certainty. Part of the uncertainty lies in the non linear physical properties of the concrete, and this information can be obtained only from comprehensive, well planned material test programs.

2.6 Seismic performance criteria

The design of earthquake-resistant concrete dams must be based on appropriate performance criteria-criteria that reflect both the desired level of safety and the nature of the design and analysis procedures. Before computers were used in design calculations, the usual criteria were merely checks on the static stability of dams; thus, they did not represent the true dynamic earthquake response behavior and did not provide any guarantee against seismic damage or collapse (Pal, N., 1976). Now with modern computer analysis procedures, it is possible to predict with some precision the linear elastic response that will result from a specified earthquake input; however, because of uncertainties in the characteristics of the expected earthquake, in the way the excitation is applied to the dam, in the dynamic strength properties of the concrete, and in the nature of the ultimate failure mechanism, it is still not possible to prescribe exactly the performance criteria to be used in the design of concrete dams. For this reason the criteria presently used in the seismic safety evaluation of existing concrete dams or in the design of new dams generally are simply stress checks in which the predicted elastic stress is compared with the expected concrete strength.

The seismic inputs in most criteria are the design basis earthquake (DBE) and the maximum credible earthquake (MCE). The DBE is defined as the greatest earthquake excitation expected to occur at least once during the life of the dam (possibly 100 years) (Fenves, G., and A. K. Chopra, 1984). It should cause no significant damage to the dam; thus, in the response to such an input it is appropriate to limit the maximum concrete stress to the strength of concrete applied with a factor of safety. In addition, the possibility of collapse may be related to the number of times and the total duration that the cracking stress threshold is exceeded.

Eventually, it is hoped that non-linear analysis capabilities will be improved to the point where the performance criteria can be stated in terms of acceptable amounts of nonlinear displacement; both the types of displacements and their location in the dam will influence the acceptable amplitude of displacement. However, significant advances will have to be made in the understanding of possible earthquake collapse mechanisms as well as in nonlinear analysis capabilities for concrete dams before such criteria may be proposed. Thus, it is evident that both analytical and experimental research on the nonlinear behavior of concrete dams continues to be of paramount importance (US Bureau of reclamation, 1966).

The seismic safety of dams is an issue that has been receiving increasing attention in many parts of the world during recent years. It has become a major factor in the planning of new dams proposed to be built in seismic regions and in the safety evaluation of existing dams in those areas. There are two main reasons for this increasing level of concern:

1) The risk of a major disaster due to dam collapse increases each year because population is increasing downstream of nearly all dams.
2) Knowledge of complex nature of the earthquake motions that may attack a dam is increasing rapidly as more earthquake records are obtained all over the world, and it is becoming apparent that the seismic safety precautions taken in the past did not fully recognize the hazard.

2.7 The hazard of dams

Dams probably were among the earliest major structures to be created by humans. The first dams undoubtedly were small earthen embankments that were designed by trial and error. However, the sizes of dams grew, leading to the increasing potential for disaster.

A notable example of dam failure that was responsible for imposition of controls on dam safety was the collapse of the St. Francis Dam, located about 45m north of Los Angeles, California. This concrete dam, 205 ft high by 700 ft long, and impounding a 35000 acre foot reservoir, is shown before and after collapse in figure 2.1. The resulting extensive property damage, and especially the loss of over 400 lives, clearly demonstrated the hazard presented by major dams. (UNESCO Committee of experts, 1968)

Example of performance of concrete dam during earthquake described below(Chopra A.K., 1976; Chopra, A.K., and P. Chakrabarti, 1971; Priscu. R., A Popivi, D. Stematiu, and C. Stere, 1985)

- Lower Crystal Spring Dam, a 38 m high gravity dam built of interlocking blocks has been subjected to San Francisco earthquake (M 8.3, 1906), withstood that earthquake without a single crack.
- A weir in California subjected to the Arvin Tehachapi earthquake (intensity IX, 1952). Three piers of the reinforced concrete bridge over the dam, as well as the sluice gates have been destroyed by the collapse of the upper portion of the slope.
- Blackbrook Dam in England, a 31m high was damaged due to consequence of an earthquake (intensity VI-II, 1957). The damage caused by the earthquake were visible cracks in the upper part of dam, displacement of the face over about 150 m, displacement of the downstream protection masonry, slight damage of the vaulting bridge over the spillway, as well as slight cracks in the foundation rock. the leakage through to the dam and foundation increased about 400 times, reaching 910 m^3/day.
- In 1967, the Koyna Dam, a 340-foot-high gravity dam in India, survived a near-field magnitude 6.4 earthquake, with an MM intensity of VIII or IX at the site. A peak horizontal acceleration of 0.51g perpendicular to the dam axis and a peak vertical acceleration of 0.36g were measured at the dam. The dam experienced major cracking on both upstream and downstream faces of the non-overflow monoliths. The overflow monoliths were not damaged. A dynamic analysis of the Koyna Dam was performed using two-dimensional finite element methods that incorporated hydrodynamic interaction and assumed linear elastic behavior in the concrete and foundation. The analysis indicated tensile stresses in the upper part of the nonoverflow monoliths exceeded the tensile strength of the concrete up to three times. Analysis of the

overflow monolith indicated maximum tensile stresses approximately equal to the concrete tensile strength.

- Pacoima Dam was strongly shaken by Northridge earthquake (M 6.7, 1994). Peak accelerations at the top of the abutment were again amplified by the local narrow ridge topography and shattered condition of the rock, and reached 1.76 g horizontal and 1.6 g vertical. The dam performed satisfactorily with a half-full reservoir. Minor horizontal cracking of concrete at the left end of the dam, and several minor horizontal and vertical block offsets occurred at the joints.
- Shih-Kang Dam in Chi-Chi Taiwan, a radial gated water supply dam was subjected to earthquake (M 7.6, 1999) at 50 km from epicenter. It failed due to differential thrust fault movement at its north abutment. Over two thirds of the dam body were uplifted about 9 m vertically, and displaced 2 m horizontally. Damage was confined to the two bays overlying the fault rupture.
- Several 200-foot-high gravity dams in Japan experienced earthquakes producing an MM intensity of VIII in the area of the dams and were not damaged. However, some examples exist of partial earthquake damage to concrete dams in Japan. In 1923, the piers of spillway gates at the top of a hydroelectric intake dam cracked at their bases. These piers were of plain concrete, indicating that reinforcing steel should be used in piers of dams built in areas where large earthquakes are anticipated. In 1943, a gravity dam for deposition of muck from a mine was sheared at a horizontal section at the elevation of approximately two-thirds up the dam height. However, the cross section of this dam was smaller than that of a water storage dam, probably because the pressure exerted by deposited muck was less than the water pressure.
- Hoover Dam, a 726-foot-high curved gravity dam, has been suspected of being the cause of moderate reservoir-triggered seismicity of Richter Magnitude M5 or less (USCOLD 1992). No damage to the dam has occurred due to these earthquakes.

In 1995, the concrete dams of Sengari and Aono, located 15 km and 30 km north of the earthquake source in Kobe, Japan, were shaken by the Great Hanshin earthquake of Richter magnitude M7.2, but suffered no damage (Krinitzsky et al 1995). These concrete dams were founded on rock and received relatively low levels of ground motion. The level of ground motion is based on the fact that there was little damage to residences with tile roofs and unstable grave markers in a cemetery nearer the closest dam were not overturned. Overturning of grave marker stones approximately 4-feet high with a 4-inch-square base indicated horizontal ground motion of approximately 0.2g. Peak rock acceleration within the source area in Kobe registered an equivalent 0.45g.

Two recent examples of dam disaster occurred with concrete thin archstructures-Malpasset in France and Vaiont in Italy. The Malpasset dam failed in a sudden burst on 2nd December, 1959, creating a huge wave that destroyed towns and villages downstream with tremendous loss of life. As was the case with the St. Franics Dam, the failure occurred not in the concrete structure but rather in a weak stem in the foundation rock supporting the dam.

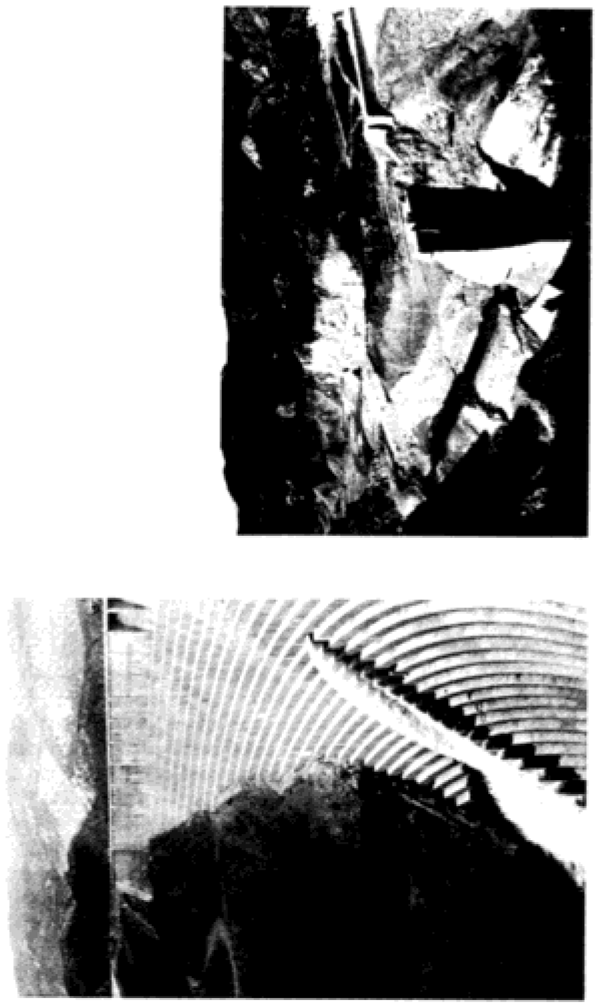


Fig 2.1 St. Francis Dam, 1928 (a) Before failure dam was 205 ft high and 700 ft long and retained 35000 acre of water, (b) the foundation failure caused dam rupture, and the reservoir release completely dam (ref: Commission on Engineering and Technical Systems, 1990)

The situation at the Vaiont disaster was entirely different in that the dam did not fail; it remained intact despite being severely overloaded when it was overtopped on 9th October, 1963, by a great wave reaching more than 300 ft over the crest. However the overtopping had the same effect on property and people downstream as if the dam had been breached; over 2000 fatalities resulted.

The failure of Teton Dam, built in Idaho, demonstrated that embankment dams too pose threat to population downstream, even when they are built by modern construction techniques.

None of the aforementioned catastrophes was related to earthquake activity, but several other events during the same time span have demonstrated the additional risk that applies to dams located in seismic regions. Two significant examples of earthquake damage to concrete dams were incidents observed at Hsinfengkiang Dam in China and at Koyna Dam in India. Hsinfengkiang Dam is located in Kwngdong province; it is 1144 ft long and 344 ft high and was completed in 1959. It was originally designed to resist ground shaking of intensity 6 on Modified Mercalli scale, but when numerous Hsinfengkiang Dam were experienced in the vicinity during and shortly after filling of the reservoir, the dam was strengthened to withstand earthquakes upto intensity 8. On 19th march, 1962, the dam was severely shaken by ground motions considerably above intensity 8, caused by an earthquake of magnitude 6.1 located very close to the dam. During the even more intense motion that occurred in the main shock, the dam developed horizontal cracks at the change of section in the non-overflow buttress blocks on each side of the spillway, but no leakage occurred. The dam was then strengthened to withstand even more intense ground motions (Chopra A.K., 1976; Chopra, A.K., and P. Chakrabarti, 1971)

Koyna Dam is a straight gravity structure built of rubble concrete in southwestern India. It is 2800 ft long and 338 ft high at the tallest block (Chopra, A. K., and P. Chakrabarti, 1973); it has an unconventional change of slope an the downstream face resulting from changes introduced while construction was in progress. It was originally intended to be built in two stages, but while the first stage of construction was in progress it was decided to build the dam to its final height in one stage. This change of plans necessitated modifications of dam cross section. On 11 December, 1967, a magnitude 6.5 earthquake occurred. Instrumentation within the dam recorded peak horizontal accelerations of .63 and .49 g in the cross channel and the stream directions, respectively, and horizontal cracks caused by these motions appeared on both the upstream and downstream faces of the tallest nonoverflow blocks (Chopra, A. K., and P. Chakrabarti, 1973). However, the reservoir was not released by this damage, even though minor leakage was observed. Thus, the cracking did not cause dam failure, and there was no flooding disaster downstream, although 180 people in the vicinity were killed by earthquake ground vibration effect. Subsequently, the dam was strengthened by the addition of buttresses on the downstream face of the non-overflow blocks.

2.8 Seismic safety of concrete dams

Possible failures for all types of causes were considered for concrete dams. For dams located in seismic regions it generally was concluded that further investigations of seismic safety were needed, due to advances in knowledge of the earthquake input as well as to better understand the dynamic response mechanisms. Accordingly, major reevaluations have been made of the seismic safety of many existing dams during the past decade. In addition, performance predictions have been made for the relatively small number of new dams built or proposed to be built in seismic regions during this time (Pal, N., 1976).

It must be noted that there is no record of any concrete dam ever having been damaged due to earthquake excitation to the extent that the reservoir was released. This record stands even though more than 100 concrete dams of all types have been subjected to measurable shaking due to earthquakes in many parts of the world(USCOLD, 1985). But it does not justify complacency, because in very few of these examples were measurements actually made of the intensity of shaking, and the severity of the tests generally is not known. Consequently, even though the earthquake safety record of concrete dams is excellent, much more research must be done if the details of their seismic behaviour are to be understood and predicted reliably.

Until two or three decades ago, the only consideration given to earthquakes in the design of concrete dams was to apply a static horizontal force specified as a fraction of the weight of the structure to represent the seismic design loading. In later stages of its use this equivalent static load design procedure often included an additional weight to represent the inertial resistance of the water behind the dam. In these analyses, a gravity dam was modeled as a simple cantilever column of varying cross section, but with the development of digital computers and the finite element method of analysis, it became possible to model the geometric configuration of the dam in a realistic way.

Subsequently, the analysis procedure was extended to treat the true dynamic character of the earthquake motions and the dynamic interaction of the dam with the reservoir water and foundation rock; then the aforementioned use of equivalent static loads was applied only in preliminary analyses. Present studies that take advantage of these advances in analytical procedures give greatly improved estimates of the seismic safety of existing or proposed concrete dams. However, uncertainties still remain in nearly every aspect of analyses, and an intensive continuing research effort is needed to reach the point where full confidence in the predictions of seismic safety will be attained.

2.9 Earthquake inputs

2.9.1 Earthquake excitation concepts

In evaluating the earthquake performance of concrete dams, it is evident that descriptions of the earthquake ground motions and the manner in which these motions excite dynamic response are of paramount importance. The procedure that leads to the selection of the seismic input for concrete dams is similar to that for large important structures such as nuclear power plants and long span bridges. It involves the study of the regional geologic setting, the history of seismic activity in the area, the geologic structure along the path from source to site, and the local geotechnical conditions. Ground motion parameters that may be utilized in characterizing the earthquake motions include peak acceleration, duration, and frequency content of the accelerogram. (USCOLD, 1985).

It is not surprising that the earliest method of defining earthquake input to concrete dams was merely to apply a distributed horizontal force amounting to a uniform specified fraction (typically 10%) of the weight of the dam body. This force was intended to represent the inertial resistance of a rigid dam subjected to the horizontal motion of a rigid foundation.

Major improvements over the rigid dam approach resulted when the dynamic effects of dam deformability, i.e., the free vibration behavior, were recognized.

- The first improvement was to convert the equivalent static force from a uniform distribution to a form related to the dam fundamental vibration mode shape.

- The second improvement was to account for the amplification effects by means of the earthquake response analysis.

The basic assumption of all these methods of analysis is that the foundation rock supporting the dam is rigid, so the specified earthquake motions are applied uniformly over the entire dam-foundation interface.

However, as the methods of response analysis improved, it became apparent that the base earthquake input no longer was appropriate. Because of the great extent of the dam, and recognizing the wave propagation mechanisms by which earthquake motions are propagated through the foundation rock, it is important to account for the spatial variations of the earthquake motions at the dam-foundation interface. The various input models are: (Fok, K. L., J. F. Hall, and A. K. Chopra. 1986)

2.9.2 Standard base input model

The dam is assumed to be supported by a rigid base boundary, as shown in the Fig2.2 the seismic input is defined as a history of motion of this rigid base, but it is important to note that the motions at this depth in the foundation rock are not the same as the free field motions recorded at ground surface.

2.9.3 Massless foundation rock model

An improved version of the preceding model is obtained by neglecting the mass of the rock in the deformable foundation region. This has the effect of eliminating wave propagation mechanisms in the deformable rock, so that motions prescribed at the rigid base are transmitted directly to the dam interface. With this assumption, it is reasonable to prescribe recorded free-field surface motions as rigid base input.

2.9.4 Deconvolution base rock input model

In this approach, as illustrated in Fig2.2, a deconvolution analysis is performed on a horizontally uniform layer of deformable rock to determine motions at the rigid base boundary that are consistent with the recorded free-field surface motions. The resulting rigid base motion is the standard base input model. It tends to be computationally expensive.

2.9.5 Free-field input model

A variation of the preceding procedure is to apply the deconvolved rigid base motion to a model of the deformable foundation rock without the dam in place, in order to determine the free-field motions at the interface positions where the dam is to be located. These calculated interface free-field motions account for the scattering effects of the topography on the earthquake waves and are used as input to the combined dam-foundation rock system.

Of these four models the free-field model is usually the most reasonable, so a key element in the input definition is the determination of appropriate free-field motions.

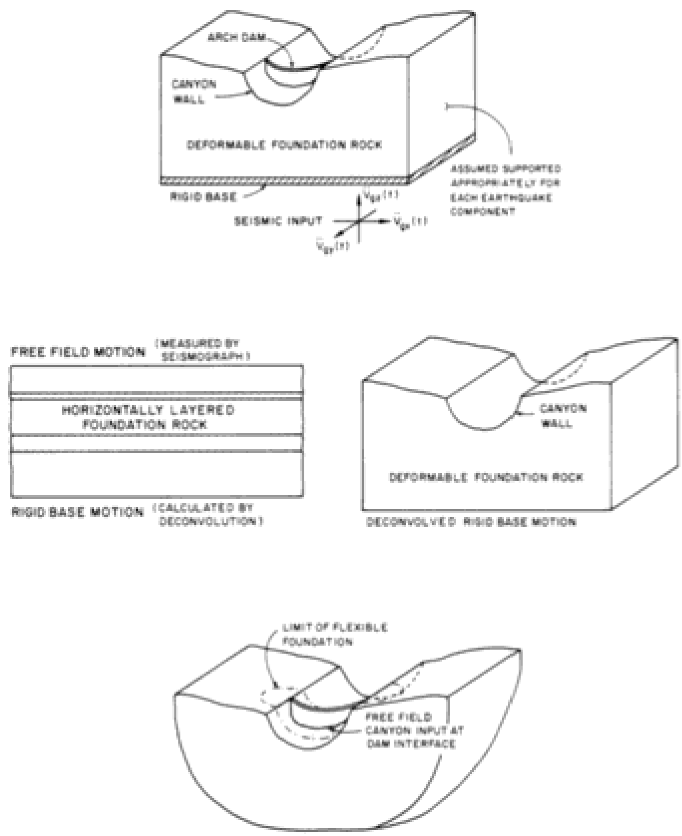


Fig2.2 Proposed seismic input models for concrete dams (a) standard rigid base input model; mass of foundation rock either included or neglected. (b) Deconvolution of free-field surface motions to determine rigid base motions. (c) Analysis of two-dimensional free-field motions to determine rigid base motions as input. (d) Analysis of three-dimensional dam-foundation system response using two-dimensional free-field canyon motions as input. (ref: Commission on Engineering and Technical Systems, 1990)

2.10 Present status of knowledge

2.10.1 Prediction of free-field motion

Definition of seismic input is very closely related to the way the dam-reservoir-foundation system is modeled. Although, existing finite element programs for dynamic analysis of concrete dams use uniform base motion as input, these programs can be modified to accept nonuniform earthquake excitation at the interface between dam and canyon wall. In this case the free-field motion is defined as the motion of the dam-foundation contact surface due to seismic excitation without the presence of the dam. (Fok, K. L., J. F. Hall, and A. K. Chopra. 1986).

2.10.2 Two-dimensional case method

If the canyons where the dam is to be located has an essentially constant cross section fro some distance upstream and downstream, it may be treated as linearly elastic half plane, and the problem of evaluating the earthquake free-field motion can be formulated as a wave-scattering problem with the canyon as the scatter. Various approaches have been used to obtain solutions to the problem. Direct numerical solution using a finite difference formulation has been employed to evaluate the scattering effects of various surface irregularities. (Trifunac, M. D. , 1973).

The theoretical free-field motions have limitations in their application due to the various simplifying assumptions made in their derivations. In the two-dimensional analyses it was assumed that the change of topography along the upstream-downstream direction was negligible; therefore, the results are valid only for prismatically shaped canyons. Moreover, the results apply only to specific wave types, and the amplification effect of wave scattering is very much dependent on the type of incident wave.

Often complicated canyon geometry requires that free-field motion varying in three dimensions to be considered, and it is doubtful that any method other than a numerical one can be expected to produce realistic results for such cases. Even with a numerical approach the various assumptions made in treating the finite boundary and in modeling the inhomogeneous media may introduce errors; thus, both two- and three-dimensional results need to be compared with actual free-field earthquake records to assess their applicability (National Academy Press, Washington, D. C. 1982).

Because of many uncertainties involved in modeling the geometry, the foundation properties, and the incident earthquake motion, it is probable that a stochastic approach to defining the free-field motions will be needed in addition to the deterministic procedures.

Fig2.3 Pacoima Dam, California, was subjected to 1971 San Fernando earthquake; the seismic motions were recorded by a seismograph on narrow rock ridge above the left abutment, at the point indicated (Courtesy of George W. Housner)

2.11 Research needs

Various theoretical models have been developed for prediction of free-field motion at the surface of a valley or canyon to be used as input to a dam system; however, no verification of such input predictions has yet been achieved by comparison with actual recorded earthquake motions. Thus, specific recommendations for research on earthquake input of concrete dams follow: (Fok, K. L., J. F. Hall, and A. K. Chopra. 1986)

1. ***Deployment of strong-motion instrumentation:***

   (a) Arrays of strong-motion instruments should be deployed at selected dam sites.Triggering of these instruments should be synchronized so that traveling-wave effects can be detected.

(b) An array of strong-motion instruments should be deployed at sites being considered for construction of concrete dams to obtain the free-field motions at canyon location without the interference of an existing dam.

2. *Use of recorded seismic motion records*

Seismic motions actually recorded at a dam-foundation interface should be utilized in analyses intended to verify the various input methods.

3. *Enhancement of two-dimensional analyses*

Current available two-dimensional theoretical free-field canyon or valley wall motions are presented in terms of the incident wave angle and wave type and are in a frequency-dependent form. These results should be synthesized to provide guidelines for defining realistic input for concrete dams.

4. *Enhancement of three-dimensional models*

The deterministic predictions of free-field motion at a dam site with three-dimensional topography can be performed by numerical methods such as finite elements, boundary elements, finite differences, or some combination of the three. The development of non-reflective boundaries for such three-dimensional model remains a high priority requirement.

5. *Stochastic approach*

In view of the many uncertainties involved, the simulation of spatially varying free-field motion may require the application of stochastic theory; therefore, methods should be developed for simulating stochastic inputs for valley and canyon topographies.

2.12 Analysis of linear response

Preliminary comments

The prediction of the performance of concrete dams is one of the most challenging and complex problems found in the field of structural dynamics due to the following factors:

1. Dams and reservoirs are of complicated shapes as dictated by the topography of the sites.

2. The response of a dam is influenced to a significant degree by the interaction of the motions of dam with the impounded water and the foundation rock.
3. A dam's response may be affected by variations in the intensity and frequency characteristics of the earthquake motions over the width and height of canyon; however this cannot be fully considered at present due to the lack of instrumental data to define the spatial variation of ground motion.

When evaluating the earthquake behavior of concrete dams, it is reasonable in most cases to assume that response to low- or moderate-intensity earthquake motions is linear. That is, it is expected that the resulting deformations of the dam will be directly proportional to the amplitude of applied ground shaking.

Such an assumption of response linearity greatly simplifies both the formulation model used to represent the dam, reservoir water, foundation rock system and also the procedures used to calculate the response. The results of linear analysis serve to demonstrate general character of dynamic response. But in case of major earthquake it is probable that calculated strains would exceed the elastic capacity of dam's concrete; in this case a much more complicated nonlinear analysis would be required. However, a linear analysis still can be very valuable in helping to understand the nature of dynamic performance.

2.13 Static analysis

2.13.1 Traditional analysis and design

In particular, the dynamic behaviour of the dam, reservoir water, foundation rock system was not recognized in defining the earthquake forces used in traditional design methods. Thus, the forces expressed simply as the product of a seismic coefficient and the weight of the dam per unit surface area expressed as a function of location. Generally, interaction between dam and foundation rock was not considered here. In the design of gravity dams, it was generally believed that stress levels were not a controlling factor, so the designer was concerned mostly with satisfying criteria for overturning and sliding stability. (U. S. Bureau of Reclamation, 1996)

2.13.2 Limitations of traditional procedures

Dynamic stresses that develop in the gravity dams due to earthquake ground motions bear little resemblance to the results given by standard static design procedures. In the case of Koyna dam, the earthquake forces based on seismic coefficient of 0.05, uniform over the height were expected to cause no tensile stresses; however, the earthquake caused significant cracking in the dam. The discrepancy in the result of not recognizing the dynamic amplification effects that occur in the dam's response to earthquake motions.

The typical design coefficient, 0.05 to 0.10, used in designing concrete dams are much smaller in the typical period range for such dams than are the ordinates of the pseudo acceleration response spectra for intense earthquake motions, as shown in fig….

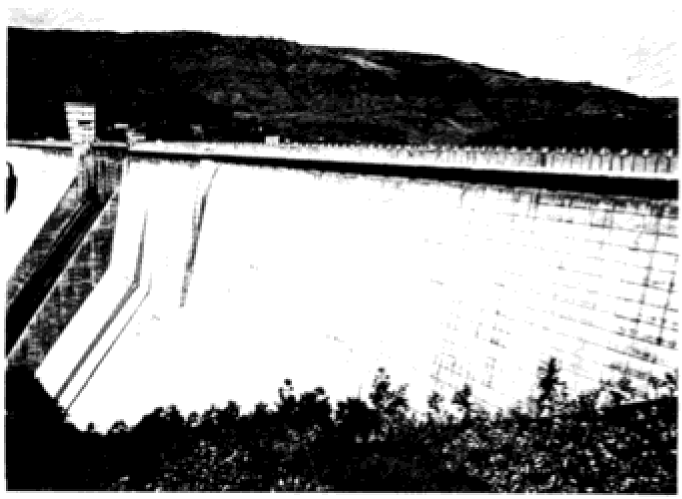


Fig 2.4 Koyna Dam, India, was damaged by a magnitude 6.5 earthquake in December 1967 (ref: UNESCO Committee of Experts, 1968)

For linearly elastic structures the dynamic aspect of the response is indicated by the response spectra and by the dynamic displacement patterns, which are conveniently expressed in terms of free-vibration-mode shapes.

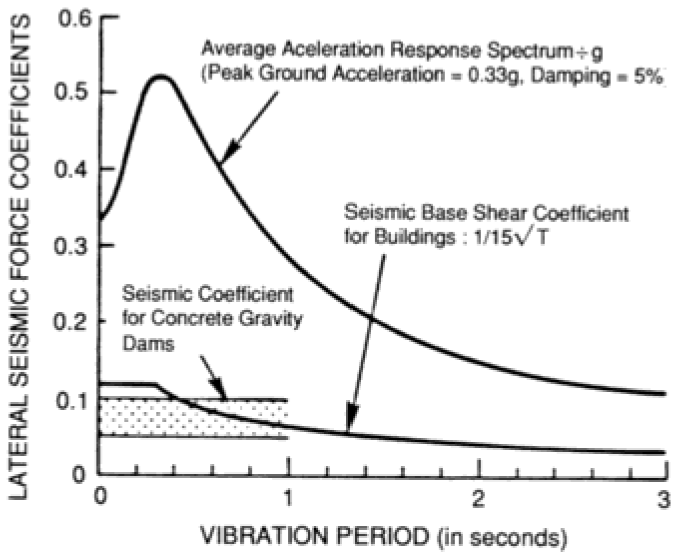


Fig2.5 Comparison of seismic coefficients used in traditional design with the response spectrum of a strong earthquake (ref: Chopra, A. K., 1978)

The seismic coefficient associated with forces in the fundamental mode of a gravity dam varies with height. One of the results of assuming a height wise-uniform seismic coefficient is that calculated stresses in gravity dams are found to be greatest at the base of the dam. This has lead to the concept of decreasing the concrete strength with increases of elevation, as has been done for some dams (e.g., Koyna Dam in India and Dworshak Dam in the United States). However, the results of dynamic analysis of Koyna Dam, as well as the location of the earthquake-induced cracks, demonstrate that the largest stresses actually occur at the two faces in the upper part of gravity dams. Therefore, those are the regions of gravity dams where the highest-strength concrete should be provided. (Structural engineers association of California recommended lateral force requirements and tentative commentary. San Francisco, California. 1988).

Another undesirable consequence of specifying a height wise constant seismic coefficient is that the detrimental effects of concrete added near the dam crest are not apparent. The hydrodynamic effects are also not properly modeled in the traditional design procedures.

Finally, it is evident that the static overturning and sliding stability criteria that have been used in traditional gravity dam design procedures have little meaning in context of the oscillatory responses produced in dams by earthquake motions.

2.14 Dynamic analysis

2.14.1 Finite element modeling

The procedures for earthquake analysis of dams began to change rapidly in the late 1960s with the adoption of finite element modeling procedures, with the advances in methods of dynamic analysis, and with the increasing availability of large-capacity, high-speed computers. (U. S. bureau of reclamation, 1976).

2.14.2 Foundation model deficiency

The two limitations of the finite element modeling of a dam-foundation system, were revealed by extensive work done during the past 20 years in the seismic analysis of nuclear power plants. (Kuo, J. S. H., 1982).

- First, the boundary hypothesized at some depth to define the foundation rock region included in the analysis is typically assumed to be rigid. Fro concrete dam sites, where similar competent rock usually extends to great depths and there is no obvious rigid boundary such as may be assumed at a soil-rock interface, the assumption of a rigid boundary may result in serious distortion of the foundation interaction effects; this distortion results mainly because energy loss (damping) associated with radiation of vibration waves beyond the assumed foundation block is not properly represented. These may significantly reduce the seismic stresses in the dam.
- Second, the earthquake input is usually applied to the dam-foundation model as prescribed motions of the rigid foundation block boundary. Clearly the earthquake motions at depth in the foundation rock will not be the same as the free-field motions recorded at the ground surface; hence, the dynamic analysis procedure should be formulated to use some different specification of the seismic input. Ideally, the earthquake inputs should be specified as spatially varying motion at the dam-foundation rock interface, but it has not

been possible for lack of appropriate instrumental records from past earthquakes.

2.15 Limitations of reservoir-water interaction models

The traditional static analysis process represent the effect of the impounding water during earthquake excitation by means of added masses calculated from Westergaard's classical formula. However, its results apply rigorously only to the case of a rigid dam with planar vertical face, and the analysis underlying the result implicitly neglects the effect of water compressibility. Although the concept has long been used in practical design, these limitations often have not been understood.(Kuo, J. S. H. fluid-structure interactions, 1982).

It was evident that the Westergaard rigid dam assumption is not consistent with the concept of dynamic analysis, so the efforts were made to formulate procedures for dynamic analysis considering dam-water interaction arising from dam flexibility. (Tarbox, G. S., K. Dreher, and L. Carpenter, 1979).

Also, extended research showed that the compressibility of impounded water has a significant contribution in the earthquake response of most concrete dams. The key parameter that determines the significance of water compressibility in the earthquake response of dams is the ratio of the fundamental natural frequency of the impounded water to the fundamental natural frequency of the dam alone. If this ratio is large enough (more than 2 for gravity dams), the impounded water affects the dam response essentially as an incompressible fluid. (Chopra, A. K., 1970)

However, this frequency ratio usually is less than 2 for dams with realistic values for the elastic modulus of concrete; hence, water compressibility is expected to be important in the earthquake response of gravity dams. (Chopra, A. K., 1968).

2.16 Present knowledge and capabilities

Conclusions about interaction effects

It is now generally recognized that:

- The earthquake response of concrete dams is increased significantly by interaction with the impounded water, with the hydrodynamic contribution being especially large in response to the vertical earthquake component.
- Hydrodynamic effects usually are more significant in the earthquake response of a slender arch dam than for a massive gravity dam.
- The assumption of waster compressibility that is commonly used in practical analysis may lead to errors on either the conservative side or the unconservative side for upstream-downstream earthquake motion, but is more likely to be unconservative in predicting the response to the vertical; and cross-stream components of motion.(Fok, K. L., and A. K. Chopra., 1987).
- An important deficiency of the incompressible water approximation is that the hydrodynamic wave absorption effects of the underlying rock or reservoir boundary sediments cannot be taken into account, and it has been shown that neglecting boundary wave absorption may lead to unrealistically large estimates of seismic response.
- Neglecting the dynamic interaction of gravity dams with deformations of the foundation rock also will generally lead to overestimation of the seismic response.

Thus, dam-reservoir water interaction, including water compressibility and pressure wave absorption at the reservoir boundaries, and dam-foundation rock interaction all should be considered in the earthquake response analysis of concrete dams.

2.17 Dynamic analysis procedures

2.17.1 Two dimensional analysis

In gravity dams with plane vertical joints the monoliths tend to vibrate independently, as evidenced by spalled concrete and water leakage at the joints of Koyna Dam during the 1967 earthquake; hence, a two dimensional plane-stress model is usually appropriate for predicting the response of such dams to moderate and intense earthquake motions. In some cases of gravity dams built in a broad valley, especially roller compacted concrete dams built without vertical joints, a plane-strain idealization may be adopted in place of plane-stress model. (Fenves, G., and A. K. Chopra, 1984).

2.17.2 Three-dimensional analysis

Eacd-3d and ADAP can be used to perform a complete dynamic analysis of a concrete dam subjected to simultaneous action of upstream, vertical, and cross-stream components of the free-field motion specified at the interface between the dam and foundation rock.(Fok, K. L., A. K. Chopra, 1986).

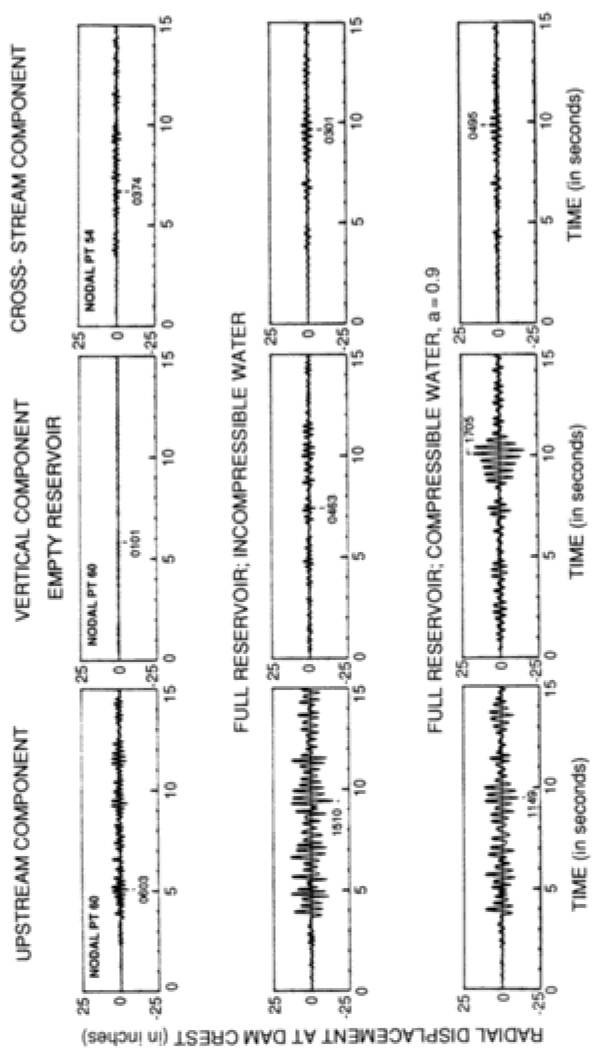


Fig 2.6 Calculated effect of water compressibility on displacement response of Morrow Point Dam due to Taft earthquake (ref: Fok, K. L., and A. K. Chopra, 1987)

Research needs

Although considerable progress has been made in the past 20 years, much additional research needs to be done to improve the reliability of methods for the seismic analysis, design, and safety evaluation of concrete dams.

- *Instrumentation*

  The instrumentation should be designed to provide adequate information on the characteristics and spatial variation of the ground motion at the site, on the response of the dam, and on the hydrodynamic pressures exerted on the dam.

- *Field forced vibration tests*

  Forced-vibration tests should be conducted on selected dams-using more than water level where feasible-and the resulting hydrodynamic pressures and the motions of the structures and their foundation should be recorded and analyzed.

- *Evaluation of analytical methods for response analysis*

  Existing analytical methods should be evaluated by comparing calculated results with the responses recorded at appropriate dam sites. If necessary, the methods should be refined programs should be used.

- *Improvement of gravity dam analysis*

  The methods used for the input of earthquake motions in present methods of earthquake analysis needs improvement. Similarly, improvements are needed in procedures used to account for the interaction between the gravity dams and their supporting foundation rock.

- *Simplified analysis procedures*

  Simplified analysis procedures should be developed that are suitable for the preliminary phase of design and safety evaluation of gravity dams.

- *Evaluation of dynamic sliding and rocking response of gravity dams*

  Analysis procedures should be developed to determine the dynamic sliding and rocking response of gravity dam monoliths. Utilizing these procedures, rational stability criteria should be derived, replacing the traditional sliding and overturning criteria that do not recognize the oscillatory response of dams during earthquakes.

2.17.3 Non linear analysis and response behaviour

If a significant damage occurs, the actual performance of a dam can be predicted only by a non linear analysis that takes account of the stiffnaess changes. A comprehensive safety evaluation of a concrete dam requires

- Identification of behaviour of materials and components under dynamic loads,
- Mathematical models that can represent the non linear behaviour,
- Efficient numerical procedures for computing the nonlinear earthquake response of the dam system, and
- Criteria to assess acceptable performance and evaluate failure modes (Karson, I. K., J. O. Jirsa., 1969)

Research needs

Recognizing the importance of evaluating the earthquake safety of concrete dams and the limited knowledge that exists concerning the non linear behaviour of concrete dams, the following items should be addressed in future comprehensive research effort:

1. *Material testing of mass concrete*

Additional data on the behaviour of mass concrete under dynamic loads is urgently needed and will require an extensive physical testing program. The testing should emphasize the tensile cracking of the mass concrete under multiaxial stress states that are representative of the in-situ stresses, including strain rate effects. Concrete samples should be representative of the actual curing conditions in dams.

2. *Development of material models for concrete*

Using the data obtained from the testing of mass concrete specimens, it is important to develop realistic mathematical models for tensile cracking under dynamic loads. Identification of a set of parameters that are predictive of non linear dynamic behaviour of concrete would be an important advance. The material model should allow multiaxial stress states, including criteria for tensile cracking and propagation of cracks, strain rate effects, and shear stress transfer by aggregate interlock.

3. *Modeling of other non linear mechanisms*

Additional research is required to develop and apply models of construction joint behaviour in earthquake analysis of concrete dams. Such models must represent the redistribution of forces as the joints open and close and allow for degradation of the joint resulting from a large number of loading cycles. Models fro concrete and rock abutments that include potential shear failure modes are also necessary.

4. *Numerical procedures for computing non linear response*

Once realistic mathematical models for concrete, joints, and foundation rock are available, they can be incorporated into several classes of numerical procedures for nonlinear dynamic response analysis of concrete dam systems, including interaction with the impounded water and flexible foundation rock and the effects of reservoir-bottom shock-wave absorption. Research should identify the methods for reducing the computational efforts, while still representing the important non linear behavior of the system, so that non linear analysis can be used for routine design and evaluation of dams.

5. *Parametric and detailed response studies*

With the development of efficient numerical procedures, response analysis of typical concrete dams can be used to improve understanding of the nonlinear behaviour during earthquake ground motion. Post-cracking stability in gravity dams should be evaluated. In addition, the influence of the following factors should be determined:

- ground motion characteristics, particularly amplitude, frequency content, duration, and occurrence of velocity pulses,
- water compressibility with respect to nonlinear response, and
- modeling issues, such as degree of spatial discretization, extent of reservoir and foundation rock domains, and radiation condition at the boundaries.

6. *Dynamic testing of dam models*

In parallel with the analytical studies, further dynamic testing of dam models is essential for verifying nonlinear behaviour and calibrating mathematical models and numerical results.

7. Identification of design criteria

Realistic design criteria for concrete gravity dams should be developed using the results of analytical and experimental studies. Comparison of the predicted nonlinear response with the more easily calculated linear response would be useful to determine the limits of assessing nonlinear behaviour based on the results of linear elastic analysis.

8. *Investigation of earthquake-resistant design measures*

Nonlinear response analysis capabilities will allow investigation of innovative measures for increasing the earthquake safety of concrete dams. Based on the results of nonlinear analyses, it may be possible to alter the geometry of dams and foundation abutments to improve resistance to earthquakes. Different jointing schemes, including use of joint materials that dissipate energy, should be investigated.

It must be emphasized that these research needs are not only important in improving the seismic safety of concrete dams but would also represent significant advances in other applications of earthquake engineering.

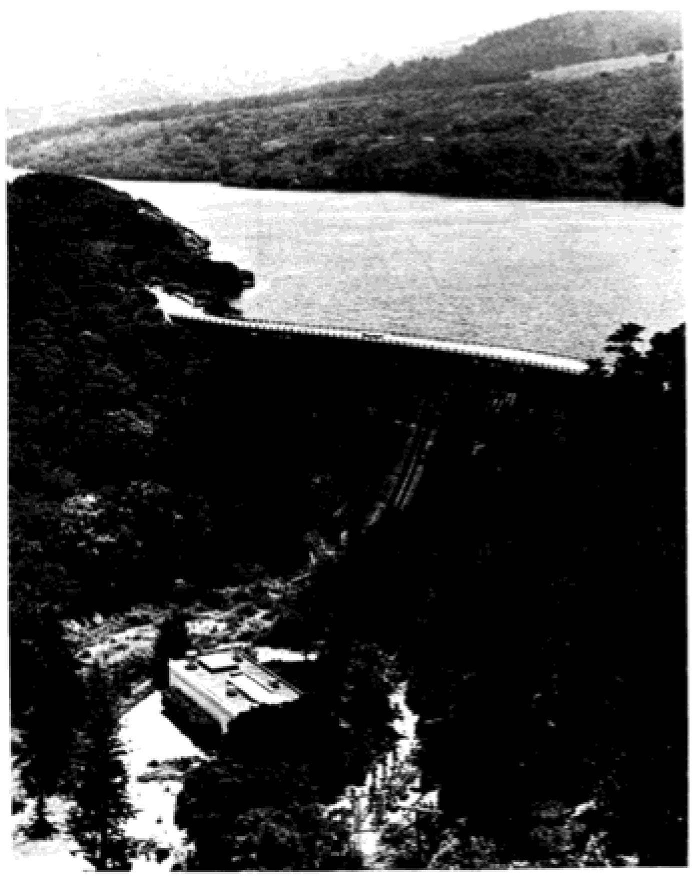


Fig2.7 Lower Crystal Springs Dam, California; the San Andreas fault lies under the reservoir (Photo courtesy of San Francisco Public Utilities commission)

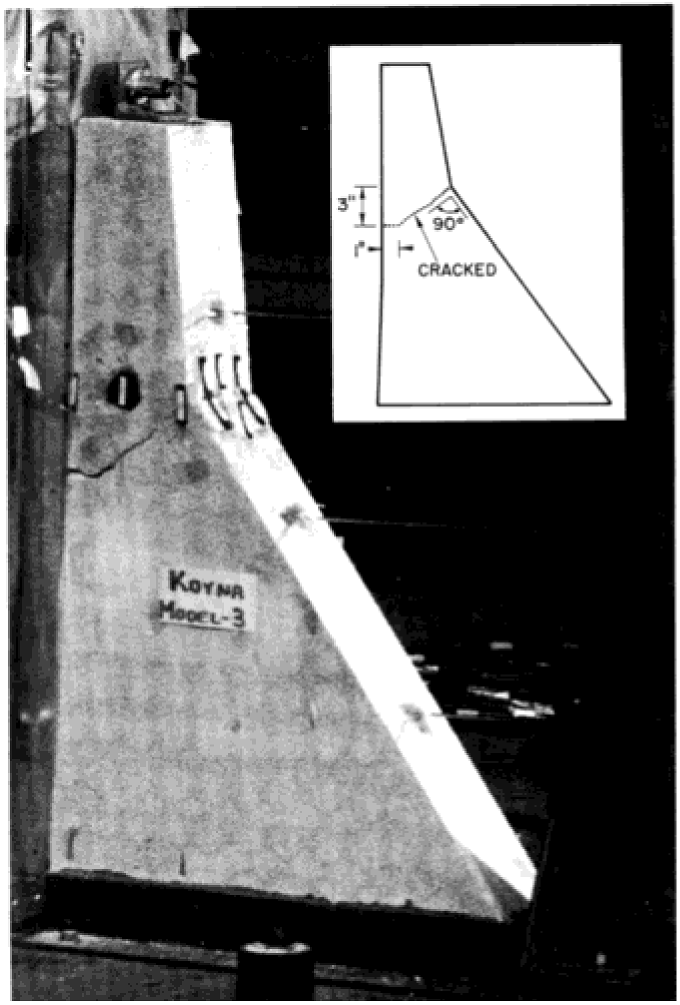


Fig2.8 Model of single monolith of Koyna Dam; crack resulted from shaking table test, including two-dimensional reservoir model (ref: Niwa, A., and R. W. Clough, 1980)

Fig 2.9 Today's photo of Navagam dam work

(

CHAPTER 3

METHODS OF DYNAMIC ANALYSIS OF CONCRETE GRAVITY DAMS & INTRODUCTION TO SAP 2000

3.1 Dynamic Analysis Process

The structural analysis for earthquake loadings consists of two parts: an approximate resultant location and sliding stability analysis using an appropriate seismic coefficient and a dynamic internal stress analysis using site-dependent earthquake ground motions if the following conditions exist:

- The dam is 100 feet or more in height and the peak ground acceleration (PGA) at the site is greater than 0.2 g for the maximum credible earthquake.
- The dam is less than 100 feet high and the PGA at the site is greater than 0.4 g for the maximum credible earthquake.
- There are gated spillway monoliths, wide roadways, intake structures, or other monoliths of unusual shape or geometry.
- The dam is in a weakened condition because of accident, aging, or deterioration. The requirements for a dynamic stress analysis in this case will be decided on a project-by-project basis in consultant.

The procedure for performing a dynamic analysis of dam includes the following (U.S. Army Corps of Engineers, 1995).

- Review the geology, seismology, and contemporary tectonic setting.
- Determine the earthquake sources.
- Select the candidate maximum credible and operating basis earthquake magnitudes and locations.
- Select the attenuation relationships for the candidate earthquakes.
- Select the controlling maximum credible and operating basis earthquakes from the candidate earthquakes based on the most severe ground motions at the site.

- Select the design response spectra for the controlling earthquakes.
- Select the appropriate acceleration-time records that are compatible with the design response spectra if acceleration-time history analyses are needed.
- Select the dynamic material properties for the concrete and foundation.
- Select the dynamic methods of analysis to be used.
- Perform the dynamic analysis.
- Evaluate the deflection and stresses from the dynamic analysis.

3.2 Interdisciplinary Coordination

A dynamic analysis requires a team of engineering geologists, seismologists, and structural engineers. They must work together to analyze a rare event like the MCE, upper bound values for the PGA, upper bound values for the design response spectra, and conservative criteria for determining the earthquake resistance of the structure. The steps in performing a dynamic analysis should be fully coordinated to develop a reasonably conservative design with respect to the associated risks. The structural engineers responsible for the dynamic structural analysis should be actively involved in the process of characterizing the earthquake ground motions in the form required for the methods of dynamic analysis to be used (U.S. Army Corps of Engineers, 1995).

3.3 Performance Criteria for Response to Site-Dependent Earthquakes

3.3.1 Maximum Credible Earthquake

Gravity dams should be capable of surviving the controlling MCE without a catastrophic failure that would result in loss of life or significant damage to property. Inelastic behavior with associated damage is permissible under the MCE (ANCOLD, August 1998; U.S. Army Corps of Engineers, 1995; USCOLD, April 1991).

3.3.2 Operating Basis Earthquake

Gravity dams should be capable of resisting the controlling OBE within the elastic range, remain operational, and not require extensive repairs (ANCOLD, August 1998, U.S. Army Corps of Engineers, 1995; USCOLD, April 1991).

3.4 Geological and Seismological Investigation

A geological and seismological investigation of all dam sites is required for projects located in high seismic active zone. The objectives of the investigation are to establish controlling maximum and credible operating basis earthquakes and the corresponding ground motions for each and to assess the possibility of earthquake induced foundation dislocation at the site. Selecting the controlling earthquakes is discussed below. (U.S. Army Corps of Engineers, 1995)

3.4.1 Selecting the Controlling Earthquakes

(a) Maximum Credible Earthquake

The first step for selecting the controlling MCE is to specify the magnitude and/or modified Mercalli (MM) intensity of the MCE for each seismotectonic structure or source area within the region examined around the site. The second step is to select the controlling MCE based on the most severe vibratory ground motion within the predominant frequency range of the dam and determine the foundation dislocation, if any, capable of being produced at the site by the candidate MCE's.

b. Operating Basis Earthquake

1. The selection of the OBE is based upon the desired level of protection for the project from earthquake induced damage and loss of service project life. The project life of new dams is usually taken as 100 years. The probability of exceedance of the OBE during the project life should be no greater than 50 percent unless the cost savings in designing for a less severe earthquake outweighs the risk of incurring (invite) the cost of repairs and loss of service because of a more severe earthquake.
2. The probabilistic analysis for the OBE involves developing a magnitude frequency or epicenter intensity frequency relationship of each seismic source; projecting the recurrence information from regional and past data into forecasts concerning future occurrence; attenuating (satisfy) the severity parameter, usually either PGA of MM intensity, to the site; determining the controlling recurrence relationship for the site; and finally, selecting the design level of earthquake based upon the probability of exceedance and the project life.

3.5 Characterizing Ground Motions

After specifying the location and magnitude (or epicentral intensity) of each candidate earthquake and an appropriate regional attenuation relationship, the characteristics of vibratory ground motion expected at the site can be determined. Vibratory ground motions have been described in a variety of ways, such as peak ground motion parameters, acceleration-time records (accelerograms), or response spectra [20] (Hayes 1980; Krinitzsky and Marcuson, 1983). For the analysis and design of concrete dams, the controlling characterization of vibratory ground motion should be site-dependent design response spectra (ANCOLD, August 1998; U.S. Army Corps of Engineers, 1995; USCOLD, April 1991).

3.5.1 Site-Specific Design Response Spectra

1. Wherever possible, site-specific design response spectra should be developed statistically from response spectra of strong motion records of earthquakes that have similar source and propagation path properties as the controlling earthquake and are recorded on a foundation similar to that of the dam. Important source properties include magnitude and, if possible, fault type and tectonic environment. Propagation path properties include distance, depth, and attenuation. As many accelerograms as possible that are recorded under comparable conditions and have a predominant frequency similar to that selected for the design earthquake should be included in the development of the design response spectra. Also, accelerograms should be selected that have been corrected for the true baseline of zero acceleration, for errors in digitization, and for other irregularities (Schiff and Bogdanoff 1967).
2. Where a large enough ensemble of site-specific strong motion records is not available, design response spectra may be approximated by scaling that ensemble of records that represents the best estimate of source, propagation path, and site properties. Scaling factors can be obtained in several ways. The scaling factor may be determined by dividing the peak or effective peak acceleration specified for the controlling earthquake by the peak acceleration of the record being rescaled. The peak velocity of the record should fall within the range of peak

velocities specified for the controlling earthquake, or the record should not be used. Spectrum intensity can be used for scaling by using the ratio of the spectrum intensity determined for the site and the spectrum intensity of the record being rescaled (USBR 1978). Acceleration attenuation relationships can be used for scaling by dividing the acceleration that corresponds to the source distance and magnitude of the controlling earthquake by the acceleration that corresponds to the source distance and magnitude of the record being rescaled (Guzman and Jennings 1970). Because the scaling of accelerograms is an approximate operation at best, the closer the characteristics of the actual earthquake are to those of the controlling earthquake, the more reliable the results. For this reason, the scaling factor should be held to within a range of 0.33 to 3 for gravity dam.

3. Site-dependent response spectra developed from strong motion records, as described in above paragraphs, should have amplitudes equal to or greater than the mean response spectrum for the appropriate foundation given by Seed, Ugas, and Lysmer (1976), anchored by the PGA determined for the site. This minimum response spectrum may be anchored by an effective PGA determined for the site, but supporting documentation for determining the effective PGA will be required (Newmark and Hall 1982).
4. A mean smooth response spectrum of the response spectra of records chosen should be presented for each damping value of interest.

3.5.2 Accelerograms for Acceleration-Time History Analysis

Accelerograms used for dynamic input should be compatible with the design response spectrum and account for the peak ground motions parameters, spectrum intensity, and duration of shaking. Compatibility is defined as the envelope of all response spectra derived from the selected accelerograms that lie below the smooth design response spectrum throughout the frequency range of structural significance (ANCOLD, August 1998; U.S. Army Corps of Engineers, 1995; Priscu. R., A Popivi, D. Stematiu, and C. Stere, 1985).

3.6 Method for analysis of dam

3.6.1 Traditional Analysis procedures

In traditional analysis procedure(U.S. Army Corps of Engineers, 1995) concrete gravity dams have been analyzed by very simple procedures. The earthquake forces are treated simply as static forces and are combined with the hydrostatic pressures and gravity loads. The analysis is concerned with overturning and sliding stability of the monolith treated as a rigid body, and with stresses in the monolith that are calculated by elementary beam theory.

In representing the effects of horizontal ground motion transverse to the axis of the dam by static lateral forces, neither the dynamic response characteristics of dam water foundation rock systems nor the characteristics of earthquake ground motion are recognized. Two types of static lateral forces are included. Forces associated with the weight of the dam are expressed as the product of a seismic coefficient and water pressure, in addition to the hydrostatic pressure, are specified in terms of the seismic coefficient and a pressure coefficient that is based on assumption of a rigid dam and incompressible water. Interaction between the Dam-Foundation rock system is not considered in computing the aforementioned earthquake forces.

The traditional design criteria require that ample factor of safety be provided against overturning, sliding, and overstressing under all loading conditions-compressive stresses should be less that the allowable values. Tension often is not permitted and even if it is, the possibility of cracking of concrete is not seriously considered. It has generally been believed that stresses are not a controlling factor in the design of concrete gravity dams; so the traditional design procedures are concerned most with satisfying the criteria for overturning and sliding stability.

Due to limitations of the traditional procedure (Asgher Vatanani Oskouei and A.A. Dumanoglu, November 2000), in most of case dams are now analyzed by using complete dynamic procedure described below.

3.6.2 Dynamic Methods of Analysis

A dynamic analysis determines the structural response based on the characteristics of the structure and the nature of the earthquake loading. Dynamic methods usually employ the modal analysis technique. This technique is based on the simplifying assumption that the response in each natural mode of vibration can be computed independently and the modal responses can be combined to determine the total response. Modal techniques that can be used for gravity dams include a simplified response spectrum method and finite element methods using either a response spectrum or acceleration-time records for the dynamic input (U.S. Army Corps of Engineers, 1995; Priscu. R., A Popivi, D. Stematiu, and C. Stere, 1985). A dynamic analysis should begin with the response spectrum method and progress to more refined methods like time-history if needed. A time-history analysis also used when yielding (cracking) of the dam is indicated by a response spectrum analysis.

3.7 Simplified Response Spectrum Method

1. The simplified response spectrum method (U.S. Army Corps of Engineers, 1995; Priscu. R., A Popivi, D. Stematiu, and C. Stere, 1985) computes the maximum linear response of a nonoverflow section in its fundamental mode of vibration due to the horizontal component of ground motion. The dam is modeled as an elastic mass fully restrained on a rigid foundation. Hydrodynamic effects are modeled as an added mass of water moving with the dam. The amount of the added water mass depends on the fundamental frequency of vibration and mode shape of the dam and the effects of interaction between the dam and reservoir. Earthquake loading is computed directly from the spectral acceleration, obtained from the design earthquake response spectrum, and the dynamic properties of the structural system. This simplified method can be used also for an ungated spillway monolith that has a section similar to a nonoverflow monolith.
2. The program SDOFDAM is available to easily model a dam using the finite element method and Chopra's simplified procedure for estimating the hydrodynamic loading. This analysis provides a reasonable first estimate of the tensile stress in the dam. From that estimate, one can decide if the design is adequate or if a refined analysis is needed.

3.8 Finite Element Methods

The finite element method (U.S. Army Corps of Engineers, 1995; Rajashekhran S.,2003) is capable of modeling the horizontal and vertical structural deformations and the exterior and interior concrete, and it includes the response of the higher modes of vibrations, the interaction effects of the foundation and any surrounding soil, and the horizontal and vertical components of ground motion.

3.8.1 Finite Element Response Spectrum Method

1. The finite element response spectrum method can model the dynamic response of linear two- and three-dimensional structures. The hydrodynamic effects are modeled as an added mass of water moving with the dam using Westergaard's formula or Zangar formula. The foundations are modeled as discrete elements.
1. Modal analysis technique used to compute the natural frequencies of vibration and corresponding mode shapes for specified modes. The earthquake loading is computed from earthquake response spectra for each mode of vibration induced by the horizontal and vertical components of ground motion. These modal responses are combined to obtain an estimate of the maximum total response. Stresses are computed by a static analysis of the dam using the earthquake loading as an equivalent static load.
2. The complete quadratic combination (CQC) method should be used to combine the modal responses. The CQC method degenerates to the square root of the sum of squares (SRSS) method for two-dimensional structures in which the frequencies are well separated. Combining modal maxima by the SRSS method can dramatically overestimate or significantly underestimate the dynamic response for three-dimensional structures.
3. The finite element response spectrum method should be used for dam monoliths that cannot be modeled two dimensionally or if the maximum tensile stress from the simplified response spectrum method exceeds 15 percent of the unconfined compressive strength of the concrete.
4. Normal stresses should be used for evaluating the results obtained from a finite element response spectrum analysis. Finite element programs calculate normal stresses that, in turn, are used to compute principal stresses. The absolute values

of the dynamic response at different time intervals are used to combine the modal responses. These calculations of principal stress overestimate the actual condition. Principal stresses should be calculated using the finite element acceleration-time history analysis for a specific time interval.

3.8.2 Finite Element Acceleration-Time History Method

1. The acceleration-time history method computes the natural frequencies of vibration and corresponding mode shapes for specified modes. The response of each mode, in the form of equivalent lateral loads, is calculated for the entire duration of the earthquake acceleration-time record starting with initial conditions, taking a small time interval, and computing the response at the end of each time interval. The modal responses are added for each time interval to yield the total response. The stresses are computed by a static analysis for each time interval.
2. An acceleration-time history analysis is appropriate if the variation of stresses with time is required to evaluate the extent and duration of a highly stressed condition.

3.9 Dam-Water Interaction

Interaction between dam (U.S. Army Corps of Engineers, 2000; U.S. Army Corps of Engineers, 1995; Chopra A.K., 1976) and water impounded in the reservoir arises from the fact that the hydrodynamic pressures affect the deformations of the dam, which in turn influence the pressures. Such interaction introduces frequency-dependent hydrodynamic terms in the equations of motion that affect the dynamic response of the dam. The hydrodynamic terms can be interpreted as an added force, an added mass, and added damping. The added hydrodynamic mass of water lengthens the fundamental resonant period. The added hydrodynamic force for horizontal ground motion has less effect on the response because it is relatively small compared with the effective earthquake force associated with the mass of dam.

Westergaard and Zanger gave the formula for finding added mass assuming that water is incompressible fluid. Hydrodynamic pressure acting on the face of dam-water interface are first obtained from the finite element solution of wave equation and then converted into equivalent added mass term. These resulting added mass terms are subsequently combined with the mass of concrete at nodal points on Dam-Water interface. For more rigorous analysis water compressibility and reservoir bottom absorption effect also consider in analysis(U.S. Army Corps of Engineers, 2000; U.S. Army Corps of Engineers, 1995).

3.10 Dam-Foundation Interaction

The interaction between the dam and flexible foundation rock affects the response of the dam in simpler manner than Dam-Water interaction (Chopra A.K., 1976; U.S. Army Corps of Engineers, 2000; U.S. Army Corps of Engineers, 1995). Dam-Foundation interaction lengthens the fundamental period of dam response. Generally in two dimensional analyses, Dam-Foundation interaction is represented by finite element modal of foundation that can account variation in rock properties in foundation region. The foundation modal, however, is massless in order to simplify the application of seismic input. The foundation mesh is needs to be extended a distance at lest equal to dam height in upstream downstream and downward direction. The nodal points at base foundation mesh are fixed in both horizontal and vertical direction and at side nodes are attached to vertical guided roller.

The two properties of the foundation rock that have a significant influence on the dynamic response are the damping ratio and the elastic modulus (U.S. Army Corps of Engineers, 1995). The damping characteristics of the foundation contribute significantly proportional to the damping of the combined Dam-Foundation system and must be considered in the analysis. When the foundation deformation modulus is low, the damping ratio of the combined system is considerably higher than the damping ratio of the RCC dam structure alone. The elastic modulus of the foundation influences the response because it directly affects Modal frequencies and Mode shape. As the elastic modulus decreases, the modal frequencies of combine dam foundation system also decrease and the mode shapes are affected by increased rigid body translations and rotation of the dam on the elastic foundation.

3.11 Time History Numerical Solution Technique

3.11.1 Introduction

Numerical solutions of equation of motion of structure are divided into two methods: Direct integration and mode superposition(Chopra, A.K., 2002; U.S. Army Corps of Engineers, 1995). In direct integration the equations of motion are integrated directly using a numerical step-by-step procedure, without transforming the equation into a different form. In mode superposition method the equations of motion are first transformed into a more effective form (modal forms) before they are solved using either a step-by-step integration in the time domain.

3.11.2 Equations of Motion

The equations of motion for a structure are formulated from the equilibrium of the effective forces associated with each of its degrees of freedom. In a matrix form these equations may be expressed by

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = p(t) \qquad (2\text{-}1)$$

Where m, c, and k are the mass, damping, and stiffness matrices, respectively; u(t), ů(t), and ü(t) are respectively nodal displacement, velocity, and acceleration vectors; and p(t) is the effective load vectors. For structures like dam the coefficient matrices and the effective load vector also include contributions from interaction with water and the foundation rock. However depending on the type of finite element formulation used, the interaction effects are treated as described below.

In finite element formulation with massless foundation and incompressible water the mass matrix m, contains both the structure mass and the added mass of water. The matrix k is the combined stiffness for the structure and the foundation. The effective load vector p(t) resulting from the earthquake ground shaking, includes inertia loads due to the mass of structure and of the water.

3.11.3 Direct Integration

In direct integration the equations of motion are directly integrated using a step-by-step numerical procedure without prior transformation to a different form. The step-by-step integration procedures provide as approximate solution at n discrete time intervals o, Δt, 2Δt, 3Δt, …t, t+Δt, ….T where T is duration of the input motion or loading, time interval Δt=T/n and n is time steps.

3.11.4 Mode Superposition

The number of operations in the direct integration method is proportional to the number of time steps used. In general the use of direct integration may be considered effective when the response is required only for a relatively short duration. However, if the integration must be carried out for many time steps, it may be more effective to transform the equation of motion into a form for which the step-by-step integration is less costly. For this purpose the equations of motion for linear analysis are usually transformed into the eigenvectors or normal coordinate system. Applying the normal accordance with Clough and Penzien (1993) to Equation leads to following decoupled equation of motion for individual modes:

$$M_n \ddot{Y}_n(t) + C_n \dot{Y}_n(t) + K_n Y_n(t) = P_n(t) \qquad (2\text{-}2)$$

Where the modal coordinate mass, damping, stiffness, and load are defined as follows:

$$M_n = \phi_n^{\,T} M \phi_n \qquad \mathbf{(2\text{-}3)}$$

$$C_n = \phi_n^T C \phi_n \qquad (2\text{-}4)$$

$$K_n = \phi_n^T C \phi_n \qquad (2\text{-}5)$$

$$P_n = \phi_n^T P(t) \qquad (2\text{-}6)$$

The modal Equation may also be expressed in the following form

$$\ddot{Y}_n(t) + 2\xi_n \omega_n \dot{Y}_n(t) + \omega_n^2 Y_n(t) = \frac{P_n(t)}{M_n} \qquad (2\text{-}7)$$

Where ξ_n is the modal damping ratio and ω_n is the undamped natural frequency. Now the time integration can be carried out individual for each decoupled modal equation. This can be accomplished using any of the above integration schemes or by numerical evaluation of the Duhamel integral

$$Y_n(t) = \frac{1}{M_n \omega_n} \int_0^\tau p_n(\tau) \exp[-\xi_n \omega_n (t-\tau)] \sin \omega_{Dn}(t-\tau) d\tau \qquad (2\text{-}8)$$

where $\omega_{Dn} = \omega_n \sqrt{\left(1-\xi_n^2\right)}$ **is the damped natural frequency. Having obtained the response of each mode is the damped natural frequency. Having obtained the response of each mode $Y_n(t)$ from Equation , the total displacement of the structure in the geometric coordinates can be computed using**

$$u(t) = \phi_1 Y_1(t) + \phi_2 Y_2(t) + \ldots\ldots\ldots + \phi_n Y_n(t) \qquad (2\text{-}9)$$

In summary, the response analysis by mode superposition requires:

- **The solution of the eigenvalues and eigenvector for transformation of the problem to the modal coordinates.**
- **Solution of the decoupled modal equilibrium equations by the Duhamel integral or other integration schemes, and**
- **Superposition of the modal responses as expressed in to obtain the total response of the structure.**

In the liner time-history analysis the choice between the direct method and mode superposition is decided by effectiveness of the methods and whether a few modes of vibration can provide accurate results or not. The solution obtained using either method is identical with respect to errors inherits in the time integration schemes and round-off errors associated with computer analysis.

3.12 Introduction to SAP 2000

SAP2000 is the latest and most powerful version of the well-known SAP series of structural analysis programs. SAP2000 is a full-featured program, which can be used for the simplest problems or the most complex projects.

3.12.1 SAP2000 Analysis Features

The SAP2000 structural analysis program offers the following features:

- Static and dynamic analysis
- Linear and nonlinear analysis
- Dynamic seismic analysis and static pushover analysis
- Vehicle live-load analysis for bridges
- Geometric nonlinearity, including P-delta and large-displacement effects

- Staged (incremental) construction
- Buckling analysis
- Frame and shell structural elements, including beam-column, truss,
- Membrane, and plate behavior
- Two-dimensional plane and axisymmetric solid elements
- Three-dimensional solid elements
- Nonlinear link and spring elements
- Multiple coordinate systems
- Many types of constraints
- A wide variety of loading options
- Alpha-numeric labels
- Large capacity
- Highly efficient and stable solution algorithms

These features and many more, make SAP2000 the state-of-the-art in structural analysis programs.

3.12.2 Structural Analysis and Design

The following general steps are required to analysis and design a structure using SAP2000:

- Create or modify a model that numerically defines the geometry, properties,
- Loading, and analysis parameters for the structure
- Perform an analysis of the model
- Review the results of the analysis
- Check and optimize the design of the structure

This is usually an iterative process that may involve several cycles of the above sequence of steps. All of these steps can be performed seamlessly using the SAP2000 graphical user interface.

3.12.3 About Manuals

Sap 2000 provides following manuals, which are design to help users quickly become productive with sap 2000.

- Sap 2000 getting started, which introduce the concepts of the structural model, the graphical user interface, and working with data tables.
- SAP2000 Basic Analysis Reference, which gives an introduction to the fundamental concepts underlying the structural model and the analysis techniques used by SAP2000
- Sap 2000 Analysis Reference, containing information about the advanced modeling and analysis features of the program.
- Sap 2000 Introductory Tutorial, which is intended to provide for users with hands-on experience using the modeling, analysis and design features of SAP2000.
- Sap 2000 design manuals, containing detailed design features specific to supported design codes
- Sap 2000 Verification Manual, containing examples showing the capabilities and verifying the accuracy of the analytical features of the program.

Sap 2000 provide following elements for the modeling the structure.

- Frame/cable element
- Shell element
- Plane element
- Asolid element
- Solid element
- Link element

Plane Element is used in this work to modal Dam-foundation combine system. Features of Plane Element are discussed below.

3.12.4 Plane element:

3.12.4.1 General:

The Plane element also called as 2D solid element is used to model plane-stress and plane-strain behavior in two-dimensional solids. The plane element is a three or four noded element for modeling two-dimensional solids of uniform thickness.

Structures that can be modeled with this element include:

- Thin, planar structures in a state of plane stress

- Long, prismatic structures in a state of plane strain

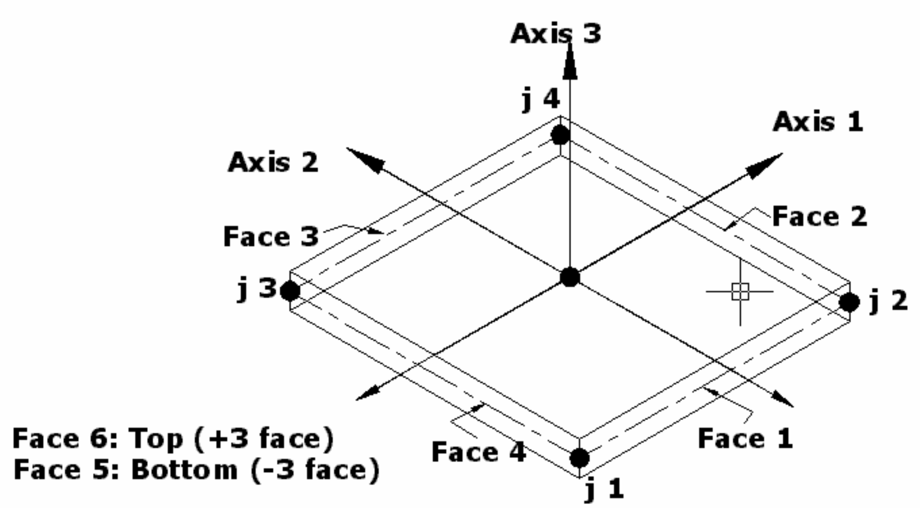
The stresses and strains are assumed not to vary in the thickness direction.

Each Plane element has its own local coordinate system for defining Material properties and loads, and for interpreting output. Temperature-dependent, orthotropic material properties are allowed. Each element may be loaded by gravity (in any direction); surface pressure on the side faces; pore pressure within the element; and loads due to temperature change.

Each plane element (and other types of area objects/elements) may have either of the following shapes, as shown in Figure3.1.

- Quadrilateral, defined by the four joints **j1**, **j2**, **j3**, and **j4**.
- Triangular, defined by the three joints **j1**, **j2**, and **j3**.

The quadrilateral formulation is the more accurate of the two.



Four-noded Quadrilatral Shell Element

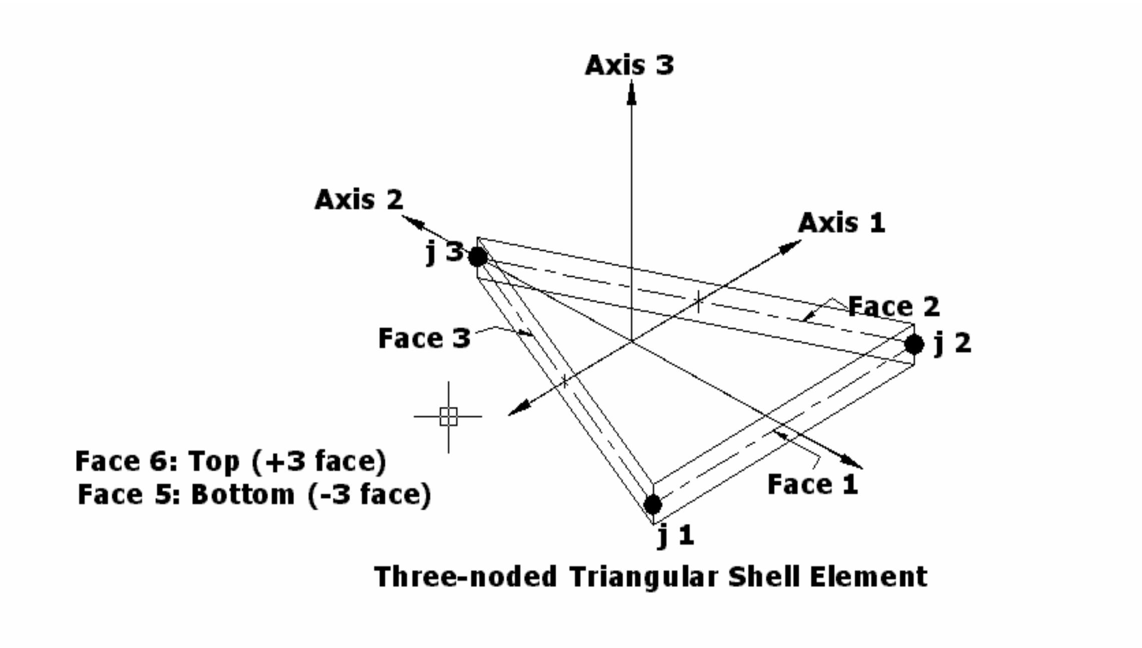


Fig 3.1: Plane elements

3.12.4.2 Degrees of Freedom

The Plane element activates the three translational degrees of freedom at each of its connected joints. Rotational degrees of freedom are not activated.

3.12.4.3 Local Coordinate System

Each plane element has its own element local coordinate system used to define Material properties, loads and output. The axes of this local system are denoted 1, 2 and 3. The first two axes lie in the plane of the element with an orientation that user specify; the third axis is normal.

3.12.4.4 Mass

The total mass of the element is equal to the integral over the plane of the element of the mass density multiplied by the thickness. The total mass is apportioned to the joints in a manner that is proportional to the diagonal terms of the consistent mass matrix. The total mass is applied to each of the three translational degrees of freedom (UX, UY, and UZ) even when the element contributes stiffness to only two of these degrees of freedom.

In a dynamic analysis, the mass of the structure is used to compute inertial forces. The mass contributed by the Plane element is lumped at the element joints. No inertial effects are considered within the element itself.

3.12.4.5 Self-Weight Load

Self-Weight Load activates the self-weight of all elements in the model. For a Plane element, the self-weight is a force that is uniformly distributed over the plane of the element. The magnitude of the self-weight is equal to the weight density multiplied by the thickness. Self-Weight Load always acts downward, in the global –Z direction.

3.12.4.6 Surface Pressure Load

The Surface Pressure Load is used to apply external pressure loads upon any of the three or four side faces of the Plane element. Surface pressure always acts normal to the face. Positive pressures are directed toward the interior of the element.

3.12.4.7 Stresses and Strains

The Plane element models the mid-plane of a structure having uniform thickness, and whose stresses and strains do not vary in the thickness direction.

Plane-stress is appropriate for structures that are thin compared to their planar dimensions. The thickness normal stress (σ_{33}) is assumed to be zero. The thickness normal strain (ε_{33}) may not be zero due to Poisson effects. Transverse shear stresses (σ_{12}, σ_{13}) and shear strains (Υ_{12}, Υ_{13}) are assumed to be zero. Displacements in the thickness (local 3) direction have no effect on the element.

Plane-strain is appropriate for structures that are thick compared to their planar dimensions. The thickness normal strain (ε_{33}) is assumed to be zero. The thickness normal stress (σ_{33}) may not be zero due to Poisson effects. Transverse shear stresses (σ_{12}, σ_{13}) and shear strains (Υ_{12}, Υ_{13}) are dependent upon displacements in the thickness (local 3) direction.

3.12.5 Analysis Cases

3.12.5.1 General

An Analysis Case defines how the loads are to be applied to the structure (e.g., statically or dynamically), how the structure responds (e.g., linearly or nonlinearly), and how the analysis is to be performed (e.g., modally or by direct-integration.). User may define as many named analysis cases of any type that he wishes. For analysis the model, user may select which cases are to be run and may also selectively delete results for any analysis case. The results of linear analyses may be superposed, i.e., added together after analysis.

The available types of linear analysis are:

- Static analysis
- Modal analysis for vibration modes, using eigenvectors or Ritz vectors
- Response-spectrum analysis for seismic response
- Time-history dynamic response analysis
- Buckling-mode analysis
- Moving-load analysis for bridge vehicle live loads
- Harmonic steady-state analysis

Each different analysis performed is called an **Analysis Case**. For each Analysis Case user has to define following type of information:

3.12.5.2 Case name

This name must be unique across all Analysis Cases of all types. The case name is used to call analysis results displacements, stresses, etc.), for creating Combinations, and sometimes for use by other dependent Analysis Cases.

3.12.5.3 Analysis type

This indicate the type of analysis (static, response-spectrum, time history, etc.), as well as available options for that type (linear, nonlinear, etc.).

3.12.5.4 Loads applied

For most types of analysis, specify the Load Cases that are to be applied to the structure. Additional data may be required, depending upon the type of analysis being defined.

3.12.6 Running Analysis Cases

After defining a structural model and one or more Analysis Cases, **run** the Analysis Cases to get results for display, output, and design purposes.

When an analysis is run, the program converts the object-based model to finite elements, and performs all calculations necessary to determine the response of the structure to the loads applied in the Analysis Cases. The analysis results are saved for each case for subsequent use.

3.12.7 Modal Definition and Analysis Results Tabular Data

Model definition data include all input components of the structural model (properties, objects, assignments, loads, analysis cases, design settings, etc.), as well as any options you have selected, and named result definitions you have created. Model definition data are always available, whether or not analyses have been run or design has been performed. These tables can be edited, displayed, exported, imported, and printed by using definition data tables on-screen in the graphical user interface, or export and import in one of the following formats:

- Microsoft Access database
- Microsoft Excel spreadsheet
- Plain (ASCII) text

Analysis results data include the deflections, forces, stresses, energies, and other response quantities that can be produced in the graphical user interface. These data are only available for analysis cases that have actually been run. Analysis results tables can not be edited or imported, but displayed, exported, and printed on-screen in the graphical user interface, or export and import in one of the following formats:

- Microsoft Access database
- Microsoft Excel spreadsheet
- Plain (ASCII) text

CHAPTER 4
THE CASE STUDY OF GRAVITY DAM

4.1 Case Study

The Tallest Overflow Section of Navagam Dam under construction across river Narmada in Gujarat, India has been taken as a case study. (Chandrasekaran A.R., I.S. Srivastava, B.C. Mathur, S. Basu, R.C. Aggarwal, 1981) The dam will be the third highest concrete dam (163 meters) in India, the first two being Bhakra (226 metres) in Himachal Pradesh and Lakhwar (192 meters) in Uttar Pradesh.

4.1.2 Properties of Dam material

- Modulus of elasticity = 2.1×10^6 t/m^2
- Poisson's ratio = 0.2
- Density = 2.5 t/m^3
- Material damping =0.07

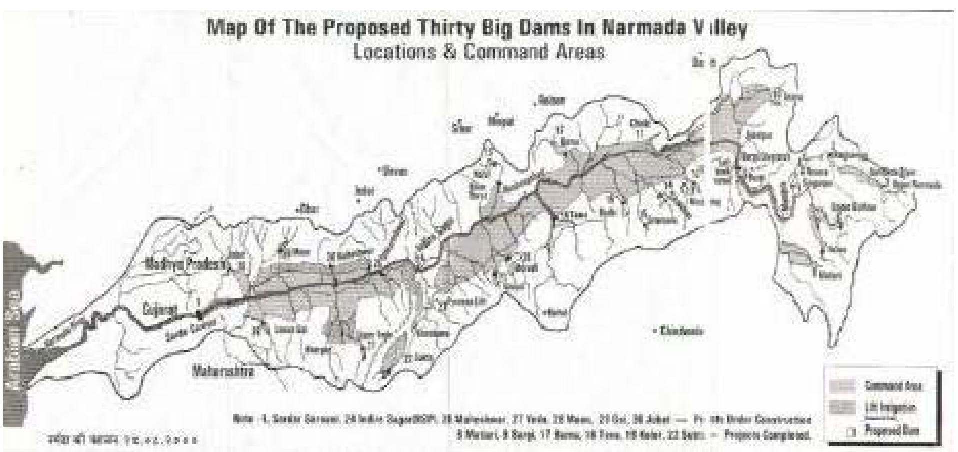


Fig 4.1 Map of the proposed 30 big dams in Narmada valley

4.1.3 Properties of Foundation

The foundation is divided into 3 zones namely zone A, zone B and zone C, as shown in fig 4.4. Properties for different zones are shown in Table 4.1. Model is analyzed for six different cases, in which foundation properties of three zones are taken as variables for five cases, also mentioned in Table 4.1. In last case dam is analyzed with fixed base condition without considering foundation.

Material damping is 0.07, which is same for all zones.

Case	Zone	Modulus of Elasticity (t/m2)	Poisson's ratio
I	A	800000	0.25
	B	50000	0.4
	C	500000	0.3
II	A	2100000	0.2
	B	50000	0.4
	C	2100000	0.2
III	A	2850000	0.2
	B	50000	0.4
	C	2850000	0.2
IV	A	2100000	0.2
	B	2100000	0.2
	C	2100000	0.2
V	A	6300000	0.2
	B	6300000	0.2
	C	6300000	0.2
VI	Fixed Base		

Table: 4.1 Foundation Properties

4.2 Mathematical model

Two dimensional finite element technique is adopted for analysis and finite element model is prepared in SAP 2000 software by using plane elements. Fig 4.2 to 4.4 shows the finite element mesh generated to model dam and foundation. 896 elements are used to represent the dam and 2240 elements for foundation geometry. The base width of the dam at foundation level is 135 m and height of dam is 125 m. The size of foundation block is 770m wide and 175m deep.

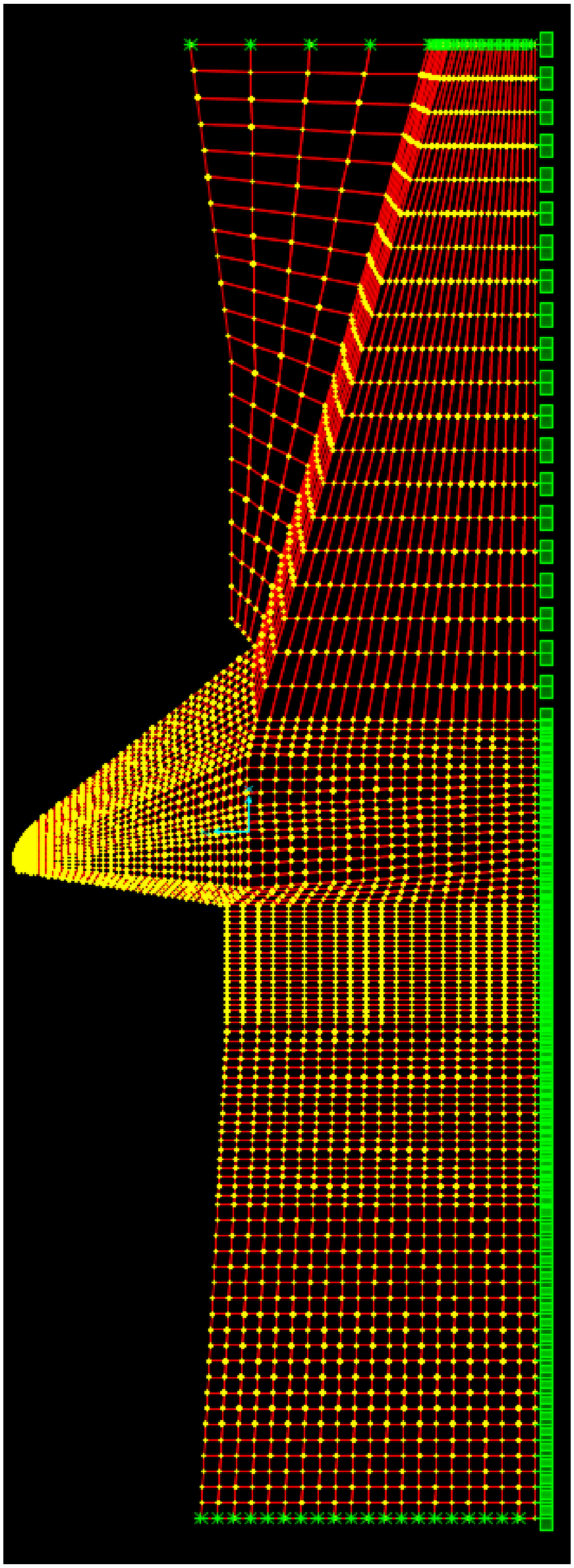


Fig 4.2: Finite element model of Dam and Foundation

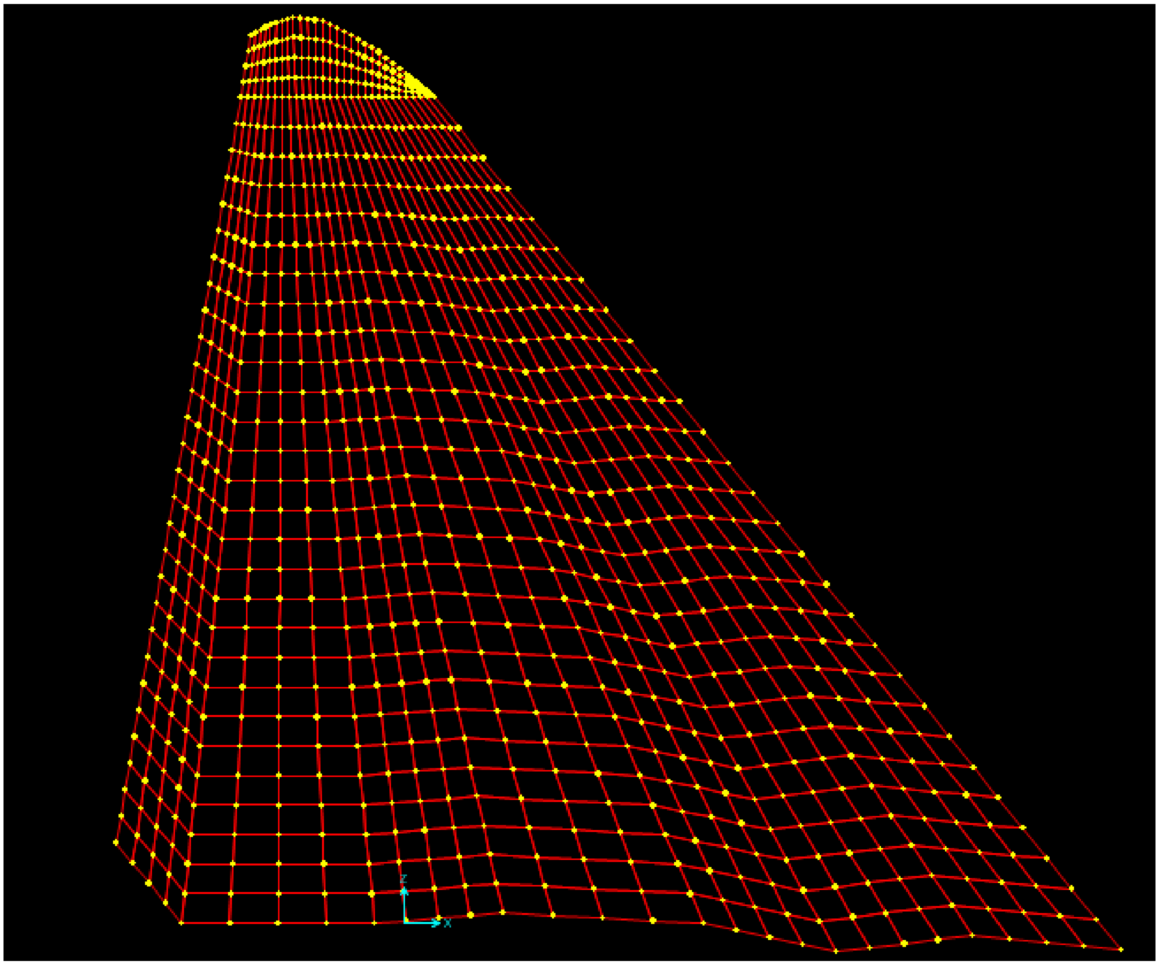


Fig 4.3: Finite element model of dam

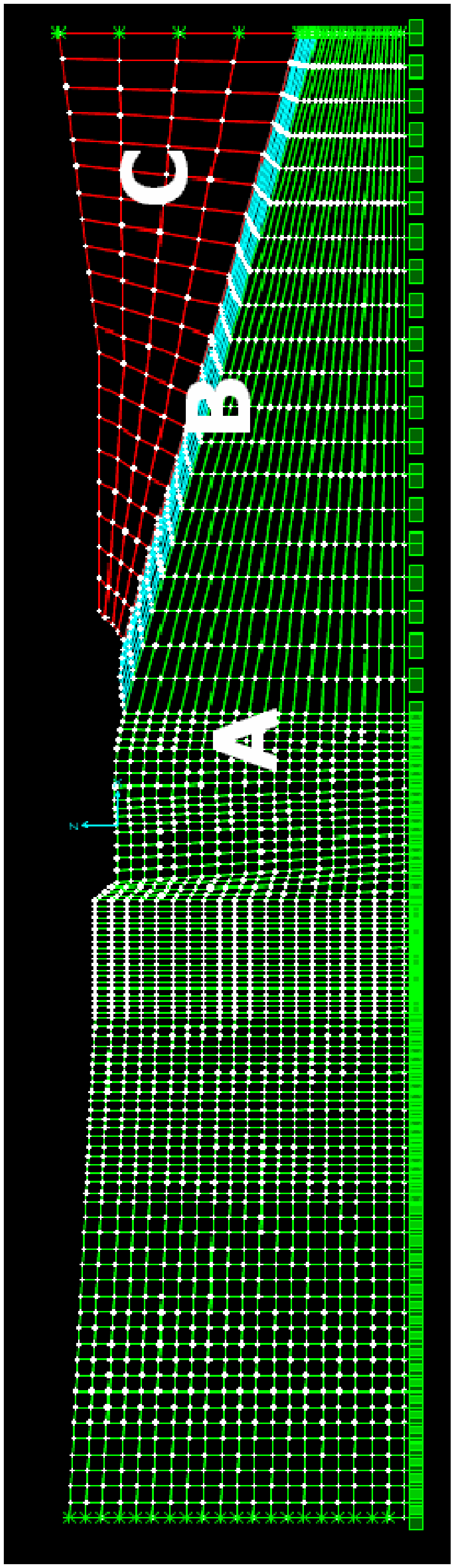


Fig 4.4: Finite element model of Foundation
Green colour for Zone A
Blue colour for Zone B
Red colour for Zone C

4.3 Load Combination

Model is analysed for following load cases:

- Self weight
- Hydrostatic pressure
- Uplift pressure
- Earthquake load (time history analysis)
- Static load combination = Self weight + Hydrostatic pressure + Uplift pressure
- Dynamic load combination = Static load combination + Earthquake load

4.4 Time History Input

Two different time-history inputs are used to perform dynamic analysis for finding response of the dam.

4.4.1 Koyna Accelerogram

Both horizontal and vertical components of earthquake are applied simultaneously. Modification factors that are used for these accelerograms are obtained as a ratio of spectral intensities of recorded Koyna spectra and proposed intensity for Navagam dam. On this basis, horizontal accelerogram would be 0.53 times horizontal component of Koyna and vertical accelerogram 0.3 times the vertical component of Koyna (Chandrasekaran A.R., I.S. Srivastave, B.C. Mathur, S. Basu, R.C. Agrwal, 1981) [8] [9]. But above study is not included in this work. Fig 4.5 and 4.6 shows Horizontal and vertical accelerogram for Koyna earthquake.

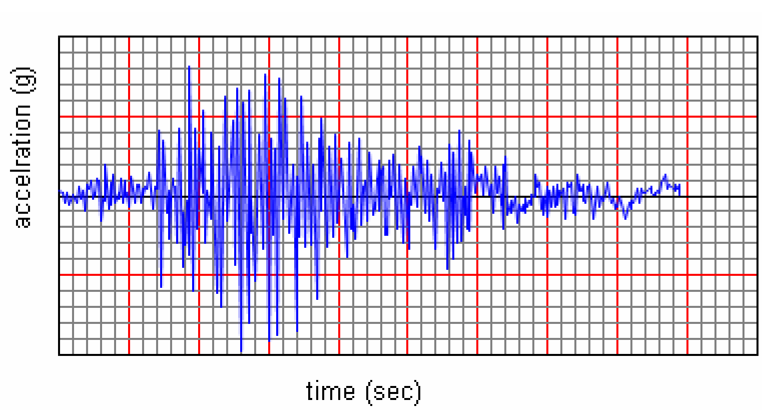


Fig 4.5: Horizontal Koyna Accelerogram

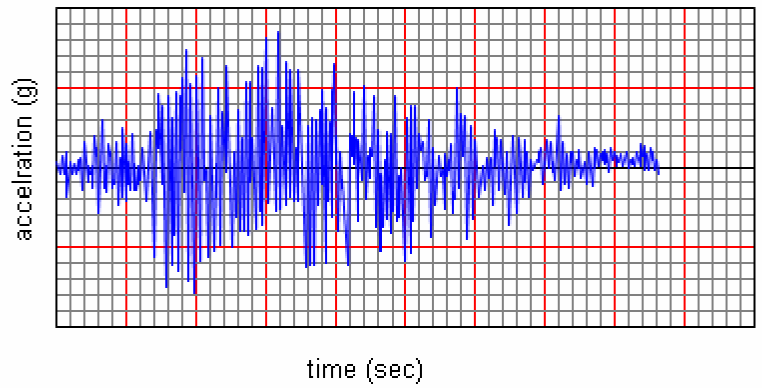


Fig 4.6: Vertical Koyna Accelerogram

4.4.2 Design Accelerogram

Design accelerogram is derived from design response spectrum, which is based on the Koyna earthquake spectrum smoothed and adjusted for geological difference at site of Navagam dam as compared with Koyna (Chandrasekaran A.R., I.S. Srivastave, B.C. Mathur, S. Basu, R.C. Agrwal, 1981)[8] [9] . But above study is not included in this work. Both horizontal and vertical components of design earthquake are applied simultaneously. Fig 4.7 and 4.8 shows horizontal and vertical accelerogram for design earthquake.

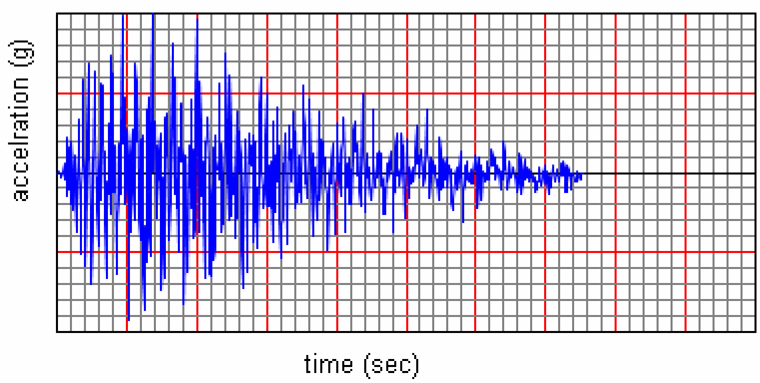


Fig 4.7: Horizontal Design Accelerogram

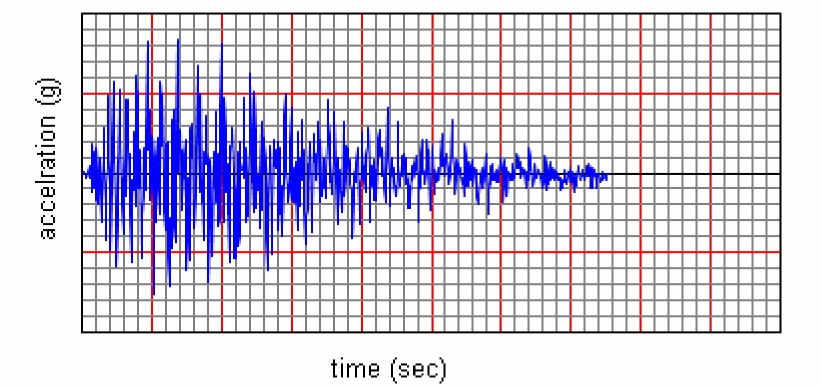


Fig 4.8: Vertical Design Accelerogram

Dynamic analysis of dam is performed by considering Dam-Foundation interaction and Dam-Water interaction.

4.5 Dam-Foundation Interaction

The Dam-Foundation interaction is considered by including foundation portion into model and assuming massless foundation. Boundary conditions for foundation are fixed at base and vertical roller supported at sides.

Model is analysed with and without considering foundation to study the effect of Dam-Foundation interaction and compare results.

4.6 Dam-Water Interaction

The Dam-Water interaction is represented by a virtual mass of water attached to the upstream face of dam. The virtual mass is evaluated using **Zanger** method and neglecting compressibility of water.

I.S. 1893-2002 (clause 7.2) gives expressions for hydrodynamic pressure at depth y below the reservoir surface on dams, which is mentioned below.

$$p = C_s \alpha w h \tag{4-1}$$

Where,**p** = Hydrodynamic pressure at depth y,

$\mathbf{C_s}$ = Coefficient which varies with depth and shape of dam.

α = Design horizontal seismic coefficient which is taken as 0.125g

w = Unit weight of water

h= Depth of reservoir

Coefficient $\mathbf{C_s}$ is found out as follows :

$$C_s = \frac{C_m}{2}\left[\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right] \qquad (4\text{-}2)$$

Where,

$\mathbf{C_m}$ = Maximum value of $\mathbf{C_s}$ obtain from graph in Fig 4.8

y = Depth below surface

h = Depth of reservoir

It can be used directly if water-structure interaction effects are ignored for dynamic analysis by treating it as uncoupled problem. As an approximation to consider coupling effects, convert pressure as equivalent mass (hydrodynamic pressure will give rise to equivalent nodal forces and from force, virtual mass is obtained) lumped at upstream nodes. In this compressibility of water is ignored (Chandrasekaran A.R and Jai Krishna , December 1976; U.S. Army Corps of Engineers, 2000).

Dam-Water interaction effect can be studied by comparison of results of dam response with empty and full reservoir condition.

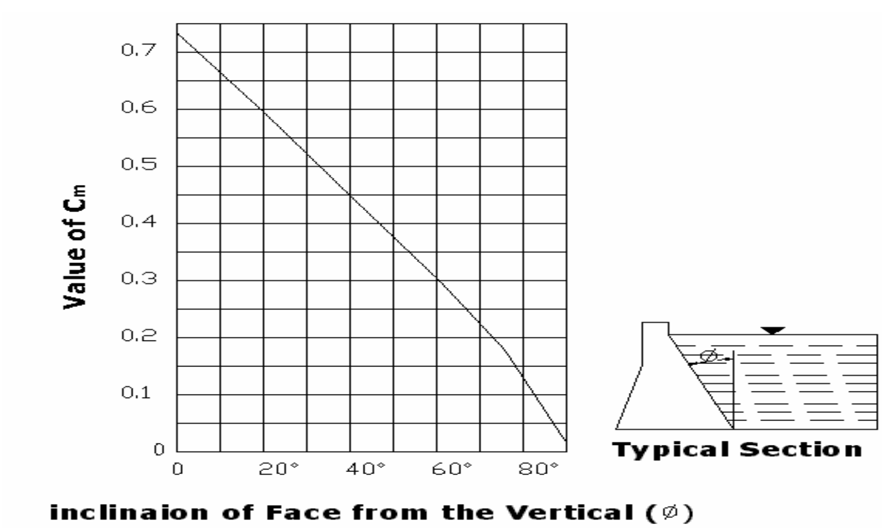


Fig 4.9: Maximum Value of Pressure Coefficient (C_m)

4.7 Modeling in Sap 2000 Software

4.7.1 Creating geometry

From studying data of dam, extreme outer points of the geometry are defined which is necessary for creating dam profile by using area element. By using Edit>Mesh Area command, geometry is discretised into finer mesh as shown in Fig.4.2.

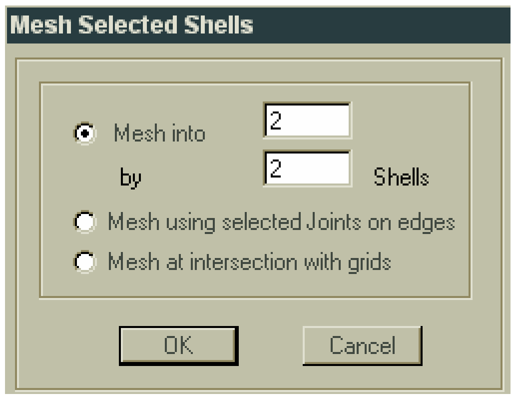


Fig4.10: Mesh command Dialog Box

4.7.2 Define material

On the Define menu, click Materials, which displays the materials dialog box with default materials like concrete and others. Then assign properties of concrete like Modulus of Elasticity, Poisson's Ratio etc. as mentioned earlier. Also define ROCK A, ROCK B and ROCK C materials and assign the properties as mentioned in Table 4.1 respectively for foundation.

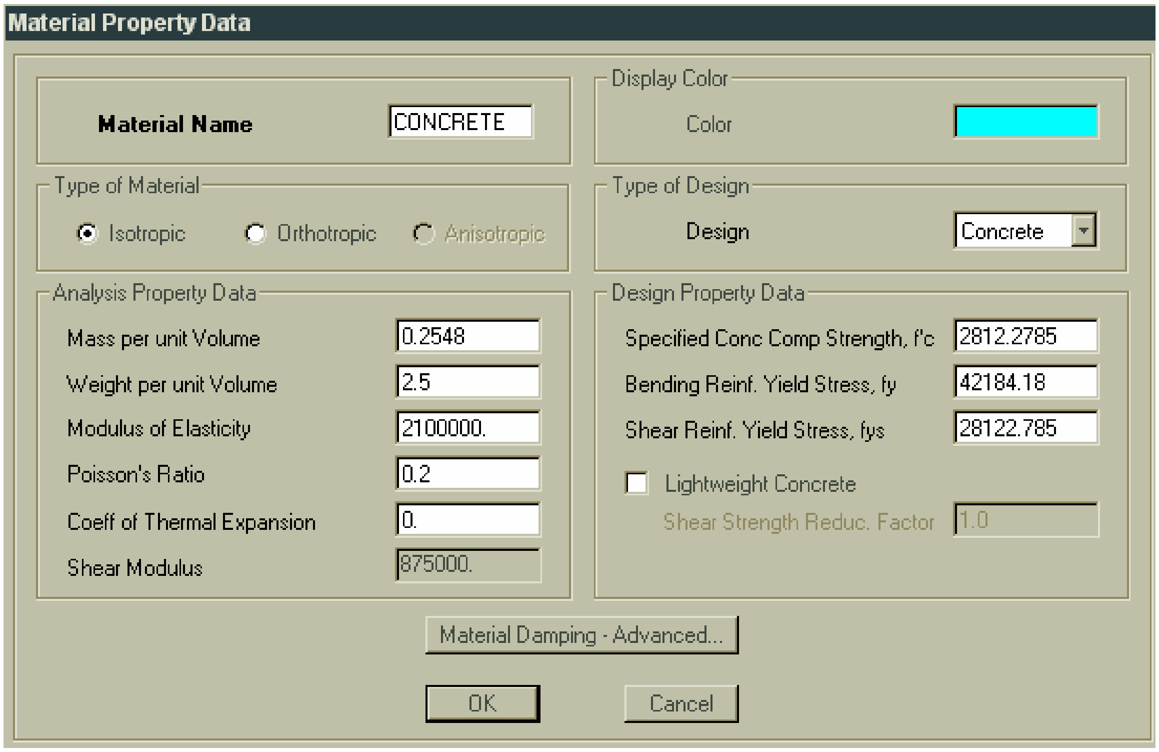


Fig4.11: Define Material command Dialog Box

4.7.3 Define Plane Element:

When we create geometry by using area element, by default the software considers it as a shell element. In this work plane elements are used so that to convert shell element into plane element, use define Menu>Area Section>Modify Option, which displays Area section dialog box. Define four types of plane elements dam, zone A, zone B and zone C each having different material i.e. concrete, ROCK A, ROCK B and ROCK C respectively. Also assign the unit thickness and define problem as plane strain problem.

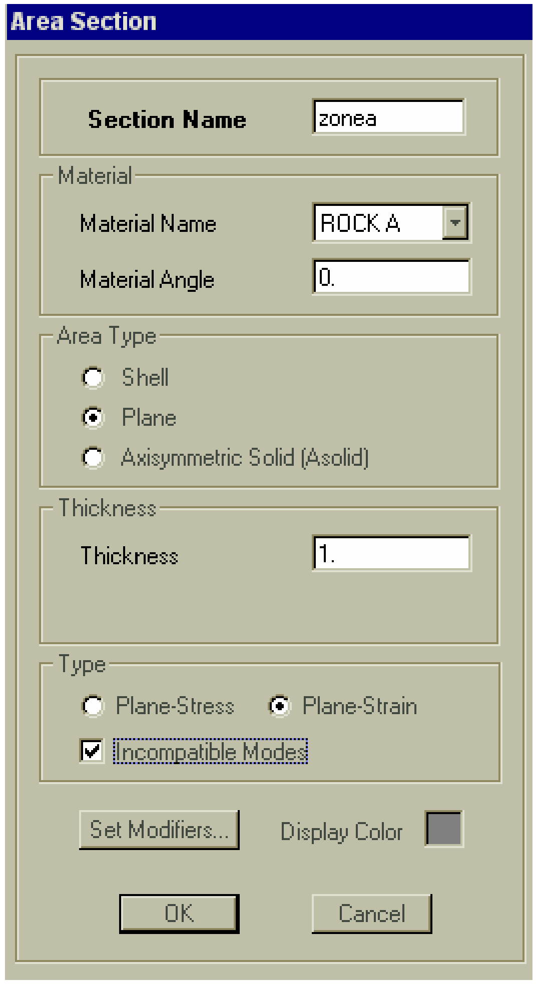


Fig 4.12: Area section Dialog Box

After defining four types of plain elements dam, zone A, zone B, zone C assign them to model to corresponding dam, and different Zones of rock A, B, and C respectively by using assign>area>section option.

4.7.4 Boundary Condition for Foundation

Assign fixed supports at base and vertical guided roller supports at sides of foundation block by using Assign>Restraints option.

Fig4.13: Joint Restraint command Dialog Box

4.7.5 Define Load Cases:

To apply loads on the model, first define load cases by using Define>Load case option. Using this option, define load cases selfweight, water pressure, downwaterload, uplift pressure, time design, and time koyna load cases.

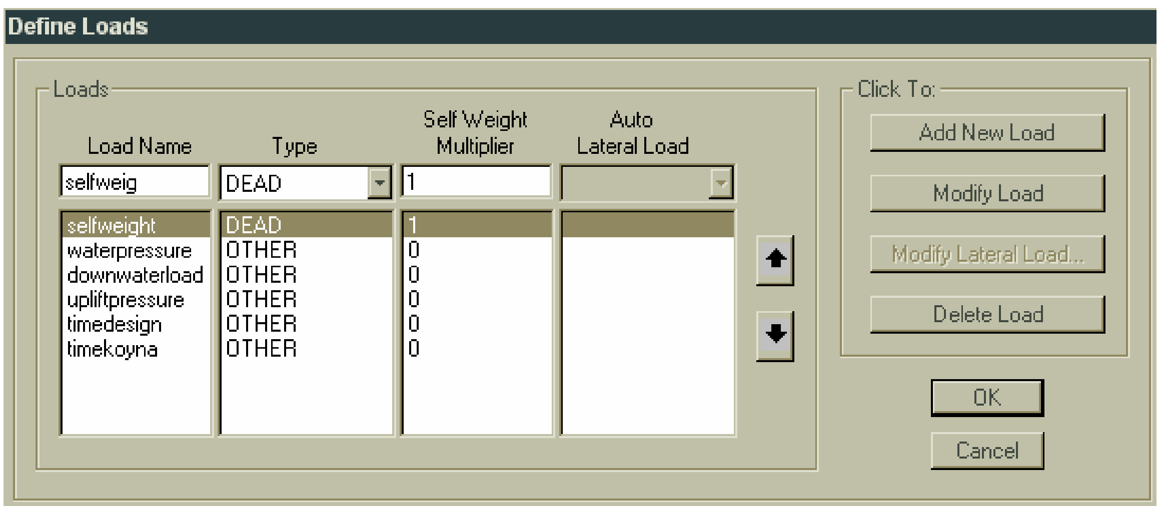


Fig4.14: Define Load Case Dialog Box

4.7.6 Self weight

When we create model, sap automatically generates load case for selfweight and assigns selfweight to model.

4.7.7 Hydrostatic Load

Two components of hydrostatic load, horizontal water pressure and vertical weight of water, which are likely to act on the upstream side, were considered in analysis.

A separate file is created where pressure in form of Uniformly Varying Load is applied on line elements used at the face on the upstream side with fixed support condition at all nodes at that face. The resulting reactions give nodal load values, which are applied as hydrostatic nodal load at upstream side. To apply nodal loads use Assign>Joint load option.

Apply nodal loads for horizontal pressure in load case water pressure and vertical weight of water load in downwaterload case.

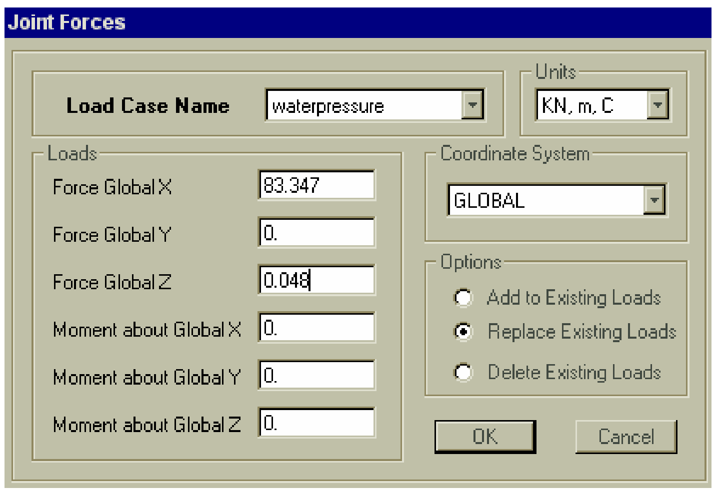


Fig4.15: Assign Joint Load Command Dialog Box

4.7.8 Uplift Load

Uplift pressure at the contact between dam and foundation is evaluated by assuming a linear pressure distribution. At upstream face uplift pressure is taken as 100% intensity of hydrostatic pressure and 33.333% intensity at the line of foundation gallery line and 0% at downstream side.

Similar method is used to found the nodal loads for uplift pressure, which were applied at the nodes at contact of dam with foundation.

4.7.9 Define Accelerogram for Time History Analysis:

To define accelerogram use define>time history function. By using this option define four time histories as mentioned earlier.

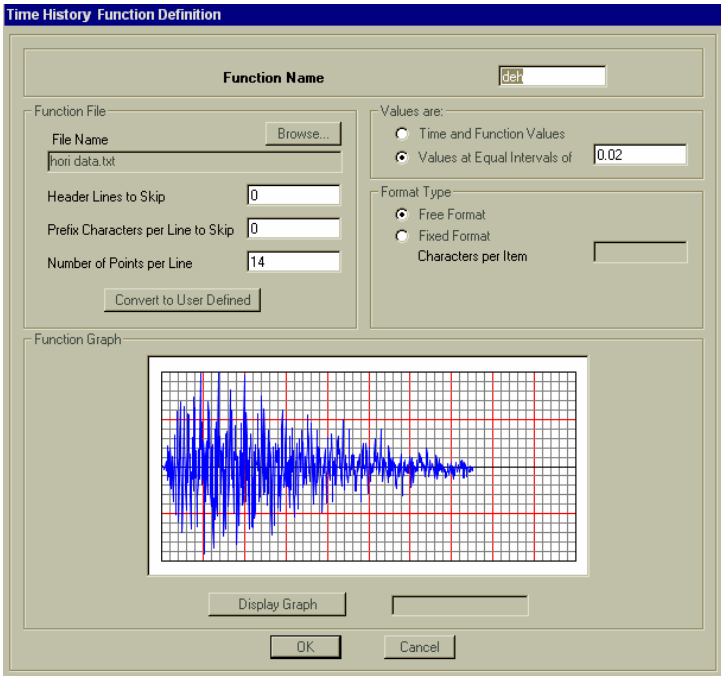


Fig4.16: Define Time History Function Dialog Box

4.7.10 Assign mass at upstream face of dam

As explained earlier, for consideration of Dam-Foundation interaction, assign lumped mass at upstream node.

To calculate pressure as per I.S. 1893 clause 7.2, excel file has been prepared and then to find nodal loads from pressure, same procedure has been used to find nodal loads from hydrostatic pressure, is adopted. Then these nodal loads are converted in equivalent mass and this mass is applied at the node on upstream face of dam by using Assign>Joint>Mass option.

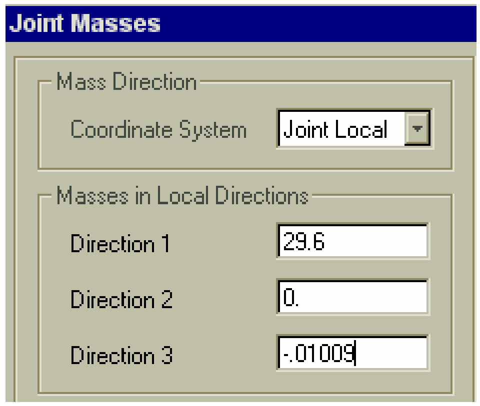


Fig4.17: Assign Joint Mass Dialog Box

4.7.11 Analysis Cases

SAP automatically generates analysis cases for loads when user defines load cases. But user has to assign type of analysis like static, time history etc. to all of these cases.

Also SAP generates modal analysis case by default. But to calculate modes of structure define Eigen vector method and also specify maximum number of modes used in dynamic analysis.

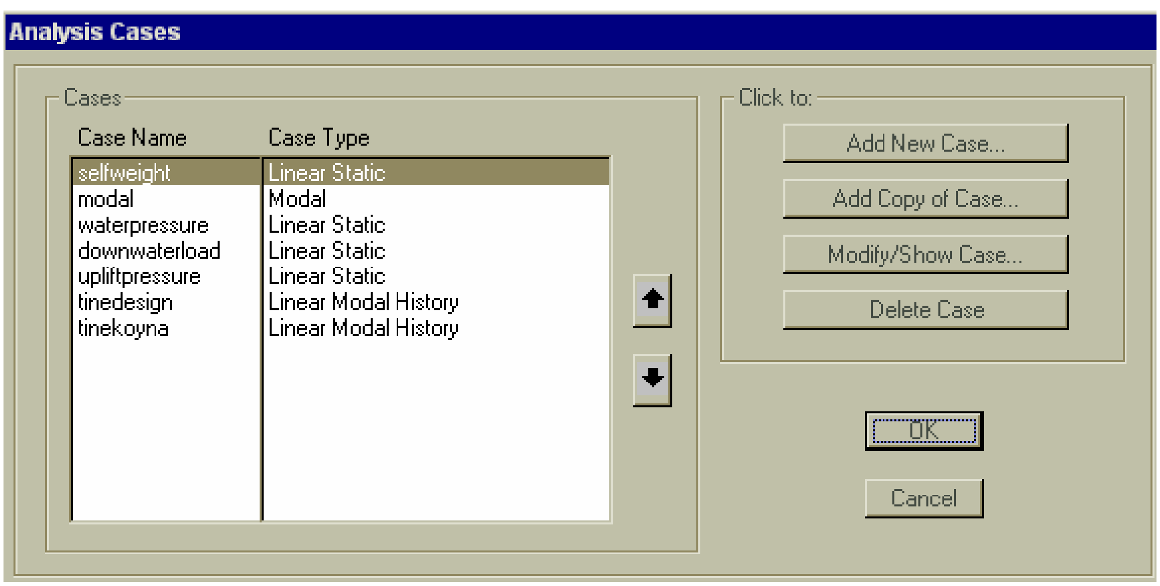


Fig4.18: Analysis Cases Command Dialog Box

For first four cases selfweight, water pressure, downwaterload, uplift pressure assign static analysis type.

For time design and time koyna load cases assign linear time history analysis type. Also specify that modal analysis used for time history and history of motion is transient. Assign load type as acceleration with direction in which earthquake was applied, which time history function (accelerogram) used in analysis, number of time step and time step size.

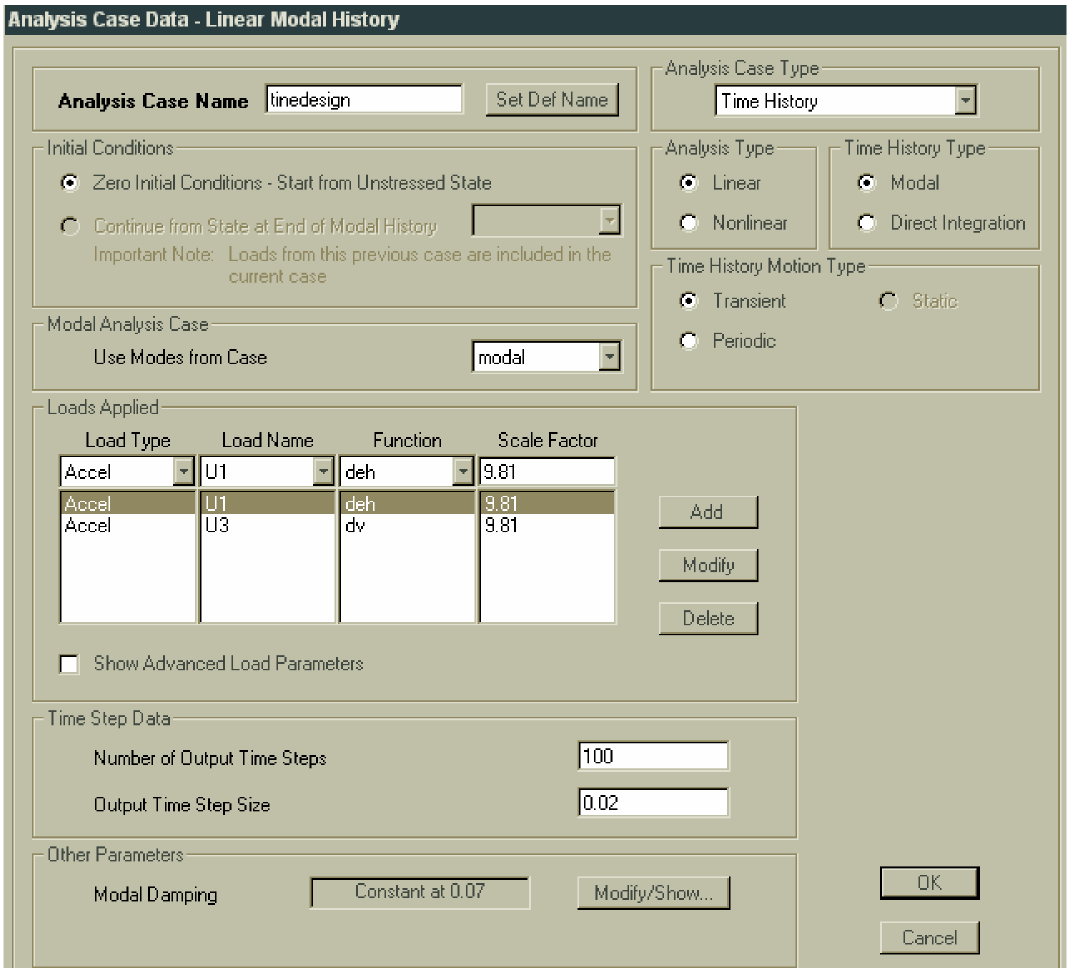


Fig4.19: Define time history Analysis Dialog Box

4.7.12 Load combination

To define load combination, use define>combination command. In this three combinations were made.

- Static: In this analysis, results of analysis cases Selfweight, water pressure, downwaterload, and uplift pressure are linearly added

- Design: In this, static combination and time design analysis case results are linearly added.
- Koyna: In this, static combination and time koyna analysis case results are linearly added.

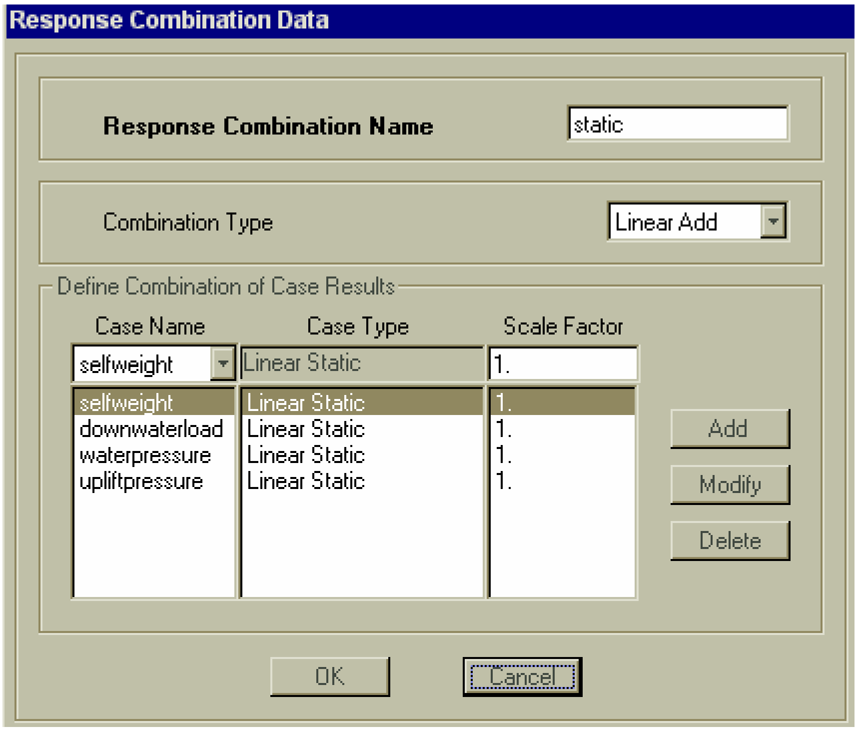


Fig4.20: Define combination Dialog Box

4.7.13 Run Analysis

To run analysis first specify that analysis is a two dimensional analysis from analysis>set analysis option. Then run the analysis by using analysis>run analysis option.

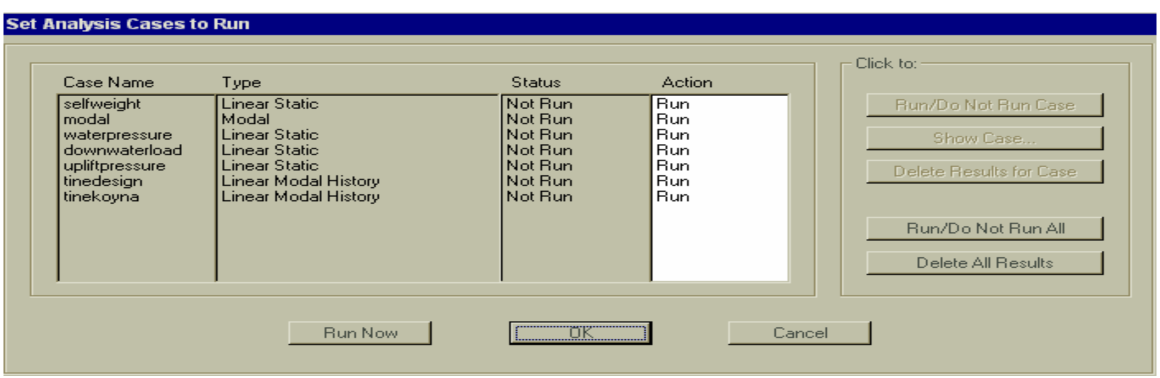


Fig4.21: Run Analysis Command Dialog Box

4.7.14 Input and Output Table

In Sap 2000 software Input information of model in form of tables obtain from display>show model definition table.

Similarly output information in form of tables obtain from display>analysis result tables.

CHAPTER 5
RESULTS AND DISCUSSION

5.1 Results and Discussion

The combined response of dam for all six cases due to static and dynamic load is presented in this chapter. Table 5.1 shows the comparison of natural periods of dam for first three modes. Fig 5.1 to 5.3 shows three vibration modes of dam for only first case. In analysis two types of time histories are used, but only results due to Koyna history are near by the results mentioned in report. So results due to Koyna history are compared with report. Table 5.2 shows the comparison of maximum horizontal and vertical displacement of dam for static and dynamic Loads.

Maximum horizontal displacement is occurred at the node 6 which is a top most point of dam at crest level for all six cases. Fig 5.4 shows the horizontal displacement response for node 6 in the first case due to Koyna history. The displacement variation with time due to Koyna history at nodal points 6, 2626 and 7, which are located at top, middle and bottom levels on the upstream side of dam are also shown in Fig 5.4. Maximum response is about 20.30 mm at 3.52 sec for node 6, 11.28 mm at 3.50 sec for node 2626 and 4.49 mm at 3.44 sec for node 7, so response become smaller at the lower level.

Maximum vertical displacement is occurred at different nodes on downstream face of the dam due to dynamic load for different cases. For first case this occurred at the node 3148 which is near the bottom of dam for dynamic load. Maximum response is about 4.706 mm at 3.54 sec due to Koyna history. Fig 5.5 shows the vertical displacement response for node 3148 for first case for Koyna history.

case	Mode 1(H)		Mode 2(V)		Mode 3 (HV)	
	Report	Sap	Report	Sap	Report	Sap
I	0.698	0.746	0.342	0.317	0.259	0.273
II	0.514	0.561	0.24	0.228	0.2	0.212
III	0.478	0.522	0.22	0.214	0.187	0.195
IV	0.473	0.522	0.231	0.223	0.197	0.208
V	0.39	0.468	0.18	0.202	0.161	0.178
VI	0.342	0.379	0.155	0.168	0.133	0.127

Table 5.1: Comparison of Time Period (Sec)

case	Loading combination	Max Horizontal		Max Vertical	
		Report	Sap	Report	Sap
I	Static	45.76	38.81	-33.77	-28.73
	Dynamic	63.81	59.11	-37.05	-33.35
II	Static	24.86	22.20	-16.52	-14.65
	Dynamic	39.16	40.09	-19.4	-18.56
III	Static	21.28	19.12	-13.44	-12.22
	Dynamic	39.67	35.35	-16.38	-15.11
IV	Static	16.9	15.76	-7.27	-12.65
	Dynamic	32.67	32.13	-8.72	-15.81
V	Static	11.27	12.51	-2.34	-9.56
	Dynamic	19.45	27.14	-2.75	-13.80
VI	Static	8.54	7.46	-0.93	-5.20
	Dynamic	16.26	19.95	-1.06	-8.34

Table 5.2: Comparison of Deflection (mm)

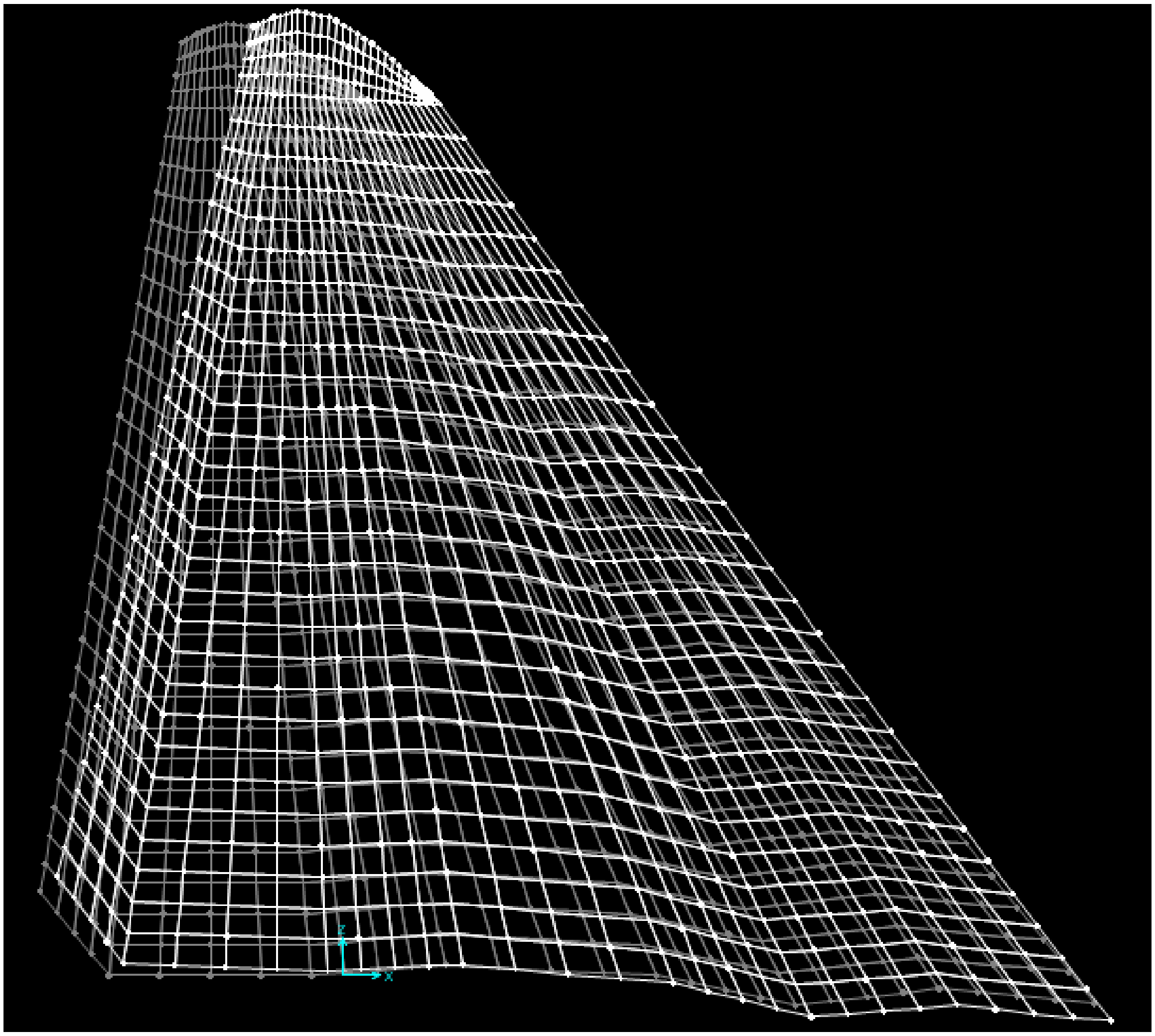


Fig 5.1: Mode 1(H) Time Period= 0.746 Sec.

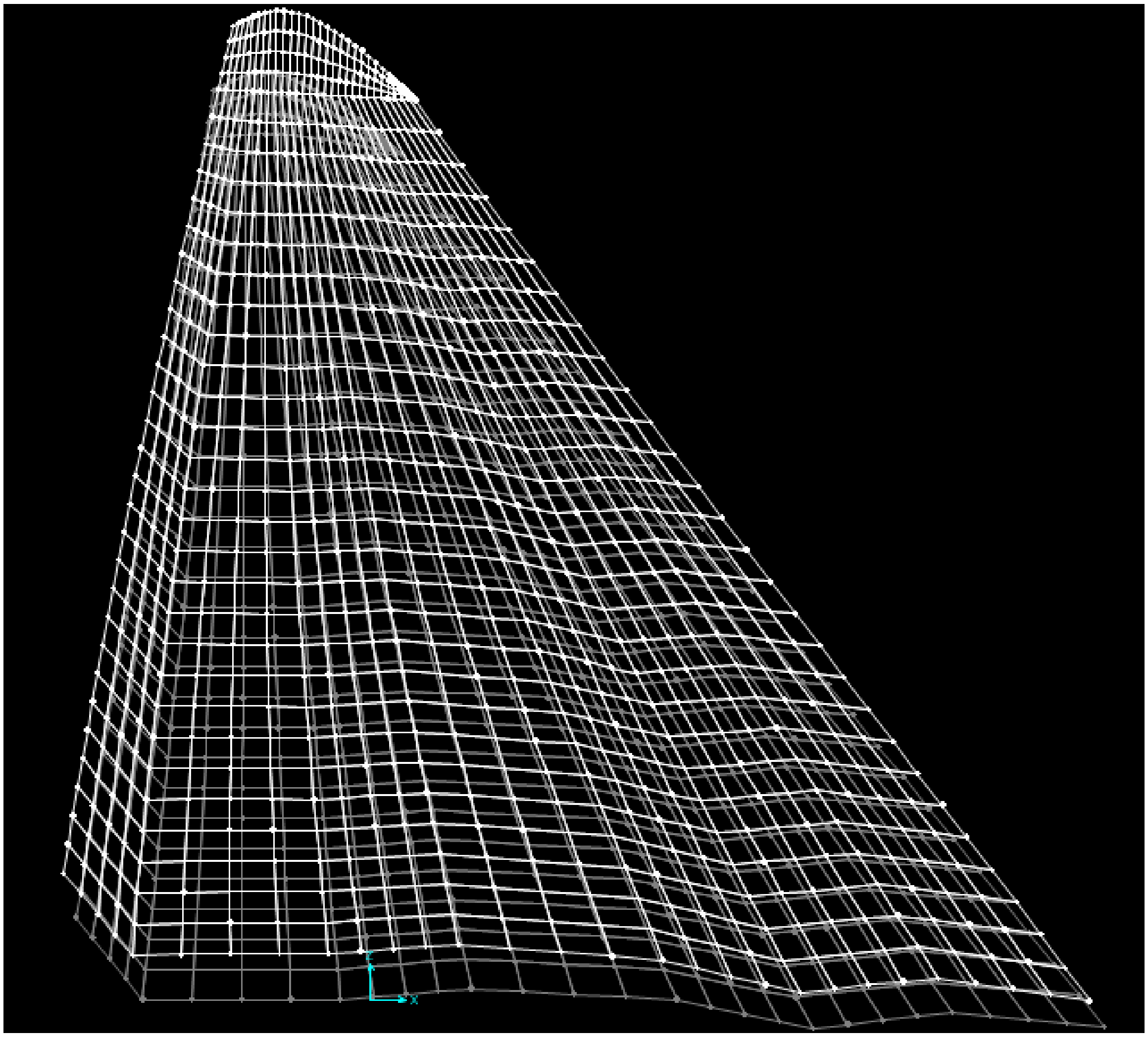


Fig 5.2: Mode 2(V) Time Period= 0.317 Sec.

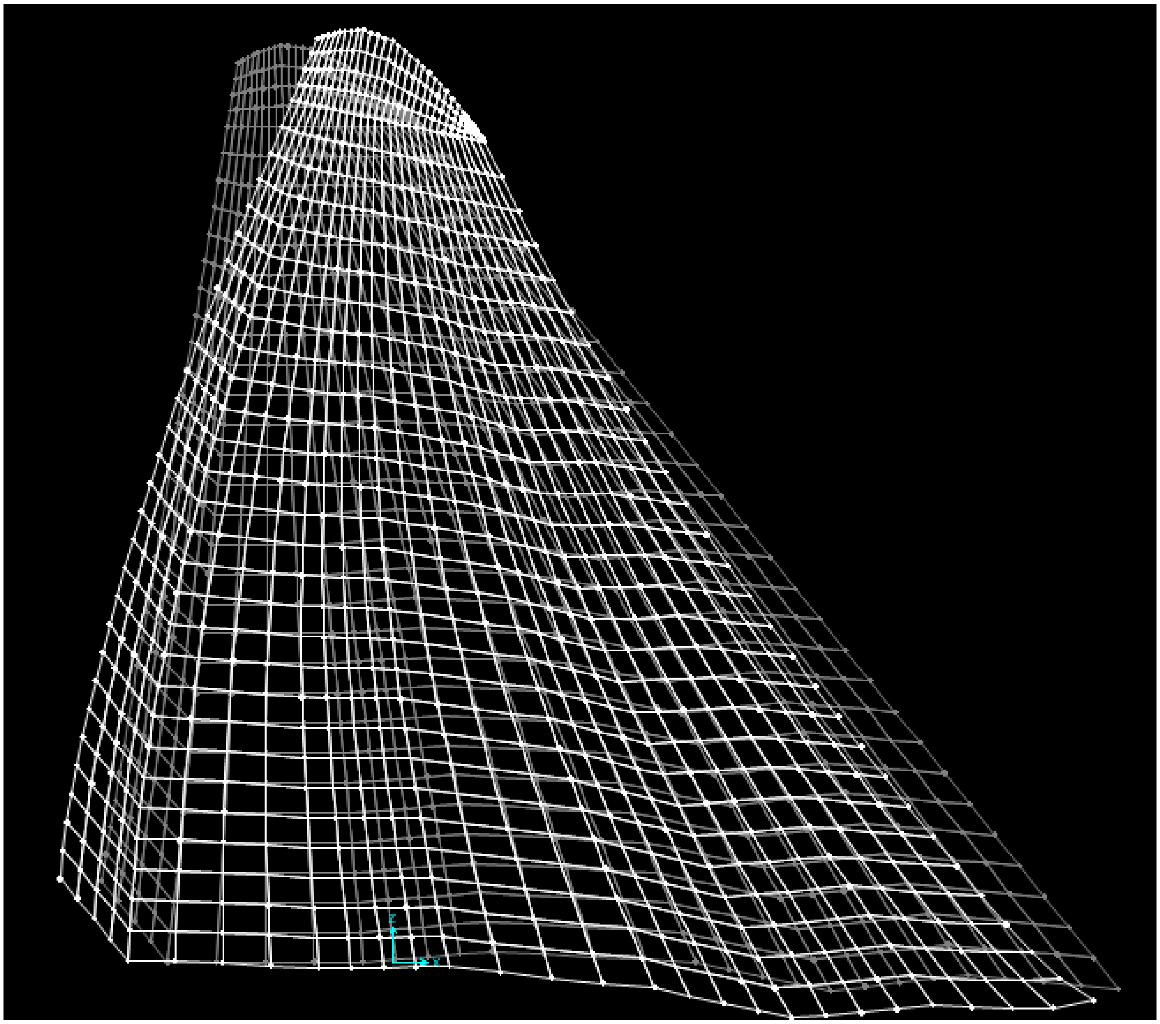


Fig 5.3: Mode 3(HV) Time Period= 0.273 Sec.

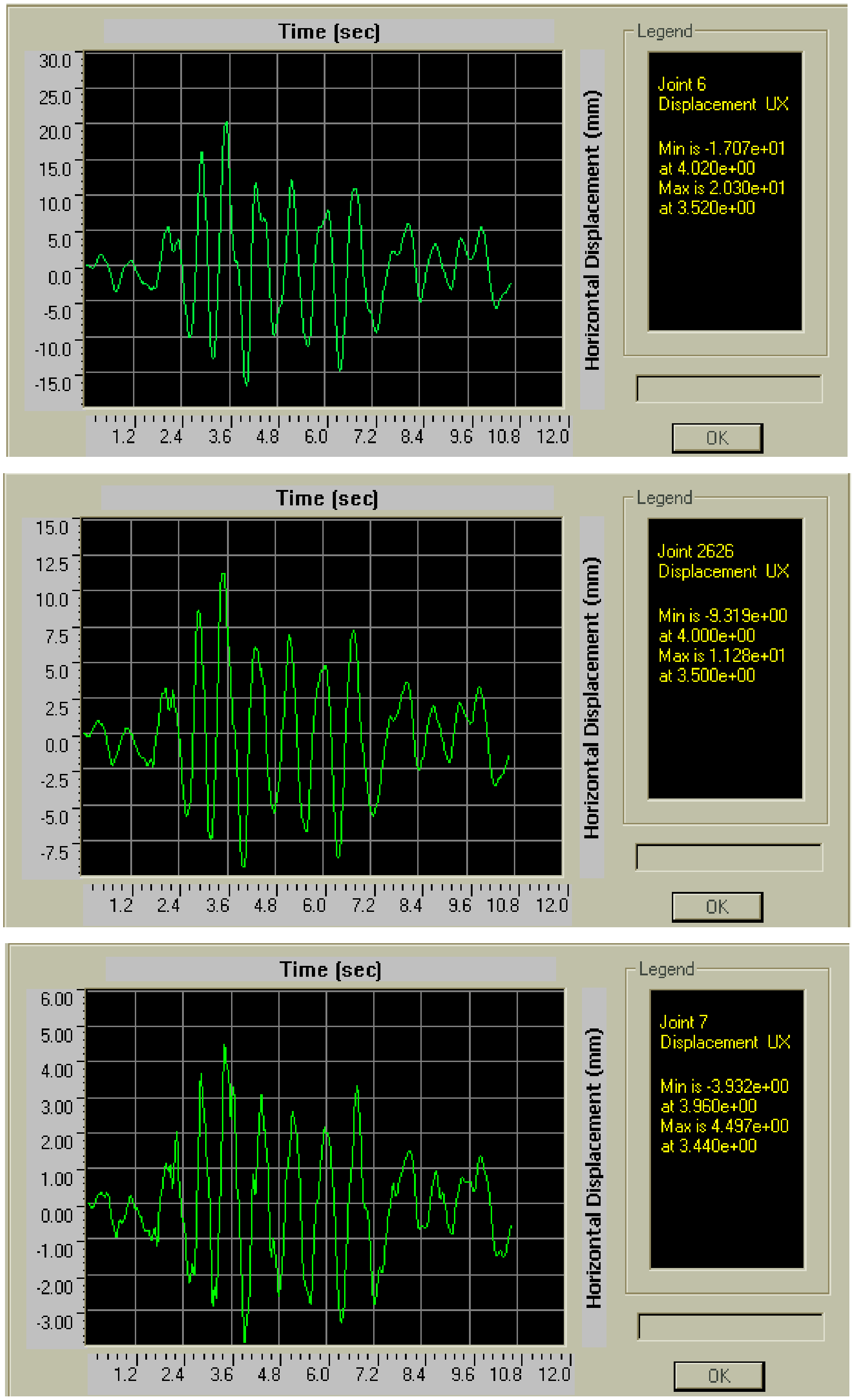


Fig 5.4: Horizontal Displacement Response of Nodes 6, 2626,and 7 due to Koyna Accelerogam.

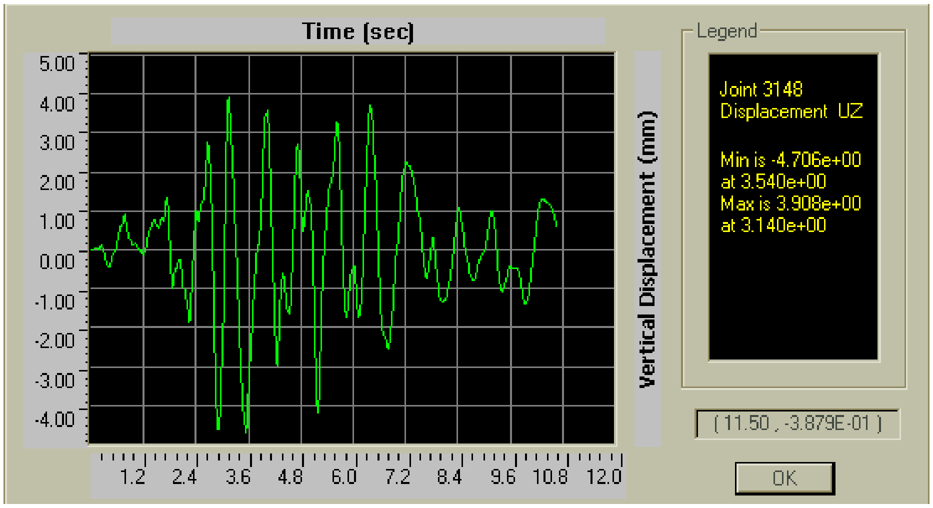


Fig 5.5: Vertical Displacement Response of Node 3148 due to Koyna Accelerogam.

Maximum horizontal acceleration is occurred at top most point of dam at crest, which is about 3.161 m/sec^2 and maximum vertical acceleration 1.968 m/sec^2 at node 2678 near the crest level. Fig 5.6 shows the horizontal acceleration response of node 6, 2626 and 7. Acceleration response increased with respect to height of dam.

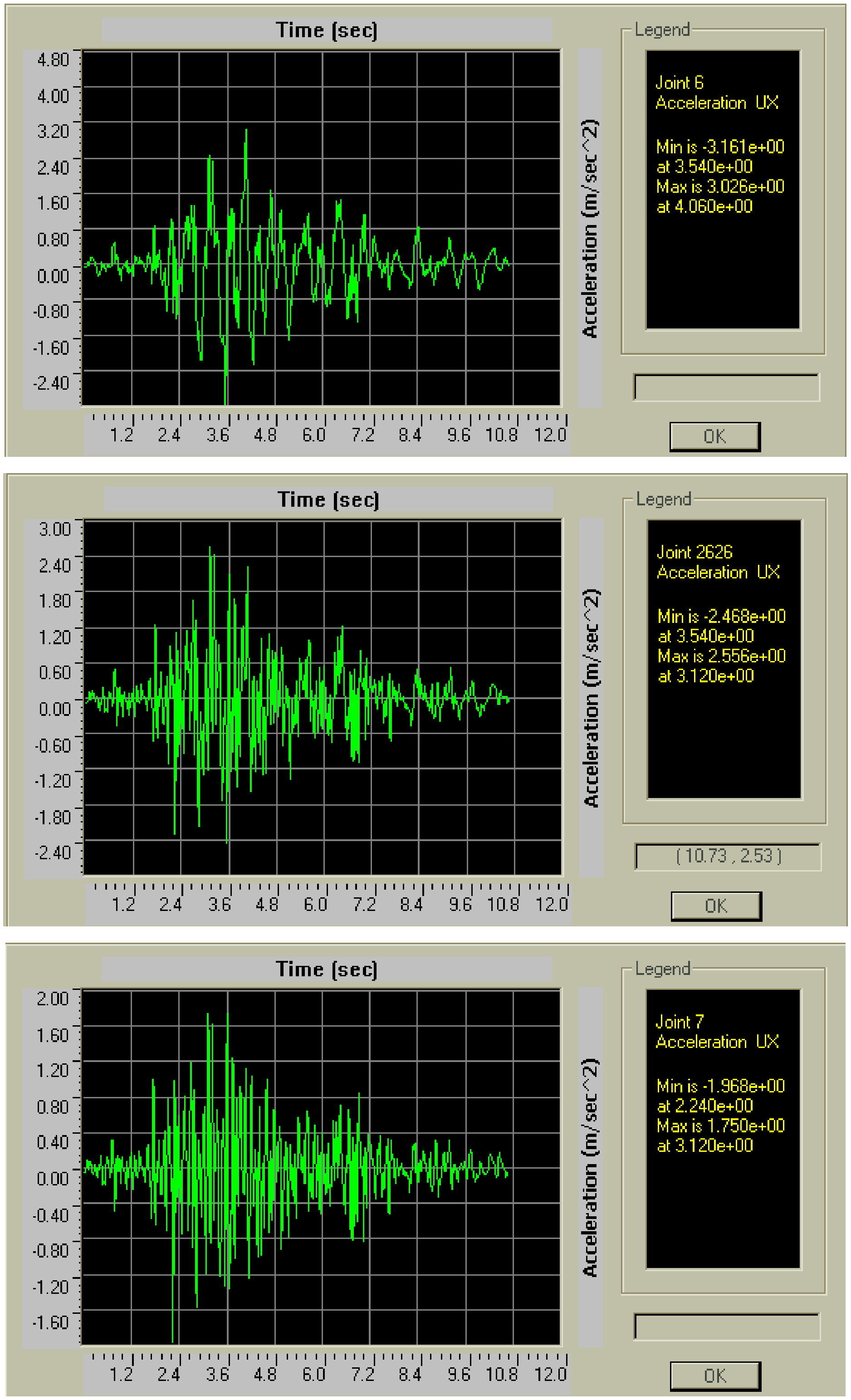


Fig 5.6: Acceleration response of Nodes 6, 2626, and 7 due to Koyna Accelerogam.

5.2 Study of Dam-Foundation Interaction

Table 5.3 show the comparison of fundamental time period and displacement response of dam when foundation portion (foundation properties as per first case) is considered and dam with fixed base condition. Fig 5.7 shows the maximum horizontal displacement response of dam for both conditions.

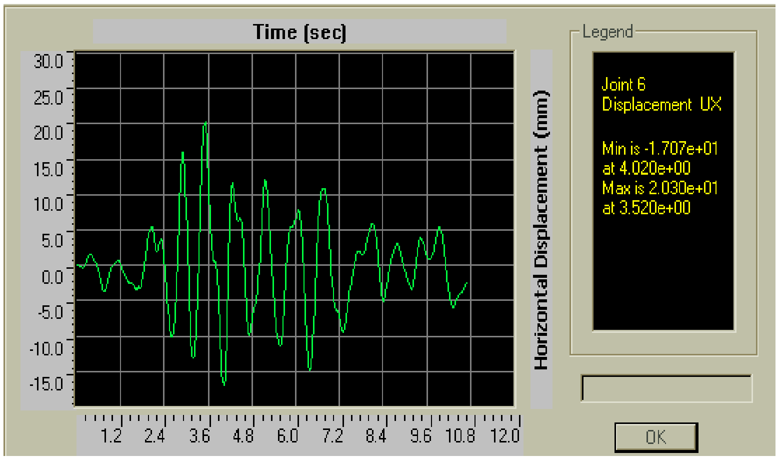
This results clearly show that Dam-Foundation interaction have significant effect on dynamic analysis of dam. It lengthens fundamental time period of dam hence increases the response of dam.

The two properties of foundation have a significant influence on the dynamic response of dam.

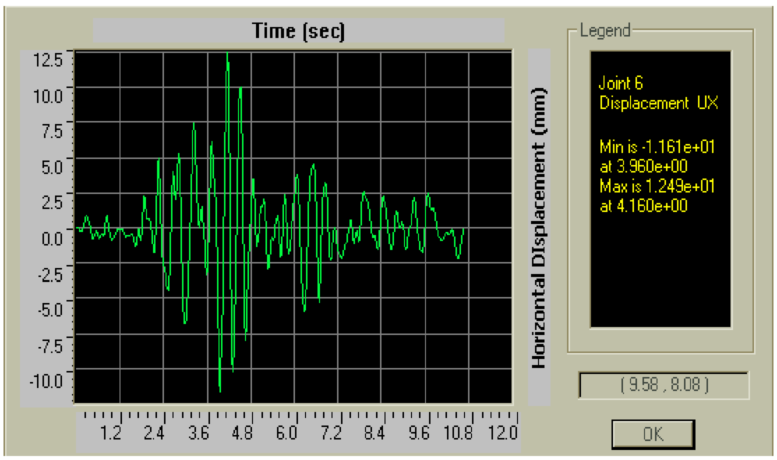
- Damping ratio: The damping characteristic of foundation contribute to the damping in combined dam foundation system. For first case model has been analysed for two damping ratio 0.07 and 0.05 of foundation. Fig 5.8 shows the horizontal displacement response of node 6 for both damping ratio, which shows that as damping ratio decreases, response of dam is increased.

Case	**Time period (sec)**			**Horizontal Displacement (mm)**
	Mode 1	**Mode 2**	**Mode 3**	
Considering foundation	**0.746**	**0.317**	**0.273**	**20.30**
Fixed base	**0.379**	**0.168**	**0.127**	**12.49**

Table 5.3: Comparison of Fundamental Time Period and Displacement of dam with Considering Foundation and Dam with Fixed Base.

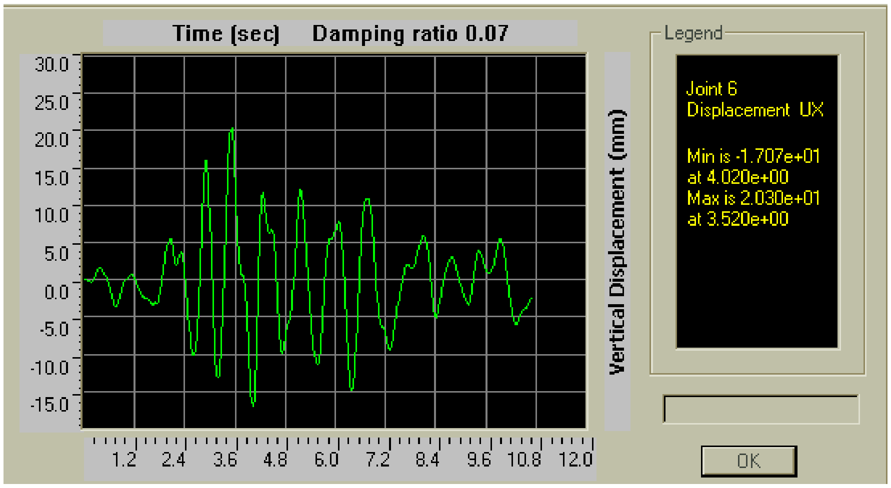


Considering Foundation

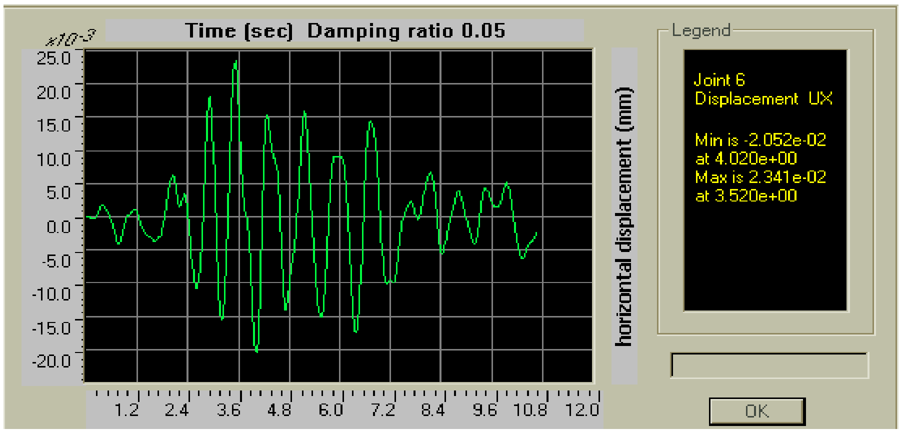


Fixed Base

Fig 5.7: Horizontal Displacement Response of Node 6



Damping ratio = 0.07



Damping ratio = 0.05

Fig 5.8: Horizontal Displacement Response of Node 6 for Damping Ratio 0.07 and 0.05 respectively.

- Modulus Elasticity: The Modulus of Elasticity influences the response of dam because it directly affects modal frequencies of dam foundation system. Modulus of Elasticity for different cases is mentioned in Table 4.1. For different cases fundamental time period of dam is shown in Table 5.1. It clearly shows that as Modulus of Elasticity increases, the fundamental time period of dam decreases i.e. the modal frequencies also increase. Table 5.2 shows that response of dam decreases as the Modulus of Elasticity increases.

5.3 Study of Dam-Water Interaction

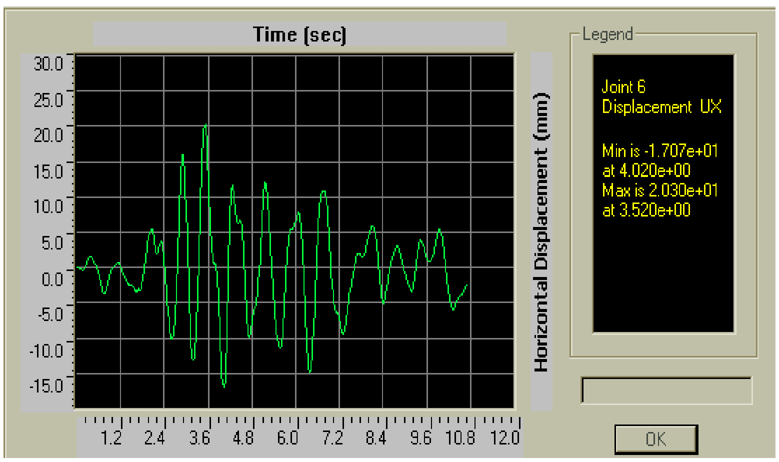
Dam-Water interaction effects can be studied by comparison of dam response with empty reservoir and full reservoir. Table 5.4 shows the comparison of fundamental time period and displacement response of dam for both conditions for only Koyna time history load case.

case	Time period (sec)			Horizontal Displacement (mm)
	Mode 1	Mode 2	Mode 3	
Full Reservoir	0.746	0.317	0.273	20.30
Empty Reservoir	0.554	0.315	0.223	17.03

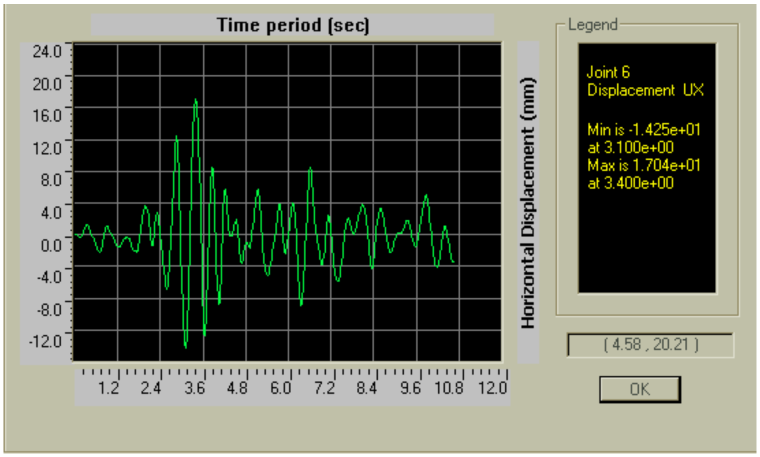
Table 5.4: Comparison of Fundamental Time Period and Displacement of dam For Full reservoir and Empty Reservoir Condition.

Fig 5.9 shows the maximum horizontal displacement response of dam for both conditions.

This result shows that Dam-Water interaction lengthens the fundamental time period of dam because of added mass of water and also affects response of the dam.



Full Reservoir



Empty Reservoir

Fig 5.9: Horizontal Displacement Response of Node 6 for Full Reservoir and Empty Reservoir.

CHAPTER 6
CONCLUSION AND FUTURE SCOPE

6.1 Conclusion

From the results mentioned in previous chapter we can conclude following points:

- Fundamental Time period and maximum deflection response of dam obtained from Finite Element Model of dam in SAP 2000 software are nearby to the results mentioned in Report EQ_81-23 “Analysis of Tallest Overflow Section of Navagam Dam”, which is done by IIT Roorkee in 1981.
- So that by using such software like SAP 2000 with finite element technique can be used for dynamic analysis of concrete gravity dam and find the response during the earthquake and also visualize behavior of dam on screen.
- Dam-Foundation and Dam-Water interaction have significant effect on the response of dam during earthquake so that above interactions should be included in the dynamic analysis of Dam, which are not considered by the IS 1893-2002.

6.2 Future Scope

- In the present analysis only gravity dam is analysed. This can be used for the other types of dams.
- Here, the analysis tool used is SAP 2000 but its inherent capabilities and limitations directly affects the analysis. In future, if better analysis software are available, one can analyze the same model with it for more real anlaysis.
- Here, Dam-water interaction effect is considered as per Zanger method, one can study similar effect by using Westergaard added mass approach.
- Water is taken as incompressible fluid in study of Dam-Water interaction, one can study the same by considering the water as compressible fluid by using A.K. Chopra method.

Reservoir bottom absorption effect is completely ignored in this analysis; one can include this effect.

REFERENCES

1. ANCOLD “Guidelines for Design of Dam for Earthquake”, August 1998, Australian National Committee on Large Dams.

2. ANCOLD “Guidelines on Strengthening and Raising Concrete Gravity Dams”, November 1992, Australian National Committee on Large Dams.

3. ANCOLD,” Guidelines on Design criteria for Concrete of Dams.”, Nov 1991, Australian National Committee on Large Dams.

4. Asgher Vatanani Oskouei and A.A. Dumanoglu, November 2000, “Nonlinear Dynamic Response of Concrete Gravity Dams: Cavitations Effect”, Soil Dynamic and Earthquake Engineering.

5. Bureau Gilles J. “Assessment of Conventional and Advance Procedures for Seismic Safety Evaluation of Concrete Gravity Dams”, Dames and Moore.

6. Bureau Gilles J. “Earthquake Engineering Handbook”, Wai-Fah Chen Charles Scawthorn.

7. Chandrasekaran A.R and Jai Krishna , December 1976, “Elements of Earthquake Engineering”, Sarita Parkashan.

8. Chandrasekaran A.R., I.S. Srivastave, B.C. Mathur, S. Basu, R.C. Agrwal, 1981, “Design Parameters for Navagam Dam in Gujarat”, Project No. P-364, 81-1, Department of Earthquake Engineering University of Roorkee, U.P., India.

9. Chandrasekaran A.R., I.S. Srivastave, B.C. Mathur, S. Basu, R.C. Agrwal, 1981, “Analysis of Tallest Overflow Section of Navagam Dam”, Project No. P-318, EQ-81-23, Department of Earthquake Engineering University of Roorkee,U.P., India.

10. Chopra A.K., 1976, “Earthquake Response Analysis of Concrete Dams”, Design of Gravity Dam, U.S Bureau of Reclaminaton, U.S. Government, Denver.

11. Chopra A.K., December 1986, “Earthquake Analysis, Design and Safety Evaluation of Concrete Gravity Dams” Paper No. 251, Vol. 23, No. 4, Bulletin of Indian Society of Earthquake Technology.

12. Chopra, A.K., 1978, “Earthquake Resistance Design of Concrete Gravity Dams”, Journal of the Structural Division, ASCE, Vol. 104, No. 6.

13. Chopra, A.K., 2002, “Dynamic of Structures: Theory and Applications to Earthquake Engineering”: Interaction “‘, Second Edition, Prentice- Hall of India Private Limited New Delhi.

14. Chopra, A.K., and P. Chakrabarti, 1971, “The Koyana Earhtquake of December 1, 1967 and the Performance of Koyana Dam”, Report No. UCB/EERC-71/1,EERC,University of California, Berkeley.

15. Chopra, A.K., and P. Chakrabarti, May 1973, “EADHI: A Computer Program for Earthquake Analysis of Gravity Dams Including Hydrodynamic Interaction “‘, Report No. EERC/73-7, EERC, University of California, Berkeley.

16. Chopra, A.K., 1967, ”Hydrodynamic Pressures on Dams During Earthquakes.”, Journal of the Engineering Mechanics Divisions, ASCE, Vol. 93 NO. 6,

17. “Dams and Earthquakes” Proceeding of a Conference Hold at Two InstitutTion of Civil Engineering London, 1-2 October 1980 Institute of Civil engineering., Thoms Telford Limited, London.

18. Desai Chandrkant and John F. Abel, 1987, “Introduction to the Finite Element Method”, CBS Publishers and Distributors.

19. Fenves, G., and A.K. Chpora, 1978, “A Simplified Analysis of Earthquake Resistant Design of Concrete Gravity Dams”, Report No. UCB/EERC/85/10, EERC, University of California, Berkeley.

20. Headquarters, U.S. Army Corps of Engineers, 1995. “Gravity Dam Design,” Engineer Manual 1110-2-2200, Washington, DC.

21. Headquarters, U.S. Army Corps of Engineers, 1995. “Factor Significantly Affecting Dynamic Response,” Chapter 7, Engineer Manual 1110-2-12, Washington, DC.

22. Headquarters, U.S. Army Corps of Engineers, 2000. “Time -History Dynamic Analysis of Concrete Hydraulic Structures,” Engineer Circular 1110-2- 6051, Washington, DC.

23. I.S. 1893-1984 “Indian Standard Criteria for Earthquake Resistance Design of Structures- Fourth Revision”, Bureau of Indian Standards, New Delhi.

24. I.S. 6512-1984 “Indian Standard Criteria for Design of Solid Gravity Dams- First Revision” Bureau of Indian Standards, New Delhi.

25. I.S. Sp 22

26. ICOLD, Bulletin 30, "Finite Element Method in Analysis and Design of Dams." 1978, International Committee on large Dams.

27. Madhavan K., June 1987, "Development in Earthquake Resistant Design of Dams", Paper No: 257, Vol. 24, No. 2, Bulletin of Indian Society of Earthquake Technology.

28. Modi P.N., "Irrigation Water Resources and Water Power Engineering", Standard Book House.

29. Pal, N., 1976, "Seismic Cracking of Concrete Gravity Dams.", Journal of Structural Division, ASCE, Vol. 102, NO. 9.

30. Priscu. R., A Popivi, D. Stematiu, and C. Stere, 1985, "Earthquake Engineering for Large Dams" John Wiley and Sons.

31. Rajashekhran S.,2003, "Finite Element Analysis in Engineering Design", S. Chand and Company Limited, New Delhi.

32. Thomas H. Henry, 1974, "The engineering of Large Dam Part-1 and Part-2", John wiley and Sons.

33. USCOLD, "Updater Guidelines for Selecting Seismic Parameters for Dam Projects". April 1991, United State Committee on large Dams.