SEISMIC RESISTANT DESIGN OF A NUCLEAR CATEGORY I EARTH DAM

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SUMMARY

An integral part of many nuclear power plants is the ultimate heat sink (UHS); the purpose of which is to retain and deliver a supply of service water to the plant when water from the primary circulating water system is not available. Obviously, all structures comprising the UHS must be designed to withstand the Safe Shutdown Earthquake (SSE) postulated for the plant site. The earth dam described herein is designed to retain the reservoir for the UHS of a nuclear power plant in Southern Europe.

In the seismic design of an earth dam, normal static design criteria are not sufficient. For an earthquake resistant dam, a few wide zones consisting of well-graded material are preferable to many narrow, uniformly graded zones as may be found in dams for which seismic loadings were not considered. A high degree of compaction is, of course, necessary to provide shear strength and to preclude liquefaction of granular zones. Additional aspects which do not increase the calculated factor of safety, but do add to the conservatism of the design, are a wide crest, a large amount of freeboard and riprap on the crest of the dam as well as on both faces.

Further, the stability of an earth dam designed to resist earthquakes is assessed with due consideration given to the dynamic nature of forces encountered during earthquakes. The usual pseudo-static analysis is only as good as the estimate for the seismic coefficient used to compute an equivalent horizontal static force on a potential sliding mass. In view of the importance of this Category I structure, as is the case of the earth dam considered herein, a more rigorous method is warranted to analyze the behavior of the earth dam to seismic loading or at least to make a more accurate computation of the seismic coefficients. A two-dimensional dynamic finite element analysis is made to predict the response of the earth dam to a Safe Shutdown Earthquake excitation which is in the form of a time history of accelerations appropriately deconvoluted from the surficial time history and applied at the base of the model.

The material properties such as shear modulus and damping are adjusted to be compatible with the level of strain obtained. Thus, non-linear behavior of soil is considered in the analysis and a more realistic response is predicted. Acceleration and stress are determined throughout the dam and are used to compute a seismic coefficient for a pseudo-static stability analysis and the dynamic strength to stress ratios at several points in the body of the dam.

Finally, the paper presents the need to design the dam to resist a progressive erosion accident resulting from postulated concentrated leaks. This is accomplished by providing a wide, well graded core protected by wide transition cores also heavily compacted.

1.0 INTRODUCTION

This paper describes the design and analysis of an earth dam which retains the Ultimate Heat Sink (UHS) at a nuclear power plant in southern Europe. The purpose of the UHS is to retain and deliver a supply of emergency service water to the plant when water from the primary circulating water system is not available; therefore, the UHS is a Category 1 structure which must be capable of withstanding the Safe Shutdown Earthquake (SSE) postulated for the plant site.[1]

As can be seen from the key plan on Fig 1, the earth dam is located in a tributary valley within the primary circulating water reservoir. It is in the unusual position of having water on both sides under normal conditions. The UHS dam serves to retain water only if the main reservoir drops below a certain elevation as a result of drought or failure of the main dam which has been designed as a Category 2 structure.

2.0 SITE GEOLOGY AND SEISMICITY

The dam site is situated on the Paleozoic shield. The basement rock is late Hercynian and is composed of granitic rocks. The dam itself is founded on deep over-consolidated Miocene soils belonging to the Tertiary sedimentary phase; whereas the thin overlying Pliocene clayey gravels have been excavated.

The procedure used for establishing the seismic design criteria at the plant site was consistent with current criteria for nuclear power plants in the United States.[2,3] The procedure involved a review of the basic geotectonics and seismic history of the region of interest. This lead to selection of the largest site Intensity, which was classified as the site SSE. Through accepted correlations between Intensity and peak ground acceleration, the peak ground acceleration corresponding to the site SSE was established as 0.1 g, since the site is located within a practically aseismic region.[2] Smoothed design response spectra for the SSE were then developed which comply with the peak ground acceleration and the local soil conditions. In addition, an artificial SSE acceleration time history which produces the SSE response spectra was computed.[4]

3.0 SEISMIC DESIGN CONSIDERATIONS

As noted by Sherard,[5] conventional non-seismic design of earth dams is guided mainly by precedent and by past experience. However, experience with the performance of dams subjected to earthquakes is limited and, therefore, a conservative design philosophy is justified.

In general, the probability of failure of an earth dam from a strong earthquake increases with decreasing strength of the foundation material, and with the decreasing ability of both the foundation and the dam itself to withstand concentrated leaks.[5] Neither the natural foundation material (stiff, partially cemented intermingled layers of sand, clayey sands and clayey silts) nor the dam materials (compacted, well-graded soils, described below in detail) are significantly affected by cyclic loading. Therefore, the potential for liquefaction was considered nil.

In addition to maintaining adequate safety factors against slope instability, the primary design criterion is the prevention of erosion from postulated concentrated leaks. Resistance to progressive erosion from concentrated leaks that may develop from an earthquake is supplied by proper gradation of all zones of the dam. The make-up and configuration of the dam zones are governed by this criterion, and are described below.

The strength and stability of the dam are evaluated quantitatively, as discussed subsequently, while the ability of the dam and its foundation to prevent excessive cracking and consequent erosion is accounted for in a qualitative manner by providing a wide, well-graded core of relatively high plasticity, protected by wide transition zones, which are also well graded and heavily compacted. The transition zone is especially wide at the top of the dam, where high accelerations and potential cracking are postulated to occur, and on the downstream face where the potential for instability and erosion is the greatest. The dam is, therefore, carefully zoned and is wider and higher than one that is not designed to withstand seismic forces.

4.0 DAM MATERIALS AND CONFIGURATION

As shown on a typical cross section on Fig 1, the dam consists of four zones, has a crest width of 15 meters, and has outside slopes of 3:1 (horizontal to vertical) on both sides. The depth of excavation varies from 3.0 meters under the core to 2.0 meters towards the edges of the base. The earth dam foundation is a Miocene deposit in the form of a very dense, slightly cemented silt and clayey sand, typically having standard penetration resistance greater than 100 blows per foot and relatively low permeability (less than 10^{-5} cm/sec). Freeboard of 2.5 meters over the normal water level of the UHS and main reservoirs is provided. The freeboard is reduced to 1.1 meters for the 10,000 year flood.

4.1 Central Core Zone

The core (Zone 4) is composed of well graded, cohesive natural soil, with 20 to 50 percent passing the No. 200 sieve, with D85 equal to 0.2 to 5 mm, and with a plasticity index of 8 to 15. This material is relatively impervious (about 10^{-6} cm/sec), self-healing, and adequate to resist concentrated leaks. With such a core material, the larger particles offer resistance against being washed out of the core, tending to wedge in a leakage channel where they act with finer grained soils to seal the leak. The 3:1 (horizontal to vertical) slope of the core on the USH side provides sufficient material for acceptable seepage gradients. Also, the 1:1 slope on the main reservoir side allows for an exceptionally wide transition zone which provides further erosion protection.

4.2 Transition Zone

The transition (Zone 3) material with 4 to 20 percent passing the No. 200 sieve, and with D85 equal to 20 to 100 mm, is composed of the natural soil and of well-graded alluvial sand and gravel, obtained by processing from the same sources as the concrete aggregate for the plant. This type of single, well-graded sand-gravel mixture should be considered as superior to several adjacent bands of different uniform gradations, as the filter protection of the latter is more susceptible to embankment deformations. This is especially true for the top of the dam which is subjected to the greatest accelerations. Both the top and the main reservoir side of the dam contain exceptionally wide transition zones for the purpose of controlling concentrated leaks. Also, due to the well-graded characteristics of the compacted transition, liquefaction potential was considered nil.

4.3 Riprap Zones

The outer zones of the dam (Zones 1 and 2) comprise rock fill for protection against waves and surface erosion, and have slopes of 3:1. On the main reservoir side of the dam, where relatively large waves may be expected, the riprap (Zone 1 material) has a maximum grain size of 500 mm. The UHS side of the dam is protected against smaller waves by riprap (Zone 2 material) having a maximum grain size of 250 mm. The crest of the dam is also protected with a 1.0 meter thick layer of Zone 1 material. The riprap and wide transition zones also reduce the chances of a progressive failure occurring from potential shallow sloughing.

5.0 SOIL PROPERTIES

The static strength parameters, permeability values and consolidation systems for the foundation, core and transition materials were determined in the laboratory with conventional testing procedures. In addition, in situ permeability testing was also performed on the foundation, core and transition materials. The dynamic properties of the foundation and core materials were investigated using in situ cross-hole measurements and a laboratory resonant column testing program. The cross-hole seismic survey[6] conducted at the site showed that for the upper layers of the dam foundation, a shear wave velocity of 650 m/sec typical for small shear strains on the order of 10^{-5} percent prevails. For the saturated foundation soils, this corresponds to a shear modulus of 1.9 to 10^4 kg/cm^2. Poisson's ratio, also determined by the cross-hole survey, equals 0.45. This yields a dynamic Young's modulus of 7.3×10^4 kg/cm^2.

The core material was subjected to laboratory resonant column testing[7] to determine the shear modulus at low strain levels as well as the reduction of the shear modulus with increasing strains. The relation between material damping and strain level was also determined in the resonant column tests. The results of this testing provide the basic data for the soil behavior included in the finite element analysis for the assessment of stability of the dam to resist an SSE.

6.0 SEISMIC ANALYSIS

The stability of the earth dam designed to resist an SSE was assessed with due consideration given to the dynamic nature of forces encountered during earthquakes. The general past practice of evaluating stability of embankments during earthquakes has been to perform a psuedo-static slope stability analysis to compute a minimum Factor of Safety against sliding along an assumed critical failure surface. In this method, the horizontal dynamic forces, which are produced during an earthquake, are accounted for in an approximate manner by introducing into a traditional stability analysis a horizontal equivalent static force that is a product of the potential sliding mass and a horizontal seismic acceleration coefficient.[8] This type of analysis, however, is only as good as the estimate for the seismic coefficient used to compute an equivalent horizontal static force on a potential sliding mass. In view of the importance of this Category 1 dam, a more rigorous method was warranted to analyze the behavior of the earth dam to seismic loading.

The analysis reported herein uses a two-dimensional finite element treatment for evaluation of both the static stresses and the seismic response of the embankment. Effective static stresses were computed on the assumption that all zones of soil within

the dam behave in a linearly elastic fashion. Appropriate values of elastic parameters such as shear modulus and damping were assigned to various zones in the dam and foundation to reflect the mean vertical effective pressure and the overconsolidation ratio.

The QUAD-4 computer program,[9] developed at the University of California at Berkeley, was used to analyze the seismic response of a section of the earth dam. In the analysis, it was possible to provide a strain compatible damping coefficient and shear modulus for each element. In an approximate way, non-linear behavior of soils was accounted for and realistic response was predicted. The seismic excitation, input at the base of the finite element model, was a time history of accelerations expected at the base elevation during an earthquake. Upon completion of the first computation cycle, the shear modulus and damping coefficient for each element were then modified in compliance with the maximum strain that occurred in the element, and another cycle of computation was performed for the same seismic event. This procedure was repeated for either a specified number of iterations on soil properties or until differences between initial and final soil properties for a cycle were negligible. In the final iteration, the stress histories were plotted for specified elements.

Figure 1 shows the typical cross-section of the dam. As noted previously, and as shown on Fig 1, the dam comprises four zones of three different materials: riprap, a transition material, and the core. In order to account for the structure-foundation interaction, it was necessary to include in the dynamic analysis a layer representing the foundation material. The boundaries of the finite element model, and hence its size, are chosen such that the response of the structure is not affected significantly by the reflections of shear waves from the boundaries. This necessitates placing the rigid base and the side boundaries far enough away from the dam to be outside its zone of influence, and, therefore, yield free field motion. The location of the boundaries was determined by analyzing three different models, each having a different total depth. The lateral extent of the foundation layer in all three cases was set at 100 m beyond either toe of the dam and, relative to the height of the dam, is in agreement with recommended values[11] and with accepted practice.[10] Each of the three models was analyzed for the appropriate time history of accelerations applied at the base of the model.

The earth dam foundation system is analyzed only for horizontal seismic excitation. The vertically propagating shear waves in the free field will give rise to horizontal motion only. In order to simulate this condition, all nodes on the vertical side boundaries were restricted to move only in the horizontal direction. Further, all nodes on the base of the model were subjected to the same prescribed time history of acceleration. This implied that the dimensions of the dam are such that the effect of traveling waves is small.

To obtain sufficient accuracy in the computation of the response, the finite element size used in the analysis was kept small with respect to the wave length of the propagating shear waves. It is generally recommended[12] that the vertical dimension of the elements be about 1/9 to 1/12 of the shortest wave length of the shear waves. The vertical dimension of the finite element in the dam was chosen accordingly to be about 1.67 m. The horizontal dimension of the finite elements was chosen such that the aspect ratio did not exceed five.[11]

The earth dam was analyzed for horizontal seismic excitation, comprising a time history of acceleration that is input at the base of the model. As the base of the analytical model is below the ground surface, the surficial time history at the site was modified before applying it to the base. This modification was performed using a conventional deconvolution technique with the SHAKE[5] Program. Briefly, the computer program SHAKE analyzes one-dimensional wave propagation in layered media. Each layer is assumed to be homogeneous and isotropic and to be infinite in horizontal extent. A linear viscoelastic system is assumed; however, non-linear behavior is considered in an approximate, piecewise-linear manner. The acceleration time history thus deconvoluted to the base of the model is shown on Fig 3 along with the surficial time history.

Maximum Accelerations

The maximum acceleration at each node point in the dam occurs at approximately the same time, i.e., between 5 and 5.5 seconds after the start of the input time history. This is to be expected, as the strongest motion in the earthquake record occurs at about 5 seconds (Fig 3). Also, the maximum of the peak acceleration occurs at the crest of the dam, having a magnitude of about 0.32 g. The variation of horizontal acceleration is presented on Fig 4. Very small vertical displacements are produced by the horizontal input motion; the computed vertical accelerations are about two orders of magnitude less than the horizontal accelerations.

With respect to the peak horizontal acceleration value of 0.32 g, at the crest of the dam, it is appropriate to consider the degree of amplification occurring at various parts of the dam. Along the centerline, the amplification factor developed between the base of the model and the base of the dam is about 2.5 (0.25 g/0.10 g), while the amplification between the base of the dam and the crest is 1.24 (0.31 g/0.25 g). These values are considered highly conservative and result from the low material damping values used in the analysis. Specifically, material damping averages only 3.3 percent of critical over the entire model, including the foundation. If the average is taken over the elements comprising the dam embankment only, the average damping value is only 6.6 percent of critical. Past experience suggests that more appropriate values are in the range of 8 to 15 percent. Hence, the maximum acceleration values computed in this analysis, as well as the subsequent stability analysis are considered to be highly conservative.

Maximum Stresses

The computer program, QUAD-4, prints out values of maximum horizontal, vertical and shear stresses that occur in all elements; peak stresses follow peak accelerations. Along the centerline of the dam, the maximum computed horizontal normal stress is 440 psf; whereas maximum peak horizontal stress is 620 psf, occurring in the shell on the UHS reservoir side.

The maximum shear stress occurs at the base of the model on the centerline of the dam, and varies from 985 psf at this point to 60 psf at the center of the crest. The variations of maximum shear stress along the centerline and the base of the dam are shown on Fig 6.

Time History of Stresses

In the final iteration, QUAD-4 computes and stores the values of stresses for every time point; these are plotted as stress time histories. A typical stress time history for shear stresses is shown on Fig 6. By converting the time history of shear stresses to an equivalent series of uniform cyclic stress applications,[17] the potential for liquefaction at points in the foundation and in the dam may be assessed. The core material of the dam, as well as the underlying foundation material, are clayey and highly cemented, however, and the potential for liquefaction does not exist.

The critical combination of static and dynamic stresses within the dam may be determined by comparing the combination to the strength of the soil. Therefore, for each element in the dam for which stress time histories were recorded, seismically induced stresses were combined with geostatic stresses at specific time intervals, and the combination was compared to the strength which may be developed under that stress combination.

Evaluation of Seismic Coefficient

Having determined the dynamic response of the dam subject to a seismic excitation, the resulting accelerations are used to compute a seismic coefficient. This is then used to assess the stability of the dam through pseudo-static methods which are described subsequently.

The time histories of acceleration are converted into an equivalent series of uniform cyclic accelerations[17] and the resulting amplitude of the uniform cycles are computed. This procedure is repeated for all elements comprising one-half of the dam, i.e., above the base elevation from the centerline to the toe on the ESW reservoir side. Having computed the equivalent acceleration amplitudes for the elements, the sum of the products of the masses of individual elements and corresponding accelerations is divided by the mass of half the dam. A weighted average of accelerations is computed[8] and this is termed the seismic coefficient.

7.0 STABILITY ANALYSIS

The stability of the maximum section of the dam was analyzed using the appropriate shear strength parameters and unit weights for the compacted soils. The stability of the dam was assessed for six static and seismic loading combinations which consider the normal operating conditions and rapid drawdown from the normal operating level and the 10,000 year flood level. The seismic loading conditions postulate an SSE of 0.1 g at the ground surface and an OBE of one-half SSE. The pseudo-static stability of the dam was assessed using the Bishop's modified method[18] and the Morgenstern-Price method.[19] All the Factors of Safety thus derived are greater than 2.0 for the static loading condition; with the minimum Factor of Safety of 1.3 when the seismic coefficient was included.

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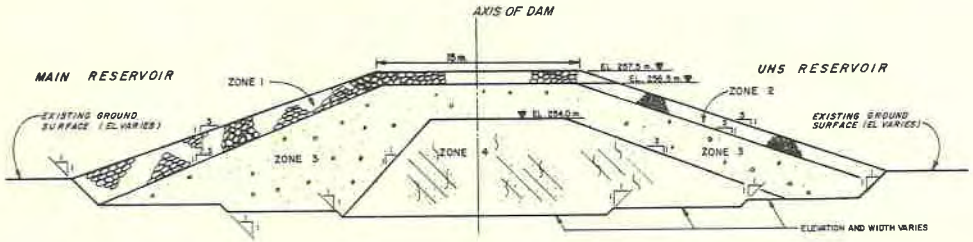
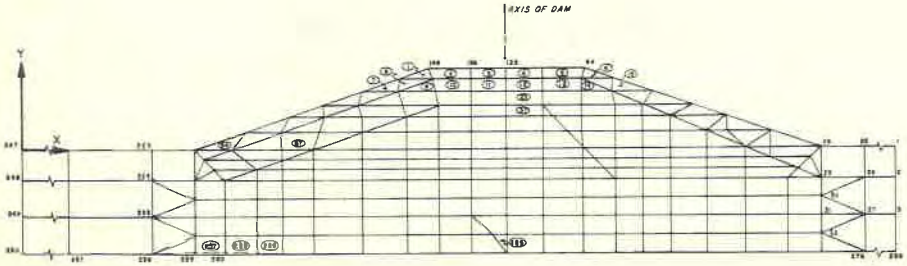
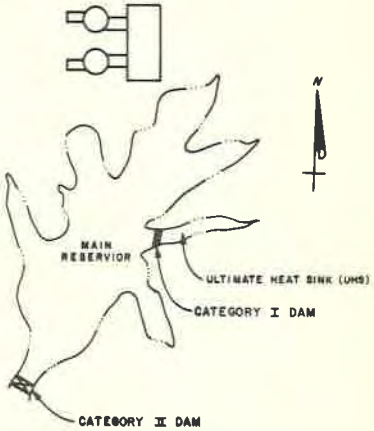


FIGURE 1. TYPICAL CROSS SECTION OF DAM

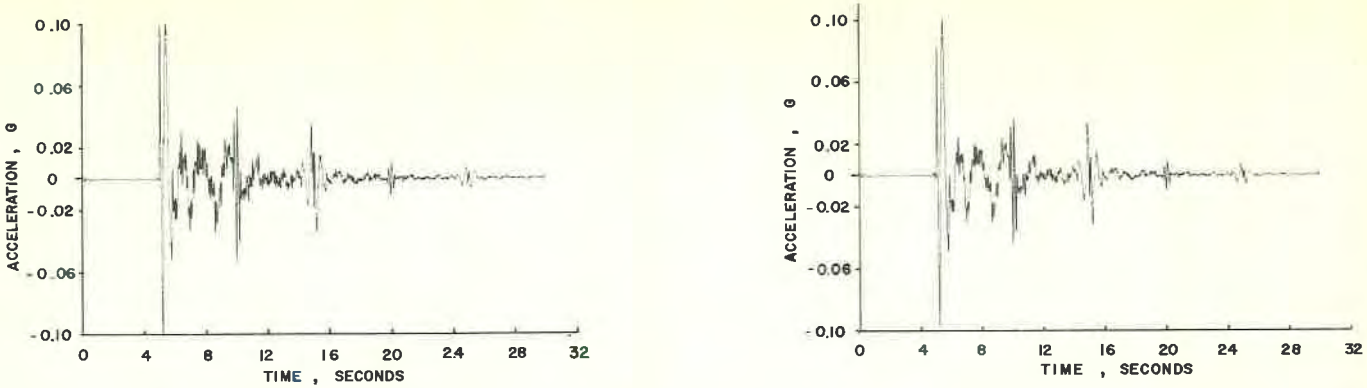


250 NODE NO (TYP)
259 ELEMENT NO (TYP)

FIGURE 2. FINITE ELEMENT MODEL



KEY PLAN



TIME HISTORY AT SURFACE

TIME HISTORY AT BASE

FIGURE 3. TIME HISTORIES OF ACCELERATION - SAFE SHUTDOWN EARTHQUAKE

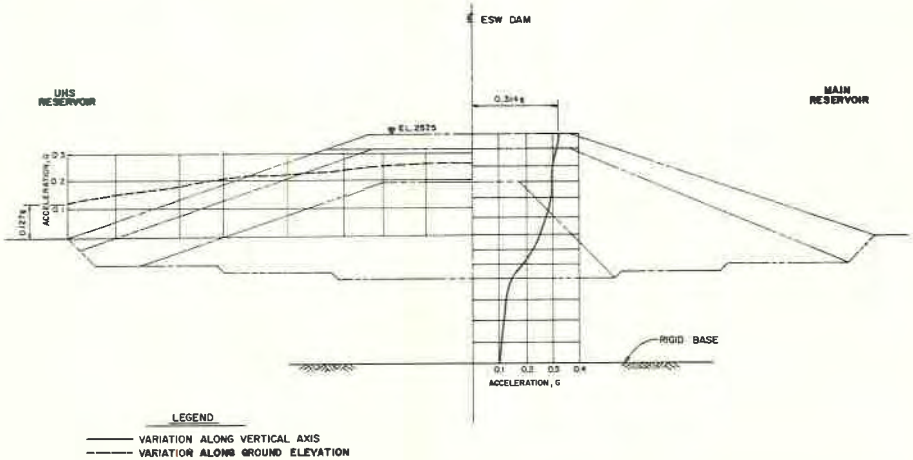


FIGURE 4. VARIATION OF MAXIMUM HORIZONTAL ACCELERATION

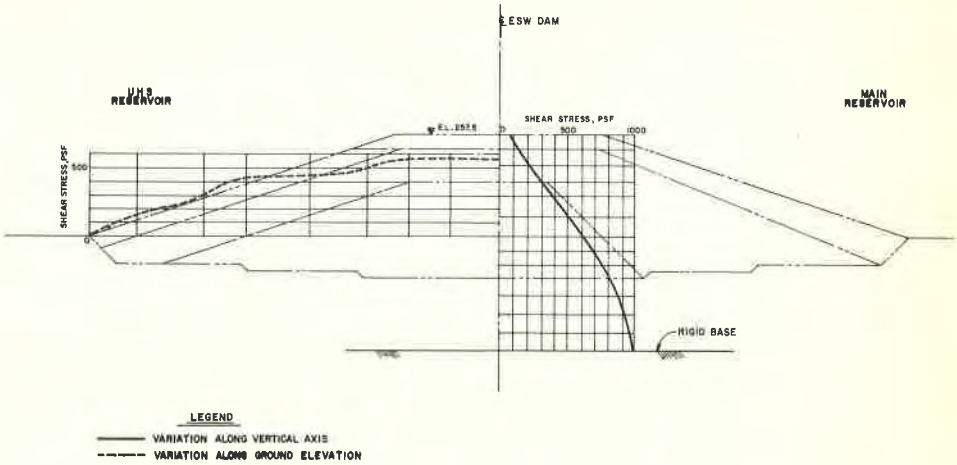
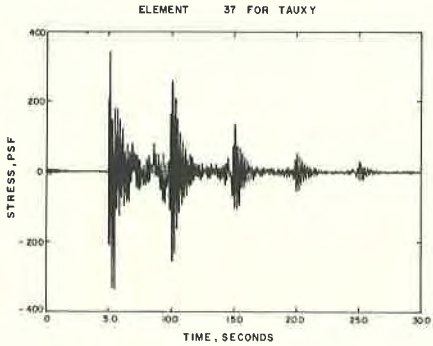


FIGURE 5. VARIATION OF MAXIMUM SHEAR STRESS



NOTE:
ELEMENT 37 IS LOCATED AT TOP OF CORE

FIGURE 6

TYPICAL TIME HISTORY OF SHEAR STRESS