Mathematically and knowledge based methods for analyzing geo-seismic related problems

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FORWARD: The papers included herein revise recent analysis approaches used in geo-seismic related problems throughout four independent contributions. The first work traces the evolution of non-linear dynamic analysis in geotechnical earthquake engineering from its beginning in 1952 to the present. Reliability of dynamic analysis is essential for performance-based design (PBD). Therefore, the paper describes major studies undertaken to validate dynamic analysis. The second work examines the influence that cracks can have over the seismic response of a hypothetical soil profile using a bidimensional finite difference model. The soil is considered to be representative of the soil materials found in most areas of Aguascalientes, and having a bilinear behavior conforming a Mohr-Coulomb model. The third work presented herein gives an outline of the advantages of applying Soft Computing, SC, techniques (neural networks, genetic algorithms and regression trees) and in particular the synergy derived from the use of hybrid SC systems (fuzzy systems tuned by neural networks and neurogenetic models), to solve geotechnical problems. Finally, the last work deals with the development of a system for the analysis of the seismic response of rockfill dams, showing that in some cases, neurogenetic techniques are a better option than analytically-based procedures. To buttress this assertion, the study of the seismic response of El Infiernillo dam, which has an ample history of being shaken by a great variety of seismic events, is conducted.
Non-linear dynamic analysis in geotechnical earthquake engineering Practice

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ABSTRACT: The paper traces the evolution of non-linear dynamic analysis in geotechnical earthquake engineering from its beginning in 1952 to the present. Dynamic analysis is crucial for the application of performance-based design (PBD) concepts in geotechnical earthquake engineering. These concepts are assuming an ever greater role in practice and two examples of their application are presented. Reliability of dynamic analysis is essential for PBD. Therefore the paper describes major studies to validate dynamic analysis. The factors controlling the reliability of non-linear effective stress analysis are examined and guidelines are suggested for conducting defensible and reliable analyses in practice.

1 INTRODUCTION
The evolution of dynamic response analysis and its role in geotechnical earthquake engineering in North American practice is traced from the first paper on dynamic analysis of dams by Hatanaka (1952), to the most complex methods of analysis in use today that are based on sophisticated plasticity constitutive models. The review is limited to North American practice. A very good picture of the role of dynamic analysis in Japanese practice has been provided by Iai (1998). A comprehensive review of European practice is not available. Potts (2003) in his Rankine Lecture has given a very interesting account of the use of finite element methods for static analysis in European practice. He gives some interesting and important insights into potential pitfalls with the use of such analysis and stresses the importance of understanding thoroughly the fundamental basis of any constitutive model to be used.

2 EVOLUTION OF DYNAMIC ANALYSIS
The procedure used 50 years ago in North American practice to evaluate the seismic safety of dams provides a cogent example of the rudimentary state of geotechnical earthquake engineering at that time. Prior to 1960 the stability of embankment dams under earthquake loading was assessed in practice using the seismic coefficient method. This involved conducting conventional slope stability analysis of a potential sliding mass while incorporating a force, acting through the center of gravity of the sliding mass, to represent the effects of earthquake shaking. This force, F, was usually applied horizontally and expressed as a fraction of the weight of the sliding mass, W, by the equation \( F = kW \). This implies that k is an effective horizontal acceleration expressed in units of gravity. Values of k ranged from 0.05 to 0.15 but no rational basis was provided for the selection of an appropriate k value.

Major analytical contributions leading to a more fundamental understanding of the seismic response of embankment dams were made by Ambraseys (1960a, b, c). He assumed that soil was a visco-elastic material and treated dams as 1-D and 2-D shear beams in his analyses. He demonstrated how the incoming motions were amplified throughout the dam, the contribution of the different modes of vibration of the dam to the global response and how the seismic coefficient varied with depth below the crest of the dam. Ambraseys (1960c) studied dams in both wide and narrow rectangular valleys and showed that when the ratio of the width of the valley to the height of the dam was less than 3, the seismic response changed significantly. This confirmed earlier results by Hatanaka (1952, 1955). Despite the limitations of the visco-elastic model of soil behavior, this analysis captured many of the important characteristics of seismic response and provided the starting point for some of the subsequent developments. Seed and Martin (1966) carried out similar analyses for a variety of dam sizes and material properties and provided a comprehensive database for selecting appropriate values of the seismic coefficients. They also drew attention to the deficiencies in the seismic coefficient method should the materials in the dam lose strength during an earthquake.

Probably one of the more significant events which contributed to the rapid development of analysis in geotechnical earthquake engineering in the 1960's was the application of finite element methods to the analysis of embankment dams for the first time by Clough and Chopra (1966). This was followed by the seismic analysis of slopes by Finn (1966a, b) and the analysis of central and sloping core dams by Finn and Khanna (1966). The latter study demonstrated the effects of the stress transfer between core and shell and showed the role of analysis in revealing the action of internal mechanisms in a complex soil structure. All these analyses were conducted using a visco-elastic, total stress constitutive model of the soil and therefore were not capable of modeling the pore water pressure development or permanent deformations. To approximate a solution to this problem, Finn (1967) outlined a procedure for interpreting the effects of the dynamic stresses computed by the visco-elastic analysis with the help of data on pore water pressures and strains from laboratory cyclic loading tests.

A major improvement in analysis occurred in 1972 when Seed and his colleagues at the University of...
California in Berkeley developed the equivalent linear method of analysis for approximating non-linear behavior. This method was incorporated in the program SHAKE (Schnabel et al., 1972). The technique was extended to two dimensional finite element analysis by Idriss et al. (1974) and Lysmer et al. (1975) in the programs QUAD-4 and FLUSH respectively. These programs led to more realistic, total stress analyses of embankment dams under earthquake loading, especially under strong shaking and the equivalent linear constitutive model has remained the backbone of engineering practice to the present day.

While this program development was going on, the capability of testing soils under cyclic loading was also being developed. The cyclic triaxial test was developed by Seed and Lee (1966) and made possible the study of the mechanics of liquefaction and seismically induced deformations. At around the same time, the resonant column test was developed for measuring dynamic shear modulus and damping at low strains (Drnevich, 1967). In the early 1970s the use of the cyclic simple shear test was pioneered by Seed and Peacock (1970) and Finn et al. (1971). Therefore by 1975 geotechnical engineers had many of the analytical and laboratory capabilities necessary for realistic analysis of the seismic response of earth structures. These developments revolutionized the assessment of the safety of embankment dams during earthquakes and the seismic response of sites to seismic excitation.

These new methods were put to the test, when Seed et al. (1973, 1975a, 1975b), undertook a comprehensive study of the liquefaction induced flow failure of the lower San Fernando Dam which occurred as a result of the San Fernando, California, earthquake of 1971. The analyses predicted that liquefaction would occur and that the dam would fail by undergoing very large deformations upstream during the earthquake, as was indeed the case. An extensive region under the upstream slope liquefied during the earthquake and the dam did develop very large deformations and failed but just after, not during, the earthquake. This delayed failure was attributed by Seed (1979) to pore water pressure redistribution.

The equivalent linear method of analysis used in the study of the San Fernando Dam is a total stress analysis and therefore did not track the development and redistribution of seismic pore water pressures during shaking and could not take into account during the dynamic analysis the effects of pore water pressures on soil properties and dynamic response. These effects were taken into account after the analysis by relating the effects of stresses calculated during shaking to pore water pressures and shear strains in laboratory test specimens under equivalent cyclic loading. The San Fernando case history provided the stimulus for the development of effective stress methods of dynamic analysis which could track the development of pore water pressures during shaking and take the effects of pore water pressures on soil properties into account directly during analysis. The Martin-Finn-Seed (MFS) model for generating pore water pressures during earthquake loading based on the strain response of the soil (Martin et al. 1975) paved the way for dynamic effective stress analysis.

The first non-linear dynamic effective stress analysis based on the MFS pore water pressure model was developed by Finn et al. (1977) and was incorporated in the 1-D program DESRA-2 (Lee and Finn, 1978). The constitutive model is based on hysteretic response. The stress-train relation is hyperbolic with Masing response on unloading and, in the latest version of the program Finn et al. (1996), contains a failure strength defined by the Mohr-Coulomb failure criterion. A rudimentary 2-D version of this program was developed by Siddharthan and Finn (1982). An updated comprehensive program TARA-3 was developed by Finn et al. (1986). TARA-3 was verified over a three year period by centrifuge tests conducted at Cambridge University in the UK on behalf of the United States Nuclear Regulatory Commission (Finn 1988). TARA-3 has the capability to conduct both static and dynamic analysis under total stress or effective stress conditions and can compute permanent deformations directly. It has the option of allowing pore water pressure dissipation and void ratio re-distribution during and after shaking. The program uses properties that are normally measured in connection with important engineering projects.

Since the mid 1980's, many non-linear effective stress programs have been developed, for the most part based on some version of plasticity theory. Detailed presentations of the constitutive models in these programs may be found in Pande and Zienkiewicz (1982), Finn (1988) and Arulanandan and Scott (1994). These programs are mathematically and analytically quite powerful but some use properties that are related to the mechanics of implementing the model rather than characterizing the soil. For example, some make different assumptions about how strains and the directions of the strains are to be determined and therefore require different parameters, which are not soil properties, to define the model. These parameters usually have to be found by adjusting them to match global response in an element test. Therefore they are related to specific directions of loading with respect to the material axes of the specimen and their performance under other loading directions is not verified. Many calibrations are done under static loading and there are well documented instances that this is no guarantee that their performance will be satisfactory under cyclic loading.

The estimation of large post-liquefaction displacements is a very important part of assessing the consequences of liquefaction in embankment dams. Evaluating large displacements means that small strain theory is inadequate. Finn and Yogendrakumar (1989) developed the program TARA-3FL to track the post-liquefaction deformations using a Lagrangian formulation. A similar technique is employed in the now widely used computing platform FLAC (Itasca 1996).
One of the most significant developments in fostering the increased use of dynamic analysis in geotechnical engineering, apart from the availability of reliable constitutive models, is the availability of the well-maintained and documented platform, FLAC, for incorporating constitutive models. More recently the program PLAXIS (2001) has become available. The analytical frameworks in FLAC and PLAXIS are different; FLAC is based on an explicit formulation and requires very small time steps, while PLAXIS is based on an implicit formulation. As a result FLAC is much slower than PLAXIS. To some extent the different formulations may have been influenced by the state of personal computing capacity at the time of development. In the 80’s RAM was small. FLAC does not require storing large matrices but advances incrementally and was very well suited for analysis on PC’s. PLAXIS in its dynamic mode, came on the scene when much larger memories and higher computing speeds were available and so an implicit formulation was feasible.

3 THE ROLE OF NON-LINEAR ANALYSIS

In the first general review of nonlinear methods for total and effective stress analysis (Finn 1988), several examples were given to show the capability of nonlinear analysis for simulating element tests and centrifuge tests. Just one example was given from practice, involving the displacement behavior of the remediated Lukwi tailings dam in New Guinea (Finn 1988). At that time the use of non-linear dynamic analysis in practice was very limited. A later review by Finn (1998) included examples from practice of the use of non-linear analysis in making design decisions regarding the remediation of major embankment dams based on tolerable displacement criteria. In the last 20 years, many papers have appeared describing the application of nonlinear dynamic analysis to embankment dams, port structures and pile foundations. With the advent of performance based design using tolerable displacement criteria to identify acceptable serviceability and safety levels, the ability to estimate displacements with sufficient reliability has become crucial to satisfactory design. Therefore the role of non-linear analysis has become central to the design process. Examples presented in this review are used to illustrate this new role for analysis. They show the different ways in which analysis empowers the designer to tackle difficult problems directly and to explore various ideas and options. For technical details on the analyses themselves, the relevant references should be consulted.

3.1 Performance Based Design

Performance based design (PBD) is based on tolerable displacement criteria and has become part of practice in geotechnical earthquake engineering. It has been widely used for developing remediation strategies for embankment dams with foundations susceptible to liquefaction under design seismic loadings. There are two crucial requirements for implementing PBD: acceptable performance criteria and a reliable method of analysis. For embankment dams the criterion of acceptable performance is specified by tolerable displacements, usually of the crest. A nonlinear analysis is essential because soil behaves as a nonlinear solid under strong shaking. If significant seismic pore water pressures are developed during shaking, the analysis must be based on effective stresses. Nonlinear dynamic effective stress analysis has many forms and its safe use requires a good technical understanding of the mechanics of the constitutive model selected for use and knowledge of its validation history based on element test data, centrifuge test data and any evidence from case histories. It also requires an adequate understanding of how the computation procedure works. These requirements will be discussed in detail later. The application of PBD to an embankment dam is illustrated by the process of selecting and designing remediation measures for Sardis Dam.

3.2 Sardis Dam

Sardis Dam is a hydraulic fill structure located in northwestern Mississippi within the zone of influence of the New Madrid seismic zone. A cross-section of the dam is shown in Figure 1.

Figure 1. Cross-section of Sardis dam.

The U.S. Army Corps of Engineers, Vicksburg District, undertook several studies to evaluate the probable behavior of the dam during and after an earthquake. The earthquake hazard was defined by the seismic design parameters:

- Peak ground acceleration - 0.20 g
- Maximum velocity - 35 cm/s - 45 cm/s
- Duration - 15 s
- Two records of the 1952 Kern County Earthquake in California were modified to provide suitable input motions for seismic response analyses. The magnitude and epicentral distances were somewhat similar to those of the selected design earthquake for Sardis.

In situ investigations revealed zones with the potential for liquefaction or significant strength loss which could threaten the upstream stability of the dam; the hydraulically-placed silt core, and a discontinuous layer of...
weak silty clay or clayey silt ranging in thickness from 1.5 m to 4.5 m located in the top stratum clay beneath the upstream slope over a 310 m long portion of the dam. The upper 3 m of sand shell along the lower portion of the upstream slope was also identified as having a potential for liquefaction. However, loss of strength in this zone has a relatively minor effect on the stability of the dam. Stability analyses showed that, although the silt core might liquefy along the entire length of dam, the factor of safety with respect to upstream stability of the dam would still be adequate except in areas where the weak layer of silty clay occurred beneath the upstream slope within 75 m of the centerline. The results of these investigations indicated that remedial measures were necessary to improve the stability of the upstream slope of Sardis Dam during seismic loading.

The studies began in 1972 and the remediation was completed in 1995. During these years, the process for evaluating the seismic safety of embankment dams evolved, driven by advances in earthquake engineering, site investigation techniques, laboratory testing and procedures for seismic response analysis. The last study phase which began in 1989 and incorporated much new technology, finally led to an innovative cost-effective, reliable method of remediation based on performance based design concepts. In this section the aim is to demonstrate how the use of comprehensive analysis helped engineers make informed decisions on complex design decisions. An interesting feature of the Sardis case history is the efforts made to assess the potential reliability of a new method of analysis and to develop some confidence in its use in a situation where a failure would have serious consequences for public safety. Reliability of analysis and guidelines for conducting non-linear analyses similar to those used for Sardis dam are discussed in more detail later in the paper.

The development of a method of post-liquefaction flow deformation analysis, TARA-3FL, by Finn and Yogendrakumar (1989), made it possible to adopt performance criteria for Sardis Dam based on allowable deformations in addition to the traditional standard of an adequate factor of safety. The early analyses focused on demonstrating the capability of the program to handle very large deformations. These early studies were conducted on the unremediated section of Sardis Dam (Finn, 1990). This early version of the program was used to determine the necessary size and average shearing resistance of a remediated zone and its optimum location to limit deformations to acceptable values. Later it was decided that the most reliable way of supplying the necessary resistance was by driving piles which would nail the upstream slope to the stable foundation soils. This necessitated modifying the program to include piles. Some of the early studies on pile behavior were reported in Finn (1993). The full process of reaching the final design for the remediation of Sardis Dam is described in Finn et al., (1998). Because the TARA-3FL analysis was a new technique, it was essential to assess its results using other studies which could provide estimates of the potential range in design parameters. These involved various kinds of 3-dimensional coupled analyses of limited sections of the remediated zone and also 2-dimensional uncoupled analyses in which the behavior of the remediated section was investigated in isolation from the rest of the dam. Stability checks were also performed on deformed sections of the dam for various values of the residual strength.

The potential post-liquefaction deformations before and after remediation were estimated using the computer program, TARA-3FL [Finn and Yogendrakumar, 1989]. The large differences between the initial and post-liquefaction strengths in Sardis Dam resulted in major load shedding from liquefied elements. This put heavy demands on the ability of the program to track accurately what was happening and on the stability of the algorithms. Therefore it was imperative to have an independent check that the computed final deformed positions were indeed equilibrium positions.

To check the performance of TARA-3FL, the post-earthquake deformed shape of Sardis Dam was computed using the specified residual strengths in the weak layer with a minimum value of 17.5 kPa. The initial and final deformed shapes of the dam for this case are shown, Figure 2. Very substantial vertical and horizontal deformations may be noted, together with intense shear straining in the weak thin layer. The static stability of the deformed shape was analyzed using the program UTEXAS2 (USACE, 1989) which satisfies both moment and force equilibrium. The factor of safety was found to be close to 1.0. The critical slip surface exited the slope near the location suggested by the finite element analysis.

![Figure 2. Post-liquefaction deformed shape of Sardis dam.: note different vertical and horizontal scales.](image)

To check the reliability of TARA-3FL more widely, a series of analyses were conducted of Sardis Dam assuming different levels of residual strength in the weak foundation layer in each analysis. The conventional factor of safety of the undeformed dam section varies over the range 1.15 to 0.68 as the constant residual strength varies from 30 kPa to 10 kPa (Figure 3). The deformed sections corresponding to the various residual strengths in this range were calculated using TARA-3FL. The factors of safety of these deformed sections were determined using UTEXAS2. In the clearly unstable region defined by a factor of safety less than one for the undeformed section, the computed factors of safety (black dots) for the...
deformed sections were in the range of $1 \pm 0.05$. This is the theoretical error band associated with UTEXAS2.

![Figure 3: Factors of safety of Sardis dam as a function of residual strength in weak foundation layer](image)

Many results of this type, for different assumptions about the residual strengths, suggest that the TARA-3FL analysis does indeed achieve equilibrium positions even for large drops in strength due to liquefaction and associated large deformations (Finn, 1990).

Analyses of this type give the loss in freeboard associated with various factors of safety based on the original configuration of the dam. For Sardis Dam, the variation of vertical crest displacement with such factors of safety, corresponding to various values of residual strength, are shown in Figure 4. For the first time, a designer could see the consequences of selecting a particular factor of safety for a dam. This information was helpful to engineers making the transition from a factor of safety based design to displacement based design.

![Figure 4: Variation of loss of freeboard with factor of safety of undeformed dam](image)

### 3.3 Remediation Studies

The selection of remedial measures for Sardis Dam focused on ways of strengthening the weak liquefiable foundation layer. The general idea was to develop a plug of much stronger material across the weak layer which would restrain post-liquefaction deformations as shown in Figure 5. The important properties of this remediation plug were its length, strength and location. Two kinds of analyses were conducted to define the properties of the remediation plug; conventional stability analyses and deformation analyses using the program TARA-3FL. Various methods of creating the plug were investigated. Because of limitations on lowering the reservoir level, any remediation would have to be placed under water. In these circumstances, nailing the upstream slope to the stable foundation layer by piles was found to be the most cost-effective and reliable method of remediation. The layout of the piles is shown in elevation and in plan in Figs 6 and 7 respectively.

### 3.4 Load Transfer Mechanism in the Pile Remediated Section

The displacement analysis of the remediated section giving the moments, shears and displacements of the piles are described in Finn et al. (1998). The distribution of pile shear between the pile rows is shown in Figure 8 and the distribution of moments in Figure 9.

![Figure 5: Location of remediation plug to stabilize upstream slope of Sardis dam](image)

![Figure 6: Elevation of pile remediation of Sardis dam (after Stacy et al., 1994)](image)

![Figure 7: Plan view of pile remediation of Sardis dam (after Stacy et al., 1994)](image)
The distributions of post-liquefaction shears and moments controlled the placement of the piles. Since the piles were not capped, the only load transfer mechanism between the pile rows was the pressure exerted by the soil between rows in response to the deflection of the pile rows. The designers were concerned by the possibility of progressive failure, if the pile rows were too far apart. Therefore the spacing of the piles in the first 3 rows was kept small. The spacing in both vertical and horizontal directions was only one third of the spacing between the other of the piles. Maximum dynamic moment occurred in the leading row of upstream piles and is shown in Figure 10. The design moment was the sum of 67% of the peak dynamic moment and the moments due to the static thrust of the embankment along the weak plane.

The use of displacement criteria for evaluating the post-liquefaction stability of Sardis dam and determining the design and placement of the remedial measures resulted in huge savings over estimated costs for other procedures. This type of displacement analysis has become very much a part of practice today. The process of probing the performance and reliability of the then new method of analysis for Sardis dam is a process that should be followed whenever a new constitutive model is used either in a stand-alone program or inserted in a platform such as FLAC.

3.5 Development of Seismic Stability Screening Relations

Performance-based design concepts may also be used as a basis for prioritizing structures for remediation. Parametric studies of the response of a particular type of soil structure such as a flood protection dike using nonlinear dynamic analysis give a data base for the potential development of simple screening methods for preliminary assessment of the behavior of a similar type of structure under strong shaking and so avoid repetitive and perhaps complicated analysis. One example is given: development of a criterion for prioritizing remediation intervention for flood protection dikes in Hokkaido, Japan.

3.6 Prioritizing Flood Protection Dikes for Remediation

The remediation of long-line structures such as flood protection dikes against the consequences of liquefaction is a very long and expensive process. Therefore it is necessary to identify the locations that are most at risk and remediate these first. A simple predictive relation based on easily measured characteristics of the dikes was developed for prioritizing the selection of locations for remediation of flood protection dikes along major rivers in Hokkaido. This project demonstrates clearly another potential role for nonlinear analysis.

The flood protection dikes along the Kushiro and Tokachi rivers suffered considerable damage during the 1993 Kushiro-oki earthquake off eastern Hokkaido, Japan. Damage included longitudinal and transverse cracks, slope failures and cave-ins. The more severely damaged dike sections were 6 m - 8 m in height, and were constructed of compacted sand fill resting on a comparatively thick peat layer. The dikes were damaged at 18 locations for a total length of about 10 km along the Kushiro river. The severest damage occurred in Kushiro Marsh (Sasaki et al., 1993; Sasaki, 1994a, b). Dike sections which failed were reconstructed, after the foundation soils had been improved by the installation of sand compaction piles.

In 1994, a major earthquake occurred off the west coast of Hokkaido, the Nansai-oki earthquake which caused failures of flood protection dikes along several river basins in western Hokkaido. After these earthquakes, the Hokkaido Development Bureau initiated a program of...
improving the diking systems. Because of the great length of dikes, they wished to develop a criterion for prioritizing the remediation work. One of the approaches considered was to use potential crest settlements as a performance criterion. The strategy to meet this objective was to conduct parametric studies on many dikes and develop an equation that linked dike characteristics to crest settlements for repeats of the 1993-1994 earthquakes. This equation would then be used to identify the dikes at greatest risk of failure and to prioritize interventions for remediation. The parametric studies were commissioned by the Hokkaido Development Bureau through the Advanced Construction Technology Center (ACTEC) in Tokyo. The analyses were conducted at the University of British Columbia, Vancouver, Canada and the resulting equation was validated by ACTEC using data from many damaged dikes not included in the parametric studies.

First simulations of some dike failures during the 1993 Kushiro-oki earthquake were conducted based on soil data and input motions provided by ACTEC (1995) and JKK (1995) respectively. After these simulations proved satisfactory, a program of parametric studies were authorized to investigate how the crest settlements of dikes correlated with slope angle, dike height, and the thicknesses of non-liquefiable and liquefiable layers. A typical dike cross-section is shown in Figure 11. On the basis of the parametric studies, the following displacement criterion was developed (Finn, 1998), for symmetrical dikes with 1:2.5 side slopes,

\[ S = 0.01 \exp (0.922 \frac{H_D}{H_{NL}} \frac{H_L}{H_{NL}}) \]  

where the normalized settlement \( S \) is the crest settlement divided by \( H_D \), the height of the dike; \( H_L \) and \( H_{NL} \) are the thicknesses of the liquefiable and non-liquefiable layers, respectively. This relationship is shown by the curve in Figure 12 in terms of the non-dimensional variables \( S \) and \( \beta \) where \( \beta = \frac{H_D H_L}{H_{NL}^2} \). The data points are the settlements computed from the parametric studies.

The measured crest settlements from a wide variety of dikes which underwent noticeable displacements during the Nansei-oki earthquake in 1994 were plotted by ACTEC engineers in Figure 13 against those predicted by the black prediction curve. The white square data points were plotted by the client after the correlation in Eq. 2 had been submitted. The 8 points close to the curve in Figure 13 are for symmetrical dikes with side slopes of 1:2.5. The agreement is very good for these dikes. The remaining points significantly to the right the curve describe the response of dikes with uniform side slopes of 1:5, and unequal slopes, 1:5 and 1:10. These slopes require different correlations.

The predictive equation for dike damage, strongly validated by data from case histories, provides a convenient method for prioritizing dikes with side slopes 1:2.5 in the Hokkaido river basins for remediation. This is another example of the way in which an appropriate analysis can help the engineer formulate a cost effective response to a challenging design problem.

Figure 12. Comparison of computed settlements with the black prediction curve

The measured crest settlements from a wide variety of dikes which underwent noticeable displacements during the Nanei-oki earthquake in 1994 were plotted by ACTEC engineers in Figure 13 against those predicted by the black prediction curve.

Figure 13. Comparison of observed settlements for all slopes against computed settlements for 1:2.5 slopes (solid curve)
4 RELIABILITY OF ANALYSIS

4.1 Computing Environment in Geotechnical Earthquake Engineering

Prior to about 1985, the equivalent linear constitutive model incorporated in the total stress programs SHAKE (Schnabel et al., 1972), QUAD-4 (Idriss et al., 1974) and FLUSH (Lysmer et al., 1975) was the industry standard for dynamic response analysis in geotechnical earthquake engineering. The need to calculate directly permanent displacements in earth structures as a result of earthquake shaking and to take into account the effects of seismic pore water pressures during analysis finally spurred a drive to non-linear effective stress analysis. However, despite thirty five years of research and development, an industry standard constitutive model has not emerged. Well over a hundred constitutive models have been developed but many have never been used in practice and the remaining models have been used only sporadically. Probably the only widely used non-linear effective stress programs over the last thirty years are DESRA-2 and the various modifications of it such as D-MOD and TARA-3 which has been used on 22 large embankment and tailings dams. The advent of the FLAC and PLAXIS computing platforms will facilitate progress towards a standard. They make computing capacity more widely available in standard formats and provide the opportunity for the evaluation of various constitutive models in practice.

The response of earth structures to seismic loading is determined by dynamic analysis using computer programs. The reliability of the computer program used for dynamic analysis, especially for critical structures, is a crucial issue for a designer. The term reliability is not used in this context in its formal mathematical sense as defined, for instance, by a reliability index. Rather it describes the confidence of the designer or analyst that the program can help to bracket the likely range of behaviour of the structure under consideration and provide him with the data on which he can exercise his professional judgement to achieve a safe and cost effective design. How does the analyst develop this feeling of confidence, especially when using a program for the first time and in the absence of an industry standard? This requires a knowledge and understanding of the constitutive model, the computational procedures to implement the model, how the model was calibrated, how a program was tested and validated and whether any verification is available from case histories in the field or in centrifuge tests.

Any constitutive model operates in an ideal environment, typically a continuum. Within this continuum with well–defined properties, all models can perform perfectly. One of the main problems in achieving a reliable response from analysis of an actual structure for a specified seismic input is the challenge of calibrating the model so that the continuum in which the model operates adequately represents the conditions in the actual structure. The difficulty in achieving satisfactory calibration is model dependent. Some programs such as DESRA-2 can operate directly using properties determined by in situ tests during site investigation but most models, especially the advanced plasticity models require laboratory tests. These tests are frequently conducted on reconstituted samples. Comprehensive research studies have clarified the conditions to be satisfied by reconstituted test specimens to be considered representative of field conditions. The main conditions are: the specimens should be formed by a process that mimics natural deposition in the field and achieves field density; the stress path in the calibration tests should be comparable to the dominant anticipated stress path in the field.

The process of sample formation has a very significant effect on the stress strain behavior of test specimens. The radically different stress-strain curves for specimens prepared by moist tamping, pluviation in air and pluviation in water are shown in Figure 14 (Vaid et al., 1998). Stress-strain curves on undisturbed test specimens of Holocene sand fashioned from samples retrieved from frozen ground and curves from reconstituted test specimens of the same sand formed by pluviation in water are shown in Figure 15. For both types of specimens the results are almost identical. Evidence of this kind clearly indicates the care that should be taken to ensure that as far as possible test specimens should be formed to mimic conditions in the field.

![Shear Stress vs Shear Strain](image1)

Figure 14. The effect of method of specimen preparation on stress-strain response

![Shear Stress vs Shear Strain](image2)

Figure 15. Comparison of stress-strain curves from undisturbed frozen specimens and from water pluviated specimens.
The effects of the stress paths followed in loading test specimens has very significant effects on stress-strain behavior also as shown in Figure 16. These stress-strain curves were developed by different loading paths in hollow cylinder torsional cyclic shear tests and demonstrate clearly the sensitivity of stress-strain behavior to stress paths in loading.

Figure 16. Effect of stress path on stress-strain relations (Yoshimine et al., 1998)

A computer program for analysis of geotechnical structures has two main components; a constitutive model and a computational model. The constitutive model is verified using data from element tests, usually triaxial or simple shear tests. The stresses in these tests are defined and the verification does not require a full, dynamic stress-strain analysis. If the constitutive model is based on material properties each of which can be independently measured, then agreement with element test data is validation that the model can simulate reliably the element response. If the model contains constants or parameters which cannot be individually measured but must be globally determined on the basis of the test data, then the validation is much weaker and there is uncertainty whether the global determination of the non-material constants from laboratory data, will be valid for the field application.

There have been 3 comprehensive prediction studies in North America to assess the capability and reliability of constitutive models and associated computer programs; Case Western Reserve University (Saada and Bianchini, 1987), VELACS (Arulanandan and Scott, 1994) and Turkey Flat (Shakal et al, 2006). These studies have provided great insight into the factors controlling the capability of constitutive models in practice and the uncertainties associated with computational models that need to be taken into account in practical applications. The principal findings from these studies are discussed below and form the basis for a set of guidelines to ensure a consistent approach to reliable, representative analysis. The issues discussed above are crucial for understanding the results of these blind prediction experiments.

4.2 Validation by Element Tests

A prediction exercise was organized in 1987 by Case Western Reserve University in Cleveland and Institute de Mechanique of the University of Grenoble, France to evaluate the capability of constitutive models to predict the response in element tests. Each predictor was given data from tests on each of three different sands: 3 compression tests, 2 extension tests and an isotropic consolidation test with loading and unloading stress paths. Each predictor was asked to predict the results of other tests on the same sands with more complex loading paths.

Of particular interest for seismic response analysis are the predictions of responses to cyclic torsional shear stresses in a hollow cylinder test (Finn, 1988). Typical results for the cyclic loading tests are given for a multi-yield surface plasticity model in Figure 17 and for a bounding surface plasticity model in Figure 18. The results for these kinds of constitutive models were selected for consideration because they are considered some of the better types of plasticity models and they are still used in practice.

Figure 17. Predicted and measured stress paths: multi-yield surface plasticity model

The predictions of stress paths are fairly crude and the volumetric strains are overestimated by a factor of 2. Both of these models were satisfactorily calibrated against the triaxial test data provided for calibration. Although the predictions were undoubtedly affected by the normal uncertainty associated with test data on soils, the salient features of the predictions suggest a more fundamental problem. The loading paths in the calibration tests and the tests for prediction are very different. There is considerable rotation of principal stresses in the torsional shear tests used for prediction but no rotation in the triaxial tests used for calibration. A reasonable conclusion is that the constitutive model parameters are strongly path dependent, despite the generality of the model formulations.
Presumably the models could have subsequently have been calibrated satisfactorily against the cyclic loading data to provide a better simulation. It is interesting to speculate how the predictions of the models in the torsional shear tests would have fared, if they had been calibrated against simple shear data.

Because of the potential for the constitutive model parameters to be path dependent, constitutive models should be calibrated by data from stress paths that follow as closely as possible the dominant stress path in the field. Triaxial test data may be more appropriate, for example, for an application involving predominantly the rocking of foundations but simple shear data would seem to be preferable for analysis of dams under excitation by shear waves propagating vertically.

4.3 Validation by Centrifuge Tests

Data from tests with non-homogeneous stress states are needed to fully test the capability of the computer analysis program that incorporates the constitutive model. Centrifuge tests provide non-homogeneous stress states under almost ideal conditions and have the most controllable environment for evaluating computer programs. The most comprehensive study of this type to date, the VELACS project, is described below.

4.4 VELACS Project

The VELACS (Verification of Liquefaction Analysis by Centrifuge Studies) project was organized under the auspices of the U. S. National Science Foundation to evaluate the capability of available effective stress based computer programs to make adequately reliable predictions of the seismic response of geotechnical structures subject to liquefaction, (Arulanandan and Scott, 1994). Scott (1994) gave a very informative summary assessment of the conduct of the VELACS project and its findings in his article “Lessons learned from the VELACS project” in the proceedings of the workshop on the project (Arulanandan and Scott, 1994). He described the function of VELACS in assessing the adequacy of computer codes as follows: “VELACS proposed to give at least a partial response to this question by substituting centrifuge for real-life, but requiring the code calculations to represent the real situation by being performed in advance of the test event. This is known in the trade as a Class A prediction.” The focus in this review is on what was learned from VELACS about the capability of the various computer programs to make reliable predictions. All the programs in the VELACS study are still around today and several have found occasional use in practice.

Nine different centrifuge models were tested, including level ground, sloping ground, dams and retaining structures. Predictions of the responses were made prior to the tests being conducted. A comprehensive data base of the properties of the soils to be used was provided. The predictions were based on the anticipated input motions and placement conditions of the soils. To ensure that the specified conditions would be realized in the tests, a uniform set of specifications had to be followed by all experimenters. However, when the tests were run, there were discrepancies between the test conditions and the anticipated conditions used in the predictions. The greatest discrepancies were in the input motions. Although different researchers could control peak acceleration and the duration of shaking, they could not control frequency content. Although there is no evidence how frequency content within the range experienced might affect the responses, it remains a source of uncertainty in the findings.

The changes in the input conditions for the tests were communicated to the predictors on completion of the test program but not the test results and they were invited to make predictions (Class B) on the updated information. Very large changes, were made in material properties in the few Class B predictions that were made compared to the corresponding properties used in the Class A predictions. These major changes were not mandated by the relatively small changes in input conditions. The predictors chose to change all the constants in the prediction model instead of just changing those constants linked to the changes from the pre-test conditions reported by the experimenter.

4.5 Findings from VELACS

The primary lesson from VELACS, as from the Saada and Bianchini (1988) prediction exercise, is that ability of a model to simulate element tests is no guarantee of how it will perform in other stress fields. Models need to be calibrated for the dominant stress paths expected in application as far as is possible with the conventional tests used in engineering practice. Smith (1994) warned about this in his discussion of the VELACS project: “A particularly insidious feature of the calibration process is that a predictor could calibrate his/her model to fit the
bulk of the (largely triaxial) data provided in the information package and still make poor predictions of seismically induced stress paths”.

Constitutive models are either inherently path dependent or can be made path dependent by how they are calibrated. If a model which includes principal stress rotation is calibrated in a triaxial test in which no rotation of principal stresses occurs, it may perform poorly in an application with pronounced rotation of principal stresses. The calibration of a constitutive model for the dominant stress path is likely to result in improved performance. Calibration of more general models with many parameters for practical use runs into the problem that data from more general stress conditions than those obtainable from the triaxial or simple shear tests used usually in practice are necessary. Furthermore more complicated models usually contain parameters not directly related to material properties and have to be calibrated against global response. This calibration may not be relevant to field conditions. The use of very general models also precludes the inference about properties from in-situ test data that are often a feature of simpler models.

Smith (1994) made another interesting observation: “Material behaviour as liquefaction is approached is probably not captured well in most models. This leads to ‘butterfly’ stress paths with excessive dilation towards the origin in stress space.” In effect, he is saying that analyses that track evolving behaviour approaching and beyond liquefaction are suspect. This is likely because of the disruption of structure and the significant changes in properties that occur when liquefaction occurs.

It is always assumed in prediction studies that the computational component of the computer program handles the computation of variables such as stresses correctly. This may not always be the case. There were VELACS predictions in which the static vertical stresses were clearly incorrect and also instances where complete liquefaction was predicted but the predicted acceleration time histories represented shaking as if the material behaviour was unchanged (Scott, 1994). Since scrutiny of the programs was not possible, the reasons for the problems could not be pursued. This highlights a weakness in validation studies. There is access only to the results but not to the process that produced them.

In selecting a model for use in the analysis of critical, high consequence structures, one cannot rely on element tests alone. Knowledge of model performance in non-homogeneous stress states is an essential requirement. Data bases from VELACS and other centrifuge tests are now available and a prudent step by any user is to check how a model under consideration for adoption performs against these data.

4.6 Turkey Flat Blind Prediction Experiment

The Turkey Flat experiment site was established in 1987 to evaluate the capability of geotechnical earthquake engineers to conduct reliable site response analyses (Tucker and Real 1986). It was an opportune time for such an experiment. A wide range of non-linear constitutive models had become available and were beginning to be used in practice (Finn 1988) and there was a clear need to evaluate the capability of these models under conditions emulating those encountered in practice. Therefore the site was chosen to be representative of those chosen for development and data on the site was limited to that normally available to practicing engineers. In many ways the Turkey Flat experiment is ideal. Turkey Flat is a dry site and therefore can be analyzed by total stress analysis. This is a fortunate choice because it allows an evaluation not only of the total stress options in effective stress programs (which have both total stress and effective stress options) but also of widely-used equivalent linear programs such as SHAKE (Schnabel et al., 1972) which operate only in the total stress environment. The site is geologically simple, the type of motions normally used for input are available from recordings and the site has been characterized in detail. Full details on the Turkey Flat site and experiment can be found in Real et al., (2006a, b); Shakal et al., (2006) and Haddadi et al., (2008).

A plan and cross-section of the site showing the strong motion sites where the data for the blind prediction study were recorded are shown in Figure 19. Downhole sensor D1 is at a depth of 24m in the same rock formation as R1. D3 is 1m below the rock surface and 24m below the ground surface at the valley center. The sensor D2 is at a depth of 11m. The distance from R1 to V1 is 800m and from V1 to V2 is 500m. Predictions were required at all sites except V2 and R2 which were optional and are not discussed here. Predictions were based on two different site characterizations: a Standard Model sent to all predictors and a Preferred Model which each predictor himself developed from the site characterization data base.

![Figure 19. Schematic illustration of the Turkey Flat site-effects test area and the strong-motion array stations (after Tucker and Real, 1986)](image)

4.7 Results of Experiment

Response spectra for V1 based on input motions from R1 are shown in Figure 20 substantially exceed the spectra of the recorded motions at V1. In practice, the most common
input motion for response analysis used in engineering practice is an outcrop motion on rock or hard soil. Predictions based on the outcrop motions at R1, therefore, most closely simulate the procedures followed by analysts in practice. It is evident that the predicted accelerations substantially exceed the recorded accelerations. The use of preferred site models resulted in only minor improvements over the standard model. Several predictions are grouped together but there are some outliers.

Better results would be expected from analyses of the response of V1 that used the recorded motions at D3 in the rock underlying the valley sediments as input motions. Predictions of peak ground accelerations and response spectra were much better. The response spectra are shown in Figure 21. Some spectra greatly exceeded the spectra of recorded motions, some greatly underestimated these values but there was a substantial clustering of predicted values around the actual spectrum.

On the basis of all predictions, Shakal et al (2006) summarized the findings of the Turkey Flat experiment as follows:

“The results of the Phase I predictions (based on the rock record at the valley edge) showed over prediction of peak accelerations around 50% and of peak response spectra by as much as 3-5 times that observed. The results of the Phase II predictions, based on the rock record D3 at the base of the valley sediments, are much closer to the observations. The results show over and under predictions of the peak acceleration and spectra. The spectra beyond 0.4 second (frequencies below 2.5 Hz) cluster around each other and the observed spectra well. An important conclusion is that the present ability to predict the motion through a sedimentary layering is much better than the ability to predict the same motion using an observed surface rock record from about 800m away, in the Turkey Flat environment.” (Italics added by the writer). The last observation is important because most analyses follow this procedure in practice.

Haddadi et al (2008) analyzed records from 8 other earthquakes recorded at Turkey between 1993 and 2004. All the motions had peak ground accelerations greater than 1%g. They found that although D1 and D3 are in the same rock formation, at the same depth and only 800m apart, the motions at D3 are consistently smaller than at D1 for most periods in the range 0.04 to 4s and concluded that this helps to explain why all predictions based on R1 over-predicted the motions at the valley center location, V1. Because the data set consisted of motions that were recorded at different distances from Turkey Flat and resulted from different magnitude earthquakes, they concluded that the lower observed motions at D3 were not likely to be caused by source or path effects.

4.8 Interpretation of Results

Two practical conclusions can be drawn from the Turkey Flat experiment: (1) the outcrop motions at the edge of the valley are not appropriate input motions in this instance and (2) even in a simple geological environment considerable scatter in ground motion predictions from different models can be expected.

The R1 site is close to the valley edge and is within the region that is likely to be affected by the topography of the rock soil interface. Topographic effects on ground motions have been the subject of many studies (Aki and Larner, 1988 and Finn, 1991) and the potential for ground motion amplification is recognized. The French building code includes a provision for taking topographic effects into account. The amplification effects have been confirmed in a number of recent studies. Psarropoulos et al (2007) have studied linear and nonlinear amplification ratios across the alluvial valley spanned by the Ohba Ohashi (bridge) in Japan. Many seismic records have been obtained at the base and surface of the valley during past earthquakes (Tazoh et al., 1984). On the basis of 2-D nonlinear analysis using Quad4M (Hudson et al., 1994), Psarropoulos et al. (2007) found that for nonlinear response the acceleration amplification ratios were about the same at the center as at the edge of the valley and were of the order of 3.
Many of the predicted response spectra based on the motions at D3 clustered around the spectrum of the recorded motions. This kind of scatter is to be expected as the parameters of the various constitutive models are deduced from the standard model of the site and the typical engineering soil properties provided to predictors. The large scatter in some of the predictions is surprising because of the simplicity of the site and the significant level of information provided. However there is no published information available that would allow an investigation of the reasons for the large scatter. As stated earlier, this is a weakness of all blind prediction exercises. Significant discrepancies between predicted and recorded data are explored without any scrutiny of programs that perform poorly. The evaluation performance is based almost entirely on a critical assessment of the data provided to predictors.

5 GUIDELINES FOR NONLINEAR DYNAMIC ANALYSIS

5.1 General Background

There are no generally accepted guidelines for conducting nonlinear dynamic analysis to ensure the most reliable assessment of dynamic response. The USACE is currently in the process of developing a manual for guidance in performing deformation analysis of earth and rockfill dams under earthquake loading (Sharp et al., 2005). The manual will focus a standard implementation of deformation analysis. A set of tentative guidelines is presented here for consideration by the profession. These guidelines relate only to the promotion of procedures that will help to ensure the reliability of the analysis for a given seismic input. The complex process of selecting appropriate input motions is also a crucial component of the analysis but is outside the scope of this paper.

The objective of the guidelines is to ensure as far as possible consistent, reliable results from non-linear effective stress analyses. To provide a context for the development of these guidelines, a worst case, if somewhat unlikely, scenario is assumed. In this scenario a designer has decided on the basis of a deformation analysis not to remediate an embankment dam with a potential for liquefaction in the foundation and the dam subsequently fails during an earthquake. Furthermore it will be assumed that the actual earthquake motions were adequately represented by the input motions used and that the site investigation met the standards of good practice. Therefore, since there is strong prima facie evidence that the analysis was wrong, the analytical process including the calibration of the constitutive model will be the main focus of any inquiry into the failure. The competence of the analyst and the procedures he followed will be subjected to the sharpest scrutiny. His understanding of the constitutive model and the computational procedures used to implement it will be probed. Superficial knowledge will not suffice for an effective defense. The analyst needs to have a thorough theoretical understanding of the mechanics of the constitutive model and of the computational process being used to implement the model in analysis or be able to demonstrate that expert help on these matters was obtained through a knowledgeable Review Board member or an outside consultant.

A thorough understanding is required not just of stand-alone programs such as DYNAFLOW (Prevost et al., 1988) but also of platforms such as FLAC. The analyst needs to understand the macros in FLAC that are used to implement a particular constitutive model including any implicit or overt constraints or limitations on their use. This is especially true for large deformation problems.

There is a need to justify why a model was selected for analysis by demonstrating its proven capability to conduct reliably the kind of analysis in contention. Therefore the analyst needs to be familiar with the validation history of the model. Here it is important to be clear on the distinction between simulation and validation. All published programs have appeared accompanied by demonstrations of how well they can simulate the response in element tests in the laboratory but most have not fared particularly well in prediction tests as discussed earlier. A notable example from the VELACS experiment is cited by Scott. The experimental test data was released at the conclusion of the prediction stage and participants were invited to submit Class C predictions that is, simulations. Only one simulation was submitted. Following significant changes in many of the model parameters, one of the worst performing models in the prediction exercise was able to simulate the test results quite well. Clearly simulation is a necessary but not sufficient condition to guarantee that a program will perform adequately in a design situation. A troublesome challenge for an analyst would be defending the selection of a model that did not perform well in a blind prediction test. Clearly the analyst should be familiar with the validation history of any model or computer program being used in practice. In addition, before adopting a model, it would be prudent for the analyst to check the model capability and its implementation himself using available data from blind prediction experiments, relevant centrifuge tests or field data.

The parameters of the constitutive model should be determined using data that is as representative as possible of field conditions. Some simpler models are based on material properties that are typically available from comprehensive site investigations. Other models have parameters that must be determined by laboratory testing. In jobs where liquefaction is an issue, very often sands need to be tested. Because of the difficulty of getting undisturbed samples, tests are conducted on reconstituted samples at field densities. How these samples are reconstituted has major effects on liquefaction potential and stress strain behavior. There is a major difference in liquefaction potential between samples formed by moist tamping and samples formed by pluviation under water.
which is representative of field deposition for Holocene sands and in addition it has been demonstrated that the stress strain behavior is very different. These aspects were discussed in the review by Finn (1998). Therefore it is imperative to form samples to be used for determining model parameters to be as representative as possible of field deposition conditions. Furthermore all models, even the most general, are stress path dependent in practice. Path dependency is probably a major factor in the difference in capability exhibited by models in simulations and validations. Therefore model parameters should be determined for loading paths that mimic the dominant actual or assumed loading paths in the field.

The designer (as distinct from the analyst) needs to understand in a physical way what elements in the model/analysis contribute significantly to the response and to assess whether these elements can be relied on, given the differences between the real environment of the dam during shaking and the ideal environment of the analysis. For example, if a coupled analysis is used, it may be that dilation provides significant resistance to displacement. Since dams subjected to liquefaction typically crack badly can the designer rely on dilation?

5.2 Tentative Guidelines

A set of guidelines reflecting the previous discussion is given below.

1) The analyst should have a good understanding of the mechanics of the constitutive model and the details of the computational procedures used to implement it.
2) The analyst should critically review the validation history of the constitutive model, particularly its performance in blind prediction tests.
3) The analyst should test the capability of the model himself using available data bases of centrifuge test data, preferably from tests with significantly non-homogeneous stress states.
4) The parameters of the constitutive model should be determined in a manner that reflects field conditions and is compatible with the predominant type of stress path in the field.
5) The sensitivity of the response to variations in model parameters should be checked by parametric studies.
6) A confirmatory analysis should be conducted using a different constitutive model.
7) The designer needs to have a physical understanding of the elements in the model that contribute to stability in order to judge whether any particular element may not be operative if the structure begins to crack as is often the case with dams prone to liquefaction.
8) The Review Board on critical projects should have an expert on dynamic response analysis to advise the Board on model selection, calibration and data interpretation.

The reviewer of an earlier version of this paper, after perusing the proposed guidelines, posed a very pertinent question: “Is there not some argument to be made that the practicing engineer can place some reliance on published model verification and validation by experts such as yourself in support of routine design decisions and analysis?” In response to this question, guideline #8 was added. The word “routine is somewhat ambiguous. It is doubtful that the complex analyses considered in this paper would be applied to very routine problems. However any published validation of a model being considered for use should be carefully scrutinized to see if it meets the conditions for being representative of field conditions and loading paths presented above. Only if it meets these requirements does published validation history have any relevance.

6 CONCLUDING REMARKS

Non-linear dynamic effective stress analysis is an essential tool in seismic design in geotechnical earthquake engineering. It is crucial to success in performance based design especially when extensive remediation is involved. But it is a complex process and requires analytical skills and a very thorough knowledge of soil behavior.

The selection of an appropriate constitutive model for the job at hand requires knowledge of the past history of the model; how it fared in blind predictions tests, how it was validated and what its track record is in practice.

The calibration of the model for the job at hand, if it cannot be done on the basis of in situ data, needs to be conducted on test specimens that are representative of field conditions and for stress paths that reflect the dominant stress paths in the field application.

The designer needs to be aware of what the main factors controlling the results of the analysis are and make judgments as to whether any of them, such for instance, dilation or the assumption of a continuum can be relied on in the field situation.

It is advisable for high consequence projects to have on the review board an expert on the important aspects of dynamic response analysis and to use this expert in the early stages of the analysis.

Non-linear analysis of critical, high consequence projects is a high risk endeavour to be conducted with the greatest vigilance.
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Seismic response of cracked soil deposits and its potential effects on structures

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ABSTRACT: Stiff clays are capable of sustaining cracks and fissures which in some cases, depending on the subsurface conditions and external environmental factors, such as long periods of drought or excessive underground water extraction from underlying aquifers, can affect the static and seismic behavior of structures located on top of them. This phenomenon may impact the structures functionality, and in some extreme cases, can even bring constructions to collapse. This paper examines the influence that cracks and discontinuities (closed cracks) can have on the seismic response of a hypothetical soil profile, using a 2-D finite difference model. The soil is considered as having a bilinear behavior using a Mohr-Coulomb model. The cracks are simulated with interface elements, where the soil stiffness is used to characterize the contact force that is generated when the crack closes. Both cases open and closed cracks (discontinuities) are considered. The nonlinear behavior was accounted for approximately using equivalent linear properties calibrated against several 1-D wave propagation analyses of selected soil columns with variable depth to account for irregularities in the basal rock geometry. Transmitting boundaries were used at the edges and bottom of the 2-D finite difference model to allow for energy dissipation of the reflected waves. The effect of cracking in the seismic response was evaluated by comparing the results of site response analysis with and without cracks, for several lengths and orientations. The difference in the response computed a single crack and a family of cracks was also evaluated. Finally, the impact that a crack may have in the structural response of a nearby structure was pointed out by solving the seismic-soil-structure interaction of a six story building using a fully coupled model. From the results of this investigation, insight was gained regarding the effect that discontinuities may have in the seismic response of soil deposits and adjacent soil-structure systems.

1 INTRODUCTION

The presence of cracks or fissures on rigid soils, such as stiff clays or sandy silts, is generally related to particular subsoil conditions combined with external environmental phenomena such as long periods of drought or excessive water extraction from underground aquifers. These natural discontinuities may affect both the static and seismic performance of structures sitting on top of them.

Although there are a number of researchers that have studied the static aspects of cracking including the concept of energetic settlement (Griffith, 1921) and intensity factor (Irwin 1957), others have looked into dynamic aspects such as wave propagation velocity in solids (Mott, 1948), kinetics energy evaluation in a cracked body (Roberts and Wells, 1954) and limit velocities of propagation in cracks (Stroh, 1957). Yoffe (1951) determined the stress field in the surrounding area of the beginning point of a crack spreading through an infinite region. Other researchers like Craggs (1960), Broberg (1960), Baker (1962) and Freund (1972) studied the curving and ramifications of cracks and the wave diffraction in solids due to the presence of stationary or propagating cracks.

Over the last decades, in recent developments computer technology and numerical methods for geo-seismic applications have allowed study the problem of fracture dynamics more rigorously, particularly to those related to wave interaction in cracked media. Chen (1975) applied the finite differences method to solve the wave propagation problem in cracked elastic media. Frangi (1998) and Rodríguez et al., (1999), solved the same problem through the boundary element method (BEM) and the finite element method (FEM). Rodríguez et al., 2004 used the indirect boundary element method to analyze the seismic response and the diffraction due to one or more cracks in an elastic media, in the presence of P, SV and Rayleigh waves’ incidence.

In this work, the influence of cracks and discontinuities (closed cracks) in the seismic response of a hypothetical soil deposit, and a six story building located near by the crack, is assessed using a two-dimensional finite difference model. In this paper a discontinuity is considered as a closed crack. Thus full contact exist between both edges. Therefore only compresional forces are transmitted normally to the interface from one edge to the other. The site response is obtained with and without open and closed cracks, considering several longitudes and orientations of the crack. Both the effect of a single crack and a family of cracks is revised.

2 DESCRIPTION OF THE NUMERICAL MODEL

2.1 Dynamic Aspects

The dynamic analysis of a given geological profile, when the finite differences technique is utilized, requires solving the global equation of motion, commonly expressed in matrix form as:

\[
[M]\ddot{u} + [C]\dot{u} + [K]u = P(t)
\]

Where [M], [C] and [K] are the global matrixes of mass, damping and stiffness, resulting from the assemblage of each individual element. The vectors \{\ddot{u}\}, \{\dot{u}\}, \{u\} are the relative nodal accelerations, velocities, and
displacements vectors, with respect to the model base, and \( \mathbf{P}(t) \) is the vector of dynamic load. The nonlinear internal force is given by the term \( [\mathbf{K}][\mathbf{u}] \). During the dynamic event, the dissipation of hysteretic energy occurs through the nonlinear soil behavior, which in this case was considered linear-perfectly plastic and associated to the Mohr-Coulomb failure criterion. The small strain damping is given by the damping matrix, which is defined based on a Rayleigh type formulation, which is only an approximation to the frequency independent damping, used in solutions formulated in the frequency domain that does not have exact solution in the time domain. This is only used to estimate the damping at the beginning of the dynamic event. In subsequent times, the damping is characterized throughout the hysteretic soil response and the nonlinear force. The mass matrix was built using an average of the consistent and lumped mass matrices to better estimate the fundamental frequencies of the system, and to optimize the ability of the element to transmit high frequencies Romo et al. (1980). The time variation of the model is solved using the Newmark integration method.

Figure 1 illustrates the model considered for the parametric study presented herein, which includes both the wave propagation analysis in the soil deposit to assess potential changes in ground response due to the presence of cracks, and the seismic soil structure interaction, SSI, analysis to revise the effect in adjacent structures.

![Figure 1: Model diagram for dynamic analyses](image)

2.2 Discontinuities Modeling

There are several instances in geo-mechanics in which it is desirable to represent planes on which sliding or separation can occur such as joints, faults or bending planes in a geologic medium; an interface between a foundation and the soil; the contact plane between a bin or chute and the material that it contains; and a contact between two colliding objects.

Discontinuities or contact points that may exist between two surfaces can be represented in a model through interface elements. There are a number of interface elements similar to that depicted in Figure 2, which are able to simulate the sliding, and opening-closing mechanism, in the case of seismic response, between two contacts. Crack opening and sliding occurs once the limit tension is exceeded at the interface between two contacts in the parallel or tangential direction with respect to the discontinuity. During crack closing the interface will have an equivalent normal stiffness and will transmit normal stresses and the corresponding strains to both sides of the crack. Equivalent normal stiffness, \( k_n \), has a direct influence on the dynamic time, required to solve the problem. The maximum \( k_n \) value can be obtained from the following expression:

\[
k_n = \frac{K + \frac{4}{3}G}{\Delta z_{\text{min}}} \quad (2)
\]

![Figure 2: Representation of an interface connected to normal and shearing rigid springs and tension and slipping elements (FLAC, 2005)](image)

Where: \( K \) and \( G \) are volumetric and shear soil modulus, respectively, and \( \Delta z_{\text{min}} \) is the minimum distance from an element of the mesh in the normal direction to the interface (c.f. Figure 2).

3 NUMERICAL MODELING

3.1 Soil profile

A typical soil deposit such as those existing at the surroundings of Aguascalientes valley, Mexico, was considered as scenario for the seismic study. A schematic representation of the deposit and the corresponding properties of each material are presented in Figure 3, which were taken from Rojas et al. (1994).
As can be noticed, the deposit is comprised of a very stiff clay layer resting on top of basal rock with a very irregular geometry. The dynamic impedance between both materials \((V_R \gamma_R)/(V_S \gamma_S)\) is about 5. It was assumed that the water table was below the maximum depth of the model (i.e. 200 m). Figure 3 also shows the assumed crack location.

### 3.2 Input motion

Based on a survey of the available seismological information, the closest seismological station on rock, identified as TONA, which is located at about 170 km towards the southwest of Aguascalientes State, was identified (Figure 4). Thus, a representative strong ground motion recorded at this station was selected for the parametric study. Accelerations time histories and the corresponding response spectra for the strong ground motion recorded at TONA station during the October 9, 1995 earthquake \((M_w 7.3)\), are depicted in Figures 5 and 6 respectively. The seismic motion duration is 80 seconds and has a predominant period of about 2.2 seconds. This ground motion was deconvolved to the base rock using the program SHAKE (Schnabel et al. 1972).

### 3.3 Model characteristics

The finite differences mesh used to represent the idealized soil profile is shown in Figure 7. A total of 840 quadrilateral plane elements were used in the analysis. Mesh elements are 15 m long and 10 high. The stress-strain relationship for soils was assumed elastic perfectly plastic with a Mohr-Coulomb failure criterion. The rock was considered as an elastic material. Transmitting boundaries were placed at the edges of the model to allow radiation energy dissipation into the surrounding media. The ground motion was applied at the base of the model considering a rigid base. The depth of the model below the base rock was increased until the effect of potential reflection of incoming waves from the soil to the base was minimized.
3.4 Dynamic properties

Equivalent linear properties were used to take into account approximately soil nonlinearities. These properties were obtained throughout one-dimensional SH waves propagation analyses conducted with the program SHAKE at the five representative soil columns A, B, C, D and E, depicted in figure 8. The normalized shear modulus degradation and damping curves proposed by Sun et al., (1988) and Vucetic and Dobry (1991) respectively, were used to represent the dependency of dynamic soil properties with shear strain amplitude, for the low plasticity clays (10 < PI < 20%) prevailing at the site. These are presented in Figure 9. Table 1 shows a summary of the equivalent linear properties obtained for each 1-D soil column. As can be noticed, for this particular case, these properties do not show a significant variation in the horizontal direction as a function of the rock depth. However, as expected, the variation along the vertical direction is considerable. The dynamic properties used in the 2-D finite difference model to define the elastic part of the Mohr-Coulomb stress-strain hysteretic behavior are presented in Table 2. These correspond to the mean values of those obtained for each independent soil column properties compiled in Table 1. Figure 10 shows a comparison between the response spectra computed using SHAKE, and FLAC for a horizontal base rock, at several locations within the mesh. As can be noticed, the response is quite similar, demonstrating that the equivalent linear properties considered, and the 2D model boundaries are in agreement with the free field response computed using the program SHAKE and that there is not wave reflexion problems at the base of the model associated with the rigid base used in the FLAC model. It is warrant to mention that the simulation takes into account approximately the potential soil non-linearities presented at the almost vertical soil-rock interface, found at the upper left hand side of the model, using the elastic-perfectly plastic hysteretic soil behavior. This vertical interface leads to a sharp stiffness contrast that can generate surface waves, which could, in turn, increase the shear strains along the interface and decrease the ground response.
Figure 9. Shear modulus degradation (a) and damping curves (b) for the stiff clay used in the analyses.

Table 1. Equivalent lineal properties for each 1-D soil column obtained with SHAKE.

<table>
<thead>
<tr>
<th>Thickness [m]</th>
<th>Depth [m]</th>
<th>Damping, $\lambda$ [%]</th>
<th>Shear modulus, $G$ [kPa]</th>
</tr>
</thead>
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<tr>
<td></td>
<td></td>
<td>Column A</td>
<td>B</td>
</tr>
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<td>10</td>
<td>0 -10</td>
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<td>144505 144552 144548 144576 144572</td>
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<tr>
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<tr>
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<tr>
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</tr>
<tr>
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<tr>
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<tr>
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<tr>
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<tr>
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<tr>
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<tr>
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</tr>
<tr>
<td>10</td>
<td>120 -130</td>
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<tr>
<td>10</td>
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<tr>
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<tr>
<td>10</td>
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<tr>
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<tr>
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<tr>
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</tr>
<tr>
<td>10</td>
<td>190 -200</td>
<td>0.050 0.05 0.050 0.050 0.050</td>
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</tbody>
</table>
Table 2. Equivalent linear properties used in FLAC

<table>
<thead>
<tr>
<th>Thickness [m]</th>
<th>Depth [m]</th>
<th>Shear modulus $G$ [kPa]</th>
<th>Damping $\lambda$ [%]</th>
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<td>30 - 40</td>
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</tr>
<tr>
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<td>40 - 50</td>
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<tr>
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<td>60 - 70</td>
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<td>70 - 80</td>
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<tr>
<td>10</td>
<td>190 - 200</td>
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</table>

3.5 Site effects and ground motion incoherence

Site effects and ground motion incoherence are generated in the soil deposit seismic response due to the irregularity of the base rock geometry, which changes both the thickness of the stiff soil layer and the ground motions at the soil–rock interface. Figure 12 shows the acceleration response spectra computed at the five points, A throughout E, indicated in figure 8, compared with the response obtained assuming a horizontal base rock with the maximum soil depth as indicated in figure 11. As can be seen, the main effect of the irregular rock base appears to be the shortening of the predominant period of the soil. This is associated with the reduction of the effective thickness of the stiff clay layer, $H_s$, and the increase of the weighted average shear wave velocity, $V_m=(H_sV_s+H_RV_R)/(H_s+H_R)$ computed at each point.

![Figure 11. Horizontal and irregular half-spaces.](image)

![Figure 12. Comparison between the computed response with horizontal and irregular base rock.](image)

3.6 Crack length influence

The influence of the crack length in the seismic response of the soil deposit was evaluated assuming a closed vertical crack located at point X, as depicted in figure 13. The closed crack implies that there is contact between both edges of the crack, thus the crack is able to transmit forces oriented perpendicularly to the discontinuity in compression but not in tension. This figure also shows the location of several stations S-1, S-2, S-3, and S-4 where the responses were obtained. The crack was assumed to be 10, 20 and 40 m long. Figure 14 shows a comparison of the response spectra obtained at both sides of the discontinuity (points X’ and X”) for the three crack lengths (10, 20 and 40 m), and the response obtained when there is not a crack in the stiff clay. From these results two conclusions can be drawn. First for crack lengths, $L_c$, smaller than 20 m, the effect of the discontinuity in the ground response is negligible, as can
be deduced by the fact that none important change in frequency content nor amplitude is observed with respect those computed with no crack. Second, for \( L_c \) closed to 40 m or longer, the response changes significantly, as can be concluded for the important amplification of the spectral ordinates at point \( X' \). This, however, is missing in the response computed at point \( X'' \). This can be explained in terms of the fact that the basal rock is closer to the ground surface at the left side of the crack, and generates surface waves that are trapped in the left side of the discontinuity. Thus, although it is important to make appropriate considerations when designing engineering works to be located in the proximity of a crack to avoid risks resulting from an incoherent seismic response of the structure foundation, this effect is only relevant for quite long discontinuities. Figure 15 summarizes the effect that a crack with variable depth (10 to 40 m) may have in the peak ground accelerations, PGA, computed along the model surface. Again, it can be observed that the only effect occurs when the crack length reaches a value of 40 m. The maximum amplification occurs at the left side of the crack, however, it seems that the presence of the crack causes an out of face response on the ground with respect that observed in the soil deposit when there is no crack, amplifying or attenuating the surface ground motions and being the source of potential ground motion incoherence.

Figure 16 shows that the effect in frequency content is again more important in stations S-1 and S-2, which are located to the left of the crack, but less significant at stations S-3 and S-4, which is located to the right of the crack.
Regarding the effect of the crack at depth, figure 17 shows that this completely vanished for depths larger than the crack length, and the cracked media practically follows the un-cracked media motions.

![Figure 17. Influence of the crack in the peak acceleration variation with depth.](image)

3.7 Crack inclination influence

In this section the effect of an inclined crack is studied. A 40 m depth of crack with inclination, $i$, of 2.15, 5, -2.15, and -5 degrees, measured with respect to the vertical, was assumed, as schematically represented in Figure 18. This figure also shows the computed PGA’s variation over the entire model. As can be noticed, for the particular case studied herein, there is practically no effect due to crack orientation, even for the largest value considered. A similar conclusion can be drawn regarding frequency content, as can be observed in figure 19, which shows the corresponding response spectra at points $X'$ and $X''$, for $i$ equal to 0 and 5.

![Figure 18. Peak ground accelerations distribution for several crack orientations.](image)

3.8 Influence of a cracks family

The influence of three 40m-cracks separated 30m away from each other is studied. Figure 20 presents a schematic representation of the cracks, indicating their relative position. Figure 21 presents the computed response spectra at points $X'$ and $X''$ for both the single and the group of cracks. It seems that the crack group tends to reduce even further the period at which the soil is responding. This can be associated with high frequencies generated by the discontinuities. The peak spectral ordinate decreases when the crack group is considered with respect to that computed for a single crack. Figure 22 shows the corresponding PGA distribution over the model. It can be noticed a very important amplification of computed PGA for the crack family with respect to both the soil without crack and a single crack (more than 200%). This effect should be taken into account for foundation design to avoid damage of structures, in particular if they are stiff.

![Figure 20. Control nodes for influence of a cracks family](image)
3.9 Effects on adjacent structures

In order to observe the potential effect that a crack may have in the seismic response of nearby structures, a hypothetical six story building was considered located 15 m away from the crack, towards the left (Figure 23). The crack was assumed to be 40 m long, and inclined 5º to the building. The structure main characteristics including mass, stiffness and damping are also included in Figure 23. The building was modeled as a shear beam (Romo and Barcena, 1994). The computed response spectra and the corresponding acceleration time histories at the top of the structure, for both the cracked and un-cracked stiff clay, are presented in figures 24 and 25 respectively. From Figure 24, it can be seen how the structural response increases up to 30% with respect the case without crack. This important amplification should be taken into account during the design process.

3.10 Response of a open crack

In all the cases analyzed above, it was assumed that compressive normal forces were able to act in both sides of the crack (i.e. the crack was closed). In this section, the response of the soil deposit of a fully open crack (without contact between edges) was studied. A 40m-long crack, opened 30 cm at the surface (c.f. figure 26), was assumed in the analysis. Figure 27, shows the computed response spectra at points X’ and X’’ assuming no crack, and opened and closed cracks. As can be seen, for this particular case, the stiffness of the soil prevented important relative movements between both edges of the crack, even when the crack was opened. Thus, the response obtained for both cases are essentially equal.
Figura 27. Computed response spectra for an opened and a closed crack

4 CONCLUSIONS

Key aspects regarding the seismic response of cracked stiff soils are presented in this paper. From the parametric study the following conclusion can be drawn: 1) the crack length is perhaps the most important factor and affects both PGA as well as frequency content. 2) For the cases analyzed not significant influence in the soil deposit response was observed as a function of crack inclination. 3) The combination of several cracks may increase significantly the PGAs, even up to four times and change drastically the frequency content. 4) Regarding the effect on structures, the presence of cracks increase up to 30% the response computed without crack.

REFERENCES

Soft computing: Applications to geoseismic engineering

Silvia Garcia., Research Engineer, Institute of Engineering, National University of Mexico
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ABSTRACT: Soft Computing, SC, is an association of computing methodologies that includes, as its most important members, fuzzy logic, neurocomputing, evolutionary computing and probabilistic computing. These tools are a great match for geoseismic applications that require the analysis of uncertain and imprecise information and where an incomplete understanding of the phenomena further compound the problem of generating models used to explain past behaviors or predict future ones. We outline the advantages of applying SC techniques (neural networks, genetic algorithms and regression trees) and in particular the synergy derived from the use of hybrid SC systems (fuzzy systems tuned by neural networks and neurogenetic models). Some successful geoseismic SC applications (estimation of liquefaction induced lateral spread, modeling of Mexico City ground motions, generation of synthetic seismic signals, prediction of peak ground accelerations from earthquake-subduction mechanisms, and spatial variation of soil - geometrical, mechanical and physical- properties) are described. They represent the basis for producing the technology and human resources necessary to transition from current geoseismic design methods to more scientifically-based procedures.

1 INTRODUCTION

Human beings built geotechnical structures long before there was a formal discipline of geotechnical engineering; however the ability of engineers to deal with geoseismic problems in analytically rigorous ways was limited. The natural geometries, materials and properties must be inferred from limited and costly observations with uncertainties that are largely inductive.

The progress in procedures for laboratory testing and techniques for field measurements has provided the most important prerequisite for technological advancement: a system of reference to catalogue observation and experience. But regarding theoretic fundamentals, traditional models that study soils and earthquakes are justifiably simplified because of the complex behavior to be analyzed. Widely used-geo.seismic methods for design or evaluation are much less scientific and direct than the rigorous approaches developed in other science fields. The result of this empirical approach is that seismic performance outcomes, as demonstrated in recent earthquakes, are highly variable and often at odds with experts’ expectations.

The geoseismic engineering is striving to meet these ever growing challenges by acquiring new knowledge, devising tools and methods for gathering requirements, designing algorithms, verifying and validating natural and human-made systems behaviors, and offering suitable means for supporting learning evolution over time. Soft Computing SC technologies have provided us with a unique opportunity to establish a coherent geoseismic analysis environment in which uncertainty and partial data-knowledge are systematically handled.

By seamlessly combining learning, adaptation, evolution, and fuzziness, SC complements current approaches allowing us develop a more comprehensive and unified framework to the effective management of uncertainty in geotechnical earthquake engineering. This paper illustrates the rapidly growing area of SC through a review of successful geoseismic applications. To begin with, we will present our interpretation of the definition and scope of SC. In the remaining of the document it is offered a concise picture of the current research and trends of using SC to address timely geoseismic problems. The neurofuzzy estimation of liquefaction induced lateral spread, an innovative neural-modeling of Mexico City ground motions, the genetic generation of seismic signals, a decision tree for predicting peak ground accelerations in subduction-zones and the neurogenetic definition of the spatial variation of soil properties, are described.

2 SOFT COMPUTING

SC is a term originally coined by Zadeh (Zadeh, 1994) to denote systems that “… exploit the tolerance for imprecision, uncertainty, and partial truth to achieve tractability, robustness, low solution cost, and better rapport with reality.” SC is “an association of computing methodologies that includes as its principal elements fuzzy logic FL, neurocomputing NC, evolutionary computing EC and probabilistic computing PC” (Zadeh, 1998). However, some authors have discussed that machine learning ML (data mining) should be considered an important SC component because of its connectionist origin. It should be noted that a consensus to the exact scope or definition of SC has not been reached (Dubois and Prade, 1998).

The main reason for the popularity of SC is the synergy derived from its components. SC’s main characteristic is its intrinsic capability to create hybrid systems that are based on a (loose or tight) integration of constituent technologies. This integration provides complementary reasoning and searching methods that allow us to combine domain knowledge and empirical data to develop flexible computing tools and solve complex problems. Extensive coverage of this topic can be found in (Bouchon-Meunier et al., 1995) and (Bonissone, 1997).

2.1 SC Components

Fuzzy Computing. Fuzzy Logic, FL, has a logical facet, derived from its multiple-valued logic genealogy; a set-theoretic facet, stemming from the representation of sets...
with ill-defined boundaries; a relational facet, focused on the representation and use of fuzzy relations; and an epistemic facet, covering the use of FL to fuzzy knowledge based systems and data bases. Fuzzy logic gives us a language, with syntax and local semantics, in which we can translate qualitative knowledge about the problem to be solved. In particular, FL allows us to use linguistic variables to model dynamic systems. These variables take fuzzy values that are characterized by a label (a sentence generated from the syntax) and a meaning (a membership function determined by a local semantic procedure). The meaning of a linguistic variable may be interpreted as an elastic constraint on its value. These constraints are propagated by fuzzy inference operations based on the generalized modus-ponens. This reasoning mechanism, with its interpolation properties, gives FL a robustness with respect to variations in the system's parameters, disturbances, etc., which is one of FL's main characteristics. A comprehensive review of fuzzy logic and fuzzy computing can be found in (Ruspini et al., 1998) and ((Pok and Xu, 1994).

Neural Computing. Neural Networks NNs are computational structures that can be trained to learn patterns from examples. They are integrated by a network of processing units or neurons. Each neuron performs a weighted sum of its input, using the resulting sum as the argument of a nonlinear activation function. By using a training set that samples the relation between inputs and outputs, and a learning method that trains their weight vector to minimize a quadratic error function, neural networks offer the capabilities of a supervised learning algorithm that performs fine-granule local optimization. The introduction of backpropagation (Werbos, 1974; Parker, 1985; and LeCun 1985) provided a sound theoretical way to train multi-layered/ feed-forward networks (nonlinear activation functions), proved as universal functional approximators (Hornik et al., 1989).

Topologically, NNs are divided into feed-forward and recurrent networks. In the context of this paper, we will only consider feed-forward NNs. A comprehensive current review of neuro-computing can be found in (Fiesler and Beale, 1997).

Evolutionary Computing. Evolutionary computing, EC, algorithms exhibit an adaptive behaviour that allows them to handle non-linear, high dimensional problems without requiring differentiability or explicit knowledge of the problem structure. As a result, these algorithms are very robust to time-varying behaviour, even though they may exhibit low speed of convergence. EC covers many important families of stochastic algorithms, including evolutionary strategies ES (Rechenberg , 1965; Schwefel, 1965), evolutionary programming EP (Fogel, 1962) and genetic algorithms GAs (Holland, 1975]), which contain as a subset genetic programming GP. As noted by Fogel (1995), ES, EP, and GAs share many common traits: “...Each maintains a population of trial solutions, imposes random changes to those solutions, and incorporates selection to determine which solutions to maintain in future generations...”. In the context of this paper, we will limit most of our discussion to GAs that emphasize models of genetic operators as observed in nature, such as crossing-over, inversion, and point mutation, and apply these to real-number-encoded chromosomes.

Data Mining. Data mining, DM, involves the use of sophisticated data analysis tools to discover unknown, valid patterns and relationships in large data sets (Adriaans and Zantinge, 1996). These tools can include statistical models, mathematical algorithms, and machine learning methods, ML. DM can be performed on data represented in quantitative, textual, or multimedia forms and use a variety of parameters to examine the data, including association (patterns where one event is connected to another event), sequence or path analysis (patterns where one event leads to another event), classification (identification of new patterns), clustering (finding and visually documenting groups of previously unknown facts), and forecasting (discovering patterns from which one can make reasonable predictions regarding future conditions). The aim of ML users is to comprehend the structures that are abstracted from a dataset. Decision, classification or regression trees (the natural generalization of decision trees for regression) are especially attractive type of models for three main reasons: i) intuitive representation, ii) nonparametric models and iii) scalable algorithms. For the reader interested in ML and particularly in regression trees, Breiman et al. (1984) is recommended.

2.2 SC Taxonomy

The common denominator of these technologies is their departure from classical reasoning and modeling approaches that are usually based on Boolean logic, analytical models, crisp classifications, and deterministic search. In ideal problem formulations, the systems to be modeled or controlled are described by complete and precise information. When we analyze real-world problems (ill-defined, difficult to model, with large solution spaces) the solution must be generated by leveraging two kinds of resources: i) knowledge of the problem (a combination of first principles and empirical knowledge) and ii) field data that characterize the behavior of the system (a collection of input-output measurements, representing instances of the system's behavior). The two main approaches in SC are knowledge-driven reasoning systems (such as probabilistic and fuzzy computing) and data-driven search and optimization approaches (such as neuro and evolutionary computing). This taxonomy, however, is soft in nature, given the existence of many hybrid systems that span across more than one field.

3 SC SOLUTIONS

Although it would be presumptuous to claim that SC solves all kind of geotechnical or seismic problems, it is
reasonable to affirm that it provides a different paradigm in terms of representation and methodologies, which facilitates these integration attempts. For instance, the traditional approach for developing models, model = structure + parameters, does not change with the advent of SC, however, we now have a much richer repertoire to represent the structure, to tune the parameters, and to iterate this process. It is understood that the search method used to find the parameter values is an important and implicit part of the above equation, which needs to be chosen carefully for efficient model construction.

In the following a brief review of the interaction of knowledge and data in SC hybrid systems is presented. To tune knowledge-derived models we first translate domain knowledge into an initial structure and parameters and then use global or local data search to tune the parameters. The task of these applications is predictive modeling: forecasting a course of events from a set of observations. Predictive modeling can be used for diagnosis (a set of fault hypotheses is identified), for control (the system has to output control actions that will impact the future state of the system) and for estimation (inference of specific parameter values based on identified attributes of the system).

The geoseismic-SC models are offered as engineering software. Undoubtedly, software has become a pervasive and critical component of our lives. It is needless to say that an even increasing spectrum of human individual and social endeavors depend, and will growingly depend, on software systems and their infrastructures. As in many areas of expertise, in geoseismic engineering human experts are needed. Usually, there are very few top level experts, and it is not physically possible for these few experts to solve all numerous related problems. It is therefore desirable to develop a computer-based system which incorporates the knowledge of the top experts and uses it either to directly solve the related problems or, at least, to provide high-level advise to engineers trying to solve these problems.

3.1 NEFLAS: Neurofuzzy estimation of liquefaction induced lateral spread

The intricacy and nonlinearity of the phenomena, an inconsistent and contradictory database, and many subjective interpretations about the observed behavior, make SC an attractive alternative for estimation of liquefaction induced lateral spread. NEFLAS (Romo and García, 2007), NEuroFuzzy estimation of liquefaction induced LAteral Spread, profits from fuzzy and neural paradigms through an architecture that uses a fuzzy system to represent knowledge in an interpretable manner and proceeds from the learning ability of a neural network to optimize its parameters. This blending can constitute an interpretable model that is capable of learning the problem-specific prior knowledge.

NEFLAS is based on the Takagi-Sugeno model structure and it was constructed according the information compiled by Bartlett and Yould (1998) and extended later by Yould et al. (2002). The input-output data considered in NEFLAS may be represented by the following function:

\[ D_h = f(M_w, R^2, A, S, W, L, T_{15}, D_{5015}, F_{15}) \]

where \( D_h \) is the horizontal displacements due to liquefaction, \( M_w \) is moment magnitude, \( R^2 = 10^{0.69 M_w - 5.64} + R \), in kilometers; \( R \) is the nearest distance from the source in kilometers; \( W \) is the free face ratio \( (H/L) \) expressed in percent; \( S \) is the gradient of the surface topography or the slope of the liquefied layer base; \( T_{15} \) is the cumulative thickness (in m) of saturated cohesionless sediments with number of blows (modified by overburden and energy delivered to the standard penetration probe, in this case 60%) \( N_{160} \), equal or lower than 15; \( F_{15} \) and \( D_{5015} \) are the average of fines content, and the mean grain size, in millimeters, for soils within the \( T_{15} \)-saturated layer, \( A \) is the maximum ground acceleration, in units of gravity, \( g \); and the length from the free-face to the point of displacement, \( L \), in meters.

One of the most important NEFLAS advantages is its capability of dealing with the imprecision, inherent in geoseismic engineering, to evaluate concepts and derive conclusions. It is well known that engineers use words to classify qualities (“strong earthquake”, “poor graduated soil” or “soft clay” for example), to predict and to validate “first principle” theories, to enumerate phenomena, to suggest new hypothesis and to point the limits of knowledge. NEFLAS mimics this method. See the technical quantity “\( M_w \)” (earthquake input) depicted in Figure 1. The degree to which a crisp magnitude belongs to LOW, MEDIUM or HIGH linguistic label is called the degree of membership. Based on the figure, the expression, “the magnitude is LOW” would be true to the degree of 0.5 for a \( M_w \) of 5.7. Here, the degree of membership in a set becomes the degree of truth of a statement.

On the other hand, the human logic in engineering solutions generates sets of behavior rules defined for particular cases (parametric conditions) and supported on numerical analysis. In the neurofuzzy methods the human concepts are re-defined through a flexible computational process (training) putting (empirical or analytical) knowledge into simple “if-then” relations (Figure 1.a). The fuzzy system uses 1) variables composing the antecedents (premises) of implications; 2) membership functions of the fuzzy sets in the premises, and 3) parameters in consequences for finding simpler solutions with less design time.

NEFLAS considers the character of the earthquake, topographical, regional and geological components that influence lateral spreading and works through three modules: Reg-NEFLAS, appropriate for predicting horizontal displacements in geographic regions where
seismic hazard surveys have been identified; Site-
NEFLAS, proper for predictions of horizontal
displacements for site-specific studies with minimal data
on geotechnical conditions and Geotech-NEFLAS allows
more refined predictions of horizontal displacements
when additional data is available from geotechnical soil
boring s. The NEFLAS execution on cases not included in
the database (Figure 2.a) and its higher values of
correlation when they are compared with evaluations
obtained from empirical procedures (Figure 2.b) permit to
assert that NEFLAS is a powerful tool, capable of
predicting lateral spreads with high degree of confidence.

3.2 NESSER: Neural Network for estimation of ground
seismic response

In the study of seismic-site response, a natural system, it is
not possible to influence the environment or to repeat
observations under fixed conditions. Traditionally, data
from this kind of phenomena are associated with
traditional (linear) mathematical models that fail to
forecast particular behaviors. As computers open the door
to more advanced modeling approaches new relationships
between the complexity of the seismic data modeled and
the techniques used to model them must be established.
Accordingly to this challenge, NESSER coupled with
RPs-EFs (Recurrence Plots RPs and Eigenfaces EFs) is
postulated as an alternative tool for the modeling of
Mexico City ground motions (García et al., 2002).

RPs are tools based on chaos postulations and their most
direct link to the real world is the analysis of time series
data. This graphic tool (Eckman et al., 1987) is helpful to
find, in huge databases, longer-range correlations and
coherent structures that describe the underlying
phenomena’s behavior. RP is a two-dimensional
representation of a single trajectory (in NESSER, the
spectral accelerations vectors). The time series spans both
ordinate and abscissa and each point \( (i, j) \) on the plane is
shaded according to the distance between the two
corresponding trajectory points \( y_i \) and \( y_j \). The pixel
lying at \( (i, j) \) is color-coded according to the distance. For
instance, if the 118th point on the trajectory is 24 distance
units away from the 6435th point, the pixel lying at (118,
6435) on the RP will be shaded with the color that
corresponds to a spacing of 24.

Figure 3 shows RPs generated from two data sets: a
time series derived by sampling the function \( \sin t \) and
white noise. The colors on these plots range from white
for very small spacing to dark blue for large inter-point
distances, as shown on the calibration bars in the figure.
With this in mind, the sine-wave RP is relatively easy to
understand: each of the blocks of color simply represents
half a period of the signal. Alternatively, the RP for noise
displays a homogenous random pattern, signifying that
the phenomena behind this signal lacks of deterministic
structures. Besides the “explanation”, the proposed
method RPs-EFs uses the plots as visual parameters for
grouping and characterizing the signals according to soil
and seismic conditions. EFs technique (Turk and
Pentland, 1991), from the pattern recognition domain, is
proposed as the most suitable tool for locating significant
configurations and for the efficient management of the
geoseismic individuals (acceleration response spectra)
expressed as RPs. EFs permit to translate the visual
information into numbers to be used in any analytical
environment (soft or hard computing).

The RPs of recorded responses on the surface of 5 sites
in Lake zone and one in the Hill zone (outcropping)
within the urban area of Mexico City are shown in Figure
4. The elastic natural periods vary from 2 to 4.2 sec and
the event selected is representative of the subduction
mechanism that affects the valley. Due to space
restrictions, the plots show responses only for a high–
intensity earthquake but NESSER is trained to work with
low- and medium-intensity inputs.

Following the four functional blocks of the RPs-EFs
algorithm (for details see García, 2007): ‘Load-Signals’,
‘Construct-RPs’, ‘Calculate-Eigenfaces-Eigenvalues’,
‘Construct RPs-Space’ and their inverse operations
“Project-Eigenfaces-Eigenvalues”, “Reconstruct-RPs’ and
“Obtain-Signals’, NESSER recognizes and classifies
seismic input motions and stratigraphic conditions for the
prediction (through a neural networks operator) of the
surface ground motion.

NESSER users must introduce four inputs that
represent the seismic conditions (\( M_w \) moment magnitude,
\( E_D \) epicentral distance, \( F_D \) focal depth and the rock
base-RP) and one geo-referenced input that describes soil
type and geographical situation (for details see García,
2007). The output’ system is the acceleration response
spectra for the specific site and event.

Results of the training and prediction stages of the
neural system are shown in Figure 5. The figure shows the
measured and computed response spectra at SCT site
(Lake zone, soft soils deposit) for weak (M<6), medium
(6<\( M < 7 \)) and extreme (\( M > 7 \)) events originated from
diverse fault mechanisms. It can be noticed that the
differences among the spectra obtained from NESSER
and those registered are minimum. The author’s findings,
as those shown in this paper, clearly indicate that
knowledge-based procedures, coupled with advanced time
series analysis tools, are capable of modeling accurately
complex seismic problems.

3.3 GENES, Genetic Generator of Signals

For nonlinear seismic response analysis, where the
superposition techniques do not apply, earthquake
acceleration time histories are required as inputs. While
the available strong motion data represents a unique and
invaluable collection for studies and research of strong
earthquake ground motion, it does no cover all the needed
ranges of the parameters commonly used in empirical
scaling laws (i.e., earthquake magnitude and focal depth,
source to station distance, percentage of rock along the
GENES (García and Romo, 2007) permits to apply seismic and soil parameters for a realistic estimation of ground motions. When GENES is applied to synthetic accelerograms construction, it is capable of i) searching, under specific soil and seismic conditions, between thousands of earthquake records and recommending a desired subset that better match a target design spectrum, and ii) through processes that mimic mating, natural selection, and mutation, of producing new generations of accelerograms until an optimum individual is obtained. The procedure is fast and reliable and results in time series that match any type of target spectrum with minimal tampering and deviation from recorded earthquakes characteristics.

GENES takes into consideration that i) a typical strong motion record consists of a variety of waves whose contribution depends on the earthquake source mechanism (wave path) and its particular characteristics are influenced by the distance between the source and the site, some measure of the size of the earthquake, and the surrounding geology and site conditions; and ii) the design spectra can be an envelope or integration of many expected ground motions that are possible to occur in certain period of time, or the result of a formulation that involves earthquake magnitude, distance and soil conditions.

The input data consist of the ordinates of the target acceleration design spectrum, the period range for the matching, lower- and upper-bound acceptable values for scaling signal shape, and a set of seismic/soil and GA parameters. The output is the more successful chromosome in terms of an accelerations vector (or a set of). Additionally some GAs parameters are required: a population size, number of generations, crossover ratio, and mutation ratio.

GENES captures the evolutionary and localized features of the soil/rock-systems nonlinear response when they are subjected to nonstationary inputs (seismic loads). The GENES procedure is fast and reliable and the results indicate that genuine ground motions (adequate temporal evolution of their frequency content and reasonable predefined image of the expected earthquake) can be obtained regardless the type of target spectrum (Figure 6).

3.4 ARETRE, Attenuation Regression Tree

Until recently, only empirical approaches could be consistently used to specify ground acceleration median values and to provide estimates of uncertainty. These numerical methods are notoriously cumbersome and extremely dependent upon a large number of poorly characterized parameters. As a result, ground motion predictions based on these techniques suffered from unknown reliability. The empirical multi-parametric attenuation regressions proposed by several researchers are highly sensitive on source description, site conditions definition and understanding regarding wave propagation processes and the ray path characteristics from source to site.

![Figure 1. Example of: a) fuzzy rules and b) membership functions.](image)
Figure 2. NEFLAS results, a) test stage and b) comparison with other empirical models (modified from Romo and García, 2007).

Figure 3. Examples of RPs
ARETRE (García and Romo, 2007), an empirical-machine learning ML formulation, uses magnitude, distance, fault type and soil conditions to predict, through a regression tree, peak ground accelerations PGA with a high level of confidence. Decision trees, either classification or regression trees, are especially attractive type of models for three main reasons: i) intuitive representation, the resulting model is easy to understand and assimilate by humans, ii) nonparametric models, no intervention being required from the user, and thus they are very suited for exploratory knowledge discovery, iii) scalable algorithms, performance degrades gracefully with the increase of the size of training data.

ARETRE utilizes a discovery approach to examine the multidimensional data relationships simultaneously and identify those that are unique or frequently represented, permitting the acquisition of structured knowledge. The particular type of regressors we are interested in, are the natural generalization of decision trees for regression (continuous valued prediction) problems. Instead of associating a class label to every node in the tree, a real value or a functional dependency of some of the inputs is used.

ARETRE serves as a basis for structuring the discussions about phenomena-parameterization policy. Notice that the values of the predicted variable (numerical PGA) are at the bottom of the tree but it is necessary to use a linear model to calculate them (Figure 7). The practical exploit of this tool is straightforward: the user comes into the attenuation-tree system and presents the basic parameters for defining the event and site conditions (even missed attributes can be declared) then each branch and node of the tree is tracked for offering, in the terminal node, a simple attenuation relationship.

To use the ML structure the analyst has to tag on the branches in line with the instance being analyzed, when it reaches a terminal node a simple functional expression is given for estimating the concept according to the attributes and values contained in the example. The driven variable is $R$ (distance in km) followed by $M$ (magnitude), but none of the other input parameters are neglected for evaluating PGA.

An essential and significant aspect of this attenuation regression tree is that, while being extremely simple, it also provides estimates of strong ground motions with remarkable accuracy (Figure 8). Additional, but important, side benefits arising from the model’s simplicity are the natural separation of source, path, and site effects and the accompanying computational efficiency. As a result, an accurate appraisal of the effects of uncertainties in source, path, and site parameters as well as any model bias can be readily quantified.

3.5 NEGIS, Neurogenetic Spatial Variation of Soil Properties

Extremely valuable for modelers and designers is the estimation of the values of a specific variable at no sampled locations, providing an objective appraisal of the spatial variation of soil properties. Based on the high cost of collecting soil attribute data at many locations across landscape, the integration of GIS (Geographical Information Systems) and neurogenetic technology offers a potential mechanism to lower the cost of analysis of geotechnical information by reducing the amount of time spent understanding data. NEGIS (García and Romo, 2005) modeling/simulation of natural systems represents a new methodology for building predictive models using nonparametric methods to analyze physical, mechanical and geometrical parameters in a geographical context.

NEGIS can handle uncertain, vague and incomplete/redundant data when modeling intricate relationships between multiple variables. In Figure 9 two application examples of an easy integration of these technologies (GIS-NN-GAs) are shown. The multidimensional model of the soils underlying a Mexico City area classifies/predicts according to geometrical (Figure 9.a, the configuration of the first hard layer within the subsoil of the lacustrine zone) and mechanical (Figure 9.b, the distribution of the cone penetration resistance $R_{100}$) parameters. The results permit to conclude that NEGIS is a rational tool for interpreting available geotechnical information and for evaluating the soil parameters space variability. The NN and GAs techniques can be used to evaluate systematically the results of soil exploration, handling the inherent in situ testing uncertainty and taking advantage of the subjective traditional stratigraphic descriptions.

4 CONCLUSIONS

Based on the results of the studies discussed in this paper, it is evident that SC techniques perform better than, or as well as, the conventional methods used for modeling complex and not well understood geoseismic problems. Soft Computing SC is having an impact on many geoseismic operations, from predictive modeling to diagnosis and control.

The hybrid SC systems leverage the tolerance for imprecision, uncertainty, and incompleteness, which is intrinsic to the problems to be solved, and generate tractable, low-cost, robust solutions to such problems. The synergy derived from these hybrid systems stems from the relative ease with which we can translate problem domain knowledge into initial model structures whose parameters are further tuned by local or global search methods. This is a form of methods that do not try to solve the same problem in parallel but they do it in a mutually complementary fashion. The push for low-cost solutions combined with the need for intelligent tools will result in the deployment of hybrid systems that efficiently integrate reasoning and search techniques.
Figure 4. Typical Recurrence Plots of Mexico City soil and rock sites.
Figure 5. NESSER results for SCT site.
Figure 6. GENES results for different target spectrum.
$LM_i$: Linear Model

$LM1$: $\text{PGA} = 50.7836M + 11.3124h + 29.4365S - 25.4547R + 106.3215$

$LM6$: $\text{PGA} = 61.6757M + 0.5312h + 39.0959S - 0.4741R - 332.4499$

Figure 7. Some examples of ARETRE operation: departing from R until getting PGA.

Figure 8. ARETRE results for some subduction zones.
Figure 9. Spatial variation of properties.
REFERENCIAS

Analytic and neurogenetic methods to estimate in situ rock fill dam dynamic properties

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ABSTRACT: Models of real systems are of fundamental importance in virtually all disciplines because they can be useful for system analysis, i.e., for gaining a better understanding of the system. Models make possible to predict or simulate a system’s behavior. In engineering, they are required for the design of new processes and for the analysis of those that exist. Advanced techniques for the design of controllers, optimization, supervision, fault detection and diagnosis components are also based on models of processes. Since the quality of the model typically determines an upper bound on the quality of the final problem solution, modeling is often the bottleneck in the development of the whole system. As a consequence, a strong demand for advanced modeling and identification schemes arises. In this paper it is shown, via the development of a system for the analysis of the seismic response of rockfill dams, that neurogenetic techniques are a better option than analytically-based procedures. To buttress this assertion, El Infiernillo dam, which has an ample history of being shaken by a great variety of seismic events, is used to this end.

1 INTRODUCTION
Many combinations and nuances of theoretical modeling from first principles, i.e., physical laws, and empirical modeling based on measurement data can be pursued. An analytically-based system to study the seismic behavior of El Infiernillo dam is presented in Romo et al., 1989 and Romo, 2002. They resorted to a theoretical approach that drew support from measured responses (data) of this dam to several earthquakes. Besides the knowledge from first principles and the information oriented in the measurement data, qualitative knowledge formulated in rules were also utilized in the development of the model. The determination of the model structure relied strongly on prior knowledge and the model parameters were mainly determined from measurement data. This model has shown to be fairly good at extrapolating and providing good understanding of the physical phenomenon. From the engineering viewpoint is reliable and scalable. However, it is time consuming and requires a degree of expertise on dam engineering to be applied in design.

During the past several years the authors have been using cognitive techniques for developing alternate procedures to solve geotechnical (e. g., Romo et al., 2001; Garcia, 1999) and geo-seismic engineering problems (Garcia et al., 2003; Garcia et al., 2004). Considering that these and further experiences have proven that knowledge-based neural techniques constitute an alternative with a number of advantages over analytical methods, it was just natural to step up the complexity of the problems dealt with. In this paper, a neurogenetic model is developed to extract information about the seismic behavior of El Infiernillo dam from its responses recorded during several earthquakes. Herein, this procedure is compared both with analytical model results and “unknown” measurement data.

2 SYSTEM IDENTIFICATION. A CLASSICAL APPROACH
Science deals with the inference of models from the comprehension of recorded data properties. System-identification handles the problem of making analytical models of dynamical systems on the basis of observed data from the system behavior. Herein system is understood as an object in which variables of different kinds interact and produce discernible signals, usually called outputs. The external signals (stimuli) that can be manipulated by the observer are called inputs. There may also be disturbances that cannot be fully controlled by the observer (Ljung, 1999).

In engineering, models are required for the design of new processes and/or the analysis of existing ones. Models of processes must draw from advanced techniques for them to be capable of performing the necessary tasks to ensure their adequate performance. In this paper only linear models will be addressed to because the response of well-built rockfill dams to seismic events has proven to be mostly linear (quasi-linear), even when shaken by severe earthquakes as the 8.1 magnitude Michoacan event of September 15, 1985, the epicenter of which was just a few tens of kilometers from El Infiernillo dam.

One of the key aspects of system-identification is the definition of the model parameters. This can be achieved by extracting the required information from observed data. The procedure of modeling is, usually, application dependent and often has its roots in tradition and specific techniques in the application area in question (Ljung, 1999). Basic techniques typically involve structuring of the process into block diagrams with blocks consisting of simple elements. Assembling these blocks is, now days, frequently done by means of computers, thus the final product is a software-system.

In the next paragraphs an example to show the procedures involved in developing a model to predict the response of a system, is given. Assume that a single-degree-of-freedom oscillator, like the one shown in figure 1, is capable of modeling the behavior a variety of structures. If this is so, then it is a matter of defining the characteristics of the model parameters such as the spring stiffness, k, the damping, c, and the mass, m, to develop a system. The equation of motion describing the response of this system is given by:
\[ m \ddot{x}(t) + c \dot{x}(t) + k x(t) = -m \ddot{y}(t) \]  \hspace{1cm} (1)

where the overdots mean differentiation with respect to time.

\[ k = \frac{1}{|H(0)|} \]  \hspace{1cm} (7)

2. From equations 3 and 7, the mass is computed from

\[ m = \frac{1}{(2\pi f_c)^2 |H(0)|} \]  \hspace{1cm} (8)

3. From equations 5 and 7, the damping ratio is given by

\[ \zeta = \frac{|H(0)|}{2|H(f_r)|} \]  \hspace{1cm} (9)

4. And, from equations 7, 8 and 9, the damping coefficient is

\[ c \approx \frac{1}{2\pi f_r |H(f_r)|} \]  \hspace{1cm} (10)

Equations 7 to 10 are the bases for much of the linear modal analysis of mechanical systems that determines the physical parameters in proposed linear differential equations of motion from measured data (Bendant et al., 1993). It is important to stress the fact that the above procedure is valid only for linear systems. Should nonlinearities were involved in the input-output data, the results would be meaningless. Interested reader in this subject may want to review a specialized book on this theme (Nelles, 2000).

When frequency response functions (i.e., equation 4) are estimated from measured input-output data both random and bias errors will be generally involved. The key to successful applications is an understanding of these errors and a diligent effort to minimize them. Approaches to minimization (optimization) are generally classified as a) supervised learning, b) reinforcement learning, and c) unsupervised learning. Of these alternatives the supervised technique is most used to optimize the so called loss function, which basically is a criterion that defines in mathematical terms what has to be optimized.

The most common choice for such criterion is the sum of squared errors or its square root. These errors come from the corruption of the outputs by noise \( n(t) \) that cause differences between the measured system, \( u(t) \), and model output \( \hat{u}(t) \), for a specified number, \( N \), of input samples. The procedure to achieve the error minimization and then optimum system is depicted in figure 2.

Supervised learning techniques are classified as linear, nonlinear local and nonlinear global optimization procedures (Nelles, 2000). The linear techniques are the straightest forward to apply and are mostly used herein for the analytical approach of system-identification. For a detailed discussion about linear and nonlinear optimization techniques the reader is remitted to (Ljung, 1999; Shepherd, 1997).

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The natural logical flow to an identification procedure is to, first, collect data, followed by selecting a model set, then the most accurate model is chosen. More often than not, the model first picked will not pass the validation test. Thus, the various steps of the procedure should be revised again as indicated in figure 3. There are several reasons why the model may be deficient: a) the numerical procedure failed to locate the more accurate model according to the convergence criterion imposed, b) the criterion was not properly chosen, c) the model was wrongly defined in the sense that it did not contain any appropriate description of the system, and d) the data set provided misleading information.

At this point it is worthy to mention that a model should be never accepted as a final and true description of the system we are trying to emulate. Rather, it should be regarded as an adequate representation of certain system facets that interest us. Real-life actual systems are entities of a different kind than analytical models. Hence, the acceptance of any model should be guided by usefulness rather than truth.

3 SYSTEM IDENTIFICATION. NEUROGENETIC APPROACH

To extract meaningful conclusions about complicated systems using information (measurement data) from a single sensor (for example, when studying seismic issues time series are recorded by an accelerometer) one of the most powerful analysis and design tools, and often one of the most difficult to develop, is a good model. In the process of identifying a dynamic model of an unknown system, any representation designed for reasoning about such system has to be both flexible enough to handle various degrees of uncertainty and complexity, and yet powerful. Formalizing the connection between data and knowledge can be addressed by two (antagonistic) ways: modeling, build a function that can mimic the data accurately and gives good results on new data sets, and abstracting, build a system that produces articulated knowledge from the data. In the first approach, emphasis is put on the ability to reproduce what has been observed. In the second approach, is fundamental achieving the ability to understand and explain the data in a human-friendly way. It is by linking the estimated model constraints to the important phenomena parameters that a designer can represent the human-originated knowledge using a numerical approximation, in what is commonly known as System Identification, SID.

Structural identification and parameter estimation depend upon input-output analysis wherein the relationship between drive and response is used to infer information about internal system dynamics (Casdagli, 1992). For nonlinear systems, parameter estimation is difficult and structural identification is even harder. Soft Computing, SC, techniques can be used to automate the former (Bradley et al., 1998), but the latter has, until now, remained the purview of human experts. One of the aims of qualitative reasoning, a branch of artificial intelligence, AI, is to automate the modeling process by abstracting knowledge, information, and reasoning to a qualitative level. Following this methodology, the aim of this work is to build a SC layer to automate the SID process (diagrammed in figure 4) around the traditional mathematical techniques and its engineering parameters. This layer automates the high-level stages of the modeling process that are normally performed by a human expert, reasoning from the input-output information to automatically choose, invoke, and interpret the phenomena data and the system results, in order to define parameters most broadly applicable (well formalized) and to generate improvements for models in current use.

Figure 2. Minimization process to model optimization

Figure 3. System identification loop

Figure 4. System identification phases
3.1 Neural Networks generated by Genetic Algorithms

SC is now a widely accepted term to cover those techniques including neural networks, NNs, fuzzy logic, FL, evolutionary computing, EC, and various probabilistic approaches (Sanchez, 1994). These methods are used in a variety of applications that demonstrate, in some way, the ability to tackle problems that contain uncertainty or imprecision in some way. NNs (Haykin, 1994) offer the ability of modeling highly non-linear relationships and have entered main stream computing with a number of well known applications in industry and commerce. Evolutionary computing (Davis, 1991) is a term that includes, for example, genetic algorithms, GAs, and genetic programming, GP, and these techniques are particularly useful for optimal search. In this work, a hybrid SC system was selected to develop the objective SID task. The structural and parametric neural learning, which are the counterpart of system identification and parameter estimation in classical system theory (Bonissone, 1997) mean the synthesis of the network topology (i.e., the number of hidden layers and nodes), while parametric learning implies determining the weight vectors that are associated to each link in a given topology.

The NN topology selection can be optimized via GAs. GAs (Goldberg, 1978) can be deployed to optimize, at the same time, different architecture’s parameters, such as activation functions, hidden nodes, and input variables, among others. In this work, an optimization process is included in the NN generation. There are many forms in which GAs can be used to synthesize or tune NN: to evolve the network topology (number of hidden layers, hidden nodes, and number of links) to find the optimal set of weights for a given topology, thus replacing BP; and to evolve the reward function, making it adaptive.

The GA chromosome needed to directly encode both NN topology and parameters is usually too large to allow the GAs to perform an efficient global search. Therefore, the above approaches are usually mutually exclusive, with a few exceptions (Maniezzo, 1994 and Patel et al., 1994) that rely on variable granularity to represent the weights. In (Montana, 1989) the use of GAs to train a feedforward NN with a given topology was proposed. Typically NNs using BP converge faster than GAs due to their exploitation of local knowledge. However, this local search frequently causes the NNs to get stuck in a local minimum. On the other hand, GAs are slower, since they perform a global search. Thus GAs achieve efficient coarse granularity search (finding the promising region where the global minimum is located) but they are very inefficient in the fine-granularity search (finding the minimum). These characteristics motivated Kitano (Kitano, 1990) to propose an interesting hybrid algorithm in which the GA would find a good parameter region which was then used to initialize the NN. At that point, Back-Propagation would perform the final parameter tuning. McInerney and Dhawan (McInerney et al., 1993) improved Kitano’s algorithm by using the GA to escape from the local minima found by the BP during the training of the NNs (rather than initializing the NNs using the GAs and then tuning it using BP). They also provided a dynamic adaptation of the NN learning rate (McInerney et al., 1993). For an extensive review of the use of GAs in NNs, the reader is encouraged to consult Schaffer, 1992 and Yao, 1992.

In this sense, the GA-NN developed here was synthesized and tuned evolving the network topology (number of hidden layers, hidden nodes, and number of links) to find the optimal set of weights for a given topology replacing the back-learning algorithm and to evolve the reward function, making it adaptive (Figure 5).

![Figure 5. The principle structure of a GA-NN system](image)

4 APPLICATIONS TO EL INFIERNILLO DAM

In spite of the tremendous advances in the field of dam earthquake engineering with the development of numerical methods such as the finite element, a series of problems have to be fully resolved for the confidence on this analytical method to be improved: first, the matter of modeling rockfill materials; secondly, the balance between the degree of complexity of the analytical technique used and the level of knowledge of the material properties; and, thirdly, the adequacy of the numerical technique in capturing the variables that determine the dynamic behavior of dams.

Regarding the modeling of rockfill materials there is still much to be learned due to the colossal difficulties of testing representative samples in the laboratory and the relatively limited applicability of geophysical field tests. Thus, one has to resort to case histories where seismic motions and the ensuing earthquake-induced dynamic movements have been recorded during a number of seismic events with various intensities and frequency contents. Two procedures are included in this paper that shows how to overcome (at least partially) this obstacle. One is based on the application of a three dimensional
finite element procedure and the other consists on the use of a neurogenetic technique.

The Hydroelectric project El Infiernillo was completed in 1964 on the Balsas River about 70 km from the Pacific Ocean (see figure 6). The maximum section of the embankment is 145 m high with average external slopes of 1.85:1 (Horizontal:Vertical) considering the up- and downstream berms (main section in figure 7). Construction details of the dam, materials used as well as their treatment are broadly described elsewhere (e.g., Marsal et al., 1964 and Marsal et al., 1967).

The seismicity of the zone is one of the highest in Mexico and since its construction the dam has been subjected to earthquake forces of different characteristics and intensities. The seismic events of table 1 include the tremors recorded at the dam site during the period 1975-1999. The statistics presented in this table indicate that there have been intervals of high and relatively low activity showing a periodic energy release, characteristic of subduction mechanisms. After the September 1985 seismic events, the activity at the dam site has decreased significantly and although numerous earthquakes have been recorded, none of them has caused appreciable damage or displacements to the dam.

Of all recorded seismic movements, the events S1, S2, S3, S4 and S5 have shaken the dam more severely. Their main characteristics are indicated in table 2. In general, these earthquakes have caused permanent displacements that have induced shallow cracking, mainly parallel to the dam axis.

The seismic instrumentation consists of digital accelerometers deployed on the embankment and the abutments as indicated in figure 7. Prior to 1985 there were only seven instruments, three on the embankment (E, F and G) and four on rock (A, B, C and D).

Later, in 1986, the vertical array (H and I) was installed.

As shown in table 1 many earthquakes have been recorded that enable the general dynamic characteristics of the dam to be estimated. In order to do this, the motions on rock and on the embankment need to have been recorded for the same seismic event. Of all the available records, the five earthquakes indicated in table 2 were selected for this purpose. It should be noted that these events occurred prior to the installation of the seismic vertical array (1986). In a later section, information recorded after this date is used to evaluate the predictive capabilities of the analytical model developed on the basis of the data discussed in the following paragraphs.

<table>
<thead>
<tr>
<th>Year</th>
<th>3 ≤ Ms &lt; 4</th>
<th>4 ≤ Ms &lt; 5</th>
<th>Ms ≥ 5</th>
<th>Total Annual</th>
</tr>
</thead>
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<td>1975</td>
<td>1</td>
<td>4</td>
<td>3</td>
<td>9</td>
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<tr>
<td>1976</td>
<td>6</td>
<td>19</td>
<td>3</td>
<td>28</td>
</tr>
<tr>
<td>1977</td>
<td>8</td>
<td>20</td>
<td>7</td>
<td>35</td>
</tr>
<tr>
<td>1978</td>
<td>6</td>
<td>15</td>
<td>9</td>
<td>30</td>
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<tr>
<td>1979</td>
<td>10</td>
<td>56</td>
<td>9</td>
<td>75</td>
</tr>
<tr>
<td>1980</td>
<td>23</td>
<td>18</td>
<td>3</td>
<td>46</td>
</tr>
<tr>
<td>1981</td>
<td>55</td>
<td>18</td>
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<td>97</td>
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<td>1982</td>
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<td>1983</td>
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<td>1</td>
<td>1</td>
<td>18</td>
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<tr>
<td>1984</td>
<td>19</td>
<td>2</td>
<td>0</td>
<td>22</td>
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<tr>
<td>1985</td>
<td>210</td>
<td>21</td>
<td>4</td>
<td>289</td>
</tr>
<tr>
<td>1986</td>
<td>-</td>
<td>3</td>
<td>3</td>
<td>6</td>
</tr>
<tr>
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<td>13</td>
</tr>
<tr>
<td>1991</td>
<td>-</td>
<td>16</td>
<td>3</td>
<td>19</td>
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<tr>
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<td>-</td>
<td>18</td>
<td>-</td>
<td>18</td>
</tr>
<tr>
<td>1993</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>6</td>
</tr>
<tr>
<td>1994</td>
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<td>14</td>
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<td>1995</td>
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<td>13</td>
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<td>13</td>
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</tr>
<tr>
<td>1998</td>
<td>-</td>
<td>6</td>
<td>2</td>
<td>8</td>
</tr>
<tr>
<td>1999</td>
<td>1</td>
<td>13</td>
<td>4</td>
<td>17</td>
</tr>
<tr>
<td>Total</td>
<td>398</td>
<td>297</td>
<td>79</td>
<td>853</td>
</tr>
</tbody>
</table>
Table 2. Main characteristics of the more significant seismic events

<table>
<thead>
<tr>
<th>Event</th>
<th>Date</th>
<th>Station</th>
<th>Transverse</th>
<th>Longitudinal</th>
<th>Vertical</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Amax (cm$^2$)</td>
<td>Vmax (cm/s)</td>
<td>Amax (cm$^2$)</td>
</tr>
<tr>
<td>S-1</td>
<td>11/X/1975</td>
<td>right bank</td>
<td>72.8</td>
<td>3.32</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td></td>
<td>berm</td>
<td>89.7</td>
<td>7.74</td>
<td>1.22</td>
</tr>
<tr>
<td></td>
<td></td>
<td>crest</td>
<td>300.2</td>
<td>9.21</td>
<td>1.85</td>
</tr>
<tr>
<td>S-2</td>
<td>15/XI/1975</td>
<td>right bank</td>
<td>40.8</td>
<td>2.38</td>
<td>0.52</td>
</tr>
<tr>
<td></td>
<td></td>
<td>berm</td>
<td>82.6</td>
<td>6.34</td>
<td>1.01</td>
</tr>
<tr>
<td></td>
<td></td>
<td>crest</td>
<td>191.6</td>
<td>6.71</td>
<td>0.77</td>
</tr>
<tr>
<td>S-3</td>
<td>14/III/1979</td>
<td>right bank</td>
<td>17</td>
<td>0.7</td>
<td>0.06</td>
</tr>
<tr>
<td></td>
<td></td>
<td>berm</td>
<td>133</td>
<td>8.2</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td></td>
<td>crest</td>
<td>371</td>
<td>14.5</td>
<td>1.54</td>
</tr>
<tr>
<td>S-4</td>
<td>25/X/1981</td>
<td>right bank</td>
<td>85</td>
<td>83</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td></td>
<td>berm</td>
<td></td>
<td>131</td>
<td>108</td>
</tr>
<tr>
<td></td>
<td></td>
<td>crest</td>
<td></td>
<td>338</td>
<td>194</td>
</tr>
<tr>
<td>S-5</td>
<td>19/IX/1985</td>
<td>right bank</td>
<td>131.7</td>
<td>6.37</td>
<td>1.46</td>
</tr>
<tr>
<td></td>
<td></td>
<td>berm</td>
<td></td>
<td>294.6</td>
<td>12.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>crest</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 7. Layout of strong motion instruments at El Infiernillo
4.1 Analitically-based System Identification

The procedure to estimate the dynamic characteristics of a system material has been discussed in Chapter 2. In case of complex structures such as a dam, finite element techniques are usually applied to compute the transfer function \( H(f) \). The methods followed to obtain the properties of this function are explained in Romo (2002). The model that resulted from the back analysis of dam responses to earthquakes given in table 2 was:

\[
G = G_{\max} \left[ 1 - \frac{(\gamma / \gamma_r)}{a + b(\gamma / \gamma_r)} \right] 
\]

\[
\lambda = \lambda_{\min} + \left[ \frac{1}{\gamma_{\min}} + \frac{1}{(\gamma_{\max} - \gamma_{\min})} \right] \gamma_r 
\]

where \( G_{\max} \) is the quasi-elastic shear modulus, \( \lambda_{\min} \) is the damping ratio for strain values of 10\%, \( \lambda_{\max} \) is the damping ratio for strain values of 10\%, and \( a, b \) and \( \gamma_r \) are soil parameters given in table 3.

<table>
<thead>
<tr>
<th>Material type</th>
<th>Model parameters</th>
<th>( a )</th>
<th>( b )</th>
<th>( \gamma_r )</th>
<th>( \gamma_{\min} )</th>
<th>( \gamma_{\max} )</th>
<th>( K_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>clay core</td>
<td></td>
<td>0.95</td>
<td>1.05</td>
<td>0.0233</td>
<td>0.051</td>
<td>0.233</td>
<td></td>
</tr>
<tr>
<td>granular filter</td>
<td></td>
<td>0.95</td>
<td>1.05</td>
<td>0.0233</td>
<td>0.036</td>
<td>0.251</td>
<td>100</td>
</tr>
<tr>
<td>and transitions</td>
<td></td>
<td>0.95</td>
<td>1.05</td>
<td>0.0179</td>
<td>0.034</td>
<td>0.236</td>
<td>100</td>
</tr>
<tr>
<td>compacted rockfill</td>
<td></td>
<td>0.95</td>
<td>1.05</td>
<td>0.0170</td>
<td>0.028</td>
<td>0.231</td>
<td>75</td>
</tr>
<tr>
<td>dumped rockfill</td>
<td></td>
<td>0.95</td>
<td>1.05</td>
<td>0.0170</td>
<td>0.028</td>
<td>0.231</td>
<td></td>
</tr>
</tbody>
</table>

The value of \( G_{\max} \) was evaluated based on recommendations for granular materials (Seed et al., 1970)

\[
G_{\max} = 1000K_2(\sigma_m)^{1/2} 
\]

where \( \sigma_m \) is the mean normal effective stress in lb/ft\(^2\) and \( K_2 \) is a soil parameter that depends mainly on the void ratio. The values indicated in table 3 were obtained from parameter identification analysis procedures (Ljung, 1999).

The maximum shear modulus for the core material was evaluated using (Seed et al., 1970).

\[
G_{\max} = 2200S_u 
\]

where \( S_u \) (in lb/ft\(^2\)) is the undrained shear strength of the clay soils defined from the envelope of the failure lines obtained from undrained, unconsolidated, laboratory tests (SRH-CFE-UNAM, 1976). The magnitude of the undrained strength, \( S_u \), was defined for the mean stress, \( \sigma_m \), values computed in the dam using finite element analyses (Romo and Villarraga, 1989) for the at-the-end-of-construction and first-reservoir-filling-conditions. In these analyses the construction procedure and reservoir filling stages as well as nonlinear (using a hyperbolic-type model) soil behavior were modeled.

The method of analysis described above was evaluated comparing the theoretical results with the dam responses measured during earthquakes that occurred after 1985. Also, the comparisons were extended to include the motions recorded by the instruments of the vertical array. This evaluation process gives more support to the predictive capabilities of the model because neither the seismic event nor one of the points of measurement was used during model development. It should be pointed out that the dynamic behavior of the materials represented by equations 11 to 14 (and parameters of table 3) as well as the finite element mesh of figure 8, were used in the evaluations analyses. Accordingly, calculated responses may in this sense be considered as true predictions.

4.2 Neurogenetic System Identification

In this section the procedures discussed in Chapter 3 are employed to predict the response of the dam and to identify the dynamic properties of its constitutive materials.

The random nature of seismic excitations, along with the limited number of sensors used to monitor system responses, make the modeling of dynamic behavior of full-scale soil-systems (such as earth dams) a quite difficult task. In this paper, a new system identification technique was developed using the relatively closely spaced accelerometers located in the vertical array (clay core). The GA-NN proposed is capable of using “indeterminate” records and the sensors spatial configuration to describe the dimensionality of the system response. The data base used for neural training/testing stages is given in table 4.

![Figure 8. Finite element model](image-url)
The identified system is the specific dam element (geometry and materials), described by given intervals of soil lying between pairs of accelerometers. The recording stations used in the model as control points, are characterized by their position - \( \{ x, y, z \} \) coordinates - and a class condition: i) boundary situation or ii) dam response information (figure 10). First class is included as an excitation node and the second one illustrates the material behavior and location. The two mechanical soil properties estimated by this system identification process are the shear modulus (G) and the damping ratio (\( \lambda \)). These "equivalent" properties are based on the effective layer values between the sensors included in the neural training stage. The acceleration records and material properties predictions are calculated at discrete points that can be located between two sensors or in any zone of the earth element.

Following the system identification process described in the preceding section, a GA-NN nonparametric framework was obtained to map the input (left abutment recordings) to the output time series (accelerations data within the dam). In figure 11, the model and actual values for unseen events (EQ5, EW component) are shown. Remarkable neural capacity to characterize the time histories of earthquake motions and successful transmission of the movements through the dam core, z direction, is proved with these results. Figure 12 shows comparisons among recorded and computed response spectra along the dam core for the 31 May, 1990 seismic event. Agreement among them is very good from the practical standpoint.

A simple identification procedure (Zeghal, et al., 1995) can be used to estimate local shear stress and strain histories from array accelerations. These estimations can be used to locally calibrate models of the constitutive behavior of soil-systems. Properties evaluated from the GA-NN accelerations histories show a good agreement with those obtained by empirical correlations and laboratory studies. This approach is obviously not feasible in analyses of a multidimensional response, and more general local identification algorithm, which is presented below, is required.

For mapping coordinates to soil properties, a more sophisticated neural model (genetic tuning of the weights and function variables) was developed for describing materials dynamic behavior via G and \( \lambda \) curves. In this model the variables that affect the phenomena are included with a double purpose, i.e., as input/output...
parameters (figure 13). In a first step the input variables are the coordinates of the recording station and the outputs are the values of the dynamic properties. Once this process is completed, $G/\lambda$ nodes can be interchanged as premises and the coordinates take the role of conclusions to corroborate the adequate description of the soil masses. The forward-back training route, permits to find the parametric changes for optimal estimation of the shear stiffness and equivalent damping ratio, describing the physical soil system (continuous mass system) without trying to adjust the observed behavior to a simple equivalent system (lumped mass models, for example).

![Event: EQ-5, EW](image)

**Figure 11. NN model results: testing stage**

![Event: 31-May-1990](image)

**Figure 12. GA-NN results: evaluations through dam core (moving along vertical direction), unseen event**

Direction (I): paralleled to river axis  
Direction (II): paralleled to dam axis
permits an appropriate model selection.

response of the distributed parameter system. Such
provide essential direct information on the dynamic
recognition analyses using nonparametric identification
It has been demonstrated that SC tools for pattern
necessities (i.e., Finite Element Method, calibration
methodology) the neuronal structure can offer a general
evaluation for each material that integrates the dam.

As can be seen in figure 14, this neural model can offer
tremendous insight into the extremely complex
soil/rockfill system behavior. Based on the user/designer
necessities (i.e., Finite Element Method, calibration

material properties (shear modulus and
damping coefficient), linear or nonlinear material’s
behavior, canyon configuration, materials zonation (dam-
cross-section geometry), grain size, etc.

Considering the uncertainties in the estimation of
rockfill dam materials and the unknown characteristics of
future seismic events, it is indispensable to carry out a
relatively large number of analyses to account for these
uncertitudes when designing rockfill dams in earthquake-
prone zones. This can be done by means of finite element
method or any alternate mathematically-based procedure.
However the cost of this endeavor would be high and
extremely time-consuming. The neuro-genetic procedure
presented in this paper offers a reliable alternate via to
fulfill this task inexpensively and expeditiously.

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5 CONCLUSIONS

It has been demonstrated that SC tools for pattern
recognition analyses using nonparametric identification
provide essential direct information on the dynamic
response of the distributed parameter system. Such
information reduces the indeterminacy problem and
permits an appropriate model selection.
The advantageous characteristic of the neurogenetic
model proposed here for evaluating material behavior at
discrete points inside the dam structure can help to reveal

Figure 13. GA-NN model results: testing stage

Figure 14. Dynamic properties: GA-NN indirect estimations

The advantageous characteristic of the neurogenetic
model proposed here for evaluating material behavior at
discrete points inside the dam structure can help to reveal

the most influential aspects related with its seismic
responses: material properties (shear modulus and
damping coefficient), linear or nonlinear material’s
behavior, canyon configuration, materials zonation (dam-
cross-section geometry), grain size, etc.
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