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The Evolution of Geotechnical Earthquake Engineering Practice in North America: 1954-1994
(State of the Art Paper)

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SYNOPSIS

This paper traces the evolution of geotechnical earthquake engineering practice in North America from 1954 to 1994. The development of the state-of-the-art has been shaped strongly by four areas of practice: assessment of seismic hazard, estimation of liquefaction potential, seismic response analysis of earth structures and seismic safety evaluation of existing dams with potentially liquefiable zones. Evolution of practice in each of these areas will be traced and the current state-of-the-art evaluated. Present capabilities in practice will be illustrated by examples from the areas of seismic response of dams, liquefaction potential and seismic safety evaluation and remediation of potentially liquefiable embankment dams.

1 INTRODUCTION

The geotechnical problems associated with destructive earthquakes were identified by Housner (1954) as:

"(i) How is the character and intensity of the surface ground motion affected by the soil properties?

(ii) What will be the behaviour of the soil itself when subjected to rapid oscillatory stresses?

(iii) What effect will the properties of the soil have upon the behaviour of a structure that is interacting with the soil during an earthquake."

Housner's summary of the state-of-the-art indicates that there was general awareness that soft soils amplified ground motions and that the oscillatory stresses lead to consolidation of loose soils. It was understood that soil-structure interaction affected the natural periods of structures, resulted in energy loss into the ground with beneficial effects for the structure and that damaging differential settlements might occur. Structural engineers with an interest in seismic design were the ones primarily interested in these questions. Traditional geotechnical engineering had little to offer in the way of solutions and this remained the pattern until the mid-1960's. Housner (1954) summed up the situation in 1954 as follows; "Research on soil mechanics problems associated with earthquakes has not been as active as might be desired." Little changed between 1954 and 1964. The significant contributions again came from individuals with a good grasp of mechanics. Kanai (1957) developed a method for the dynamic analysis of elastic layered systems. Hatanaka (1952, 1955) studied the earthquake resistant properties of earth dams and investigated 3-D response.

Geotechnical earthquake engineering was in a rudimentary state in the early 1960's. The seismic stability of an embankment dam was evaluated using a conventional slope stability analysis in which an inertia force was assumed to act on any potentially sliding mass. 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 equivalent horizontal acceleration expressed in units of gravity. Values of K ranged from 0.05 to 0.15 but no scientific basis was provided for the selection of an appropriate K value. A gradual shift to more innovative approaches to seismic design became evident in the late 1950's. Clough and Pirtz (1958) conducted shake table tests on models of the 90 m high Kenny Dam in British Columbia to develop an appreciation of modes of failure and the pattern of deformations. This study also drew attention to the problems associated with retaining similitude in the testing of earth structures on shake tables.

Two major developments provided a focus for research in geotechnical earthquake engineering: the development of civilian nuclear power and the technical challenges raised by the widespread damage caused by liquefaction during the major earthquakes in Alaska and Japan in 1964. These two earthquakes focused the attention of the geotechnical engineering profession on the problem of seismically induced liquefaction and exposed the need for fundamental studies of how earth structures and foundations respond to earthquake loading.

The evolution of geotechnical earthquake engineering in North America, as it relates to the seismic safety evaluation of embankment dams, will be traced in broad outline from 1960 to 1993. This approach is adopted to allow a focused, coherent description. However, the techniques used in the seismic safety of dams are widely applicable for estimating ground motions, topographic and basin effects on ground motions, foundation response and seismic soil-structure interaction. This broad review is intended to provide a framework for understanding the evolution of practice to its current state.

A historical overview of developments will be presented first and then the state-of-the-art in the major areas of practice will be reviewed. These areas of practice are: seismic hazard evaluation; evaluation of dynamic soil properties; liquefaction potential; dynamic analysis; validation of constitutive models; and evaluation of post-liquefaction behaviour. The state-of-practice will be illustrated by a few important case histories which demonstrate the most recent developments.

2 HISTORICAL OVERVIEW

2.1 Analysis

Major contributions in analysis 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 viscoelastic material and treated the dams as one-dimensional (1-D) and two-dimensional (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 varies along the height of the dam. Ambroseys (1960c) studied the elastic response of dams in both wide and narrow rectangular valleys and showed that if the ratio of the width of the valley to the height of the dam was greater than 3, the seismic response changed significantly. This confirmed earlier results by Hatanaka (1952, 1955). Despite the limitations of the viscoelastic model of soil behaviour, this analysis captured many of the important characteristics of seismic response and provided the starting point for 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 seismic coefficients. They also drew attention to the deficiencies in the seismic coefficient method that should the materials in the dam lose strength during an earthquake.

Newmark (1965) clarified many aspects of the problem of seismic stability of slopes. He pointed out that, although the factor of safety in an equilibrium analysis incorporating the seismic coefficient might show a factor of safety less than 1, this need not imply that the performance of an embankment dam would be unsatisfactory or its stability compromised. The factor of safety was less than 1 only for short intervals during which the dam underwent some deformation. Newmark stressed that what counted was whether the deformations that the dam suffered during the earthquake were tolerable or not. Obviously, large deformations that resulted in loss of freeboard and extensive cracking of the dam were not acceptable. The level of tolerable deformation should be based on the particular characteristics of the dam under study, judgement of experienced dam designers, and an appreciation of the reliability with which the deformations can be estimated.

One of the more significant events which contributed to the rapid development of geotechnical earthquake engineering and the estimation of seismic displacement 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 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. All these analyses were conducted using a viscoelastic constitutive model of the soil and therefore were not capable of modelling the porewater pressure development or permanent deformations. To overcome this problem Finn (1967) outlined a procedure for interpreting the effects of the dynamic stresses computed by the viscoelastic analysis with the help of data on porewater 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 at Berkeley developed the equivalent linear method of analysis for approximating nonlinear behaviour. This method was incorporated in the 1-D shear wave propagation program SHARE (Schnabel et al., 1972). The technique was extended to 2-D finite element analysis by Idiss et al. (1973) and Lysmer et al. (1975) in the programs QUAD-4 and FLUSH, respectively. These programs took into account the strain dependence of damping and shear modulus. However, the analysis was still elastic-plastic, as a result, permanent deformations could not be estimated directly. Despite the limitation of elastic behaviour, these programs led to more realistic analyses of embankment dams under earthquake loading and have 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 liquefaction potential. The test also made possible estimations of seismically induced deformations from the strains developed in the samples. Later, it was discovered that the cyclic triaxial tests was first developed in China in 1956 by Wang Wengshao, but this was unknown in the West until the late 1970's (Finn, 1982).

During the 1960's, the resonant column test for measuring dynamic shear modulus and damping at low stress levels was developed and improved (Hagiwara, 1967; Hardin and Music, 1965; Hardin and Black, 1966, 1968; Drnevich, 1967; Hardin, 1970; Hardin and Drnevich, 1970). In the early 1970's, the use of the cyclic simple shear test was pioneered by Seed and Peacock (1971) and Finn et al. (1971). This test was particularly appropriate for modelling the excitation of level ground by shear waves propagating vertically and provided the basis for estimating settlements in sands (Seed and Peacock, 1971) and seismic porewater pressures (Martin et al., 1975).

By 1975, geotechnical engineers seemed to have many of the analytical and laboratory capabilities necessary for realistic assessments of the seismic safety and deformation behaviour of embankment dams. These methods were put to the test when Seed et al. (1973, 1975a, 1975b), undertook a comprehensive study of the liquefaction induced slide in the Lower San Fernando Dam which occurred as a result of the San Fernando earthquake of 1971. The analytical techniques to predict a stronger response than actually occurs. Because the strains in the dam were estimated from triaxial tests simulating the estimated loading, the stronger response leads to greater deformations during the earthquake. Another major factor resulting in very different strains in the triaxial test compared to those in the dam, is the radically different boundary conditions on soil elements in the test compared to the corresponding elements in the dam. The strains deduced from the triaxial tests are incompatible with conditions in the dam. The difference between predicted performance and field performance of the San Fernando dam provided the stimulus for the
development of both nonlinear and effective stress methods of dynamic analysis which could take nonlinear response and the effects of porewater pressures into account directly. The Martin-Finn-Seed (MFS) model for generating porewater pressures during earthquake loading based on the strain response of the soil was developed by Martin et al. (1975) and paved the way for dynamic effective stress analysis and the direct estimation of displacements.

The first nonlinear dynamic effective stress analysis based on the MFS porewater pressure model was developed by Finn et al. (1975, 1976) and was incorporated in the 1-D program DESRA-2 by Lee and Finn (1978). An updated version of the program, DESRA-2R, incorporating a more convenient form of the MFS porewater pressure model by Byrne (1991a), a modified nonlinear stress-strain law incorporating yield and a joint element for simulating water accumulation beneath impermeable layers was developed by Finn and Yoshida (1993). 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 has the capability to conduct both static and dynamic analysis under total stress or effective stress conditions and can compute stresses and strains directly. The program uses properties that are normally measured in connection with important engineering projects.

Since the mid 1980's, other nonlinear effective stress programs have been developed, for the most part based on some version of plasticity theory and Biot's consolidation equation. These programs are mathematically and analytically quite powerful but use some properties which are not routinely measured in the laboratory or the field. Detailed presentations of some of these programs may be found in Pande and Zienkiewicz (1982) and comprehensive critical reviews in Finn (1988a,b) and Marcuson et al. (1992). A number of these models were recently used in numerical predictions of centrifuge earthquake induced experiments (Arulanandan and Scott, 1993). The evaluation of these experiments is still under way and is too early to draw conclusions about the relative merits of the various models.

The estimation of post liquefaction deformations is an important part of assessing the consequences of liquefaction in embankment dams. Finn and Yogendrakumar (1991) developed the program TARA-3PL to track large post-liquefaction deformations using an updated Lagrangian technique for coping with the large strains and deformations.

2.2 In-Situ Testing

Since 1975 there has been a steady development of in situ tests in an attempt to obtain soil properties without the disturbance to the soil structure occasioned by drilling, sampling, transportation and testing. In engineering practice today there is heavy reliance on data from in situ tests.

The Standard Penetration Test (SPT) is widely used in site investigations to determine stratigraphy and to recover samples for soil identification purposes. The standardized blowcounts per 30 cm, \( N_1 \), have been correlated with shear modulus, shear wave velocity, strength parameters, and relative density. However, these correlations generally exhibit wide scatter. Sykora and Koester (1988) have compared relationships proposed by various researchers for the estimation of shear wave velocity. The correlations are adequate for \( N_1 \leq 25 \), but the scatter increases as \( N_1 > 150 \).

The SPT is the primary tool for assessing liquefaction potential in sands and silts (Seed, 1979b; Seed et al., 1986). In gravelly and coarse soils, the Becker Penetration Test (BPT) is used (Harder and Seed, 1986). The determination of liquefaction potential is reviewed in Section 5.

One of the more versatile in situ tests is the Cone Penetration Test (CPT). The cone can be equipped to measure bearing resistance, skin friction, porewater pressure, shear wave velocity and conductivity. These parameters provide soil identification, measures of shear modulus through shear wave velocity and indirect measures of moduli and strength parameters through well documented correlations. The CPT is also used to evaluate liquefaction potential and its use in this area will increase as the database of case histories expands.

Downhole and crosshole methods have been used for many years to measure shear wave velocity and these methods can be used in situations in which it is not feasible to push a cone.

In coarse gravels or rock fill, the Spectral Analysis of Surface Waves (SASW) which does not require a borehole is a useful alternative (Nazarian and Stokoe, 1984). Recent developments in the method have reduced costs and accelerated the procedure so that it has become a viable tool for routine use in practice (Robertson et al., 1992). A database is accumulating linking shear wave velocity and liquefaction potential, and as a result, shear wave velocity is beginning to be used as an index of liquefaction.

3 SEISMIC HAZARD EVALUATION

The seismic demand on an embankment dam is established by an appropriate seismic hazard analysis. The first requirement of a hazard analysis is that the procedures followed should satisfy the regulations of the agency with the basis or safety level earthquake, DBE or SLE. Design is usually controlled by the SLE. Design has been based often on the Maximum Credible Earthquake which is defined as the maximum earthquake that the seismogenic source may be capable of producing, usually without explicit regard for the probability of occurrence. This method of hazard analysis is deterministic. There is a gradual shift from the MCE to what is called in California a Safety Evaluation Earthquake, SEE. In selecting this earthquake, consideration is given both to the probability of occurrence and the consequences of failure in addition to the factors usually taken into account in determining the MCE. There is limited experience with this procedure. At present in California the procedure tends to consideration of return periods between

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10,000 and 100,000 years (Bolt, 1994). These levels of probability have been used by the US Bureau of Reclamation to establish magnitudes and distances for design events in random seismogenic areas for safety evaluation of dams. Bolt has noted that such procedures have led to design earthquakes in California somewhat smaller than the MCE (Bolt, 1994).

For most dams the SEE ground motions are selected such that the annual probability of exceedance is about 1 in 10,000. There have been notable exceptions. The probability of exceedance of the design ground motions at the High Aswan Dam have been estimated at 1 in 500,000 (Cluff and Cluff, 1990).

Procedures for conducting probabilistic seismic hazard analysis have been presented by the Earthquake Engineering Research Institute Committee on Seismic Risk (EERI, 1989). The basic elements of a seismic hazard study are the delineation of seismogenic sources, both faults and aerial sources and establishing the occurrence rates for earthquakes of various magnitudes and the maximum earthquake for each source. The scale of effort required to establish these parameters depends on the perceived threats from failure, the demands of the regulatory authority, the process of public scrutiny, and the historical and instrumental data available. Temporary instruments may be deployed to record current seismic activity which will help in identifying active faults and their characteristics.

Geological investigations including aerial reconnaissance of geological structures, low sun angle photography, evaluation of surface characteristics of faults, trenching across faults and dating of geological materials to establish earthquake recurrence rates are usually necessary to establish reliably the seismic hazard for important projects.

The conduct of these studies is beyond the competence of design engineers but they must be able to understand the process by which the conclusions of the seismological consultants are reached. This is essential to guard against excessive conservatism being introduced at the early stage to be compounded with other conservative estimates at later stages in the design. The designer should be given best estimates of seismic parameters and measures of their uncertainty rather than extreme values. The appropriate design criteria should evolve from a joint consideration of the seismic environment, the probable dynamic characteristics of the dam and the consequences of failure. Design motions should be selected from the patterns of seismic ground motions developed by the seismologists after due consideration of the design criteria.

A major consideration in seismic hazard assessment is the nature of the seismic environment. There are four major environments to be considered: plate margins such as the San Andreas Fault System; subduction zones such as the coasts of Chile, Japan and the Pacific Northwest in the USA and Canada; intraplate regions such as eastern North America and collision environments such as the Anatolia Fault System in Turkey and the Himalayan region in India.

The plate margin environments are best understood both in terms of earthquake occurrence and attenuation of motions with distance from the fault. Knowledge of subduction events has increased since the major earthquakes in Chile and Mexico in 1985. However, it is still uncertain to what extent experience is transferable from one subduction zone to another. For instance in the evaluation of seismic safety of dams in the Pacific Northwest, there is considerable uncertainty regarding the appropriate attenuation law to be used since no major deep subduction earthquake has occurred there in historic times. There are major problems with seismic design in intraplate regions because of the infrequent occurrence of large earthquakes and the almost total lack of strong motion data. Source and path modelling are now being used to define ground motions in these areas (Boore and Atkinson, 1987; McGuire et al., 1988).

Collision environments are among the most active seismic sources and have produced several earthquakes in the magnitude M = 8+ range. Yet there are few strong motion data for these regions and hence no well constrained attenuation laws.

The seismic hazard at a site is usually specified by peak ground acceleration, response spectra corresponding to 1%, 2%, 5%, 10% and 84% levels of ground motion and of representative ground motions. Appropriate spectral values may be established by spectral attenuation laws (Joyner and Boore, 1980; Idriss, 1993; Campbell, 1985, 1993). Design response spectra may also be determined from a statistical analysis of response spectra derived from a selection of representative ground motions. There is a shift towards using uniform hazard response spectra established by probabilistic analyses following the procedures outlined in EERI (1989). Synthetic motions with spectra that match a target design spectrum in some average sense may be determined using computer programs such as RASCAL (Silva and Lee, 1987). To achieve time histories consistent with field observations, a phase spectrum from a field record of approximately the same magnitude and distance as the design earthquake should be incorporated. The phase spectrum controls the distribution of energy in time and models path effects and surface wave contributions. This combination of response spectra with selection or generation of synthetic motions evaluating seismic response is a major evolution from the procedures of the 1960's based either on regulatory prescriptions or limited ground motion studies.

The seismic safety evaluation of the High Aswan Dam (Cluff and Cluff, 1990) is an important example that illustrates most of the techniques described above.

4 EVALUATION OF DYNAMIC SOIL PROPERTIES

The cyclic triaxial test has been the basic laboratory test for exploring the response of soils to cyclic loading because it allows the testing of both reconstituted and undisturbed samples including frozen samples. Other tests such as the simple shear test and the hollow torsional shear test cannot be used for testing high quality undisturbed samples.

Laboratory tests play a major role in spanning the strain range between the low strain response that can be characterized by geophysical methods to large strain response characteristic of strong
shaking. Laboratory tests are also necessary for exploring the soil response to the variety of stress paths encountered in practice from the rocking of foundations to the horizontal shearing typical of horizontal shear waves propagating vertically.

It would be highly desirable to measure the response of soils to cyclic loading in-situ. The self-boring pressuremeter holds promise of permitting this but the accuracy of measurements need to be improved and more comprehensive interpretations of the data need to be developed (Martin, 1992).

Techniques for measuring soil properties in the laboratory and the field are at a mature state of technology and no development is anticipated in the next five years which would lead to a major improvement in the quality of the data from these tests except perhaps in the case of the self-boring pressuremeter.

What is most lacking at present is documentation of how representative moduli, damping and strength parameters predicted by laboratory and in situ arrays and instrumenting earth structures at high seismic hazard sites. What can be achieved by these arrays is exemplified by the data coming from the downhole strong motion array at Lotung in Taiwan (Chang et al., 1991). An example is given in Fig. 1 in which back-calculated shear moduli are compared with those developed initially by laboratory testing to characterize the site. The shear moduli deduced from the response data are clearly more strain dependent than the data from the resonant column tests would suggest. The cyclic shear test data at the larger strains compare very well with the field data but this may just reflect a decreasing sensitivity of the moduli to strain at the larger strains.

Data from the Turkey Flat seismic array in California for small earthquakes M < 4 showed that low-strain damping was much less than that predicted by laboratory tests.

These examples show the need for well instrumented sites and earth structures in high seismic hazard areas to provide the data for verifying and calibrating procedures for characterizing earth structures for seismic response analysis. These instrumented structures will also provide the data base to validate procedures for dynamic analyses.

One of the most convenient ways of determining the low strain shear modulus is to use the seismic wave cone to measure the shear wave velocity. This is usually the most economical procedure, since the cone may also be used to define the stratigraphy of the embankment and foundation of the dam. The shear wave velocity measurements can be made during pauses to install additional rods for further penetration, typically at every metre of penetration. Downhole and crosshole methods are used extensively to measure shear wave velocities. However, these methods require boreholes and therefore are more time consuming and expensive and generally require greater technical competence to yield reliable results. In very coarse gravels or rockfill, the spectral analysis of surface waves method (SASW) which does not require a borehole, is a viable alternative.

The low strain shear modulus can be determined using the resonant column test when good undisturbed samples can be taken. The strain dependence of the shear modulus must be determined by laboratory tests either on good high quality undisturbed samples when possible or on samples reconstituted as closely as possible to field conditions. In recent years, on projects where reliable safety assessment was paramount, samples have been cored from frozen ground to determine properties for the verification of estimates of moduli, strength parameters, and liquefaction potential obtained from correlations with penetration test data. A major study of the technology for recovering and testing frozen samples is planned for the Canadian Geotechnical Test Site in Alberta. The study is also expected to provide data on the quality of "undisturbed" samples of cohesionless soils recovered by the best conventional techniques.

One of the most difficult problems in practice is the determination of the residual strength of liquefied soils for use in post-liquefaction stability analysis. This topic is discussed in the next section dealing with liquefaction potential.

It is very important to obtain site specific properties for dynamic analysis. Although generally the low strain moduli are obtained from shear wave velocities measured directly at the site, there is a tendency to use generic data on the variation of modulus with strain in the equivalent linear methods of analysis. Critical analysis of the behaviour of soils during earthquakes has shown that the use of generic data can lead to very misleading conclusions. A classic example is the response of the Mexico City clays to earthquake loading. Laboratory tests show that the strain dependence of the shear modulus of Mexico City clay is unusual. The clay remains elastic over a much larger strain range than most clays. Subsequent studies by Sun et al. (1988) confirm that for clay soils the variation

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**Fig. 1.** Comparison of back calculated shear moduli and data from laboratory tests (after Chang et al., 1991).
of modulus with strain is a very strong function of the plasticity of the clay. Data on other soils also show the need to develop site specific curves showing the strain dependence of shear moduli (Gohl, 1988).

The damping of soils, especially under strong shaking, is measured by cyclic loading tests in the laboratory. The damping is a function of the area under the hysteresis loop which, in turn, is a function of strain level. The damping is usually associated with stable loops established after a certain number of cycles. Because the determination of the damping can be time consuming, generic data curves are often used. For strong shaking of important structures site specific damping curves should be developed for equivalent linear analysis.

5 LIQUEFACTION POTENTIAL

5.1 Liquefaction Potential in Sands

Seed's liquefaction assessment chart (Fig. 2) is the primary tool in current practice for determining the liquefaction potential of saturated, cohesionless soils (Seed, 1979b; Seed et al., 1986). The curves in Fig. 2 define critical conditions that are lower bounds separating sites which liquefied during past earthquakes from those which did not. The soil conditions are defined by the normalized standard penetration resistance \( N_l \) and the effects of the earthquake by the equivalent cyclic stress ratio \( \tau_e / \sigma'_0 \) where \( \tau_e \) is the equivalent uniform shear stress mobilized by the earthquake shaking and \( \sigma'_0 \) is the vertical effective confining pressure. Procedures for determining \( (N_l)_{60} \) and \( \tau_e / \sigma'_0 \) have been presented by Seed and Idriss 1971, Seed et al. (1986) and Seed and Harder (1990). The curves in Fig. 2 apply to earthquakes of magnitude \( M = 7.5 \). Critical stress ratios for other earthquake magnitudes can be generated by using the scaling factors in Table 1 (Seed et al., 1986; Seed and Harder, 1990).

<table>
<thead>
<tr>
<th>Earthquake Magnitude</th>
<th>Number of Equivalent Cycles</th>
<th>Duration Scaling Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.5</td>
<td>26</td>
<td>0.89</td>
</tr>
<tr>
<td>7.5</td>
<td>15</td>
<td>1.0</td>
</tr>
<tr>
<td>6.75</td>
<td>10</td>
<td>1.13</td>
</tr>
<tr>
<td>6</td>
<td>9</td>
<td>1.92</td>
</tr>
<tr>
<td>5.25</td>
<td>2-9</td>
<td>1.5</td>
</tr>
</tbody>
</table>

The liquefaction resistance depends also on fines content and the measured \( (N_l)_{60} \) values for different fines content can be corrected to equivalent clean sand values with the aid of the curves for different fines content in Fig. 2.

There are two important limitations associated with Fig. 2. The field data correspond to level ground conditions with no initial static shear stresses on horizontal planes and to effective overburden pressures less than 150 kPa. Seed (1983) outlined procedures for making corrections when these conditions are violated.

The first estimate of the liquefaction resistance of a soil element in a dam or slope is determined using the in situ \( (N_l)_{60} \) penetration resistance and the appropriate curve for critical conditions in Seed's liquefaction assessment chart in Fig. 2. This resistance must then be corrected for deviations from the standard conditions of the database underlying the chart.

It is generally recognized that the rate of increase of resistance to liquefaction decreases with increase in effective confining stress. Liquefaction resistance derived from Seed's liquefaction resistance chart (Seed et al., 1986) has been normalized to an effective overburden pressure of 100 kPa. The liquefaction resistance at higher confining pressures is derived by multiplying the resistance at 100 kPa by a factor \( K_p \). Seed and Harder (1990) have plotted \( K_p \) for many sand types against confining pressures up to 800 kPa and suggested a general conservative curve for use in practice (Fig. 3).

Vaid and Thomas (1994) and Thomas (1992) report detailed studies on the effects of confining pressure on the liquefaction resistance of Fraser River sand. Their study confirms in a general way, the findings of Seed and Harder (1990), but they demonstrate clearly that \( K_p \) is a function of relative density. Their results are shown in Fig. 4. At 59% relative density and a confining pressure of 1200 kPa, the value of \( K_p \) is about 0.75, compared with a value of 0.4 for the Seed and Harder (1992) curve at a pressure of only 800 kPa. These results strongly suggest that job specific values of \( K_p \) should be determined. In

![Liquefaction Assessment Chart (after Seed et al., 1986).](attachment:image.png)
Conservative estimates of field liquefaction resistance curves may be obtained by entering Fig. 2 and determining the critical stress ratios for the different earthquake magnitudes corresponding to the (N1)60 of the soil, using the scaling factors in Table 1. These stress ratios may then be plotted against the equivalent number of cycles of significant motion associated with each magnitude to give the liquefaction resistance curve. The equivalent number of cycles are obtained also from Table 1.

The critical SPT values as defined by Seed et al. (1986) are supported by the largest database and hence must be considered the most reliable. Although the CPT is considered to be a more reliable test, the direct database supporting critical values of the penetration resistance, Qc, is still inadequate.

The shear wave velocity has also been used as an index of liquefaction potential (Bierschawle and Stokoe, 1984; Tokimatsu et al., 1991; Finn et al., 1990). Robertson et al. (1992) proposed that the shear wave velocity normalized to a standard pressure of 100 kPa, Vsl', be used as an index of liquefaction potential as shown in Fig. 6.

The attractive aspect of the shear wave velocity procedure is that it can be used in materials which are difficult to sample such as loose clean sands or difficult to penetrate, such as gravels and boulders. In the latter situations, the spectral analysis of surface waves (SASW), is used to determine the distribution of velocity with depth (Nazarian and Stokoe, 1984; Addo and Robertson, 1992).

5.2 Liquefaction Potential in Gravels

The presence of large gravel and cobble particles preclude the use of the SPT in the evaluation of liquefaction potential. Misleading high SPT blowcounts would be recorded as the 5 cm sampler bounced off the large particles. A much larger and heavier penetration tool is required to penetrate gravels and provide a continuous record of penetration resistance. Harder and Seed (1986)
developed the Becker Penetration Test (BPT) for this situation. SASW, the spectral analysis of surface waves, has emerged as a potential alternative to the BPT but still lacks an adequate direct correlation with liquefaction potential (Nazarian and Stokoe, 1984; Stokoe et al., 1990; Addo and Robertson, 1992).

5.2.1 Becker Penetration Test (BPT)

In the BPT a double-walled casing is driven into the ground with a diesel pile hammer (Fig. 7). The casing comes in three sizes: 140 mm O.D. - 83 mm I.D.; 170 mm O.D. - 110 mm I.D. and 230 mm O.D. - 150 mm I.D. The casing can be driven with an open bit and reversed air circulation to obtain disturbed samples or with a plugged bit and driven as a solid penetrometer. The blows required for each foot of penetration are recorded and provide a record of penetration resistance for the entire profile. The test is standardized by reducing the blow counts to constant standard combustion conditions, following procedures established by Harder and Seed (1986). Two types of rigs are in use, the HAV-180 rig and the AP-1000 developed later in the 1970's. Both rigs use the same model of diesel hammer, an ICE Model 180. Harder and Seed (1986) showed that the older HAV-180 rig was more efficient with the result that the blow counts from AP-1000 were 50% higher than those from the HAV-180 rig. Harder and Seed (1986) developed the BPT procedures using the AP-1000 so blowcounts obtained by the HAV-180 rig must be adjusted to equivalent AP-1000 blowcounts unless correlations are developed specifically for the HAV-180 rig.

There is no direct correlation between BPT blowcounts and liquefaction potential. Instead the standardized BPT blowcounts are converted to equivalent \( (N_{100}) \) values using the correlation developed by Harder and Seed (1986) or a site specific correlation. Liquefaction potential is then determined from the \( (N_{100}) \) values using the liquefaction assessment chart in Fig. 2.

The BPT procedure has been used to evaluate the liquefaction potential of foundation gravels in the Mormon Island Dam near Sacramento, California. An excellent description of the use of the BPT on this project may be found in Wahl et al. (1988).

A critical assessment of the reliability of the BPT procedure for liquefaction in assessment is not possible because of the lack of field data directly relating the BPT blowcounts to site performance during earthquakes. However, increasing use in practice has revealed a number of problems with the BPT. Harder and Seed (1986) found that even for standard combustion conditions the blowcount using the AP-1000 drill were 1.5 times the blowcounts from the HAV-180. More disturbing Harder (1993) found substantial differences in blowcounts, up to 30%, between two AP-1000 rigs. Clearly constant combustion conditions are not enough to ensure the transmission of the same driving energy into the Becker casing.

Another problem is related to friction on the Becker casing. Since the BPT is a continuous
driving type of penetration, it accumulates increasing frictional resistance with increased depth of penetration. The increasing impact of friction can invalidate correlations based on shallower depths of penetration. This can be serious for two reasons, it may lead to underestimation of the potential for liquefaction by giving misleadingly large penetration resistances and to overestimation of the effectiveness of remedial measures such as densification.

A solution to the standardization problem has been proposed by Sy (1993). He measured strains and accelerations in the top of the Becker casing in a number of boreholes and calculated the energy transferred to the casing by the diesel hammer from force-velocity data. Sy found that about 30% of the rated energy of the hammer was transferred to the casing and proposed this energy level as a standard. Comparison of equivalent SPT blowcounts derived using this standard and the Harder-Seed correlation showed that the Harder-Seed correlation corresponds to a much higher but unspecified energy level. The Sy standardization criterion gave similar Becker blowcounts for different rigs (Fig. 8).

Sy also advocates the use of the pile driving analyser to estimate the friction between the Becker casing and the soil.

5.2.2 Residual Strength

The controversy surrounding residual strength makes it difficult to assess reliably the consequences of liquefaction. There are two methods generally recognized for determining the residual strength. The first due to Poulos et al. (1985) consists of testing good undisturbed samples in static triaxial tests and correcting the results for the disturbance effects of sampling, transportation and testing. Residual strengths derived in this way appear to be too high compared to those derived from the back analysis of past flow slides. On the basis of such analyses Seed (1987) developed a correlation between \((N_{1})_{60}\) and the undrained residual strength, \(S_{ur}\). An updated version of this correlation was proposed by Seed and Harder (1990) which is shown in Fig. 9. Lower bound strengths from this correlation are very often used in practice although occasionally values approaching the 33rd percentiles have been used. In either case these values are small at low \((N_{1})_{60}\) values and frequently result in the prediction of instability and the need for substantial remediation.

An alternative approach which has recently come into practice consists of relating the residual strength to the vertical effective overburden pressure, \(p'\), so that \(S_{ur} = Cp'\). In the safety evaluation of Sardis Dam, Mississippi, Finn et al. (1991) used \(C = 0.075\). This value gave residual strengths substantially higher than the lower bound values from the Seed correlation at low \((N_{1})_{60}\) values. A re-examination of Seed's case histories by Lo et al. (1991), and McLeod et al. (1991) suggests that \(C\) may have a value of about 0.1. Since then values of \(C\) ranging from 0.06 to 0.08 have been used in other dams.

The wide divergence in estimates of the residual strength could be resolved by a focused research program. Part of the answer may lie in the effects of stress paths (Vaid et al., 1989; Finn,
The work by Vaid et al. has shown factors of 6 to 10 between residual strength measured in compression and extension testing modes. On a curved failure surface the stress conditions vary roughly over this range so that using compression values, as has been the custom, may lead to an overestimation of the average mobilised residual strength. Vaid and Thomas (1993) have shown that the residual strength in extension is a function not only of void ratio but of the confining pressure (Fig. 10). In a test sequence on Fraser River sands they showed that irrespective of void ratio the ratio of residual strength to initial effective overburden pressure is about 0.15 on the average. Baziar and Dobry (1991) report a ratio of 0.12 for loose silty sand from undrained static compression tests. These test data support the alternative approach discussed above.

Fig. 10. Effects of overburden pressure on residual strength (Vaid and Thomas, 1993).

Another factor that confuses the issue of residual strength determination is the method of sample preparation used in fundamental research studies. Some researchers (Poulos et al., 1985) have used moist tamped samples which might be considered representative of compacted fills. Vaid et al. (1989) and Vaid and Thomas (1993) used water pluviated samples which would be more representative of natural deposition and placement by hydraulic fill methods. There is grave need for a research program to resolve these controversial issues surrounding the evaluation of residual strength not least because estimates of residual strength have a major impact on the costs of remediation.

6 EMPIRICAL AND SEMI-EMPIRICAL METHODS FOR ESTIMATION OF DEFORMATIONS

6.1 The Newmark Method

Newmark (1965) developed and published a method based on a sliding block analogy for estimating earthquake induced relative displacements. It is interesting to note that on a project for the U.S. Army Corps of Engineers, the late D.W. Taylor (letters from D.W. Taylor, 1953) appears to have independently developed a similar model for a similar purpose. The Taylor model was first implemented by R.V. Whitman at M.I.T. in 1953. A remarkable sentence is taken from the 20 May letter; "The procedure therefore cannot be expected to have much validity if, as in the writer's opinion, the threat of damage from, earthquake action lies not in an increase of activative force but in a progressive decrease in shearing resistance as a result of many cycles of application of the activating force."

Deformation of a dam is modelled as the displacement of a rigid block sliding on an assumed failure surface under the action of the ground motions at the site. Various potential sliding surfaces in the embankment are analyzed statically to find the inertia force \( F_I = (W/g)a_y \) required to cause failure (Fig. 11). The average yield acceleration \( a_y \) is then deduced from this force. The sliding block is assumed to have the same acceleration time history as the ground. The yield acceleration is deducted from the acceleration time history, and the net acceleration (the shaded area in Fig. 11) is available to generate permanent displacements. The analysis is conducted on the equivalent model of a horizontal sliding block on a plane with only one-way motions allowed (Fig. 11).

Fig. 11. Elements of Newmark's deformation analysis.

Makdisi and Seed (1978) modified the Newmark method by taking the flexibility of the dam into account. The average acceleration time record of the sliding block is obtained usually from a QUAD-4 analysis. The method differs from the Newmark approach in generating relative displacements by the net accelerations above the sliding surface, whereas Newmark used the net accelerations below the sliding surface. The QUAD-4 accelerations in the sliding block are determined without taking the yield accelerations into account. Therefore, in many cases of strong motions, estimates of displacements would probably be conservative.

Byrne (1991b, 1992) modified the Newmark approach by deriving an "equivalent" seismic coefficient which allows the pattern of displacements to be estimated by a 2-D finite element analysis.
The Newmark method was introduced at a time when there were no direct methods of computing permanent deformations. It is still widely used despite all the evidence that the sliding block model is not a very good representation of how embankment dams deform, especially embankment dams with low factors of safety. This was shown as early as 1958 by Clough and Pirtz (1958) in their shake table tests on models of Kenny Dam. The method is useful in comparing the deformation potential of alternative design proposals, or in comparing a design with that of an existing dam. However, in many cases, similar results to those of the Newmark method can be achieved by using Maksidi and Seed's (1978) simplified approach. Using current technology and finite element analysis, permanent deformations can be calculated directly without restrictive assumptions about the mode of deformation.

7 DYNAMIC ANALYSIS OF SOIL STRUCTURES: OVERVIEW

The state of the art of earthquake analysis procedures for concrete and embankment dams was summarized in Bulletin 52 of the International Commission on Large Dams (Zienkiewicz et al., 1986). The Bulletin outlined a general framework for analysis in both total and effective stress modes applicable to embankment dams using equations which coupled the response of soil and water. It recommended three levels of analyses:

1) Simple total stress methods including pseudostatic analysis using seismic coefficients when porewater pressures are negligible and no significant degradation in soil properties occurs.

2) The equivalent linear method of analysis coupled with the use of laboratory data (Seed et al., 1973; Seed, 1979) when substantial porewater pressures are generated.

3) Effective stress analysis conducted in "a direct and fundamental manner."

Pseudostatic analysis with seismic coefficients might be used safely in areas where a long history of use has calibrated the seismic coefficients to reflect experience with dam behaviour during earthquakes, such as Japan. It is not recommended where such direct experience is not available. The equivalent linear method is still the most widely used in practice, but "direct and fundamental" methods are finding increasing application. This is especially true in dealing with the complex problems that must be faced when evaluating the safety of existing dams which contain potentially liquefiable soils.

7.1 Equivalent Linear Analysis

The dynamic response of an earth dam is usually computed in engineering practice using an equivalent linear (EQL) method of 2-D analysis such as that incorporated in the computer programs QUAD-4 (Idriss et al., 1973) or FLUSH (Lysmer et al., 1975). The results may be corrected approximately for three dimensional (3-D) effects (Mejia and Seed, 1983). These corrections were used in the back analyses of Oroville Dam for the 1975 earthquake (Vrymoed, 1975). The correction is based on altering the shear modulus in the 2-D analysis so that the fundamental 2-D period matches the equivalent 3-D period.

Dakoulas and Gazetas (1986a,b) studied the problem again and Gazetas (1985) points out that, despite matching the fundamental period, the contributions of higher harmonics may be substantially underestimated. Therefore assessing the seismic response of embankment dams in narrow valleys requires the exercise of engineering judgement, since the higher harmonics are likely to have their greatest effect at the crest of the dam.

The EQL analyses are conducted in terms of total stresses and the effects of seismically induced porewater pressures on elemental shear stiffness are not reflected in the computed strains, stresses, and accelerations. Since the analyses are elastic, they cannot predict the permanent deformations. Therefore, equivalent linear methods are used only to get the distribution of maximum accelerations and maximum shear stresses in the dam. Semi-empirical methods are often used to estimate the permanent deformations using either the acceleration or stress data from the equivalent linear analyses.

7.1.1 Deformations from Acceleration Data

Deformations are often estimated from the acceleration data using the Newmark method as modified by Maksidi and Seed (1978). The resulting deformations do not represent the deformation patterns of embankment dams under strong shaking, but they may provide a useful index of potential deformation. If a sliding wedge can be found which undergoes large deformations, one would expect to estimate large deformations by an appropriate nonlinear finite element analysis. However, the deformations computed by the Newmark approach should not be used for estimating whether the seismic deformations will satisfy displacement criteria.

7.1.2 Deformations from Stress Data

A more detailed picture of potential strains and deformations is obtained using Seed's semi-empirical method (Seed et al., 1973a). The computed dynamic stresses in soil elements in the dam are converted to equivalent uniform stress cycles and are applied to laboratory specimens in consolidated states similar to corresponding elements in the dam. The resulting strains in the laboratory specimens are assigned to the corresponding elements in the dam. This procedure gives an incompatible set of strains which, are an indication of the potential for straining at selected locations within the dam.

These procedures were used to investigate the slide in the Lower San Fernando Dam during the 1971 earthquake (Seed et al., 1973, 1975a,b). Large upstream displacements were predicted to occur during the earthquake. In fact, the failure occurred under static loading conditions shortly after the earthquake shaking had ceased. A major motivation for the development of more general constitutive relations has been the need to model nonlinear behaviour in terms of effective stresses and to provide reliable estimates of porewater pressures and permanent deformations under seismic loading.

7.2 Nonlinear Methods of Analysis

A hierarchy of constitutive models is available for the direct and fundamental analysis of the dynamic response of embankment dams to earthquake
loading. The models range from the relatively simple equivalent linear model to complex elastokinematic hardening plasticity models. Detailed critical assessments of these models may be found in Finn (1988a,b) and Marcuson et al. (1992). This review presents only the main procedures used in current practice and outlines their advantages and limitations.

7.2.1 Elastic-Plastic Methods

It is generally recognized that elastic-plastic models of soil behaviour under cyclic loading should be based on a kinematic hardening theory of plasticity using either multi-yield surfaces or a boundary surface theory with a hardening law giving the evolution of the plastic modulus. These constitutive models are complex and incorporate some parameters not usually measured in field or laboratory testing. Soil is treated as a two-phase material using coupled equations for the soil and water phases. The coupled equations and the more complex constitutive models make heavy demands on computing time (Finn, 1988b).

Validation studies of the elastic-plastic models suggest that, despite their theoretical generality, the quality of response predictions is strongly path dependent (Sadao and Bianchini, 1987; Finn, 1988b). When loading paths are similar to the stress paths used in calibrating the models, the predictions may be good. As the loading path deviates from the calibration path, the prediction becomes less reliable. In particular, the usual method of calibrating these models, using data from static triaxial compression and extension tests, does not seem adequate to ensure reliable estimates of response for the dynamic cyclic shear loading paths that are important in many kinds of seismic response studies. It is recommended that calibration studies of elastic-plastic models for dynamic response analysis should include appropriate cyclic loading tests, such as triaxial, torsional shear, or simple shear tests. The accuracy of pore pressure prediction in the coupled models is highly dependent on the accurate characterization of the soil properties. It is difficult to characterize the volume change characteristics of loose and saturated soils which control porewater pressure development because of the problems of obtaining and testing undisturbed samples representative of the field conditions. As a check on the capability of these models to predict porewater pressure adequately, it is advisable to use them to predict the field liquefaction resistance curve as derived from normalized Standard Penetration test data (Seed et al., 1986).

Typical elastic-plastic methods used in current engineering practice to evaluate the seismic response of embankment dams are DYNAFLOW (Prevost, 1981), DIANA (Kawai, 1985), DSAGE (Roth, 1985), DYNA (Moriwaki et al., 1988), FLAC (Cundall and Board, 1988), DYSAC2 (Muraleetharan et al., 1989,1991) and SWANDYNE (Zienkiewicz et al., 1990a,1990b). There is no published information on the current version of DIANA which is an extensive modification of the earlier program. Programs DSAGE, DYNA, and FLAC are proprietary to their developers.

The constitutive model of DYNA-FLOW is based on the concept of multi-yield surface plasticity. The initial load and unload (skeleton) stress-strain curve obtained from laboratory test data is approximated by linear segments and the curves for loading, unloading and reloading follow the Masing criteria (Masing, 1926). The procedure can include anisotropy. The program allows dissipation and redistribution of porewater pressures during shaking. Validation of the program has been by data from centrifuge tests. The computational requirements of the code are quite intensive.

DSAGE is predecessor of the program FLAC. The latter is a microcomputer implemented code based on the explicit finite difference method for modeling nonlinear, static and dynamic problems. The program uses an updated Lagrangian procedure for coping with large deformations.

DYNA, DYSAC2, SWANDYNE 4 is a general purpose elastic-plastic computer code which permits a unified treatment of such problems as the static and dynamic nonlinear drained and undrained response analyses of saturated and partially saturated soils to earthquake loading. The formulations and solution procedures, upon which the computer code is based, are presented in Zienkiewicz et al. (1990a,1990b).

7.2.2 Direct Nonlinear Analysis

The direct nonlinear approach is based on direct modelling of the soil nonlinear hysteretic stress-strain response. The WES has been working with the direct nonlinear dynamic effective stress analysis methods of Finn for more than ten years. This approach is represented here by the program TARA-3 (Finn et al., 1986), which is proprietary to Finn.

WES has extensive experience using this method in practice and a number of field studies are available. Some of these studies are used in the remainder of this paper to simply illustrate the use of dynamic effective stress and seismic deformation analyses in evaluating and/or remediating the seismic safety of embankment dams.

The objective during analysis is to follow the stress-strain curve of the soil in shear during both loading and unloading. Checks are built into the TARA-3 program to determine whether or not a calculated stress-strain point is on the stress-strain curve and corrective forces are applied to bring the point back on the curve if necessary. To simplify the computations the stress-strain curve is assumed to be hyperbolic. This curve is defined by two parameters which are fundamental soil properties, the strength $\tau_{\text{max}}$ and the in situ
small strain shear modulus, $G_{\text{max}}$. The response of the soil to uniform all round pressure is assumed to be nonlinearly elastic and dependent on the mean normal effective stress.

The response of the soil to an increment in load, either static or dynamic, is controlled by the tangent shear and tangent bulk moduli appropriate to the current stress-strain state of the soil. The moduli are functions of the level of effective stress, and therefore, excess porewater pressures must be continually updated during analysis and their effects on the moduli taken progressively into account.

During seismic shaking, two kinds of porewater pressures are generated in saturated soils, transient and residual. The residual porewater pressures are due to plastic deformations in the sand skeleton. These persist until dissipated by drainage or diffusion and therefore they exert a major influence on the strength and stiffness of the soil skeleton. These pressures are modelled in TARA-3 using the MFS porewater pressure model (Martin et al., 1975).

Validation of Constitutive Models

8.1 Element Tests

Constitutive models are normally validated by using them to predict response in single element tests such as the static or cyclic triaxial test. However, single element tests may be a necessary but not a sufficient test because they do not provide an adequate validation of the predictive capability of a model. The stresses or the strains are known a priori and there is no need to solve the boundary value problem using the constitutive model to predict the response. All practical applications involve the solution of the equilibrium equations and the continuity equations under a prescribed set of boundary conditions and a prescribed input. In other words, adequate model validation requires an inhomogeneous stress field which is not the case in the element test.

8.2 Centrifuge Tests

The centrifuge test offers the best opportunity for validating models by the solution of boundary value problems. Centrifuge models can be extensively instrumented, prepared under controlled conditions and shaken by prescribed input. Constitutive models and procedures and finite element models can be clearly tested by seeing how well the performance of the centrifuge model can be predicted. Also, numerical models and procedures can be calibrated and improved or modified for phenomena that may not have been adequately accounted for in a model.

The TARA-3 model has been subjected to validation studies on the centrifuge over a three-year period, 1984-86. The tests were conducted at Cambridge University on behalf of the Nuclear Regulatory Commission of the United States under the auspices of the National Science Foundation called the VELACS program. The acronym arose from the title Verification of Liquefaction Analysis by Centrifuge Studies. A conference on predictions made under this program was held at the University of California at Davis in October 1993 (Arulanandan and Scott, 1993).

8.3 Case History from Field

Opportunities for quantitative validation by case histories in the field are quite limited, primarily because structures are not generally adequately instrumented and earthquakes are rare. The 1987 Edgecumbe Earthquake in New Zealand M = 6.7 provided an opportunity to see whether the acceleration response and the permanent deformations could be adequately modelled in Matahina Dam.

The dam is located on the Rangatake River in the eastern Bay of Plenty Region of New Zealand about 23 km from the earthquake epicenter and about 11 km from the main surface rupture.

Founded on rock, the dam is 86 m high and has a crest length of 400 m (Fig. 12). The core is low plasticity weathered greywacke and slopes upstream. Dam shells are compacted rockfill of hard ignimbrite. The transition zones adjacent to the core are the fine and soft ignimbrite stripping from the rockfill, quarry and left abutment excavation (Finn et al., 1992, 1994).

Matahina Dam was instrumented at three locations along the crest to measure accelerations at the top of the crest, at the mid point between the crest and the base and at the base. Lateral and vertical displacements of the downstream slope have been monitored consistently at many locations since the dam was constructed. Readings were taken shortly before the earthquake and immediately afterward and the earthquake induced permanent deformations were determined as shown in Fig. 12. These deformations provided the basis for checking the capability of TARA-3 for estimating permanent deformations.

The 1987 Edgecumbe earthquake (Finn et al., 1992). Analyses assumed that no pore pressure developed in the rockfill and the core deformed under undrained conditions during the earthquake. The properties of the clay core were obtained by laboratory testing and in situ measurements. Stiffness of the rockfill was estimated by measuring average shear wave velocities at various locations. Strength was conservatively taken from the literature, and as a first step, volume change properties of the rockfill were estimated by inverse analysis from the measured deformations.

The computed and recorded accelerations at the crest for the Edgecumbe earthquake are shown in Fig. 13. Recorded and computed deformations during the earthquake at points on the downstream slope are shown in Table 2.

There is good agreement except for the node at the crest. The discrepancy here may be due to the fact that appurtenant structures on the crest were not modelled. The model was considered to be satisfactory for estimating the response under the design earthquake.
Fig. 12. Cross-section of the Matahina Dam showing the locations of displacement measurements. The vertical and horizontal components are given for the estimated displacements during the earthquake.

Table 2: Measured and Computed Seismic Displacements of Matahina Dam in mm.

<table>
<thead>
<tr>
<th>Node</th>
<th>X meas</th>
<th>X comp</th>
<th>Y meas</th>
<th>Y comp</th>
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</thead>
<tbody>
<tr>
<td>215</td>
<td>-</td>
<td>85</td>
<td>-34</td>
<td>-44</td>
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<tr>
<td>235</td>
<td>268</td>
<td>234</td>
<td>-102</td>
<td>-99</td>
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<td>323</td>
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<td>340</td>
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<td>-42</td>
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<td>366</td>
<td>33</td>
<td>33</td>
<td>-22</td>
<td>-22</td>
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<td>202</td>
<td>-</td>
<td>10</td>
<td>-11</td>
<td>-5</td>
</tr>
</tbody>
</table>

In the context of this section, liquefaction is synonymous with strain softening of sand in undrained shear as illustrated by curve 1 in Fig. 14. When the sand is strained beyond the point of peak strength, the undrained strength drops to a value that is maintained more-or-less constant over a large range in strain. This is called the undrained steady state or residual strength.

If the driving shear stresses due to gravity on a potential slip surface in an embankment are greater than the undrained steady state strength, deformations will occur until the driving stresses are reduced to values compatible with static equilibrium. The more the driving stresses exceed the steady state strength the greater the deformations needed to achieve equilibrium. Clearly, the residual strength is a key parameter controlling the post-liquefaction behaviour.

If the strength increases after passing through a minimum value, the phenomenon is often called limited liquefaction and is illustrated by curve 2 in Fig. 14. Limited liquefaction may also result in significant deformations because of the strains necessary to develop the strength to restore stability.

9 EVALUATION OF POST-LIQUEFACTION BEHAVIOUR

A challenging technical problem for geotechnical earthquake engineers involves the post-liquefaction behaviour of existing dams with potentially liquefiable zones in the structure or foundation. Two major challenges are: (1) estimating the post-liquefaction behaviour of the dam, and (2) planning cost-effective remedial measures.
Major difficulties are associated with estimating the residual strength of the liquefied soils. The most basic approach to the problem is to investigate the stability of the embankment by limiting equilibrium analysis which incorporates the residual strength of the liquefied soils. The major uncertainties associated with residual strength were noted earlier. Usually a factor of safety 1.1 to 1.2 is considered acceptable. Reliance on acceptable factors of safety alone is not adequate. Test data (Vaid and Thomas, 1994) show that large strains may be necessary to mobilize the residual strength or a significant level of post-liquefaction shearing resistance. The associated deformations can result in unsatisfactory behaviour of the dam despite adequate factors of safety. Additionally, as the thickness of a liquefied soil deposit increases, the assumption of well-defined failure surfaces becomes less reliable, and the dam may significantly deform from bearing capacity failure in the deposit. Therefore, it is necessary to conduct post-liquefaction deformation analyses to investigate the full consequences of liquefaction.

Fig. 14. Types of liquefaction behaviour.

Procedures for assessing post-liquefaction behaviour will be presented here and illustrated by a number of case histories.

9.1 Post-Liquefaction Response

An independent assessment of the equilibrium of the final position should be conducted using a conventional static stability analysis. The factor of safety determined in this way should be unity or greater depending on whether the deformations occurred relatively slowly after the earthquake or during it when inertia forces were acting.

When liquefaction is triggered, the undrained shear strength will drop to the residual strength. The post-liquefaction stress-strain curve cannot now sustain the pre-earthquake stress-strain condition and the unbalanced shear stresses are redistributed throughout the dam. This process leads to progressive deformation of the dam until equilibrium is reached.

A computer program, TARA-3FL, which is a variation of the general computer program TARA-3, has been developed by Finn and Yogendrakumar (1989) for estimating large post-liquefaction deformations based on the above concepts. Since the deformations may become large, it is necessary to update progressively the finite element mesh. Each calculation of incremental deformation is based on the current shape of the dam not the initial shape as in conventional finite element analysis.

The application of post-liquefaction analysis in practice will be illustrated by two case histories; Sardis Dam and the Upper San Fernando Dam. A detailed study will be presented in the next section on Mormon Island Auxiliary Dam. This example is especially useful for illustrating the procedures for assessing proposed remediation measures and evaluating the remediated dam. This example, therefore, is treated in detail separately.

9.3 Case Histories

9.3.1 Sardis Dam, Mississippi

Sardis Dam is a U.S. Army Corps of Engineer dam constructed in the late 1930's located in northwestern Mississippi, 16 km southeast of the town of Sardis on the Little Tallahatchie River. The dam is approximately 4,600 m long with a maximum height of 36 m. It was constructed by hydraulic filling and consists of predominantly a silt core surrounded by a sand shell (Fig. 15). The foundation consists of a 3 m to 6 m thick zone of natural silty clay called the topstratum clay as shown in Fig. 15. The top stratum clay is underlain by pervious alluvial sands (substratum sands) approximately 12 m thick which in turn are underlain by Tertiary silts and clays.

The U.S. Army Engineer District, Vicksburg, evaluated the seismic stability of Sardis Dam for a maximum credible earthquake having a peak horizontal acceleration of 0.20 g. Field and laboratory testing and seismic stability analyses indicated significant strength loss or liquefaction which threatens upstream stability may occur in (1) the hydraulically placed silt
core, (2) a discontinuous layer (1.5 m to 4.5 m thick) of clayey silt located in the topstratum clay, and (3) the upper 3 m to 9 m of sand shell along the lower portion of the upstream slope (U.S. Army Engineer District, Vicksburg, 1988; Finn et al., 1991; Finn and Ledbetter, 1991).

The liquefaction or strength loss potential of the silty clay was judged on the basis of a modification (Finn et al., 1991) to the Chinese criteria developed by Wang (1979). The residual strength \( S_{ur} \) of the silty clay was estimated from field vane tests and laboratory investigations to be 0.075 times the effective overburden pressure \( p' \), \( S_{ur} = 0.075p' \) (Finn et al., 1991). The following discussions are for a section where the weak clayey silt layer is 1.5 m thick.

9.3.2 Deformation Analysis of Sardis Dam

Deformation analyses by TARA-3FL supplemented by slope stability analyses were used to investigate the post-liquefaction response of the dam and to develop the remediation requirements.

The initial and final deformed shapes of the dam are shown in Fig. 16 for a particular distribution of residual strengths. Substantial vertical and horizontal deformations may be noted, together with intense shear straining in the weak thin layer. Different deformed shapes resulted from different assumptions about the distribution of residual strengths.

The static stability of each of these deformed shapes was analyzed by Spencer's method (1973) using the program UTEXAS2 (CAGE, 1989). In the clearly unstable region defined by a factor of safety less than one for the undeformed section, computed factors of safety for deformed sections were in the range of 1.0 - 0.05 (Fig. 17).

The variation of vertical crest displacement with factor of safety of the undeformed dam is shown in Fig. 18. This type of plot gives much more meaning to the factor of safety by associating with each factor an index of overall critical displacement such as loss of freeboard.

9.3.3 Remediation Requirements for Sardis Dam

The deformation analyses supplemented by slope stability analyses were used to investigate various proposals for remediation. Driven reinforced concrete piles were selected to control the post-liquefaction deformations of the dam (Fig. 19). The location of the zone of remediation is controlled by the conservation level of the pool and a desire to avoid driving the piles through riprap on the upper slope above the slope break.

During shaking by the design earthquake, the saturated portion of the core and the weak foundation clay outside the remediated zone are still expected to liquefy. This will result in increased lateral forces against the piles. Therefore, the piles must fulfill two functions: they must have sufficient strength to prevent shearing along the level of the weak clay layer and also have sufficient stiffness to prevent significant horizontal bending deformations that could lead to unacceptable loss of freeboard.

The static and dynamic loads for design of the piles were estimated by TARA-3FL and TARA-3 analyses. The time history of peak moments in the leading row of piles is shown in Fig. 20 under the assumption that liquefaction occurred at the beginning of the earthquake. The final design of the pile installation was based on limiting vertical deformations of the crest to about 1.5 m. The piles selected for remediation are 0.6 m square steel reinforced concrete piles placed 1.2 m on center perpendicular to the dam axis and 2.4 m on center parallel to the dam axis for the first three rows closest to the dam center line and 3.7 m on center for the remaining seven rows.

9.3.4 Upper San Fernando Dam

Inel et al. (1993) incorporated a simple soil model in the general purpose program FLAC (Cundall and Board, 1988) to investigate the deformations of the Upper San Fernando Dam during the 1971 San Fernando earthquake. The program uses an updated Lagrangian procedure similar to TARA-3FL for coping with large deformations. The constitutive model incorporated the Mohr-Coulomb failure criterion and elastic shear and bulk moduli dependent on the mean normal effective stresses.
Fig. 16. Deformed cross-section of Sardis Dam.

Fig. 17. Factors of safety for Sardis dam.

Fig. 18. Variation of loss of freeboard with factor of safety of undeformed dam.

The cross-section, soil properties and modified Pacoima Dam record used by Seed et al. (1973) in a previous study of the dam were used in FLAC analysis. The cross-section used is shown in Fig. 22.

As shown in Fig. 23, the porewater model satisfactorily predicted the laboratory-based liquefaction resistance curves. The deformed mesh at the end of shaking is shown in Fig. 24. A settlement of the crest of about 1 m was predicted and a bulging of the downstream toe of between 0.3 and 1 m. These agree well with the measured data. However, the computed overall deformation pattern differs significantly from the field pattern. The field data suggest the entire upper portion of the dam moved downstream. The computed deformation pattern suggested two different deformation modes with the upstream and downstream slopes moving apart creating significant deformations both in the upstream and downstream directions.

10 MORMON ISLAND AUXILIARY DAM: REMEDIATION MEASURES

Deformation analyses using TARA-3 and TARA-3FL were used to guide the remedial treatment of Mormon Island Auxiliary Dam (Ledbetter et al., 1991, Ledbetter and Finne, 1993). The deformations in the dam were evaluated for earthquake-induced strength degradation and liquefaction in the embankment and foundation soils. Critical regions of high strains were identified which contributed to unacceptable deformations and thereby controlled the performance of the dam. A cost-effective remediation plan was developed using results of the deformation analyses to keep post-earthquake deformations within tolerable limits by optimizing the location and degree of remediation.

10.1 Description of Mormon Island Auxiliary Dam

The Folsom Dam and Reservoir Project is located on the American River about 32 km upstream of Sacramento, California. Seven kilometers of man-made water retaining structures comprise the Folsom project and include the earth-fill Mormon Island Auxiliary Dam. The Folsom project was designed and built by the Corps of Engineers in the period 1948 to 1956, and is now owned and operated by the U.S. Bureau of Reclamation.
Mormon Island Auxiliary Dam was constructed across an ancient channel of the American River which is about 1.6 km wide at the dam site.

The channel gravels are about 20 m in thickness and were dredged for their gold content in the deepest portion with the cobbly-gravelly-tailings placed back into a partially water-filled channel in a loose condition. The tailings are variable with particle size distribution in the ranges: (1) 5-55% cobbles, (2) 5-75% gravel, (3) 5-75% sand, and (4) 5-40% silt and clay.

Mormon Island Auxiliary Dam is a zoned embankment dam 1,469 m long and 50 m high from core trench to crest at the maximum section. The narrow, central impervious core is a well-compacted clayey mixture grounded directly on rock to provide a positive seepage cut-off. Two 3.7 m wide transition zones flank the core. Dam shells were constructed of dredged tailing gravels and are founded on rock, undredged alluvium, and dredged alluvium. The alluvium was excavated to obtain slopes of approximately 1 vertical to 2 horizontal.

The seismic threat to the Polson project was determined to be an earthquake with peak acceleration of 0.35 g. Material zones and properties for Mormon Island Auxiliary Dam were derived from extensive field, laboratory, and geophysical investigations (Hynes-Griffin et al., 1988; Wahl et al., 1988).

Equivalent linear dynamic stress analyses (Hynes-Griffin et al., 1988; Wahl et al., 1988) resulted in the idealized section with zones of liquefiable and excess pore pressure soils shown in Fig. 25. Extensive liquefaction is expected in the dredged gravel foundation over a 245 m long section of the dam.

10.2 Deformation Analysis of Mormon Island Auxiliary Dam

Large differences between the initial and post-earthquake strengths in Mormon Island Auxiliary Dam result in major load shedding from those elements undergoing strength loss due to excess residual pore pressures and liquefaction. This results in substantial deformations throughout the dam. The computed deformation pattern was used to identify: (1) severely strained regions, (2) locations for remediation which are critical to control dam performance, (3) the likely geometric extent of remediation treatment areas, and (4) the strength requirements for the remediated zones.

Figure 26 shows a typical projected deformation pattern for the dam during the process of progressive loss of strength to 50 percent of initial strength in the zones predicted to liquefy in Fig. 25. The pore pressures were progressively increased to 30-40 percent of the effective vertical stress in the regions identified by $P$ in Fig. 25.
Fig. 21. Upper San Fernando Dam.

Fig. 22. Upper San Fernando Dam representative cross-section (Seed et al., 1973).

Fig. 23. Upper San Fernando Dam cyclic shear strength curves, predicted and measured.

As shown in Fig. 25, the critical areas controlling the dam performance are in the foundation materials beneath the shells of the dam. These materials are losing strength, deforming, and trying to move from beneath the dam. The largest strains are developing in the foundation soils just above bedrock. Remedial measures must contain and control the areas of large strain in order to control the performance of the dam.

10.3 Remediation for Mormon Island Auxiliary Dam

Deformation studies to provide guidance and insight for engineering judgements in the development of remediation requirements involved studies of the effects of variations in location, width, depth, and strength of areas to be treated.

These led to the remediation plan shown in Fig. 27, which includes both foundation treatment and an upstream berm.

10.3.1 Upstream Remediation

Due to the drought in California and the resulting low reservoir, dynamic compaction was a viable cost-effective method for strengthening the foundation materials upstream.

A 32,000 kg weight was dropped up to 30 times from a 30 m height in a grid pattern with a minimum spacing of 4 m on center. Upstream post-treatment evaluation is continuing. Results are being incorporated in analyses with expected downstream treatment to obtain the anticipated global behaviour of the dam. Significant increases (more than 50) in \( (N_1)_{60} \) values have been achieved in the top 12 m with decreasing improvement from depths of 12 m to 20 m.

10.3.2 Downstream Remediation Design Studies

TARA-3 and TARA-3FL were used again to evaluate the effects of various remediation zone widths and locations in the downstream foundation on crest movement. Typical results are shown in Fig. 28.

These data indicate the desirability of the treatment zone extending as far as possible upstream beneath the shell.

The effects of pore pressure migration on effective stresses were studied. Results showed the potential for excess pore pressure to rise to significant levels within the treated zone due to migration leading to strength loss in the zone.

Figure 29 indicates the sensitivity of crest displacement to treatment zone width and to excess residual pore pressure in the treated zone from both generation within the zone and migration from outside. Pore pressures must be controlled in the
treated zone to limit crest movement to less than 3 m. Parametric analyses showed that excess pore pressures in the treated zone during the earthquake should be limited to about 30 percent of the vertical effective stress and should be dissipated quickly after the earthquake.

The control of excess pore pressures by stone columns was a remediation option studied. Simultaneous generation and dissipation of pore pressures during the earthquake was simulated.

Results showed that clean-high-permeability stone columns could be used as drains to excess pore pressures within the treatment zone below the target 30% of effective vertical stress.

10.3.3 Downstream Remediation

Data from test sections showed that significant densification could be achieved during the construction of stone columns. This leads to the current downstream remediation plan is shown in
in pore pressure dissipation. At this writing, downstream remedial treatment is in progress.

Due to static ground water conditions the clean stone columns are anticipated to remain clean until an earthquake occurs that induces excess pore pressures.

Field evaluation of remediation achievements is being made by penetration and shear wave tests. The penetration criteria were based on limiting the pore pressure generation potential to about 30 percent of the vertical effective stress. A factor of safety against liquefaction (FSL) can be calculated as the ratio of cyclic stress ratio causing liquefaction (Seed et al., 1986) to cyclic stress ratio induced by the design earthquake. The FSL for different values of (N₁)₆₀ has been related to residual excess pore pressure by Seed and Harder (1990). From these relations, (N₁)₆₀ criteria can be developed which limits pore pressure generation potential.

10.3.4 Remediated Dam Response

Analyses of the dam in the proposed remediated state indicate that during the design earthquake excess pore pressures in the downstream treated zone do not exceed 30 percent of the vertical effective stress and are dissipated quickly after the earthquake, Fig. 31. The four curves in Fig. 31 are for the first five layers from the bottom of a mesh similar to that shown on Fig. 27. The top curve is for a drainage blanket.

Shown in Fig. 32 are typical results of the estimated deformations for the proposed remediated dam. The behaviour results from the progressive loss of strength to the liquefied state and residual strength values in all materials labelled L (Fig. 27) outside the treated zones. The pore pressures were progressively increased to 30 to 40 percent of the vertical effective stress outside the treated zones for the foundation and embankment materials labelled P in Fig. 27.

A comparison of Fig. 32 with Fig. 28 shows that the fully and partially liquefied soils between the dam center line and the treated zones are expected to be contained, and that the performance of the dam is controlled to acceptable limits.

Dynamic analyses of the proposed remediated dam show that the deformations are mainly post-earthquake and gravity driven. A study was made
Fig. 30. Downstream remediation plan.

The scaling parameter $K_o$ used to extrapolate the existing database on the incidence of liquefaction to the depths and stress conditions associated with embankment dams shows a wide variation with respect to soil type and relative density. Since this parameter has a major impact on the requirements for remediation if liquefaction is allowed to trigger, it should be determined on a project specific basis.

The post-liquefaction behaviour of dams should be assessed using both limiting equilibrium analysis and deformation analysis. The extent and location of remediation should be determined primarily on the basis of calculated deformation patterns. For many dams, especially those with substantial freeboard, criteria based on factor of safety alone can result in unnecessary remediation costs.

Whether equilibrium or deformation procedures are used, the post-liquefaction undrained behaviour of the liquefied material is the essential factor controlling the cost of remediation. It has two elements which should be well defined, the residual strength and the strain level required to reach it.

The controversy surrounding residual strength carries significant cost penalties for the remediation of embankment dams. A focused research program is required to resolve issues.

11 CONCLUSIONS AND RECOMMENDATIONS

The technology is available today for constructing safe embankment dams in any seismic environment. Indeed even in the extreme cases where a major fault passes under the dam it has been confidently asserted that "an embankment dam can be theoretic-ally made safe against any feasible fault displacement", (Sherard et al., 1974).

The major geotechnical problems facing dam designers in a seismic environment arise in the evaluation of the safety of existing dams. The most common factor leading to potential instability is the presence of loose saturated cohesionless soils in the dam itself and/or in the foundation which may liquefy during an earthquake. There are three difficult technical problems associated with potential instability induced by liquefaction. Will liquefaction be triggered? If so what will be the consequences? How can cost-effective remediation measures be designed to mitigate or prevent the consequences?

Analyses are planned to evaluate the earthquake deformation performance of the remediated dam again after completion of remediation.
The residual strength has traditionally been considered a function of void ratio only. The key question that has surfaced lately is, does it also depend on the loading path and the confining pressure?

The dynamic response analyses of embankment dams are still largely based on technology developed in the 1970's and represent our first attempts to carry out nonlinear analyses by equivalent linear procedures. The stresses and accelerations determined in this way are input into other procedures for determining the performance of the dam. These procedures appear to work quite well provided the behaviour of the dam is not strongly nonlinear and significant pore pressures do not develop. More comprehensive methods are available which can deal with these problems directly especially for evaluating the permanent displacements resulting from strong shaking with or without the presence of liquefaction. These procedures should be used when appropriate.

The Newmark procedure for estimating permanent deformations based on sliding block analysis is widely used despite all the evidence that deformations do not occur in this way. These methods are particularly inappropriate when a large zone has liquefied in the embankment or foundation.

The seismic safety evaluation of dams has evolved from very empirical procedures in 1960 to a mature sophisticated professional practice in 1993. As pointed out above, the evaluation of well designed dams is well within the capabilities of the profession. The assessment of existing dams which may have potentially liquefiable zones can be done safely but the uncertainties associated with the critical elements of the procedure are such that conservative judgements are being made. The challenge for the profession in the immediate future is to reduce these uncertainties and thereby the extent and cost of the remediation of embankments.

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