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N.Z. GEOMECHANICS NEWS

No. 39

DECEMBER 1989

A NEWSLETTER OF THE N.Z. GEOMECHANICS SOCIETY

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C O N T E N T S

THIS IS A REGISTERED PUBLICATION

"NZ Geomechanics News" is a newsletter issued to members of the NZ Geomechanics Society. It is designed to keep members in touch with recent developments. Authors must be consulted before papers are cited in other publications.

Persons interested in applying for membership of the Society are invited to complete the application form at the back of the newsletter. The basic annual subscription rate is $24.00 and is supplemented according to which of the international societies, namely Soil Mechanics ($11.00), Rock Mechanics ($12.00) or Engineering Geology ($9.00) the member wishes to be affiliated. Members of the Society are required to affiliate to at least one International Society.

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EDITORS NOTES

I heard the following comment recently: "When we were busy we were really active : We held several technical meetings and produced several technical papers. Now that we are not so busy, we don't seem to have the time." Once more I have had little success in persuading our members to produce copy for Geomechanics News. I have been hounding potential contributors and I always get a positive answer. Without fail, I am assured that something will be coming very shortly. It seems, however, that New Zealand is full of half-written articles or papers!

This issue reproduces a state-of-the-art paper on "Geotechnics Aspects of Earthquake Engineering", presented by Geoff Martin at the 5th ANZ Geomechanics Conference in Sydney (1988) and printed in Australian Geomechanics. This is a major contribution to the subject and will be particularly relevant to New Zealand. We have an agreement with the Australian Geomechanics society that each can reproduce articles of the other in the respective News journals. However, this is the soft option and would rather not rely on the practice to 'pad' out our own newsletter.

Readers are reminded that NZ Geomechanics News is a newsletter. I seek contributions of any sort for future editions. The following comments are offered to assist contributors:

. Technical contributions can include any of the following:

- Technical papers which may, but need not necessarily be of a standard which would be required by the international journals and conferences,

- Technical notes,

- Comments on papers published in Geomechanics News,

- Descriptions of geotechnical projects of special interest.

. General articles for publication may include:

- Letters to the NZGS,
- Letters to the Editor,
- Articles and news of personalities.

Submission of text material in camera-ready format is not necessary though typed copy is encouraged. Diagrams and tables should be of size and quality for direct reproduction. Photographs should be good contrast black and white gloss prints and of a suitable size for mounting to magazine format. Authors and other contributors must be responsible for the integrity of their material and for permissions to publish.

Tim Sinclair
<u>EDITOR</u>

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REPORT FROM THE MANAGEMENT SECRETARY

1. NEW MEMBERS

A large number of new members have recently joined the NZ Geomechanics Society. I trust they will find their membership enjoyable and productive. A warm welcome to:

Julie McMann
Michael Smith
Hugh Fendall
Maurice Fraser
Edward Marsh
Kosmas Ampualam
Mary Halliday
Ann Williams
Jan Plutecki

2. CONFERENCES

The organisation for our three major conferences is now well underway. You should have received or soon receive information about the 1990 Groundwater Symposium. Trevor Matuschka is the convener so contact him for further details.

The 1992 ANZ Conference and the 1992 Internation Symposium on Landslides will both be held in Christchurch. Planning is now well underway. These conferences are the largest venture our society has undertaken and the NZ Geotechnical community should derive enormous benefits and status from holding the International Symposium on landslides. Please provide Don Elder and David Bell with all possible support.

3. GEOMECHANICS NEWS

Not surprisingly, our editor Tim Sinclair has a great deal of difficulty obtaining articles for Geomechanics News. Geomechanics News is the only tangible benefit of membership for many of our members. We do not want to establish a high quality geotechnical journal. Geomechanics News is for publication of News from your region, a summary of branch meetings, gossip about the Geotechnical Community and is a forum for technical notes describing interesting projects, equipment and/or case histories.

What about a note from Professor Pender describing interesting features of his sabbatical in North America, a report from Bruce Riddolls telling us about engineering geology activities in Arrowtown. Because I have had the bad taste to single out a couple of our members for contributions I should also produce a note for the next volume. Let's hope the editor is swamped with articles.

Merry Christmas and Happy New Year to all our members.

Chris Graham
MANAGEMENT SECRETARY

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REPORT OF THE AUSTRALASIAN VICE PRESIDENT OF ISSMFE

My term of duty as Australian vice-president of ISSMFE is over and the cares of office are on Harry Poulos' shoulders. May they rest lightly on him! But before I disappear I must report to you on the recent Board and Council meetings of ISSMFE and the vice-presidential part I played in the XII ICSMFE.

Both Board and Council meetings are vital to the running of the business of ISSMFE. The Board is an advisory body to the President on which all vice-presidents sit and the Council has the power to take binding decisions. It is very important that Member Societies are represented regularly at Council meetings. If they are not present in person or by proxy then they cannot complain when ISSMFE goes against their wishes. It is incumbent on every Member Society to attend, whenever possible, to put their case for a particular course of action, to listen to the arguments of others, and then to vote in the best interests of ISSMFE. These interests do not necessarily match their local point of view.

A good illustration of this occurred in Rio! The venue for the next Council meeting had to be chosen and there were four candidates; Bangkok, Chile, Florence and Lesotho. The advocate for each made an impassioned plea extolling the beauty of the setting, the enticing amusements to be found there, the marvellous study tours that were planned, and the excellent facilities and accommodation that were available. At the same time they scored amusing debating points off their rivals. All good fun! But then the vice-presidents began to ask serious questions! The main purpose in holding a Council meeting was to conduct the business of ISSMFE. An effective business meeting required both a good attendance and a good cross section of the membership. How much these serious considerations weighed in the choice I do not know, but Florence was chosen by a large margin. Was it individual convenience or art galleries that swung the choice? Or is Florence, in fact, the best choice, this time, for ISSMFE?

Other important decisions taken by Council were, in my opinion, the adoption of the Statutes and By-laws, the acceptance of a realistic budget, and the adoption of a comprehensive set of guidelines for the President concerning the setting up of Technical Committees. ISSMFE now has a clear set of rules to work by with rational procedures for amendment should this ever be necessary and the unsatisfactory alternative of returning the draft for further amendment has been avoided. Not only does this show maturity on the part of Council but the energies of the incoming Board will not be taxed by a further round of consultations and painstaking debate over further changes.

Passing a realistic budget also shows the business maturity that the Council has developed, and the guidelines they have given the President embody the experience of the past few years and the expressed desires of several Member Societies. The guidelines should make it much easier for everyone to appreciate the difficult decisions that a president has to face in setting up committees.

The decisions taken by Council are given in the attached summary prepared by the Secretary General.

While the Council is an open forum the Board is, in effect, a closed committee. Only its recommendations appear publicly. During my term the Board has had many useful discussions and has overseen the development of ISSMFE into an effective business unit. A great deal of time was spent in discussing and polishing the Constitution and By-laws which have been adopted. Future Boards should have much more time to spend on looking at and planning ways in which ISSMFE can develop. A realistic budget development process has been devised. Budgets are now prepared on a four year rolling cycle so that longer term expectations and plans are recognised in good time. And a rational set of guidelines has been developed for the guidance of the President in setting up Technical Committees. It fell to me at the Rio meeting to draft the recommendation which the Council later adopted.

In drawing up the budget the Board gave a lot of consideration to the level of support that it was appropriate to provide to Board members. We were unanimous in our opinion that the Board had a vital part to play, and in the best interests of ISSMFE had to continue to meet regularly. That being the case it was incumbent on ISSMFE to provide substantial support to help Board members attend meetings, and this has been recognised in the new budget. If the Region wishes its vice-president to carry out certain duties then it must have a formal funding system to support him in carrying out those duties. Casual funding is just not good enough!

One final point of ISSMFE business that I would like to mention is the system of Young Geotechnical Engineers' Conferences originated by President Broms. These are a major new development of great significance and we should all be grateful for Bengt Broms' excellent idea and the energy with which he pursued it.

As regards my vice-presidential duties at the XII ICSMFE, they were minimal. To sit on the platform for the openings ceremony and to receive a commerative scroll at the closure is hardly onerous. While I believe that it is important for the good name of a Region that its vice-president be present, surely there are some useful tasks that could be given him to perform? This would help justify the cost of getting him there!

But there is one significant task that he can do, provided his successor is also at the conference. This is to pass on to his successor a picture of the concerns of the outgoing Board and how these might affect the incoming Board's thinking. I was able to do this at Rio (thanks, Harry, for listening so patiently to me at dinner!) and this time further continuity is provided by the fact that our new President was a member of the old Board. This possibility of enhanced continuity is good reason for an incoming vice-president to attend an ICSMFE. There is also another good reason. This is to allow the incoming Board to meet and get to know each other before they take up office. This can only make the task of the incoming Board easier. I therefore recommend to you that in the interests of the Region and ISSMFE you should plan on funding the incoming vice-president to attend future ICSMFE. It is, I believe, a very necessary part of his duties to be there.

Before signing off there is one lesson that needs to be learnt from XII ICSMFE. This is the importance of timely issue of information and the observation of deadlines. Conferences are run for the benefit of the participants and it is vital that the information needed by intending contributors, participants and purchasers of proceedings goes out in good time. In the case of international events these people may not be familiar with the location of the venue, the proximity of accommodation, the cost of transport between them etc. Thus they need to be provided with comprehensive information on such matters, and about the itinerary, cost and similar details of study tours. Deadlines for sending out all this information must be well chosen (and observed) in terms of the time scale of participants. They are not mere internal benchmarks for the convenience of the organising committee! I feel that perhaps the Brazilians forgot this basic fact and the attendance at Rio suffered accordingly.

And so, good colleagues, I wish you goodbye. I hope I have met at least some of your expectations and managed to achieve something for the Region and for ISSMFE. It has been a stimulating experience to be your vice-president and I thank you for your support and confidence in me. May Harry Poulos have an equally stimulating, enjoyable and well supported incumbency.

J.H.H. Galloway
VICE-PRESIDENT (RETIRED)

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ISSMFE COUNCIL MEETING

Rio de Janeiro, August 12 1989

DECISIONS TAKEN

The decisions taken at this meeting are given below. The full minutes will be prepared and distributed to all Member Societies within two months.

1. New Member Societies admitted to ISSMFE since 1987 are Korea R, Sudan and Vietnam

2. The suspension on the Member Society of Ecuador was lifted and the Society readmitted into full membership.

3. The membership of the Dominican Republic is annulled.

4. Resolution regarding membership:

   "The Secretary General should write to any Member Society whose payments are 3 years or more in arrears stating that unless an attempt is made to pay these arrears by September 30, 1990 it will be recommended to the 1991 Council Meeting that their membership should be annulled."

5. Dr N.R. Morgenstern was elected President of ISSMFE for the period 1989-1994 (January).

6. Regional Vice-Presidents 1989-1994 (January):

Mr G. Donaldson	-	Africa
Prof. K. Ishihara	-	Asia
Prof. H. Poulos	-	Australasia
Prof. U. Smoltczyk	-	Europe
Prof. J.K. Mitchell	-	North America
Mr L. Decourt	-	South America

7. Resolution regarding Technical Committee Activity.

   "The President is asked to bear in mind and as appropriate to implement the following broad principles in establishing Technical Committees and appointing members to them.

   (a) The Committee should have a finite life preferably not exceeding six years.

   (b) The Committee should have a definite task commensurate with its life and the resources at its disposal.

   (c) The relevant Member Societies and Vice-Presidents should be advised and consulted on the intended membership of Committees.

(d) The size of a Committee should be appropriate to its task and may include corresponding and liaison members.

(e) Chairmen should be reminded of the advantages of regular face-to-face meetings of their Committee and of the need to make the best use of the skills and resources of all their members".

8. Resolution regarding Soil Classification System.

"While waiting for general agreement about a classification system that will have to be used in papers presented to ICSMFE, each author should be obliged to identify - and describe if necessary - the soil classification system used in the paper".

9. The April 1989 draft Statutes and By-laws were approved, to come into effect immediately following the Council Meeting.

10. The audited accounts for 1987, 1988 were approved.

11. The Budget for 1989-1993 prepared by the Finance and Budget Committee was approved.

12. The next ISSMFE Council Meeting will be held in 1991 in Florence, Italy (May 25), immediately before the 10th European Regional Conference (May 27-June 1).

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REPORT OF THE AUSTRALASIAN VICE PRESIDENT FOR ISRM

ROCK MECHANICS ACTIVITIES IN AUSTRALASIA AUGUST 1988 - AUGUST 1989

GENERAL

Rock mechanics in the Australasian region has continued with its usual high level of activity over the last year, with numerous technical meetings throughout the region on topics related to a range of interests.

One of the initiatives taken up by the Australian Geomechanics Society has been the expansion of the member countries of the region. Papua New Guinea is a country that has long been very active in mining and rock mechanics, but the practitioners have usually been ex-patriate Australians. However, it was felt that there is now sufficient local expertise to merit the formation of a geomechanics society in that country and talks have been initiated for this purpose. Should such a society be formed, then it would join the Australasian region of the three International Societies.

Consideration has been given to forming similar societies in other Pacific island states, but it was felt at this stage, there was insufficient local interest.

ISRM PUBLICITY AND PROMOTION

The ISRM has continued to receive wide ranging publicity throughout the region with the full circulation of the ISRM "News", periodic reports on activities by mailings from the Vice-President, and the regular publication of the two principal news vehicles, "Australian Geomechanics" and the New Zealand equivalent, "N.Z. Geomechanics News". In addition, the various local groups of the Australian Geomechanics Society have organised technical meetings at Universities and Colleges on topics most likely to interest students. This initiative has been aimed at increasing membership of the local society.

The Australian Geomechanics Society has also produced a pamphlet which describes its activities, including those of the international societies. This pamphlet has been widely circulated amongst engineering organisations, including educational, in order to increase membership.

CONFERENCES AND SYMPOSIA

With the region's principal conference taking place immediately prior to the 1988 Madrid Symposium, although there has been numerous small local events, there has been no major conference or symposium in the region in the last year. However, in the next year a major symposium on Groundwater and Seepage will be held in Auckland in May 1990 and the Third International Symposium on Rock Fragmentation by Blasting will be held in Brisbane in August 1990.

On a more general note, it is commonly accepted that the principal event of the Australian Region's conference calendar is the Australia-New Zealand Geomechanics Conference which is held once every four years. This event was first held in the 1950's under the general title of Conference on Soil Mechanics and Foundation Engineering. However, in the 1960's, the name was changed to Geomechanics Conference in recognition of the formation of the ISRM and the IAEG, and the desire to include substantial contributions from these areas of geomechanics.

This conference has long been recognised by the ISSMFE and as such, it has co-sponsored the event along with the Engineering Institutions of the two countries involved. However, despite the name change, the substantial contribution of papers and discussions on rock mechanics subjects and the official status given to the ISRM Vice-President, this series of conferences has never received the endorsement of the ISRM. This would seem a great pity as the ISRM has much to offer, and the region has a special close relationship between the practitioners of soil mechanics, rock mechanics and engineering geology. In order to rectify this situation, a formal request has been placed before the board to sponsor the next ANZ Geomechanics Conference to be held in Christchurch, New Zealand in February 1992. The theme for the Conference is "Geotechnical Risk - Identification, Evaluation and Solutions". There has also been a similar request for the 6th International Symposium on Landslides to be held at the same venue in the following week.

ISRM COMMISSIONS

Two of the ISRM Commissions have their Presidents residents of the region. The Commission of Interpretation of Hydraulic Fracturing Records, under Mr J. Enever, has been very active and it is understood that a draft of the compendium of experience on interpretation of hydraulic fracturing records will be tabled at the Commission Meeting in Pau. It is intended that the final version of this compendium will be published soon after and, along with the publication of the proceedings of the workshop held in Minneapolis, will represent the completion of the Commission's activities.

The Commission on Rock Boreability, Cuttability and Drillability under Dr W.E. Bamford has been relatively inactive over the last year. However, it is understood that some changes to the organisation of the commission are to be proposed. At the time of preparation of this report, details were not available, but hopefully will be at the time of the Board Meeting in Pau.

PUBLICATIONS

"Australian Geomechanics" and the "N.Z. Geomechanics News" continue to appear regularly with various items of news, information and discussion on various subjects of general interest to the region. In addition, the New Zealand Geomechanics Society has published a booklet entitled "Guidelines for the Field Description of Soils and Rocks in Engineering Use". It is hoped that this carefully constructed publication will be of considerable use to the industry in general.

AWARDS

The John Jaeger Medal is the principal honour of the Australian Geomechanics Society. This medal is awarded for contributions of the highest order in the field of Australian Geomechanics and was first awarded in 1980 in memory of Professor John Jaeger who contributed so much to geomechanics, and in particular, rock mechanics. The medal is presented once every four years at the Australia-New Zealand Geomechanics Conference, where the recipient also delivers the John Jaeger Memorial Lecture. This occasion is recognised throughout the region as the major lecture of the region. There is currently a move to extend the conditions of this award to include New Zealand Geomechanics.

It recently came to the attention of the National Committee of the Australian Geomechanics Society, that the U.S. National Committee for Rock Mechanics had instigated a Jaeger Lecture Series, the first one being presented by Professor Neville Cook at the U.S. Rock Mechanics Symposium in Minnesota in June 1988. Consequently, the chairman of the Australian National Committee, Dr Neil Mattes, wrote to the U.S. National Group to bring to its attention that there already was an award and a lecture to honour the memory of Professor Jaeger, and that any award made by the U.S. National Group would be likely to overshadow the original Australian award. The U.S. National Group responded through their Chairman, Douglas D. Bolstad, and a copy of that letter is attached.

It should be stressed that the sentiments expressed in Dr Bolstad's letter are understood. It is, of course, appreciated that Professor Jaeger had a great influence on the development of rock mechanics throughout the world and not only in the U.S. It is also appreciated that Professor Jaeger had strong ties with the U.S. It would be agreed that Professor Cook is an excellent choice for such an award and that it would be fitting for it to be presented before a larger international audience.

However, the fact remains that the Jaeger Award/Lecture was first adopted by the Australian Geomechanics Society in recognition of a great Australian. If the U.S. National Group also has a similarly named award or honour, the significance of the original Australian honour will become eclipsed by the size and influence of the U.S. honour. Furthermore, although it was indicated that the U.S. Committee would not appoint a Jaeger during the years that the Australian award was made, the Australian award is made only once in four years. Therefore, there could be three U.S. awards to one Australian. This imbalance would merely make the situation worse.

The U.S. Committee has many more international figures that could be honoured by the naming of a lecture series. Australia has had much fewer such international figures, particularly of the calibre of Professor Jaeger.

It is recognised that the National Committee of the Australian Geomechanics Society does not possess a copyright on the name "Jaeger", and it is further recognised that the Australian Society cannot prevent the U.S. Committee from using the name. However, in the interests of international co-operation, and in recognition that the smaller Australian Geomechanics Society would like to keep its principal award as its own and not confused by a similarly named U.S. award, the U.S. National Committee is requested to discontinue the Jaeger Lecture series under that name.

Ian W. Johnston
VICE-PRESIDENT FOR AUSTRALASIA

LOCAL GROUP ACTIVITIES

1. AUCKLAND BRANCH

Since the last issue of Geomechanics News, the Auckland Branch has held one meeting with the title "Engineering Hazards" and addressing the questions:

Will the slope on my property stand?
Could my house sink into the ground?
What would happen in an earthquake?
Could the nearby stream rise and flood my land?
Am I insured for these and other hazards?

The meeting included four speakers with the following topics.

GEOLOGICAL AND GEOTECHNICAL HAZARD MAPS	Dr. W. Prebble - Senior Lecturer Geology Dept. University of Auckland
HAZARD IDENTIFICATION AND DEVELOPMENT ZONING TWO EXAMPLES	Mr B. Horide - Senior Engineer Beca Carter Hollings & Ferner Ltd
HYDROLOGICAL HAZARDS - A LOCAL AUTHORITY VIEW	Mr K. Oldham - Senior Engineer Auckland City Council
HAZARD AND DISASTER INSURANCE	Mr J. O'Brien - Managing Director Applied Geology Associates Ltd

Malcolm Stapleton has recently resigned as Auckland Branch member to travel the world. John Sekula has taken over his duties for the remainder of the year.

EDITOR

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2. WELLINGTON BRANCH

The Wellington Branch had a successful meeting on the 14th November 1989. John Galloway was presented with his Life Membership Certificate of the Society.

A presentation on the Dynamic Compaction carried out at the BP Terminal at Seaview was the subject of the meeting.

The presentation was led by Ted Malan, Director of Tonkin and Taylor Ltd, the project consultants accompanied by John Stares, Project Manager BP Oil (NZ) Ltd, Wellington Terminal Development Manager and Dave Jewell, Wellington Manager for Brian Perry Ltd.

John Stares opened the presentation with a light heart view from the clients' perspective, Ted Malan followed on the technical issues and Dave Jewel closed with a view from the contractor.

C. J. NEWTON

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3. CHRISTCHURCH BRANCH

The only recent meeting has been a presentation by Dr John Berrill, of the University of Canterbury, on the San Francisco Earthquake.

John was one of the 5 member New Zealand party returning from Armenia and in San Francisco when the earthquake struck. He gave a fascinating account of his immediate impressions during and after the earthquake. This was primarily a slide show with comments and anecdotes, since he had only been back in New Zealand for two days.

Some of the main points he made were:

- the organisation of authorities after the earthquake
- the rapid mobilisation of scientific investigations
- the particular vulnerability of poor soils
- the widespread extent of liquefaction and general importance of site specific effects.

The lecture was very well attended with over 80 present.

Organisation of the 6th Australia-new Zealand Geomechanics Conference, and the 6th International Symposium on Landslides, is progressing well.

DON ELDER

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4. SUMMARY REPORT OF WELLINGTON BRANCH MEETING

Dynamic Compaction (14 November 1989)

The evening panel discussion was presented by Mr John Stares of BP Oil NZ Ltd, Mr Ted Malan of Tonkin and Taylor, Panel Leader and Mr Dave Jewel of Brian perry Ltd.

Mr Stares opened the meeting with a brief introduction to the history of the terminal and the construction stages it will undergo in its upgrading.

The terminal comprised essentially two areas, the northern zone which contains the present oil storage tanks and an area to the south which had been reclaimed from the Wellington harbour. This reclaimed area was to accommodate new storage tanks.

Mr Stares also gave a light hearted view of a Client's Project Manager undertaking a task of this nature.

Ted Malan continued and outlined the technical aspects of the work. Tonkin and Taylor undertook a site investigation to determine the suitability of the new site to locate the storage tanks. The site was essentially suitable for the new tanks.

However under the design earthquake, it was found that the underlying sands and silts could be expected to liquefy. These materials extended below the surface materials to a depth in the region of six to seven metres.

Densifying the materials was considered the most appropriate method of overcoming the liquefaction problem. Several methods of densification were considered. The most suitable method identified was to dynamically compact the site. This involved the dropping of a large weight from a height. Design calculations indicated that a 100 tonne-metres of energy would be required to achieve densification to the required depth. The most suitable combination was found to be a nine tonne weight, dropped from a height of eleven metres.

A field trial was undertaken to determine the effectiveness of this method. The trial was satisfactory and the method of dynamic compaction was adopted to densify the liquefiable sands and silts underlying the site.

The selection of concrete as opposed to steel for the weight was made on economic and fabrication time criteria. The initial block was constructed using steel fibre reinforcement. The block shattered during the field trial after a small number of drops.

A suitable crane test weight was located as a replacement. This proved to be ideal and survived the estimated 20,000 drops with only minor cracking.

The ground accelerations at various radii from the point of impact were measured to determine the possible effects on structures. The accelerations were found to be very low even at short distances and hence compaction could be undertaken adjacent to existing structures. The vibrations could only just be recognised by personnel close to the drop point and hence the work did not interfere with nearby workers in surrounding buildings.

The block was dropped from four heights, eleven, six, three and one metres. The majority of drops were from eleven metres with the one metre drops being used for densification of surface layers which unravelled during the higher energy drops.

The site was marked out in a six metre grid for each of the six passes required to cover the site. The layout of the secondary and subsequent grids were located within the initial grid so that the site was completely covered.

Between passes, the site was graded to remove the indentations from the previous pass. The indentations were over 150 mm deep and after periods of rainfall, they ponded water. Without the regrading, the site quickly deteriorated as muddy water was sprayed around the site.

The result of the dynamic compaction was a substantial increase in density of the liquefiable silts and sands. Measurements taken using a cone penetration rig before and after the compaction revealed a dramatic increase in strength of these materials.

CPT's were preferred due to the soft ground conditions encountered and the rapid rate at which the tests could be undertaken compared to carrying out SPT's.

Mr Jewel concluded the discussion with a brief outline from the contractor's view. The type of work was found to be very demanding on the crane and operator.

The weight was allowed to free fall with the cable still attached. The operator was required to limit the amount of over spin on the winch without snapping the cable before the weight hit the ground. The crane required only eight replacement cables to complete the job which involved in the region of 20,000 drops.

The crawler mounted crane was found to be very suitable for the task. The crane moved along the length of the site carrying out three drops, one each side and a central drop, before moving on to the next set. There was no identifiable damage to the lattice boom or mechanical components of the crane.

The work was completed on time and within the estimated cost.

C. J. Newton

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PUBLICATIONS OF THE SOCIETY

The following publications of the Society are available:

(a) From the Secretary, IPENZ, P.O. Box 12-241, Wellington North

- Proceedings of the Palmerston North Symposium "Geomechanics in Urban Planning", April 1981. Price $20.00.
- "Stability of House sites and Foundations - Advice to Prospective House and Section Owners". (Published for the Earthquake and War Damage Commission). Price $0.50.
- Proceedings of the Third Australia-new Zealand Conference on Geomechanics, Wellington, May 1980. Price $20.0 for the three volume set to members, $30.00 to non-members.
- Proceedings of the Second Australia-New Zealand Conference on Geomechanics, Brisbane, July 1975. Price $25.00.
- Proceedings of the Wanganui Symposium "Using Geomechanics in Foundation Engineering", September 1972. Price $8.00 to members, $10.00 to non-members.
- Proceedings of the Alexandra Symposium "Engineering for Dams and Canals", November 1983. Price $40.00 to members, $50.00 to non-members.
- Copies of all back-issues of "New Zealand Geomechanics News", are available to members at a nominal price of 50 cents per copy plus 50 cents post and packaging per order.

NOTE To reduce stocks, all the above publications costing over $10.00 will now be sold at 1/2 price - while stocks last!

(b) From Government Bookshops and the Secretary IPENZ:

- "Slope Stability in Urban Development" (DSIR Information Series NO. 122). Price $2.00. (Also available from Government Bookshops).

The following publications of the Society have been sold out:

- Proceedings of the Nelson Symposium "Stability of Slopes in Natural Ground", 1974.
- Proceedings of the Wellington Workshop "Lateral Earth Pressures and Retaining Wall Design", 1974.
- Proceedings of the Hamilton Symposium "Tunnelling in New Zealand", November 1977.

(c) Newer publications, also available from the Secretary, IPENZ.

- Proceedings of the Hamilton Symposium "Piled Foundations for Engineering Structures", November 1986. Price $20.00 to members, $25.00 to non-members.
- From the Institution of Engineers, Australia, Guidelines for the Provision of Geotechnical Information in Construction Contracts. A 20-page booklet. Price $10.00.

DICK BEETHAM
<u>PUBLICATIONS OFFICER</u>

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FORTHCOMING CONFERENCES

1. CONFERENCE DIARY

1990

February 13
Brussels, Belgium, Journee D;Etude-Approche Probabiliste des Problems en Geomecanique.

March 27-28
Paris, France, Conference on Foundations for Large Works.

April 2-6
Oxford, UK. Third Int. Symp. on Pressuremeter.

April 18-20
Vienna, Austria. International Conference on Jointed and Faulted Rock.

April 25-28
Beijing, China + International Conference on Structural Engineering and Computation.
Topics: include soil mechanics and foundation engineering aspects.

May 38-June 1
The Hague, The Netherlands. Fourth Int. Conf. on Geotextiles and Geomembranes.

June 4-6
Loen, Western Norway ISRM Regional Conference on Rock Joints.

June 10-13
Trondheim, Norway. Second Symposium on Strait Crossings.

June 18-21
Ithaca, NY, USA. ASCE Spec. Conf. on Design and Performance of Earth Retaining Structures.

August 6-10
Amsterdam, The Netherlands. Sixth International Conference of the IAEG.

August
Bucharest, Romania. International Colloquium on Viscoplasticity of Geomaterials.

September 3-7
Glasgow, UK. 3rd Int. Symp. on Reclamation Treatment and Utilisation of Coal Mining Wastes.

September 3-7
Chengdu, Sichuan, China. International Congress on Tunnel and Underground Works - Today and Future.

September 10-12
Glasgow, UK. International Reinforced Soil Conference.

October 11-13
Cracow, Poland. 9th National Conference on Soil Mechanics and Foundation Engineering.

October 17-20
Caracas, Venezuela. 3rd South American Congress on Rock Mechanics.

1991

March 11-15
University of Missouri-Rolla, USA. Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

May 27-June 1
Florence, Italy. 10th ISSMFE European Regional Conference.

Bangkok, Thailand. 9th ISSMFE Asian Regional Conference.
(Date to be announced).

August 26-30
Santiago, Chile. 9th ISSMFE Pan Am Regional Conference.

September 23-27
Maseru, Lesotho. 10th ISSMFE African Regional Conference.

1992

February 3-7
Christchurch, New Zealand. 6th ANZ Conference on Geomechanics.

February 10-14
Christchurch, New Zealand. 6th Int. Sym. on Landslides.

May 10-16
Rostock, GDR. 3rd Baltic Conference on Soil Mechanics and Foundation Engineering.

May 28-31
Aalborg, Denmark NGM-92, XIth Nordic Geotechnical Meeting

1994

January
New Dehli, India. XIII International Conference on Soil Mechanics and Foundation Engineering.

2. GROUNDWATER AND SEEPAGE SYMPOSIUM

24-25 May 1990 - University of Auckland

PURPOSE

The New Zealand Geomechanics Society is organising a Symposium on Groundwater and Seepage in Geotechnical Engineering to review appropriate theory and present practice, to provide a forum for the presentation of case histories and to enable the exchange of concepts, information and experiences. A feature of the Symposium will be a keynote address by Professor D. K. Todd from University of California, Berkeley. Professor Todd is distinguished in the field of groundwater, has published text books on the topic and has his own consulting engineering practice.

FURTHER INFORMATION

For further information or to request registration details when available please contact

Trevor Matuschka (Convenor)
Engineering Geology Ltd
P O Box 33-426
Takapuna.

or

Barry Williams (Symposium Administrator)
University of Auckland
Private Bag, Auckland

PROGRAMME

(see separate attachment)

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NEW ZEALAND GEOMECHANICS SOCIETY

GROUNDWATER AND SEEPAGE SYMPOSIUM

"Draft Programme"

THURSDAY, 10 MAY 1990

8.30 am	Registration
9.15 am	Welcome (Chairman, Geomechanics Soc.)

HYDROGEOLOGY REVIEW AND SOLUTIONS TO GROUNDWATER FLOW PROBLEMS

9.30 am	The Occurrence, Movement and Chemistry of Groundwater - D.H. Bell
10.00 am	Morning Tea/Coffee
10.30 am	The Solution of Seepage Problems - L. D. Wesley
11.00 am	Calculating Seepage Flow with a free Surface. Same Old Methods and some New Ones - J. Fenton
11.20 am	The Method of Fragments - T. Larkin
11.40 am	Discussion
12.00	Lunch

FIELD INVESTIGATIONS AND INTERPRETATIONS

1.00 pm	Making Sense of Permeability Testing - A.P. Kortegast
1.20 pm	Planning and Design of a Groundwater Programme - W. Russell
1.40 pm	to be confirmed
2.00 pm	An Application of Instrumentation to Groundwater Monitoring - M. T. Mitchell
2.20 pm	Discussion
2.40 pm	Afternoon Tea/Coffee

CONSTRUCTION DEWATERING

3.00 pm	Construction Dewatering - B. Perry Ltd
3.15 pm	Nearsurface Hydrogeology and Excavation Dewatering in Granular Christchurch Soils - D. Elder and I. McCahon
3.30 pm	Dewatering Marina Reclamations - G. Ade
3.45 pm	Discussion

KEYNOTE ADDRESS

4.00 pm	Professor D. K. Todd, University of California, Berkeley
5.00 pm	Free
6.30 pm	Social
7.30 pm	Symposium Dinner

FRIDAY, 11 MAY 1990

SEEPAGE PROBLEMS IN NATURAL GROUND

8.45 am	Monowai Mine Redevelopment: Prediction of Groundwater Inflows -I.M. Parton and M.L. Plested
9.00 am	Subsurface Tunnel Seepage and Erosion in Loessial Soil - M.D. Yetton
9.15 am	Ground Stabilisation with Counterfort Drains. Design Installation and Monitoring of Drawdown Performance - P. Riley
9.30 am	The Collapse and Deviation of a 3.2 m Diameter Tunnel in Saturated and Faulted Andesitic Alluvium - W. Prebble
9.45 am	Discussion
10.05 am	Morning Tea/Coffee
10.25 am	Groundwater and Opencast Mining - D. Jennings, C. R. Henderson, A. D. Pattle
10.40 am	Farming Use of Seepage Water from a Basaltic Cone Aquifer - J. Sekula, D.V. Toan
10.45 am	Groundwater in Loessial Soil - M.D. Yetton and D.M. Elder
11.00 am	Tidal Response Method for Aquifer Characteristics - T.J.E. Sinclair
11.15 am	Discussion

EMBANKMENT SEEPAGE

11.35 am	Seepage Control at the toe of the Patea Dam - D. V. Toan
11.40 am	Embankment Seepage - Theory vs Observation for the Johore Bahru Dam - L.D. Wesley
11.50 am	Discussion
12.00 pm	Lunch

GROUNDWATER CONTAMINATION

1.00 pm	Underground Petrol Contamination - A Case History on Investigation, Remediation and Risk Evaluation - B. Krown and P. Brown
1.15 pm	Synthetic Liners - The Ultimate Solution - A. Nelson
1.30 pm	Investigation and Removal of Fugitive Rolling Oil from Groundwater - R.J. Burden and K.W. Tearney
1.45 pm	The Use of Risk Assessment in Evaluating the Hydrogeological Aspects of a Hydrocarbon Spill - P. Brown and S. Wood
2.00 pm	Discussion
2.20 pm	Afternoon Tea/Coffee

DISPOSAL OF WASTES BY GROUND SOAKAGE

2.50 pm	The Role of Evapo-Transpiration Seepage (ETS) in On-Site Wastewater Disposal from Households and Institutions - I. Gunn
3.10 pm	Effluent Disposal Systems - A Local Authority View - R. Lorden
3.25 pm	Computer Modelling of Seepage Flow to Study the Environmental Effects of Disposal of Septic Tank Effluent by Deep Soakage in Environmentally Sensitive Areas - N. Mark-Brown and D. Jamieson
3.40 pm	Urban Stormwater Disposal by Ground Soakage - C. Manley, K. Oldham.
3.55 pm	to be advised - B. Kelsey
4.10 pm	Discussion

CLOSING REMARKS

4.30 pm	Chairman, NZ Geomechanics Society.

3. SIXTH AUSTRALIA - NEW ZEALAND CONFERENCE ON GEOMECHANICS

Geotechnical Risk - Identification, Evaluation and Solutions
Christchurch, February 3-7, 1992

Organisation of the conference by the Christchurch based committee is proceeding smoothly. The first bulletin, including an advance call for abstracts, is due to be distributed via NZGS by Christmas.

Assistance is sought with organising 'Case History Prediction' sessions at the conference. These were a major highlight of the 1988 conference in Sydney, Australia. Information packages were made available prior to the conference on 6 geotechnical projects.

Subjects included:

- Pile loads and deflections
- Cut slope stability and failure
- Soft soil Settlements
- Earth pressures on culverts

Actual Behaviour was recorded and compared at the conference to predictions made by delegates. The exercise was not only informative but also a lot of fun!

Anyone with ideas is invited to provide them to the conference committee via the convenor.

Dr Don Elder
Convenor

C/- Soils & Foundations Ltd
P O Box 451
Christchurch

Fax: (03) 667 780
Ph : (03) 798-432

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LETTERS TO THE EDITOR

30 October 1989

Dear Sir,

PUBLIC ACCEPTANCE OF APPLIED GEOLOGICAL SKILL

I was interested to read Ormiston and Carryer's article "ENGINEERING REGISTRATION ACT, END OF AN ERA". They have tended to concentrate on one aspect of a complex problem which affects economic or mineral geologists, engineering geologists and hydrogeologists. Hitherto in new Zealand none of these groups are sufficiently large to support professional bodies of their own and they seem unwilling or incapable of joining together to form one professional body with various groupings within it.

The Australasian Institute of Mining and Metallurgy looks after some of the concerns of the mineral geologists but it does not, unfortunately, much concern itself with the question of professional recognition in New Zealand. Resulting from the lack of public recognition, as distinct from recognition within the industry, of a group of mineral industry professionals, mineral exploration and mining are now moving towards control by local government "environmental scientists" generally lacking professional associations and civil engineers. For the industry the result will probably be disastrous until common sense finally prevails.

Whether it will be best to admit engineering geologists to corporate membership of IPENZ or to include them within a professional body of applied geologists is a matter for a debate which should take place. Failure to properly debate this issue, together with those of the mineral geologists, hydrogeologists, and others and to mutually resolve them will ensure that appropriate geological input will often be lacking where it is most needed.

I agree with them when they say that the engineering geologist should be concerned with "where to build" while the engineer should be concerned with "how to build". While engineers insist on having the final say on both I have no doubt that we will have more Abbotsfords, Whaeos, Ruahihis etc. than we might otherwise. The presence of engineering geologists as subordinate personnel within large engineering practices will do little to alleviate the situation. In my view only where engineering geologists are full partners within these practices should these practices be entitled to include "engineering geology" in their supermarket-like listing of expertise.

Those whose principal activity is groundwater supply investigation and development are probably fewer in number than the geologists concerned with slope stability and the location of engineered structures. As one of this group I find that there is little recognition, among the public or among civil engineers, that geology has a very significant role in the identification, development and exploitation of groundwater resources. The influence of aquifer size, well spacing and exploitation rate on the cost of water are poorly understood. In contrast to mineral resources the quantification and exploitation of groundwater resources cannot readily be separated. Unless groundwater resources are considered pari passu with surface water resources water supplies will often cost more than is necessary and require higher levels of chlorination than might otherwise have been the case.

Personally, I anticipate that the problems that I have indicated will continue until such time as the general public, local government councillors and board members, and central government itself accept that, generally speaking, expertise in matters of the earth lies with geologists rather than engineers. This acceptance or recognition will not come about until the applied geologists I have referred to form their own professional body with standards of training and experience required for entry as high or higher than those of IPENZ and then undertake a vigorous campaign to educate the uninformed.

Note the ACENZ advertisement in every telephone directory, "Engage a professional engineer for professional results". The engineers can keep their advertisement short and sweet because the general public knows what they do. The general public does not know what geologists do, therefore they must be more explicit. I would like to see in every telephone directory, "Engage a professional geologists for minerals, quarries, sand, gravel, groundwater, land stability and environmental problems. The Earth is OUR business".

Yours faithfully,

Roger Dewhurst
M.App.Sc., M.I.M.M., M.Aus.I.M.M., C.Eng., F.G.S.
Consulting Geologist
Member Mineral Industry Consultants Association

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"SHAMSHER PRAKASH FOUNDATION"

Roorkee (UP) 247667 INDIA

Honorary Secretary
Dr. V.H. Joshi

George Gazetas receives 1989 Award

George Gazetas, Professor of Civil Engineering, State University of New York, Buffalo, (NY) has been selected to receive the 1989 Shamsher Prakash Research Award of US $1111.00.

Dr. Gazetas has made significant contributions to the understanding of the dynamic response of foundations, the effects of foundation shape and embedment, and has developed simple procedures for evaluating these effects. He has studied vibrations of pile foundations and has made major contributions to the studies of the seismic response of dams.

He has delivered several state of the art papers to International Conferences. He has been invited to make a special presentation on "SEISMIC EFFECTS ON EARTH DAMS" to the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, March 1991.

Dr. Gazetas has received several prizes and awards for his original contributions.

The jury consisted of Professors Ralph B. Peck, M.T. Davisson, A.S. Arya and Dr. R.K. Bhandari.

V.H.J.
(V.H. Joshi)
Hon. Secretary

Shamsher Prakash, Ph.D., P.E.

Anand Kutir
1111 Duane Avenue
Rolla (MO) 65401

October 19, 1989

Dear Sir,

We are distributing a user friendly computer program to compute natural frequencies and amplitudes of motion of rigid blocks supporting 1) a reciprocating machine and 2) a hammer. It will, therefore, be appreciated if you please insert the following listing in your forthcoming newsletter.

Thank you.

Sincerely,

Shamsher Prakash
Professor of Civil Engineering
University of Missouri-Rolla

FOR LISTING

SOFTWARE FOR MACHINE FOUNDATIONS

Two computer programs for determination of natural frequencies and amplitudes of motion of a rigid block foundation for 1) a reciprocating machine and 2) a hammer are available free. These programs are written for personal computers using FORTRAN 77 compiler. To receive a disk, please remit US $7.50 to cover shipping, handling, and duplicating costs to:

Secretary
Shamsher Prakash Foundation
Anand Kutir
1111 Duane Avenue
Rolla, MO 65401 USA

Telephone # (314) 364-5572

9/S.P.

Telephone: (314) 341-4489 (Off) (314) 364-5572 (Res) Telefax (314) 341-4729

UNIVERSITY OF MISSOURI-ROLLA

School of Engineering

Department of Civil Engineering
Butler-Carlton Civil Engineering Hall
Rolla, MO 65401-0249
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(FOR LISTING)

ADVANCE NOTIFICATION FOR INTERNATIONAL PARTICIPANTS

FOURTH SHORT COURSE
on
SOIL DYNAMICS AND FOUNDATION ENGINEERING

APRIL 2 - 6, 1990

July 15, 1989

(St. Louis, Missouri USA)

Invitation

The Fourth Short Course on Soil Dynamics and Foundation Engineering will be conducted by the Department of Civil Engineering, University of Missouri-Rolla from April 2-6, 1990, in St. Louis, Missouri. Civil, structural, and geotechnical Engineers, seismologists, scientists, and teachers all over the world are invited to join this short course. In the previous three short courses, participants had joined from Canada, China, England, Ghana, Japan, Mexico, Trinidad, and USA.

Course Description

Dynamic loads due to earthquakes and other sources pose a serious hazard for structures and foundations. Understanding of dynamic behavior of foundations and soils is of great importance in developing earthquake resistant design of foundation systems.

In this course, dynamic soil structure interaction, retaining structures, mat and pile foundations, liquefaction of soils, earth dam stability, and selection of design soil parameters will be covered. Emphasis will be placed on behavior and design of structures. Workshop sessions will be devoted to problem solving both with the computer and by manual methods.

State-of-the-art information on the behavior and analysis of foundations under dynamic loads will be presented.

Elementary knowledge of soil and/or structural dynamics is desirable.

Fees

The course fee is approximately US $1000.00.

The Text

"SOIL DYNAMICS" by Shamsher Prakash, published by McGraw-Hill Book Company, New York, 1981, will be furnished to course participants. The text will be supplemented by detailed lecture notes.

Please send requests for admission with 2 page resume:

Shamsher Prakash, Course Director
Fourth Short Course on Soil Dynamics
Professor in Civil Engineering
University of Missouri-Rolla
Rolla, MO 65401 USA

Telephone # (314) 341-4489/4461
Telefax # (314) 341-4729

ARTICLES AND TECHNICAL PAPERS

WHAT SHOULD I TELL MY STUDENTS?

IN THE INTRODUCTORY LECTURE

The subject I am about to teach you has a variety of names, the best known of which are "soil mechanics" and "geotechnical engineering". Nearly all the text books and reference literature related to the course uses these names along with the additional term "foundation engineering".

In New Zealand however, we have adopted our own term - "geomechanics" for this subject; precisely why we use a term which is little used by the rest of the world is not clear to me.

ON TERMINOLOGY

I will endeavour to stick to the terminology commonly used in soil mechanics throughout the world, and which you will find used in the text books you may refer to. You will need to be aware, however, that in new Zealand we use some terms which are not used elsewhere. For example, your text books all use the term **specific gravity**. In New Zealand, our soil testing standard has replaced this with a term called **"solid density"**. Our friends across the Tasman (also suffering from a bout of nationalistic fervour) have adopted the term **"soil particle density"** (Aust. Standard AS1289 - C8.3 - 1984). British and American standards retain the term used by your text books, and is the term I will generally use in this course.

ON SOIL CLASSIFICATION AND DESCRIPTION

The Unified Soil Classification system is undoubtedly the best known and most widely used system on a worldwide basis. For this reason, I think you should be thoroughly familiar with its principals and use.

There are a number of variations of the Unified system in use around the world; for example the British have produced a system which divides soils up into a lot more categories, based on the L.L. value. I do not propose to describe these other systems in any detail.

In New Zealand you may perchance come across a document called "Guidelines for the Field Description of Soils and Rocks in Engineering Use". This is a strange hotch-potch of the Unified and British systems with some specific New Zealand variations; unsatisfactory features include the following:

- the use of unified symbols while adopting a 35%/65% criterion for division of fine and coarse grained soils, instead of the 50/50 criterion of the unified system

- the introduction of a "soil weathering" scale not found in any other soil classification system

- failure to distinguish clearly between classification of the material itself and description of the state in which it exists in the ground

- heavy reliance on laboratory test results for a system titled "field description"

- a very misleading descriptive category called "moisture condition".

The above comments may be unduly critical and the issues they relate to might possibly be considered trivial. However, it does seem to me that we in New Zealand have not done our subject a good turn by insisting on "doing our own thing" and not seeking to keep in step with the rest of the world. It is inevitable that virtually all of the reference material we use will originate from overseas, and it is highly likely that many of our graduates will work overseas. All major developments and advances in the subject will originate from overseas. It seems therefore that there would have to be very strong reasons indeed for not keeping in step with the rest of the world. Is it not time the Geomechanics Society looked carefully at this question?

L.D. Wesley

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COMMENTS ON THE 12TH INTERNATIONAL CONFERENCE
ON SOIL MECHANICS AND FOUNDATION ENGINEERING

(HELD IN RIO DE JANEIRO IN AUGUST 1989)

Most of us probably imagine Rio de Janeiro to be one of the great glamour cities of the world, with its spectacular natural setting and equally spectacular man made edifices such as the Christ the Redeemer statue and the sugar loaf cable car, and therefore a great place to stage a conference. However, Rio is also something else, as demonstrated by the box insert taken from the Sept issue of "Ground Engineering". The article is highly emotive and very biased, but it does highlight the fact that concerns about personal safety tended to mar what would otherwise have been a very pleasant conference. Personally, I had no direct experience of street pickpockets or muggers, nor did any of the other N.Z. delegates as far a I am aware. However warnings from travel agents in N.Z. before departure had made me wary and I took reasonable precautions when walking the streets (such as removing my watch and concealing my camera).

Rio conference first report

The major geotechnical event of 1989 was the conference of the International Society for Soil Mechanics & Foundation Engineering held in Rio de Janeiro, Brazil from 13 to 18 August. Mike Winney reports.

BRAZIL'S DESPERATE economic straits dominated proceedings at the 12th international conference on soil mechanics and foundation engineering – held under siege conditions last month in Rio.

Violent assaults and armed robberies were directed at delegates and their partners venturing beyond the protective cordon ranged around the Hotel Nacional conference centre. Such discreditable mementos have left many respected members of the geotechnical community questioning whether a developing country should again host ISSMFE's conference.

New Delhi, India, has long been lined up for the January 1994 event. But now a host country with a more secure venue is likely to be chosen.

Brazen muggings are not prominently featured as an attraction in tourist brochures extolling the unique beauty of Rio's superb combination of city, mountains and beaches.

But where the maps show rolling hills and a plain as the backdrop there are filthy *favelas* – shanty towns. From these, muggers go out to hunt in the coastal territory of the rich who live in well guarded and caged homes, and who have found other ways of coping with inflation running at 1% a day.

The conference delegates, like other tourists, turned out to be unsuspecting targets for persistent vicious street and beach attacks on a scale that rates New York city's reputation as nothing.

Gun and knife threats divested them of valuables in broad daylight. Anyone unwary enough to display a camera, handbag or even a watch or ring was at risk from young gangs with no compunction about maiming to get what they wanted.

The conference itself was well run and several of the invited special lectures were very interesting. The overall impression however was that the conference was rather "routine" and perhaps organised without much imagination or innovation. I may be expressing a heretical view but it seems to me that the heyday of soil mechanics has passed; the "upward curve" of the post war era has peaked or flattened out and advances in the subject from here on will be rather hard won and unspectacular. The conference reflected this situation.

I have only been to one other of the international conferences and that was in 1965 in Montreal. At that conference all the well known figures except Terzaghi were present - Casagrande, Peck, Lamb, Skemptom, Bishop, Bjerrum, Roscoe etc. while at Rio none of these were present. New figures do not appear to have arisen to take their place and I think their absence had an influence on the conference.

Regardless of the threat from pickpockets and the rather routine nature of the conference, I thoroughly enjoyed Rio de Janeiro; it is every bit as beautiful and spectacular as the tourist publicity suggests and has an atmosphere all its own, made up of a strange mixture of hedonism, religious belief, and the struggle to survive. Because of the once weekly flights of Argentinean Airlines to Auckland I had several days to fill in after the conference; I spent one night at Iguassu falls (featured in the recent film "The Mission") and two nights in Buenos Aries. The latter city, at least the central city area, is the opposite to Rio - very elegant and cultured with much fine architecture, and apparently quite free from threats to personal safety.

L. D. Wesley

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The following pages, numbered 42 to 77, are reproduced with the permission of the Australian Geomechanics Society.

The paper entitled "Geotechnical aspects of earthquake engineering, State of the Art Report No. 2" was first presented by Geoff Martin at the 5th ANZ 1988 Geomechanics Conference held in Sydney on 21-26 August 1988 and printed in Australian Geomechanics, Special Issue devoted exclusively to the Conference.

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G:\GEONEWS

GEOTECHNICAL ASPECTS OF EARTHQUAKE ENGINEERING STATE-OF-THE-ART REPORT No 2

Geoffrey R. Martin

SUMMARY A broad "state of the practice" overview is presented of the many aspects of earthquake engineering for which geotechnical problems play a significant role. Selected observations are made on ground and earth structure response to earthquakes, ground settlement and liquefaction, embankment and slope stability, retaining structures and pile foundations.

1. INTRODUCTION

The behavior of soils during historic earthquakes has played a dramatic role in causing major structural damage, ground, embankment and slope failures, and disruption of port facilities and transportation systems. A well presented summary of the types of failures attributed to geotechnical factors during earthquakes and the corresponding engineering lessons learned, has been published by The Earthquake Engineering Research Institute (1986).

Efforts to develop improved understanding and predictive capabilities for evaluating geotechnically related earthquake behavior or performance increased significantly in the early 1960's. The thrust of this effort in the United States was fueled by dam safety programs, the catastrophic liquefaction failures arising from the 1964 Niigata and Alaskan earthquakes and nuclear power plant siting programs. The dam safety programs resulted in considerable improvement in the approaches to assess earth dam response and slope stability during earthquakes. The Niigata earthquake caused more than one billion dollars in damage, most of it due to widespread soil liquefaction. Liquefaction and ground failures generated by the great Alaskan earthquake triggered numerous landslides and ground stability problems that caused extensive damage and disrupted transportation routes for many months. The stringent safety regulations related to nuclear power plant siting resulted in considerable improvement in our understanding of site response and soil structure interaction during earthquakes.

The continued need for improved prediction capabilities were further reinforced by damage observations in more recent earthquakes such as:

- The 1971 San Fernando Earthquake, which focused attention on the seismic design bridges, and the near failure of an earth dam due to liquefaction.
- The 1977 Tangshan Earthquake in China again demonstrated the destructive effects of liquefaction failures.
- The 1978 Miyagi-Ken-Oki Earthquake in Japan which led to considerable damage to port and harbor structures such as bulkheads.
- The 1985 Mexico City Earthquake where the site response of the soft Mexico City clays was a major factor in inducing structural damage.
- The 1987 Bay of Plenty Earthquake in New Zealand which provided the first opportunity for New Zealand engineers to directly compare the liquefaction performance of New Zealand soils with the historic observations of other countries.

Whereas major advances in our ability to predict the earthquake performance of foundation systems and earth structures have been accomplished in the past three decades, the prediction process remains complex and challenging. The complexity is illustrated schematically in Figure 1. The initial challenge rests with the definition of the design earthquake event. This challenge requires consideration of the historic seismicity, the tectonics of the site region, attenuation relationships, and the overall risk adopted for the project. Discussions of these problems are beyond the scope of this paper.

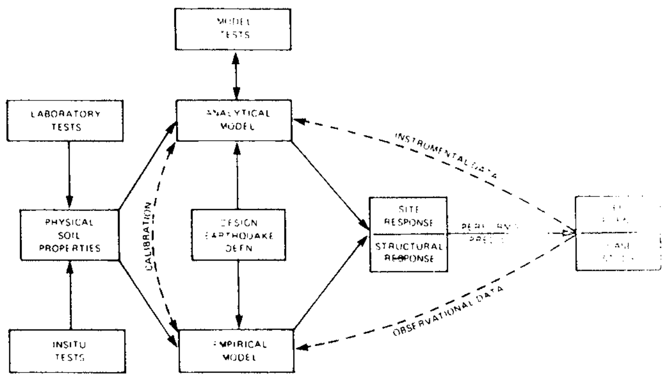

Figure 1 Performance Prediction

Performance predictions may be based on analytical models or on empirical models. Empirical models are based on observed earthquake performance linked to measured physical soil properties at the site. The input data on dynamic soil properties required for the models is obtained through laboratory or in situ tests or from generic data in the literature. Geotechnical performance predictions relate to site response (that is, the levels of ground acceleration as influenced by local site soils or prediction of liquefaction potential) or to assessment of performance of earth structures such as embankments, (slope deformations and stability) or foundation behavior.

Observational data from subsequent earthquake events lead to case studies, which in turn form the building blocks for improvements to empirical models. Alternatively, where instrumental data is available, this may be used to validate analytical models and hence further model improvements. In this respect, such earthquake performance predictions are generally "after the fact" predictions, as recorded ground acceleration data is needed to evaluate the success of the performance prediction model.

Clearly the problem of having to wait an uncertain length of time to validate the performance prediction is frustrating. Albiet there has been a recent emphasis on placing an increasing amount of geotechnically related instrumentation at project sites, the recent development of centrifuge modeling capabilities incorporating small shake tables is seen as a major advance. The use of centrifuge techniques provides the means for more rapid improvement and validation of analytical models.

In light of the conference theme of performance prediction, the paper primarily addresses the "state-of-the-practice" as opposed to the "state-of-the-art" and attempts to cover through a broad overview, a wide range of geotechnical problems. Discussion is also placed in a historic context where appropriate, to give the reader a broader perspective. With this emphasis, it is clearly difficult to do justice to the extensive research presently ongoing at many universities, which continually leads to improvements in the "state-of-the-art."

The discussions following provide selected observations of the "state-of-the-practice" for problems related to:

- site response
- ground settlement
- ground liquefaction
- response of earth structures
- embankment and slope stability
- retaining structures and
- pile foundations.

2. SITE RESPONSE

2.1 Background

Local site soil conditions (depth and stiffness of soil layers) may have a significant influence on the acceleration and frequency characteristics of earthquake ground motions, that is, the site response. Information on the characteristics of earthquake ground motions may be obtained from accelerographs recorded during past earthquakes such as that shown in Figure 2. Whereas damage directly associated with soil behavior, such as that due to ground settlement or liquefaction, is primarily dependent on the amplitude of accelerations and duration of the ground motion, the frequency characteristics of the earthquake have a significant influence on the response of structures founded on the site soils, and on structural damage. For example, studies of building damage caused by ground shaking during the 1967 Caracas Earthquake, indicate that the characteristics of ground motions resulting from different soil conditions were a significant factor in reconciling the observed damage (Seed and Alonzo, 1974).

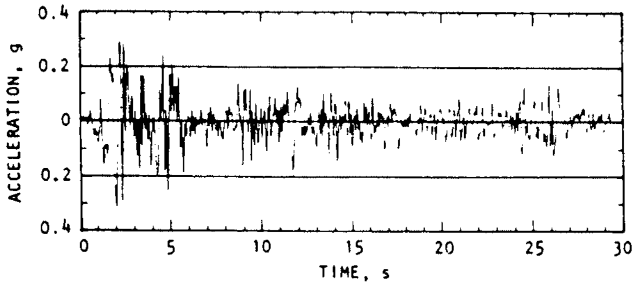

Figure 2 El Centro May 18, 1940, earthquake accelerogram (N.S. Component)

The combined influence of the amplitude of ground accelerations and their frequency components on the response of structures may be represented by acceleration response spectra such as those shown in Figure 3. Acceleration response spectra represent the maximum acceleration response induced by a given ground acceleration time history on a single-degree-of-freedom structure having different natural periods of vibration.

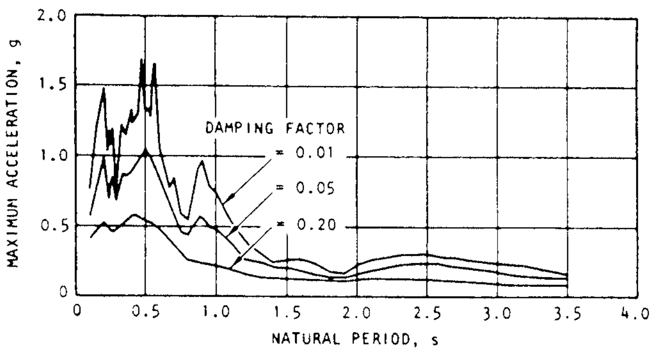

Figure 3 Acceleration response spectrum
El Centro May 18, 1940, earthquake (N.S. Component)

The characteristics of earthquake ground accelerations at a given site are influenced by a number of factors including: (1) the magnitude of the earthquake, (2) the source mechanisms of the earthquake including the direction and speed of fault rupturing, (3) the geologic characteristics of the wave transmission path from the source to the site, (4) the distance of the site from the source of energy release, (5) the geologic topography underlying site soils, and (6) local soil conditions (soil type, stiffness, layering and depth) at the site. Whereas the above complexity makes it very difficult to quantitatively evaluate the influence of local soil conditions as described above, groups of accelerograph records obtained for different soils conditions in a local region for a given earthquake, have shown that soil conditions can significantly influence the shape of the response spectrum. Figure 4, for example, shows an analysis of 104 spectra where average normalized spectral shapes are plotted for four different soil categories (Seed et al., 1976). For the soft to medium clay and sand sites, wide variations in spectral shape have been observed. For engineering purposes, particularly for the development of structural codes, such shapes are often simplified as shown in Figure 5 (Applied Technology Council, 1978).

With respect to peak ground accelerations, for rock, stiff soil and deep cohesionless soil sites,

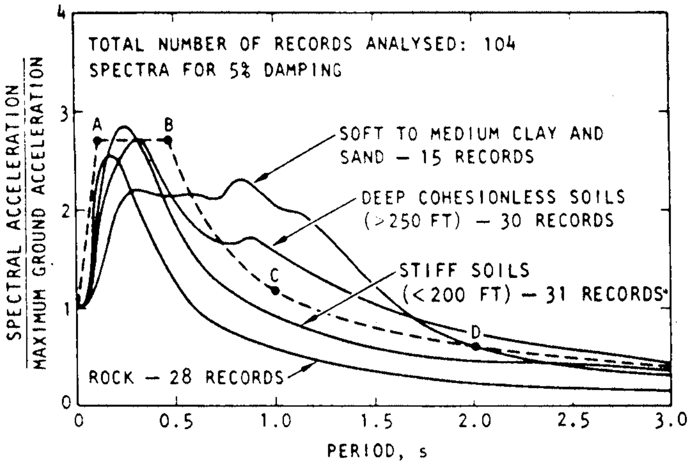

Figure 4 Average acceleration spectra for different site conditions (After Seed et. al., 1976)

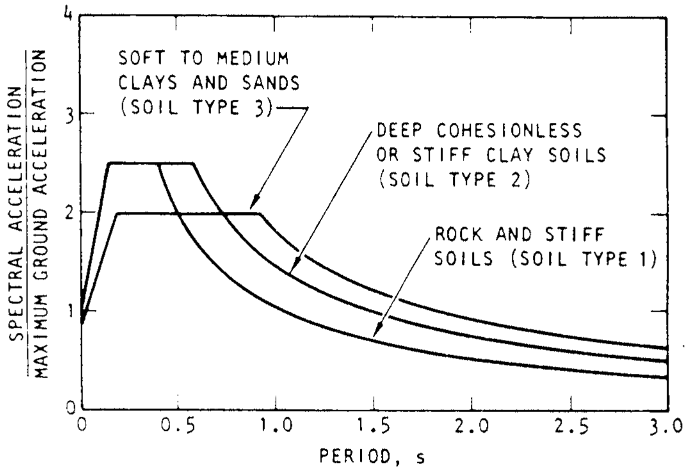

Figure 5 Normalized spectral curves for use in building code (Applied Technology Council, 1978)

studies have shown that there is no significant influence of soil conditions on peak acceleration values. In the case of soft to medium clay sites or for cohesionless soil sites of shallower depths, however, site conditions can influence maximum accelerations. For lower ground accelerations, say less than 0.1 g, amplification of peak ground acceleration with repsect to rock sites may be possible whereas for very high ground accelerations, say in excess of 0.3 g, soft clays may attenuate maximum ground accelerations as shearing stresses approach shear strengths and high energy dissipation occurs.

It must be emphasized that for any given local site soil condition, the characteristics of earthquake motion can vary significantly from average trends depending on the characteristics of the seismic source and its location relative to the source. Depending on a given situation, variations in ground motions due to source effects can overshadow the effects of local soil conditions or vice versa (Singh, 1985).

An example of such a situation was seen in Mexico City during the September 19, 1985 earthquake. The magnitude 8.1 event occurred in the subduction zone off the Pacific coast of Mexico, and caused catastrophic damage in Mexico City over 300 kilometers away. The La Villita accelerogram shown in Figure 6, was recorded 45 kilometers from the hypo-center and the rupture plane was thought to have passed 20 to 30 kilometers below the site. Peak accelerations were 0.12 g. The UNAM Mexico City record was recorded on a hard soil site. Peak ground accelerations were on the order of 0.03 g and showed a relatively strong response period of around 2

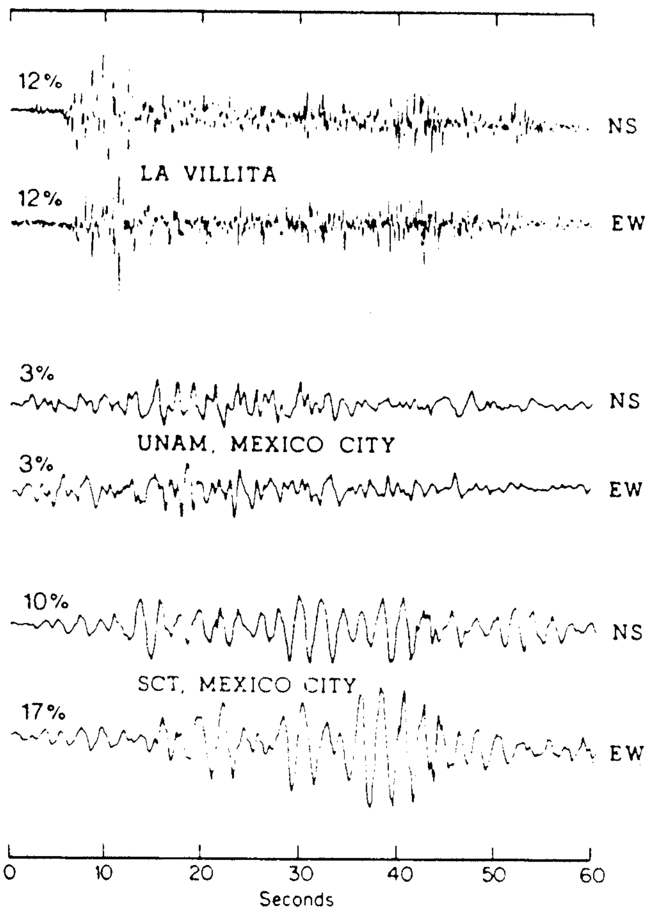

Figure 6 Accelerograms, Mexico City Earthquake, September 19, 1985 (After Mitchell, et al., 1986)

seconds. This relatively long period has been attributed to both the source mechanisms as well as the magnitude and distance from the Mexico City site. The SCT Mexico City record was recorded on the surface of soft clay lake bed deposits overlying dense sands and stiffer clays with shear wave velocities comparable to those of soft rock. The depths of the lake bed deposits were 35 to 40 meters with an average shear wave velocity on the order of 75 to 80 meters per second (Romo and Seed, 1986). Because the natural period of the site was also about 2 seconds, vertically propagating shear waves in the soft clay deposits were significantly amplified. The peak horizontal accelerations of incoming rock motions at depth (≃0.03g), reached peak accelerations of up to 0.17 g at the lake bed surface. The combination of strong shaking and long duration was devastating to longer period structures founded on the lake bed soils.

2.2 Site Response Analyses

Most current procedures for determining acceleration time histories and acceleration spectra for design place strong emphasis on the use of existing earthquake records. Records are selected which have been recorded at sites with similar soil conditions and for similar tectonic environments and magnitude-distance parameters. Scaling of existing records is often required to develop a sufficient number of representative earthquake records to establish smoothed spectra for design.

In many situations where unusual site soil conditions are encountered, such as shallow soil sites overlying bedrock or soft soil sites such as those in Mexico City, suitable earthquake records may not be available, particularly for very strong levels of earthquake ground shaking. Under these circumstances a site specific response analysis is often performed to evaluate the influence of local soil conditions on levels of ground acceleration and spectral characteristics. Such analyses may also be required where time histories of shearing stresses or strains are required for ground-settlement or liquefaction analyses as discussed in subsequent sections.

Whereas recorded ground accelerations reflect the complex superposition of may wave types (body and surface waves), in many cases, the higher frequency ground motions associated with maximum surface accelerations, may be attributed primarily to the vertical propagation of horizontal shear waves from underlying rock formations. For site soil deposits which essentially comprise horizontally stratified layers in the region of interest, analyses may be reduced to a one-dimensional problem. Assuming linear visco-elastic properties for soil layers, wave equation solutions such as those used by Kanai (1950) and Herrera and Rosenblueth (1965) have been used to analyze site response. For the simple case of a horizontal linear elastic layer of depth h having a shear wave velocity of v_s, the natural period of vibration equals $4h/v_s$. Hence, for example, a soft clay layer with a depth of 40 meters and a shear wave velocity of 80 meters per second has a natural period of 2 seconds (compare Mexico City lake bed deposits).

In situ shear wave velocities for site soils may be determined from geophysical measurements such as downhole or crosshole seismic surveys. Representative shear wave velocities for natural soil deposits in the Los Angeles basin region, for example, are shown in Figure 7 (Lew, et. al, 1981). The soft natural soils comprise Holocene and

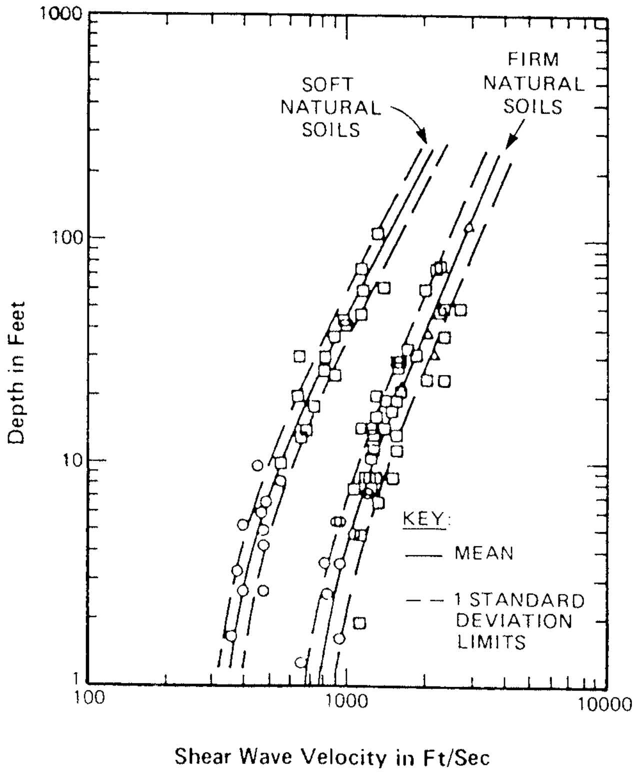

Figure 7 In situ shear wave velocity characteristic of natural soils (After Lew et al., 1981)

Pleistocene clays or very recent Holocene flood plain or delta deposits. The firm natural soils comprise high density Holocene or Pleistocene silty and gravelly sands.

Research conducted in the late 1960's identified the importance of including the nonlinear stress strain characteristics of soils in site response analyses for strong earthquake motions. The range of shearing strain amplitudes which may be expected during strong earthquake shaking vary between 10^{-3} to 10^{-1}% as shown in Figure 8, whereas shear wave velocities generated by downhole seismic tests involve shearing strains of less than 10^{-3}%.

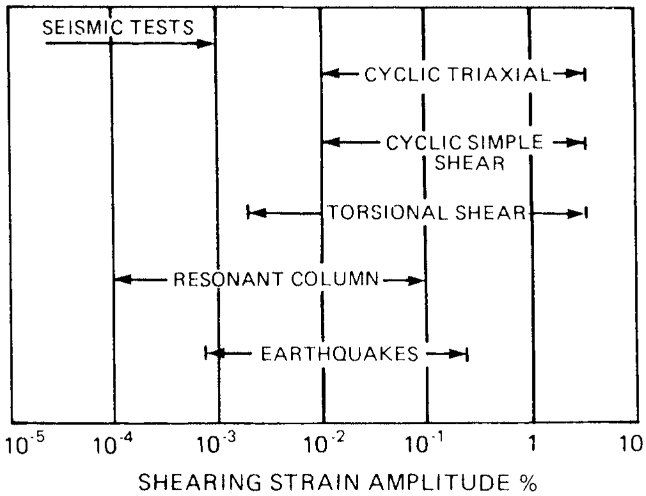

Figure 8 Ranges of shearing strain amplitude

Numerous laboratory studies have been undertaken to study the effects of shearing strain amplitude on the shear modulus G. (The shear wave velocity $v_s = (G/\rho)^{1/2}$ where ρ is the soil density). For vertically propagating shear waves, field loading conditions are best simulated by cyclic simple shear tests as illustrated in Figure 9. However, resonant column, torsional shear and cyclic triaxial tests are more commonly deployed.

Representative hysteresis loops illustrating the nonlinear cyclic behavior of soils are shown in

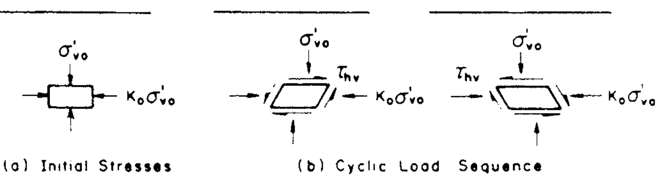

Figure 9 Idealized field loading conditions - vertically propogating shear waves

Figure 10. The variation of shear stiffness with shear strain may be represented by the variation of an equivalent linear shear modulus with strain amplitude. Normalized curves showing the typical range of equivalent linear modulus values as a function of shearing strain for a variety of soils are shown in Figure 11 along with the variation of equivalent viscous damping ratios. Representative values of shear modulus and damping factors for use in response analyses are widely available in the literature. Seed and Idriss (1970) and Hardin and Drnevich (1972) provide comprehensive data bases. A more recent study of shear modulus and damping ratios for sands and gravelly soils is presented by Seed, et. al. (1986). The variations of shear modulus with shear strain may be obtained from

cyclic laboratory tests on site specific soil samples or from databases such as those noted above. However, it is important to recognize that the effects of sample disturbance can have a significant effect on the magnitude of modulus values. In many cases, the most practical approach is to first determine in situ low strain shear wave velocities using downhole or crosshole seismic tests or using correlations between shear wave velocity and standard penetration blowcounts, such as those described by Seed, et. al. (1986). Normalized curves such as those shown in Figure 11 obtained from site specific laboratory tests or an appropriate database may then be keyed to the measured in situ low strain modulus values.

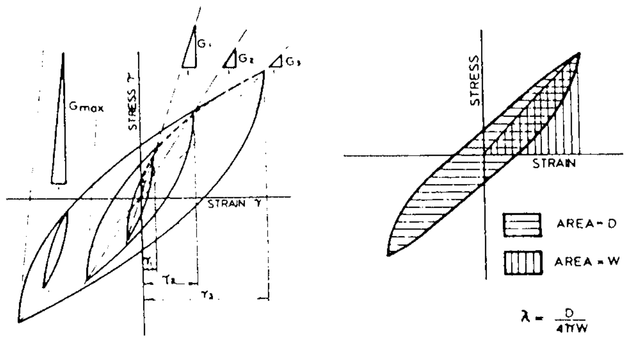

Figure 10 Nonlinear hysteretic shear strain behavior of soils

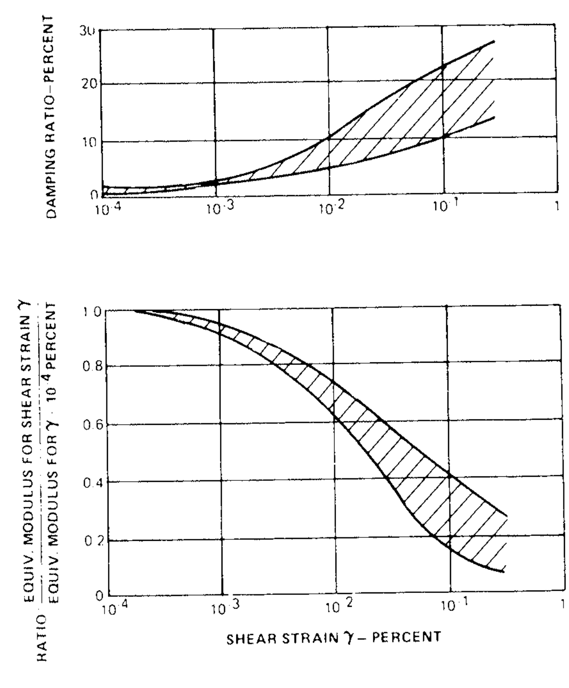

Figure 11 Equivalent linear modulus and damping characteristics of soils

The incorporation of nonlinear soil stiffness characteristics into site response analyses is most commonly performed by representing soil deposits as a series of horizontal layers connected by nonlinear shear springs and dashpot elements. The computer program SHAKE (Schnabel et. al, 1972) encompasses this approach and solves for earthquake induced site response using conventional dynamic analysis procedures coupled with an equivalent linear representation of soil moduli and damping values. The latter values are adjusted iteratively to be compatible with the computed strain amplitudes developed in the deposit. The program SHAKE remains in common use today to study earthquake site response. Representative examples indicating the method of application are given by Romo and Seed (1985) and Valera et. al. (1977). Typically, representative rock or firm ground accelerograms are used as input to the site soil profile, at a depth determined by site investigations.

The application of the SHAKE program to evaluate the site response in Mexico City has recently been described by Romo and Seed (1986). A site on the soft clay lake bed (the SCT Site) was analyzed using the acceleration records obtained on firm ground at the UNAM site (see Figure 6). Recorded and computed response spectra at the SCT site are shown in Figure 12. Agreement between response spectra shapes are very good, although computed spectral accelerations are significantly less than those measured. This has been attributed to evidence suggesting that as ground motions passed from the surrounding rock basin into the lake bed, they acquired a strong directional bias and increased amplitude (Finn and Nichols 1988).

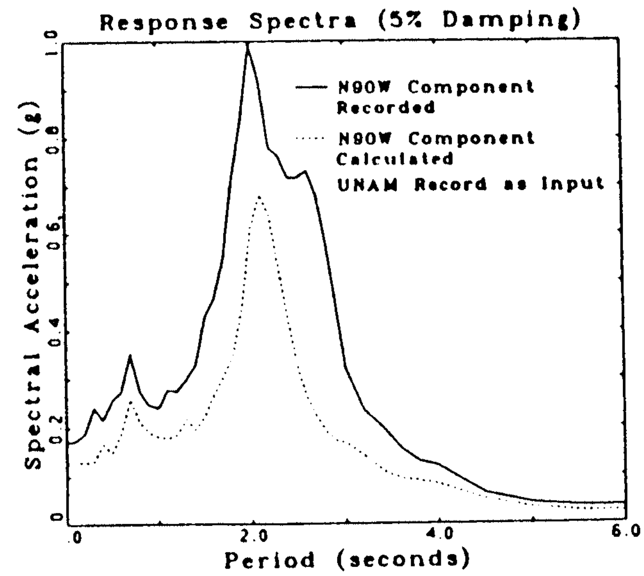

Figure 12 Recorded and computed spectra at SCT site: N90W component, Mexico City earthquake, September 19, 1985 (After Romo and Seed, 1986)

The equivalent linear elastic representation of the nonlinear behavior of soils utilized in the program SHAKE has limitations in some situations. For example, solutions may exhibit strong resonant effects when the predominant period of the earthquake ground motions corresponds to the fundamental period of the site. For soft soil sites and for very strong shaking, maximum shear stresses cannot be limited by the available shear strength of the soil. These difficulties are overcome by the use of nonlinear site response programs, where a true hysteretic nonlinear stress-strain formulation is used.

A number of nonlinear computer programs are available including CHARSOIL (Streeter, et al., 1973), DESRA (Finn, et. al. 1978), DESRA 2 (Lee and Finn, 1978), Taylor and Larkin (1978), and LASS (Dikmen and Ghaboussi, 1984). Typically the nonlinear formulation is based on an initial loading curve as shown in Figure 13. The nonlinear curves shown assume a hyperbolic formulation as used in the program DESRA. Unloading from any state is generally described by the Masing criteria. This implies that if the loading curve is $\tau = f(\gamma)$ then when unloading from (τ_r, γ_r), the unloading path is given by the equation $(\tau - \tau_r)/2 = f[(\gamma - \gamma_r)/2]$. The hysteresis loops developed from this formulation can very closely match given soil properties expressed as shear modulus versus strain amplitude and equivalent viscous damping factor versus strain amplitude curves, as illustrated by Finn, et al., (1978). An overview of nonlinear soil models and the use of Masing rules is given by Pyke (1979).

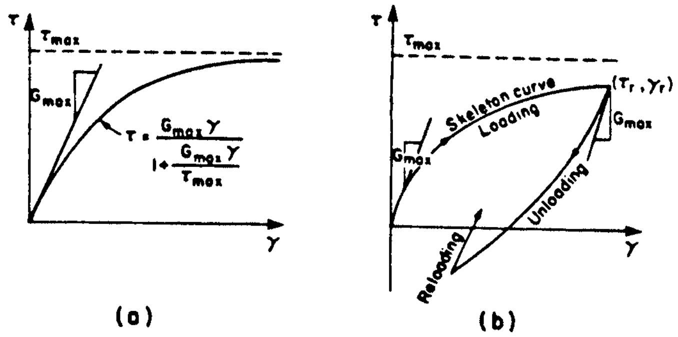

Figure 13 Nonlinear response models:
(a) initial loading curve;
(b) Masing stress strain curves for unloading and reloading

Figure 14 shows a comparison of acceleration response spectra computed by SHAKE, CHARSOIL and DESRA, for a 50 feet layer of sand subjected to the El Centro 1940 accelerogram scaled to 0.1 g. Whereas computed surface acceleration time histories were very similar, the SHAKE response spectrum shows greater magnification than the nonlinear programs in a period range near the natural site period.

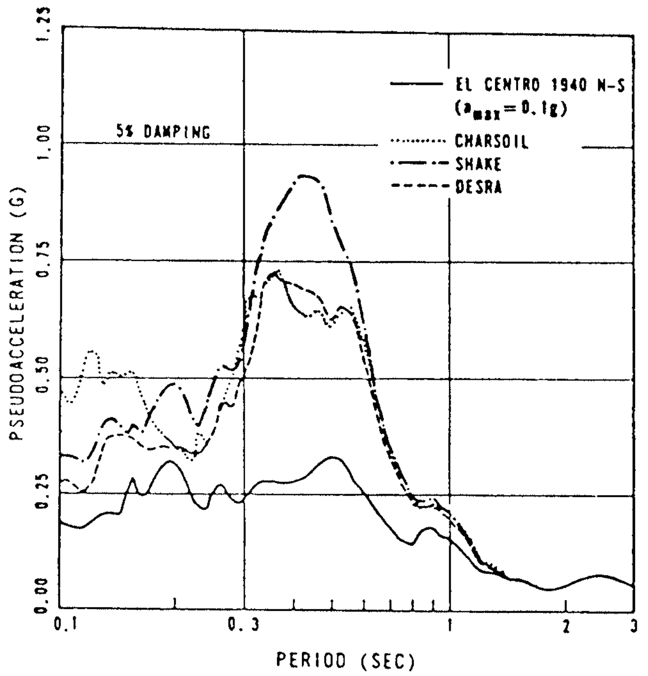

Figure 14 Comparison of acceleration response spectra computed by SHAKE, CHARSOIL, and DESRA - total stress analyses (After Finn et al., 1978)

For very soft clays, the shear modulus may degrade during repeated cycles of loading as shown in Figure 15. Modulus degradation can also be incorporated in nonlinear programs such as in the programs described by Singh et al., (1981), and Lam et al., (1978). Nonlinear deformation and degradation characteristics of clays under cyclic loading have been extensively studied by Taylor (1971). Modulus degradation may also occur in saturated sands during earthquake shaking due to increases in pore water pressures leading to potential liquefaction. Dynamic effective stress site response analyses for saturated sands are described in later sections.

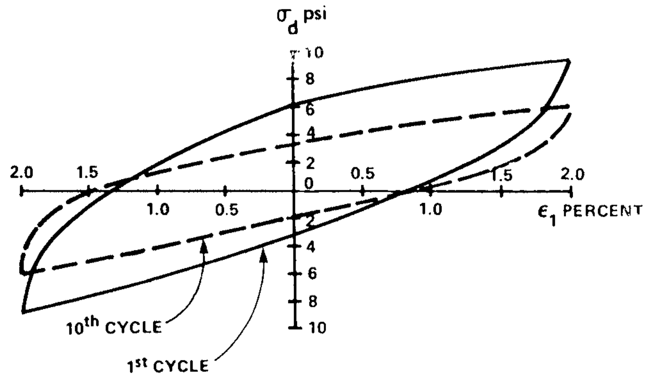

Figure 15 Nonlinear degrading hysteresis load-deformation behavior of saturated soft clays (After Idriss et al., 1978)

For deep soil sites, where a well defined boundary between the soil layers and underlying rock is not well defined at reasonably shallow depths, the concept of a transmitting boundary formulation (Joyner and Chen, 1975), may be used. The concept is illustrated in Figure 16. In this procedure, a firm ground motion recorded at the ground surface is used to provide input excitation at depth.

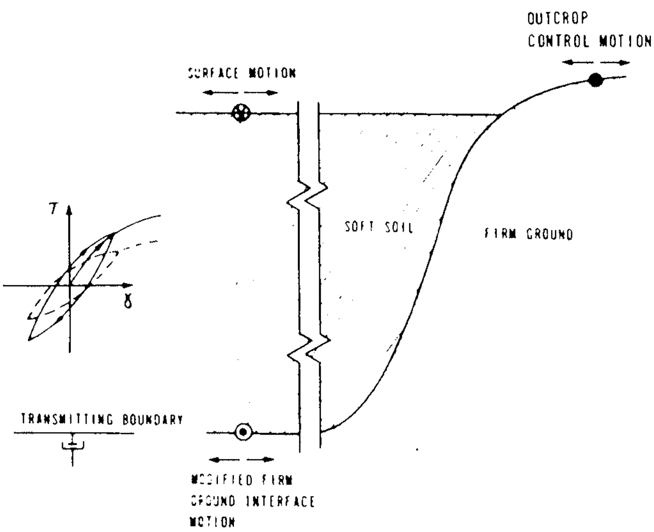

Figure 16 Transmitting boundary model

However, with the transmitting boundary the energy is allow to propagate up and down in proportion to the relative rigidity of the soil above and below the input point. The application of this procedure to evaluate the seismic site response of deep deposits of cohesive soils is described by Tsai et al., (1980).

In using one dimensional site response analyses for estimating the response of soil sites for which no representative strong motions are available, it must be remembered that there are a number of major uncertainties. One of the major uncertainties is in the selection of representative input motions. It is important that an adequate number of earthquake records be selected to bracket the required magnitude and epicentral distance. The motions should also cover the frequency range of interest and should have been generated by a source mechanism similar to that anticipated. For critical facilities such as offshore structures, the selection of 5 to 10 candidate motions for input is not uncommon. A large number of records allows a wide range in frequency content to be explored. When rock motion input records are used, it must be remembered that computed surface accelerations will not include the effects of surface waves. The latter waves may be particularly significant for longer period structures.

2.3 Centrifuge Model Tests

Recently developed capabilities for performing shaking table tests on a centrifuge offer new capabilities for performing validation of one dimensional codes. Nonlinear codes for very soft soils in particular are in need of validation at strong shaking levels where peak shear strengths maybe mobilized. In a centrifuge model, stresses and strains that exist in a full scale structure, can be produced at corresponding points in a centrifuge model by spinning at an angular speed to give a model centrifuged acceleration of ng where n is the model scale.

A schematic drawing of the centrifuge deployed by the California Institute of Technology is shown in Figure 17. During centrifuge spin up, the soil container swings into a horizontal position. Electrical and hydraulic power, air pressure and instrumentation signals to and from the container are conducted through electrical slip rings and rotating unions. Prescribed earthquake input motions may be delivered to the base of the centrifuge model by an electro hydraulic shaking apparatus capable of subjecting test models to vibration, which with proper scaling, can simulate the duration, intensity, and spectral shapes of real earthquake accelerograms. Input and measured surface accelerations in a test simulating a 12 meter deposit of sand, is shown in Figure 18 (Gohl and Finn, 1987). Applications of centrifuge testing for saturated sands are discussed in a following section on ground liquefaction.

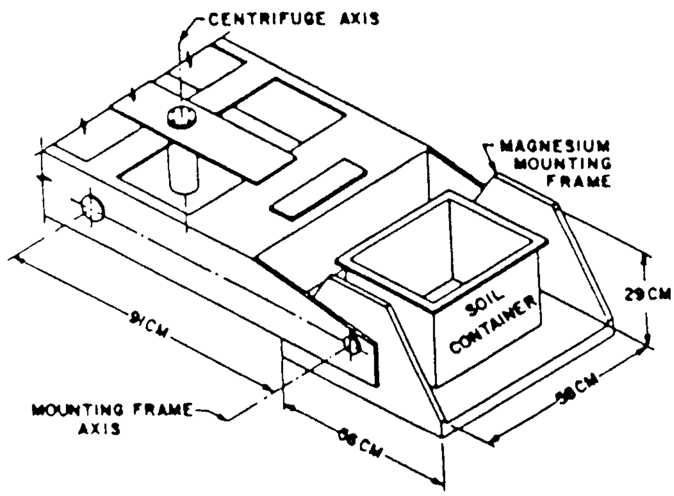

Figure 17 Schematic drawing of centrifuge arm (After Allard, 1983)

3.0 GROUND SETTLEMENT

3.1 Background

Vibration has long been recognized as an effective means for compacting cohesionless soils. Hence, it is not unexpected that significant ground settlements have occurred as a result of ground shaking produced by strong earthquakes. Clearly the looseness and fabric of the soil structure, the thickness of cohesionless soil layers and the intensity and duration of strong ground shaking can be expected to be the principal factors governing the amount of ground settlement. Large earthquakes with long durations of shaking such as the 1906 San Francisco and 1964 Alaska events, produced about a meter of settlement for deep alluvial strata. Settlements of 5 to 30 centimeters have commonly been observed. Associated differential settlements may cause damage to building and engineering structures and disrupt utilities such as buried pipelines. Settlements of approach fills to bridges can also disrupt transportation networks.

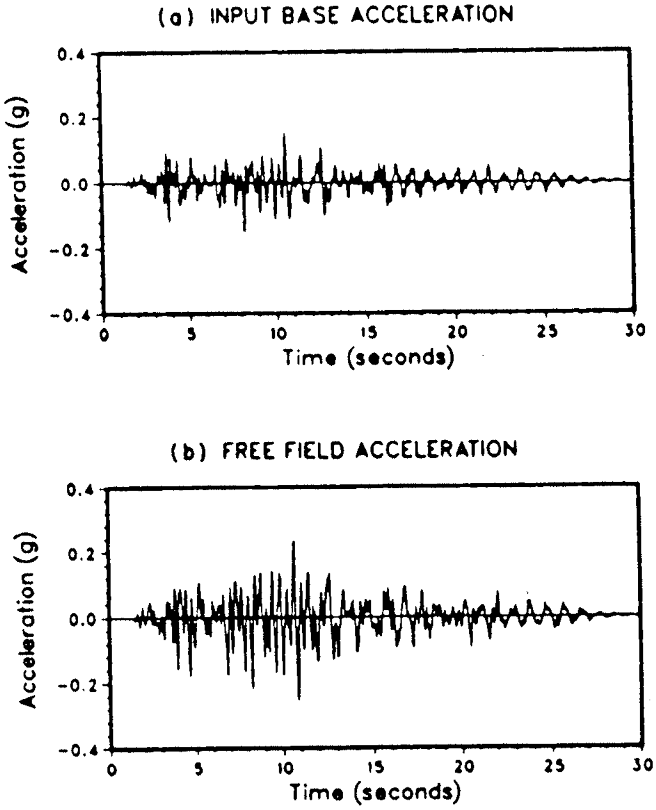

Figure 18 Acceleration time history centrifuge model test on a sand layer (After Gohl and Finn, 1987)

In many cases where settlements have occurred, the cohesionless soils have been saturated. If no significant drainage occurs during earthquake ground shaking, progressive build-up in pore water pressures may occur, leading to potential liquefaction. When shaking ceases the excess pore pressures dissipate and settlements occur as the soils reconsolidate. The subject of liquefaction and the related post earthquake settlement of saturated soils is discussed further in the next section. In this section, attention is focused on settlement of nonsaturated cohesionless soils.

3.2 Laboratory Studies

Several experimental studies have shown that vertical vibration causes little or no densification of sand samples until the vertical acceleration exceeds one g. Hence, it would appear reasonable that settlement is primarily due to the horizontal components of ground shaking which can be reproduced in the laboratory by cyclic simple shear tests. Tests results reported by Silver and Seed

(1971) have shown that volumetric strains or settlements which result from cyclic shearing are a function of (1) the relative density of the soil, (2) the magnitude of the cyclic shear strains, and (3) the number of strain cycles. Figure 19 for example, shows the effect of relative density on settlement after 10 cycles of constant cyclic shear strain. These tests were conducted using a modified GEONOR simple shear device which utilizes wire bound rubber membranes to confine the sample laterally. As lateral strains are zero, measured vertical strains during tests represent the total volumetric strain. It was further found that for a given density and number of cycles, the settlement depended only on the cyclic shear strain amplitude and was independent of the vertical stress as shown in Figure 20. Tests also show that for a given density and number of cycles, volumetric strains are reasonably proportional to the cyclic shear strain amplitude for strain amplitudes less than about 0.3 %. Cyclic simple shear tests conducted by Youd (1972) confirmed the above findings and also showed that settlements are independent of the frequency of loading and are similar for tests on both air dried and drained saturated samples.

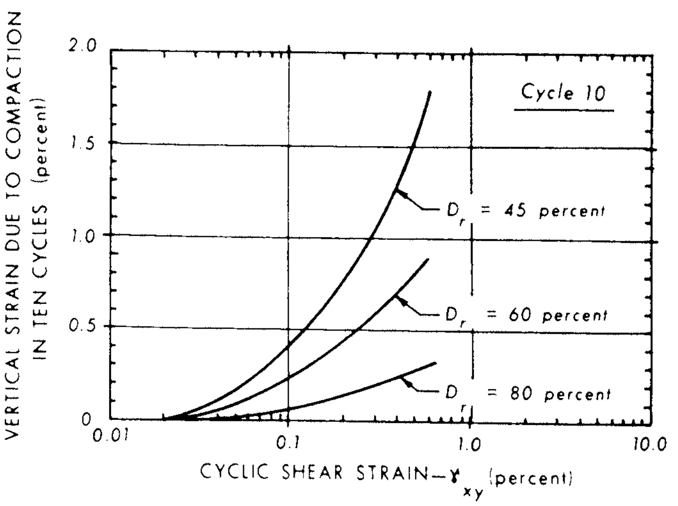

Figure 19 Effect of relative density on settlement in 10 cycles (After Silver and Seed, 1971)

The above data was obtained from uniform cyclic strain amplitude tests. For nonuniform earthquake loading, the volumetric strain increments per cycle need to be computed for nonuniform cyclic strain amplitude sequences. The establishment of such a method together with an experimental study is described by Martin, et. al. (1975). Cyclic simple shear tests with uniform strain amplitude were first performed to establish the basic volume change characteristics as shown in Figure 21. The method assumes that the volumetric strain increment for a given cycle depends on the total accumulated volumetric strain from previous cycles and the magnitude of the cyclic shear strain amplitude for the current cycle. Thus the accumulated volumetric strain at any time may be envisaged to represent a strain history parameter, and a series of nonuniform cyclic strain amplitude tests confirmed the hypothesis. To enable such a procedure to be conveniently used for numerical calculations, the curves shown in Figure 20 may be replotted in terms of incremental volumetric change or settlement per cycle $\Delta\varepsilon_{vd}$, as a function of cyclic shear strain amplitude γ, and accumulated volumetric strain ε_{vd}, as shown in Figure 22. To permit these curves to be utilized by computer they may be fitted to an analytical function of the form:

$$\Delta\varepsilon_{vd} = C_1 (\gamma - C_2 \varepsilon_{vd}) + \frac{C_3 \varepsilon_{vd}^2}{\gamma + C_4 \varepsilon_{vd}} \quad (1)$$

The evaluation of the four constants C_1 through C_4 using data plotted from 2 or 3 constant strain amplitude cyclic tests, completely defines the volume change or settlement behavior under cyclic simple shear loading. The above constitutive relationship is used in the nonlinear one-dimensional site response programs DESRA (Finn et. al., 1976), and DESRA-2 (Lee and Finn, 1978). DESRA-2 includes a transmitting boundary.

Measured settlement characteristics of sands from cyclic simple shear tests can be used to compute earthquake induced settlements from one-dimensional site response analyses. For each layer of a soil deposit, the time history of shear strain may be computed and either represented by an equivalent number of uniform strain cycles to compute settlements from cyclic simple shear stress results, or alternatively, settlements may be computed directly using incremental volumetric strain curves (Figure 22) in conjunction with the computed shear strain time histories.

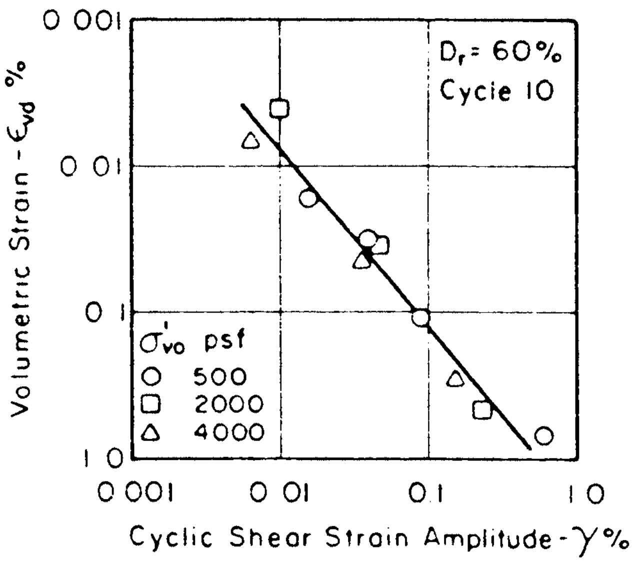

Figure 20 Effect of vertical stress and strain amplitude on settlement in 10 cycles (After Silver and Seed, 1971)

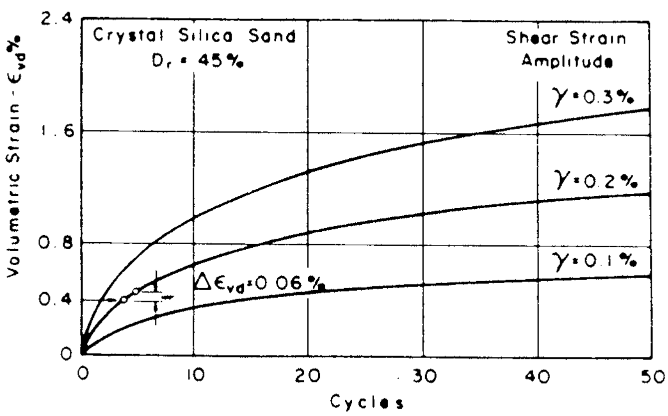

Figure 21 Volumetric strain curves for constant cyclic shear strain amplitude tests (After Martin et al., 1975)

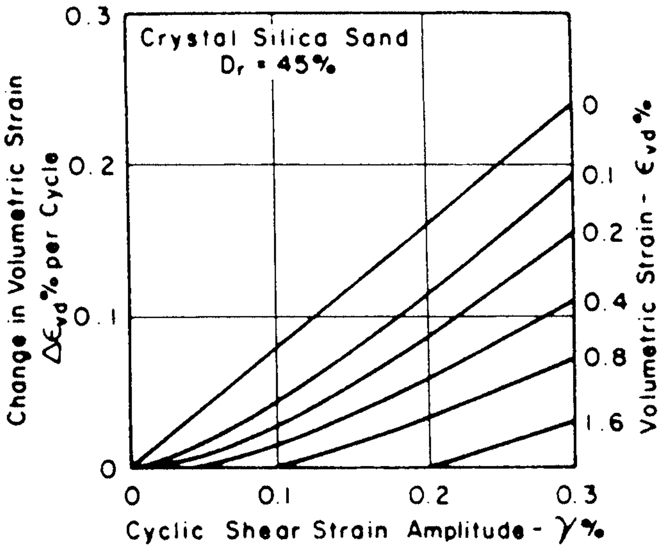

Figure 22 Incremental volumetric strain curves (After Martin et al., 1975)

3.3 Effects of Multi-Directional Shaking

The test results described above were obtained for cyclic simple shear in one direction only. However, actual earthquakes involve multi-directional shaking and the question might be posed as to how this influences earthquake induced ground settlement.

Settlement measurements made at the Jensen Filtration Plant following the 1971 San Fernando earthquake provided a valuable case history for study. A cross section through the main control building of the Plant is shown in Figure 23. The fill comprised a uniformly compacted well graded clayey sand to depths of about 50 feet. Five inches of settlement were observed; however, lateral movement of the site also occurred due to liquefaction in the underlying alluvial formation. As a result only 3-1/2 to 4 inches of settlement are attributed to compaction of the fill soils during the earthquake.

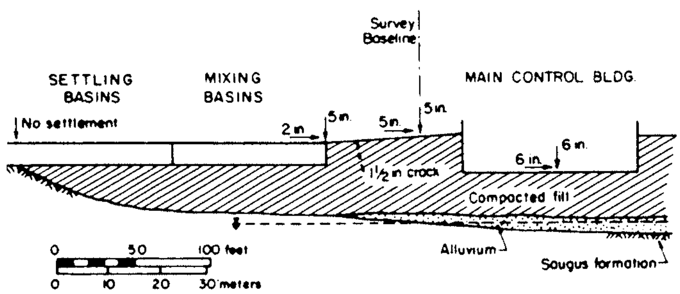

Figure 23 Settlement and displacements at section through main control building, Jensen Filtration Plant (After Pyke et al., 1975)

A detailed study of the effects of multi-directional shaking and of the settlement of the Jensen Filtration Plant is reported by Pyke et al. (1975). In this study a series of strain controlled cyclic were performed on recompacted samples of the fill material. Representative results are shown in Figure 24.

Estimated firm ground earthquake base motions at a depth of 55 feet were obtained from the Pacoima Dam record scaled to yield the maximum acceleration of 0.6 g. The computed time histories of shear strain at a depth of 25 feet are shown in Figure 25 and were obtained using the computer program SHAKE. Assuming 10 cycles of uniform shear strain equal to 2/3 of the maximum shear strain, integration of computed settlements for each layer gave a total settlement of about 1.6 inches, significantly less than the observed settlement of 3-1/2 to 4 inches.

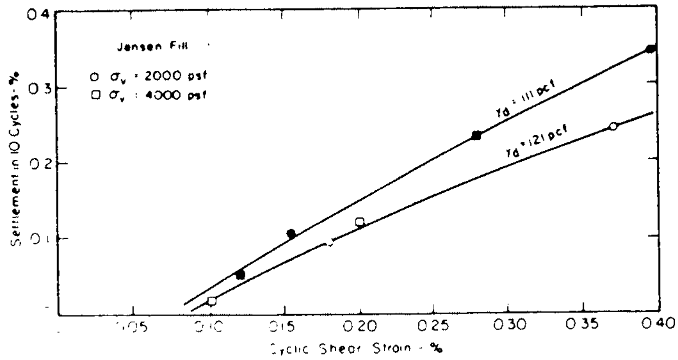

Figure 24 Settlement of Jensen fill as function of cyclic shear strain (After Pyke et al., 1975)

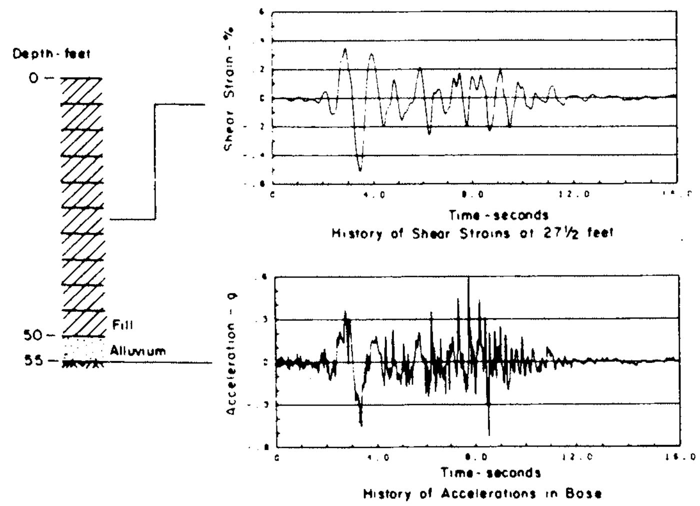

Figure 25 Analysis of soil response, Jensen Filtration Plant (After Pyke et al., 1975)

To investigate the possibility of the effects of multi-directional shaking increasing the settlement, shaking table tests were undertaken and are reported by Pyke, et al. (1975). Multi-directional ground motions were applied to a layer of sand on a small shaking table by mounting the table on an underlying second larger shaking table. The large 24 feet square University of California, Berkeley, shaking table was used and is capable of producing motions in one horizontal direction and a vertical direction. Motion in the second horizontal direction was added by mounting the smaller shaking table transversely on the large table.

A program of systematic experiments showed that settlements caused by combined horizontal motions are equal to the sum of the settlements caused by the individual components of horizontal motions acting separately. While vertical accelerations of less than 1 g cause no settlement when applied alone, it was found that vertical accelerations superimposed on horizontal accelerations caused a marked increase in the settlement. For the Jensen fill material, it was concluded that the originally computed settlement of 1.3 inches would be approximately tripled to yield a value of 3.9 inches when allowance was made for the 3 components of earthquake motions. The latter figure compares more favorably with the observed settlements.

4.0 GROUND LIQUEFACTION

4.1 Background

While liquefaction has been reported in numerous earthquakes, the extensive damage resulting from the 1964 earthquakes in Niigata, Japan, and in Alaska, was clearly the cornerstone for initiating major research efforts to better understand the phenomona and to develop prediction methodologies. A representative soil profile in Niigata is shown in Figure 26 and is characterized by loose sands with blow counts of less than 15 extending to depths of up to 40 feet. The epicenter of the Niigata earthquake (Magnitude 7.5) was about 55 miles from Niigata. The maximum recorded ground accelerations were about 0.16 g. Extensive liquefaction in the low lying area of the city occured with water flowing out of cracks and boils during and immediately following the earthquake.

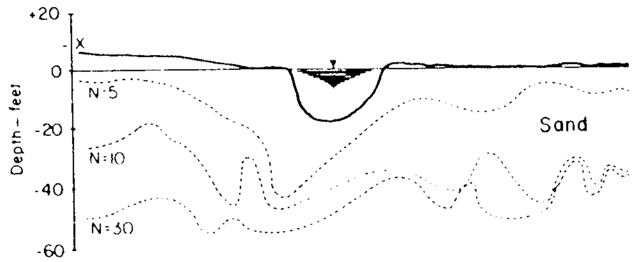

Figure 26 Representative Niigata soil profile (After Seed and Idriss, 1967)

Many structures settled more than three feet and thousands of buildings collapsed or suffered major damage.

The published research literature on liquefaction since the initiation of more intense research in the 1960's is very extensive, and major advances in our understanding have occurred. Perhaps the most comprehensive research document on liquefaction is the U.S. National Research Council publication on "Liquefaction of Soils During Earthquakes" (1985). The report reviews the state-of-the-knowledge on the causes and effects of liquefaction of soils during earthquakes, documents the state-of-the-art of analyses, and recommends future directions for research.

The brief overview in this section, focuses on the problem of earthquake induced liquefaction of level ground. Problems related to embankment or slope stability associated with pore pressure buildup will be addressed in later sections.

4.2 Laboratory Studies

A basic understanding of liquefaction phenomona can be gained from tests on saturated sand samples subjected to cyclic stresses in laboratory test apparatus. In these tests, samples of saturated sand are consolidated under a confining pressure and then subjected to a sequence of constant amplitude cyclic shear stresses under undrained conditions. This approach was first used by Seed and Lee (1966) using cyclic triaxial equipment. However, tests using cyclic simple shear equipment, such as those described by Peacock and Seed (1968) and Finn et. al. (1971), more closely simulate field stress conditions associated with vertically propagating shear waves. Cyclic torsional shear tests on hollow cylindrical samples such as those described by Ishihara (1985), also simulate field loading conditions.

A typical laboratory test record on loose sand is shown in Figure 27. The pore water pressure builds up steadily as cyclic stress is applied and eventually approaches a value equal to the initially applied confining pressure, resulting in large cyclic shearing strains. The stress path and stress strain curves for the above test are illustrated in Figure 28. For loose sands, once the state of initial liquefaction is reached, large deformations occur rapidly with shear strains as high as 20%. When the sand undergoes unlimited deformation without mobilizing significant resistance to deformation, the sand is said to have liquefied.

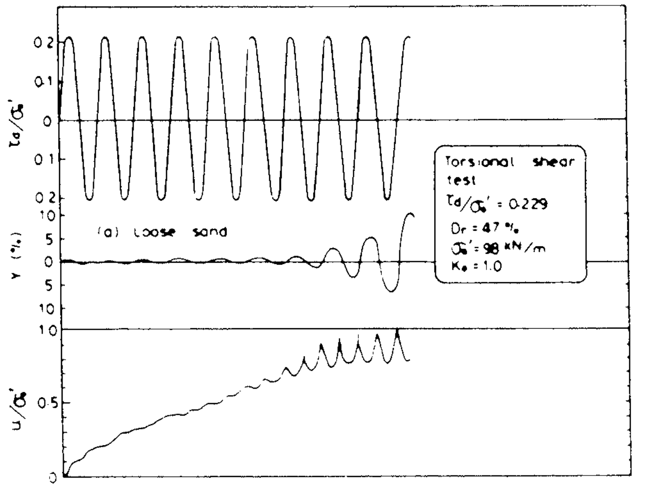

Figure 27 Cyclic torsional shear test for saturated loose sand (After Ishihara, 1985)

For dense sands, the pore water pressure increases more slowly during cyclic loading. As peak pore pressure values become equal to the initial confining pressure, the large shear strains mobilize significant dilation of the sand structure, and significant cyclic reductions in pore pressure to maintain constant volume. The consequent mobilization of undrained strength causes the hysteresis loop shapes as shown in Figure 29. Shear strain amplitudes never become larger than a certain limit. (Referred to as cyclic mobility by Castro, 1975, and limiting strain potential by Seed, 1979). For loose sands, the state of initial liquefaction coincides with the incipient state of liquefaction and is accompanied by large deformations. For dense sands, a state of initial liquefaction does not produce large deformations but rather a degree of softening takes place.

Figure 30 shows representative simple shear liquefaction test results, expressed as the cyclic stress ratio (τ/σ'_0) required to cause initial liquefaction for varying relative densities and load cycles, where τ=the cyclic shear stress amplitude and σ'_0=the initial vertical effective stress. Such curves are commonly referred to as liquefaction strength or liquefaction resistance curves. Figure 31 illustrates the concept of limiting shear strain. The curves show the cyclic stress ratio required to cause initial liquefaction and limiting strains in 10 load cycles as a function of relative density.

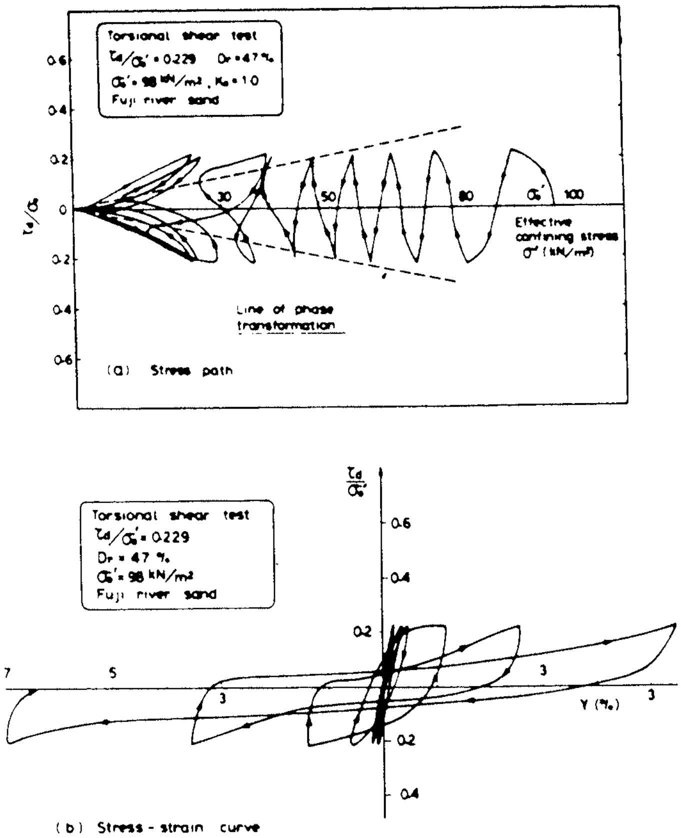

Figure 28 Stress path and stress-strain curve for loose sand obtained from cyclic torsion shear test (After Ishihara, 1985)

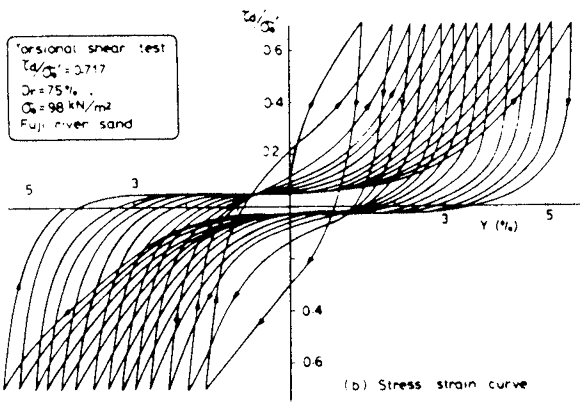

Figure 29 Stress-strain curve for dense sand obtained from cyclic torsion shear test (After Ishihara, 1985

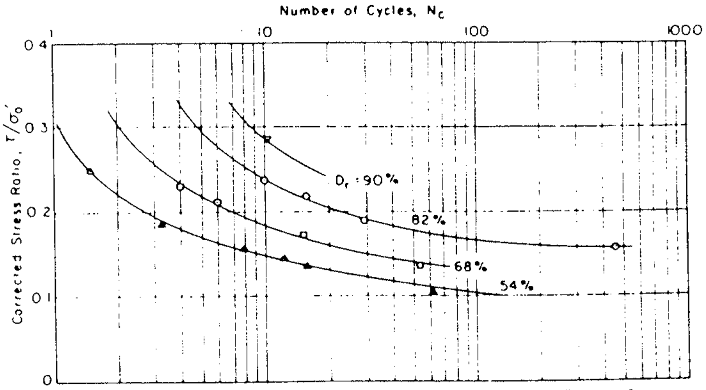

Figure 30 Stress ratio versus number of cycles to initial liquefaction (After DeAlba et. al., 1976

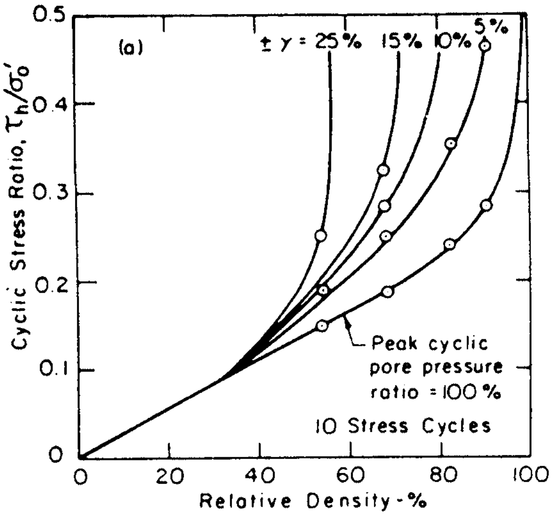

Figure 31 Limiting shear strain during liquefaction tests (After Seed, 1976)

To evaluate liquefaction potential in the field from the results of laboratory tests, it is necessary to construct a field liquefaction strength curve from the laboratory test results. This requires consideration of a number of factors as summarized by Seed and Idriss (1982). These factors include: (1) The effects of sample disturbance. It is very difficult to obtain undisturbed samples of most sands unless expensive freezing techniques are used prior to sampling. Laboratory test data must hence be interpreted with a degree of judgement to allow for the possible effects of sample disturbance. (2) The effects of multi-directional shaking. Analyses have shown that strength ratios from laboratory tests should be decreased by about 10% to provide results representative of multi-directional shear conditions. (3) The conversion of cyclic triaxial test data. Whereas simple shear tests provide a reasonable simulation of earthquake loading conditions, the more common isotropically consolidated cyclic triaxial test does not reproduce field stress paths. Correction factors to measured cyclic deviator stress ratios are needed to convert to cyclic shear stress ratios in the field. These factors can vary from about 0.5 to 1 depending on field K_0 conditions and the type of soil (silts or sands).

Seed and Idriss (1971) developed a simplified procedure for evaluating field liquefaction potential based on in situ liquefaction strength curves determined from a laboratory test program. A simple expression was developed for evaluating maximum earthquake induced cyclic shearing stress (for depths of less than 40 feet) as a function of peak ground acceleration, the soil density and a depth factor. As the actual time histories of shear stress at any point in a soil deposit will have an irregular form it is necessary to determine an equivalent number of cycles of a uniform cyclic shear stress amplitude. The average uniform cyclic shear stress amplitude is taken as 65% of the maximum cyclic shear stress, and the number of cycles of loading assumed a function of the earthquake magnitude, as shown in the table below.

Earthquake Magnitude	No. Of Significant Stress Cycles, N
5-1/4	2-3
6	5
6-3/4	10
7-1/2	15
8-1/2	26

Hence a direct indication of liquefaction potential may be obtained by comparing the cyclic shear stress amplitude to cause liquefaction (for the number of cycles associated with the earthquake magnitude under consideration), with the average earthquake induced cyclic shearing stress amplitude for the depth under question. This concept is illustrated schematically in Figure 32.

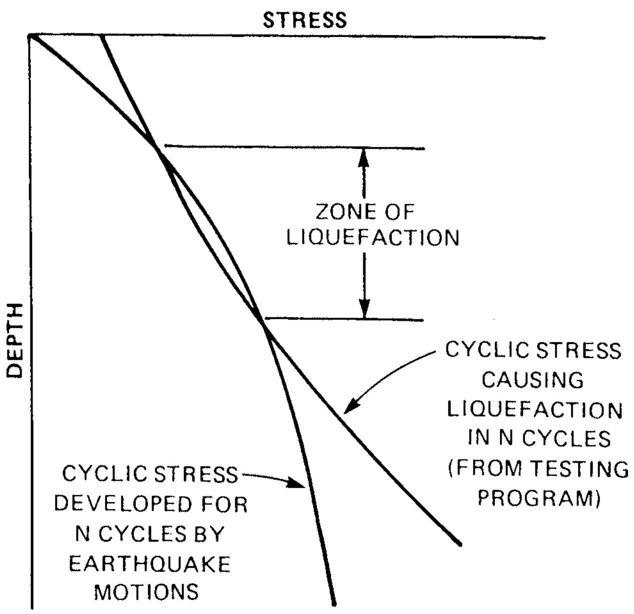

Figure 32 Method of evaluating liquefaction potential

Whereas case histories using the above approach have resulted in reasonable correlations with observed performance, (Seed and Idriss, 1982), the difficulties related to converting laboratory test results to equivalent field liquefaction strengths leads to considerable uncertainty for design. Hence in the present state-of-the-practice, much greater emphasis is placed on the use of empirical correlations between liquefaction strengths in the field and in situ test data obtained from standard penetration tests or cone penetration tests. These correlations are briefly described below.

4.3 Liquefaction Potential - The Empirical Approach

Questions about the accuracy of determining field liquefaction resistance from laboratory tests may be overcome by the use of an empirical observational approach, where known values of average cyclic stress ratios from earthquake events are correlated with a convenient index obtained from in situ measurements of soil properties. Japanese engineers were the first to use this procedure following the Niigata earthquake, where standard penetration test (SPT) results were used as a means of differentiating between those sands which liquefied and those which did not. This procedure has been progressively refined since that time, as reported by Ishihara, (1985), and the progressive research by Seed and his research colleagues (Seed et at., 1983; Seed et al., 1985).

A recently published form of the SPT correlations, is shown in Figure 33 for sands with D_{50}>0.25mm and for earthquakes with a magnitude of about 7.5. Each point corresponds to one boring record and some particular earthquake. The intensity of ground motion at the sites is represented by the equivalent uniform stress ratio evaluated at the depth in the particular deposit associated with liquefaction. The resistance of the soil is represented by modified SPT blow counts. The curve separating those points which liquefied and those which did not, defines the liquefaction resistance of site soils for any measured modified penetration resistance. Figure 34 shows the generalized form of such curves for a range of earthquake magnitudes. As increasing magnitude is associated with an increasing number of equivalent uniform cycles, liquefaction resistance reduces with increases in earthquake magnitude.

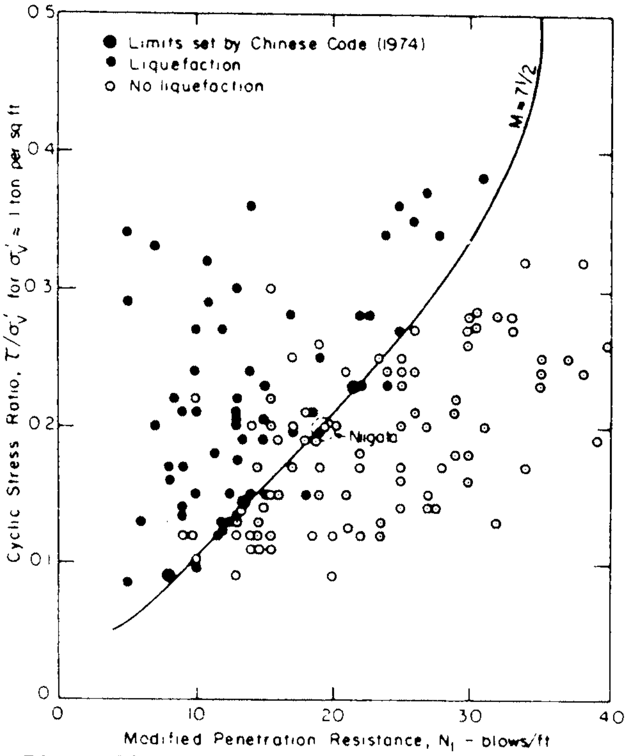

Figure 33 Correlation between cyclic stress ratios and standard Penetration resistance for sands (D_{50}>0.25mm) (After Seed et. al, 1983)

As SPT procedures have not been standardized and there are important differences between the procedures used in different countries, it is clearly important to correct for these differences when using data such as that shown in Figures 33 and 34. The SPT procedure for use in liquefaction correlations and correction factors for other procedures are documented by Seed et. al. (1985).

The empirical procedures described above for assessing liquefaction resistance or liquefaction potential for a given earthquake are now widely used by engineers. Corrections to the empirical relationships to account for the percentages of fines encountered in silty sands, are also available, Seed et al., (1985). However, it is still not possible to evaluate the likelihood of liquefaction of a silty sand or silt with the same confidence as for a clean sand and in many cases, a cyclic laboratory test program is recommended to gather additional confidence.

The use of the cone penetrometer test (CPT) as an empirical index has also received increasing attention in recent years and has the advantage in providing a more consistent and continuous evaluation of soil properties with depth then the SPT. As little CPT data is available at sites where earthquake liquefaction has occurred, the direct development of correlation charts is not possible at this time. However, the use of correlations between CPT resistance and SPT blowcount, enables use of the standard blowcount correlation charts for

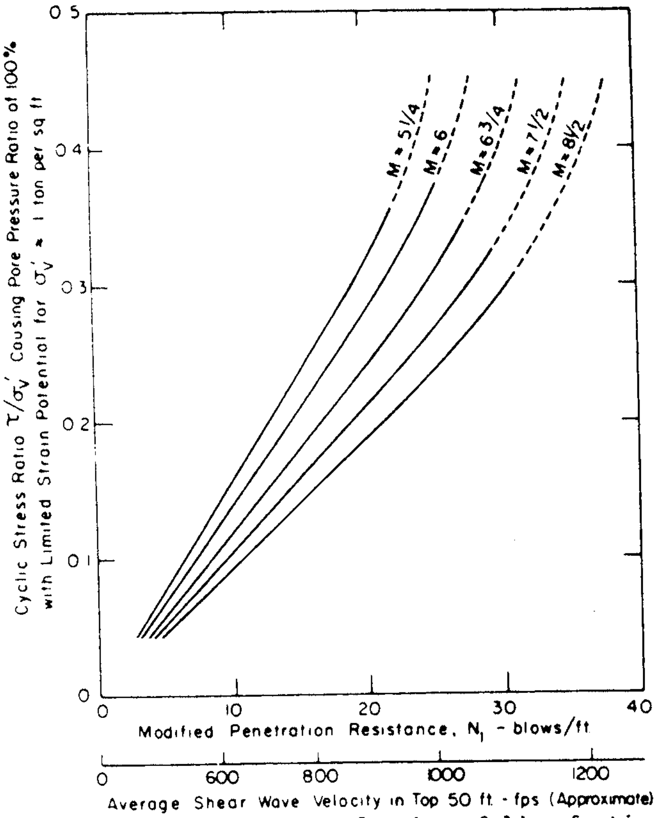

Figure 34 Chart for evaluation of liquefaction potential of sands (After Seed and Idriss, 1983)

liquefaction resistance. Research on correlations between cone resistance and SPT blowcount have been presented by Douglas et. al. (1981), Robertson and Campanella (1985) and Seed and DeAlba (1986).

Correlations between SPT and CPT at a site which liquefied during the 1968 Inangahua earthquake, have been described by Berrill et al., (1987). As the CPT is a more sensitive device for locating thin strata of loose sand within a heterogeneous deposit, it also has the ability to provide better control over field methods used to improve the resistance of in situ soils to liquefaction.

A number of case histories have been published where empirical procedures for evaluating liquefaction resistance have been validated. For example, publications by Clough and Chameau (1983), Youd and Bennett (1983), and Seed et. al. (1983). Other studies have also been conducted and are summarized by Ishihara (1985), for conditions that relate to the occurrence or nonoccurence of ground damage arising from liquefaction. These studies suggest that the occurrence of ground damage depends on the thickness of the liquefiable sand layer, the thickness of the overlying surface layer and the level of ground acceleration. Figure 35 shows proposed boundary curves distinguishing between conditions leading to ground damage and conditions where ground damage will not occur. It is clear that the thickness of the nonliquefiable surface layer can have a significant effect in preventing ground damage.

4.4 Liquefaction Potential - A Mechanistic Analytical Approach

A qualitative understanding of the mechanism underlying progressive pore water increases during cyclic loading of sand is necessary to develop a reliable analytical approach for assessing field liquefaction potential. A framework for such an understanding was developed by Martin et. al., (1975). As discussed in Section 3, the application of cyclic shearing stresses to dry sands, results in progressive decreases in volume, even in the case of relatively dense sands. For saturated sands, if drainage is unable to occur during the time span of the loading sequence, the induced volume changes must be compensated for by an equal elastic volume rebound at grain contacts as shown schematically in Figure 36. The amount of elastic rebound required to maintain constant volume in effect defines the progressive increment in pore water pressure during a loading cycle. The practical application of this concept hinges on the fact that values of the incremental volume change per cycle during drained cyclic tests have been found to be independent of vertical stress. Therefore the mechanistic theory in its simplest form, implies that if a sample of saturated sand loaded to an initial vertical effective stress of σ'_{vo} has a recoverable volumetric strain of ε_{vr}, then liquefaction will occur under an applied cyclic strain history that produces a total volumetric strain $\varepsilon_{vd} = \varepsilon_{vr}$ under drained conditions.

On an incremental basis this may be expressed by an equation of the form:

$$\Delta u = \bar{E}_r \, \Delta\varepsilon_{vd} \qquad (2)$$

where $\Delta\varepsilon_{vd}$ = the incremental volume change occurring during a drained cycle of loading (equation (1)) of shear strain amplitude γ.

$\bar{E}r$ = the secant rebound modulus corresponding to the level of vertical effective stress σ_v'.

Δu = The pore pressure increment occurring in an undrained cycle of loading of shear strain amplitude γ.

An analysis of one dimensional unloading characteristics of sands (Martin et al, 1975) has shown that $\bar{E}_r$ may be expressed as a function of the initial vertical effective stress σ_{vo}' and the current vertical effective stress σ_v'. Representative unloading curves are shown in Figure 37 both before and after cyclic loading. Elastic rebound after cyclic loading has been observed to be slightly greater than the virgin rebound. Hence, a knowledge of the material constants governing volume change under cyclic loading and elastic rebound, enable the complete time history of pore pressure increases to be determined for a given cyclic shearing strain history.

The mechanistic approach described above allows the computation of progressive pore water pressure increases during nonuniform load cycles. By incorporating the physical relationships between shearing strain, volume change and pore pressure in a nonlinear site response analysis, a coupled effective stress analysis for saturated sands becomes possible, allowing the prediction of earthquake induced progressive pore water pressure increases in a one-dimensional dynamic response analysis. The procedure is incorporated in the program DESRA (Finn, et. al., 1977; Finn et. al. 1978; Lee and Finn, 1978). The DESRA program also allows for the simultaneous redistribution and/or dissipation of pore water pressures during analyses. An extensive verification of the DESRA-2 program was conducted by Finn and Bhatia (1980) under cyclic simple shear conditions using a variety of loading patterns.

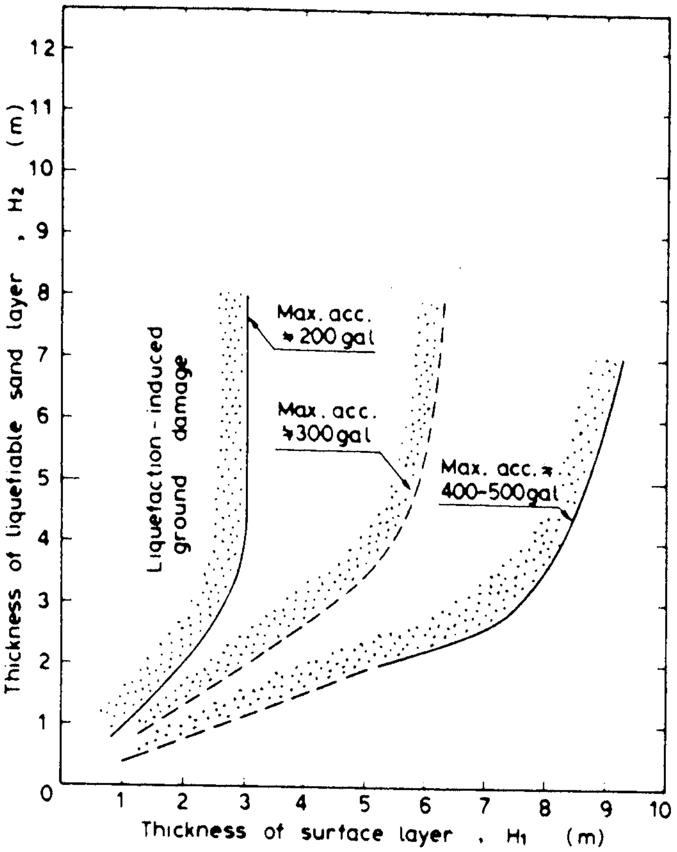

Figure 35 Proposed boundary curves for site identification of liquefaction induced damage (After Ishihara, 1985)

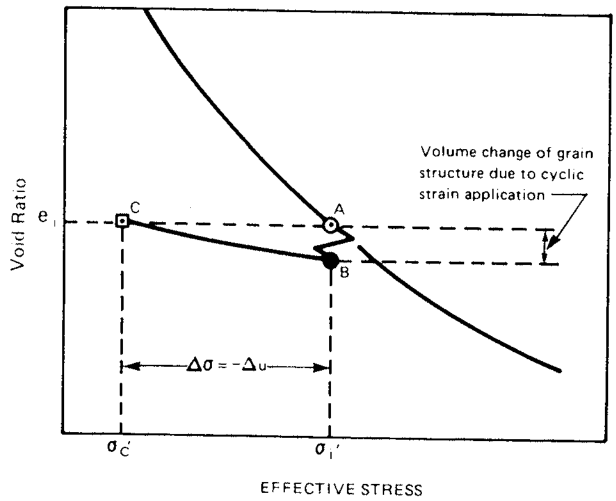

Figure 36 Schematic illustration of mechanism of pore pressure generation under cyclic loading

The application of the program DESRA to a 50 foot layer of saturated medium dense sand (Finn, et. al., 1978) is illustrated in Figures 38 through 41. Figure 38 shows surface acceleration time histories for three cases. Note that total stress response analyses do not take into account the shear modulus degradation which arises as pore water pressures progressive increase. Also, the DESRA program cannot reliably predict post liquefaction response, as the constitutive relations do not include the effects of cyclic dilation which occurs at larger shearing strains. However, approximate estimates may be made by assuming values of residual strength and stiffness following initial liquefaction. Figure 39 shows how pore pressure redistribution or dissipation in the case of more permeable soils can have a significant effect on the liquefaction potential. This is illustrated further in Figure 40 which shows the distributions of pore pressure at a particular time (seven seconds) after the beginning of the earthquake. Acceleration response spectra are shown in Figure 41 where it may be seen that the effective stress analysis shifts the fundamental site period due to pore pressure induced degradation of soil stiffness.

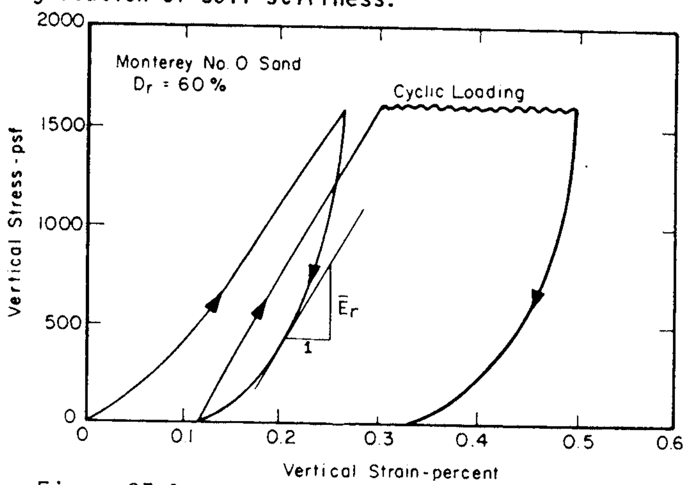

Figure 37 One-dimensional loading and unloading data obtained in simple shear apparatus (After Seed et. al., 1978)

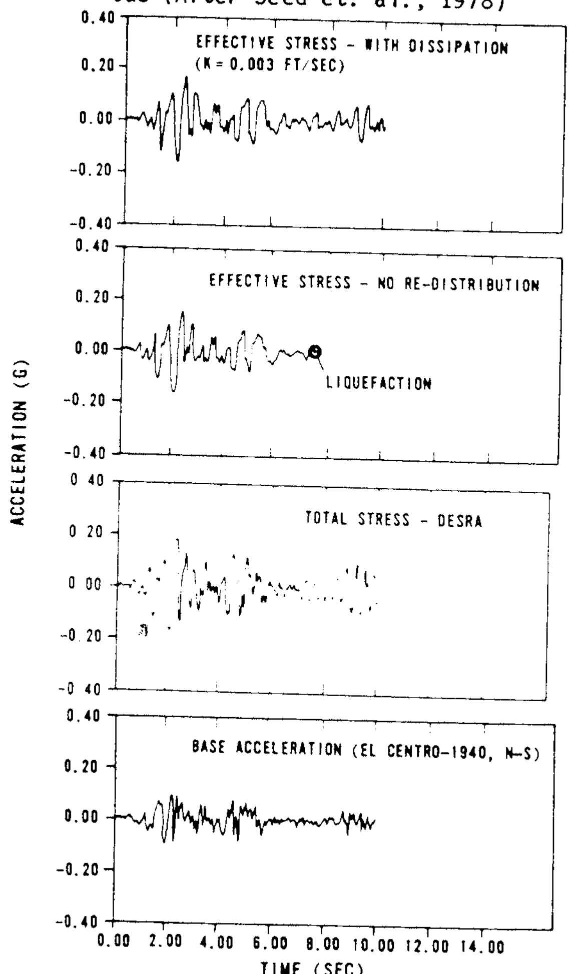

Figure 38 Effect of drainage conditions on surface acceleration response (After Finn et. al., 1978)

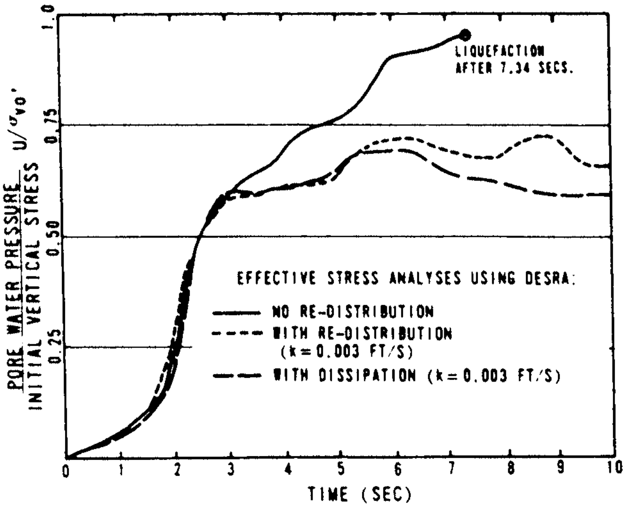

Figure 39 Pore pressure time history from DESRA effective stress analyses (After Finn et. al., 1978)

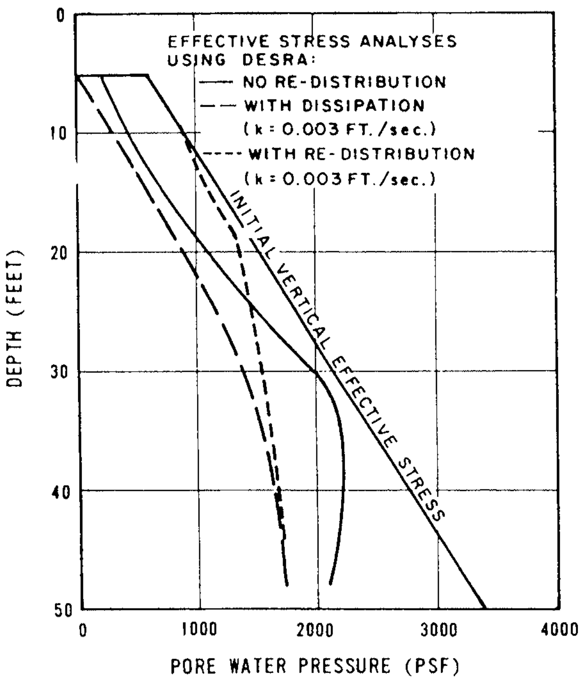

Figure 40 Pore pressure distributions from DESRA effective stress analyses (After Finn et. al, 1978)

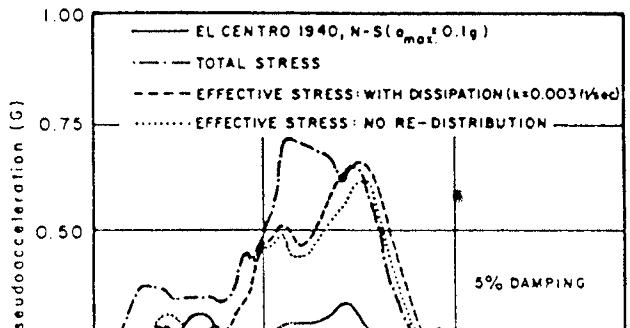

Figure 41 Comparison of acceleration response spectra computed using effective stress analyses versus total stress analyses (After Finn et. al., 1978)

The mechanistic approach also provides the means for evaluating the effects of two-directional shaking on pore pressure increases using the two-directional volume changes characteristics determined by Pyke, et. al., 1975. Evaluations of the liquefaction resistance in both one-directional and two-directional cyclic shear using the mechanistic approach are reported by Seed, et. al., 1978. Representative results are shown plotted in Figure 42. Reduction in liquefaction resistance of the order of 10 to 20% to account for two-directional shaking seems reasonable.

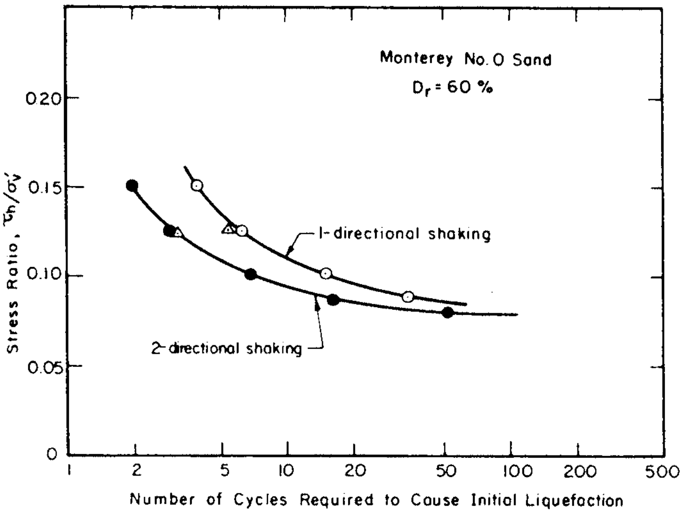

Figure 42 Comparison of stress causing initial liquefaction under one and two-directional shaking (After Seed et. al., 1978)

The practical application of the program DESRA to field liquefaction assessments during earthquake loading do not necessarily require laboratory measurement of the volume change and rebound constants during cyclic loading. Clearly this would be difficult and expensive due to problems of sample disturbance and the numerous tests required in the case of a non-uniform layered site condition. In addition, the rebound characteristics of sand following cyclic loading differ to those prior to cyclic loading as illustrated for example in Figure 37. For practical purposes, a more pragmatic approach to determining the needed physical soil constants may be adopted as described below:

1. Field liquefaction strength curves are first assessed for potentially liquefiable layers for the site under consideration. These curves are generally determined from SPT or CPT measurements using the empirical correlations previously discussed.

2. Based on available insitu test data for the site soils, (SPT or shear wave velocity data), shear modulus values as a function of shear strain amplitude and confining stress are determined. This data enables the nonlinear shear stress/shear strain functions to be determined.

3. Assuming a reasonable set of physical constants to describe the rebound characteristics of the soils, through a trial-and-error procedure a set of volume change constants (equation 1) are determined which are compatible with the field liquefaction strength curves. This compatibility is determined by performing a series of theoretical stress controlled cyclic simple shear tests with the assumed parameters.

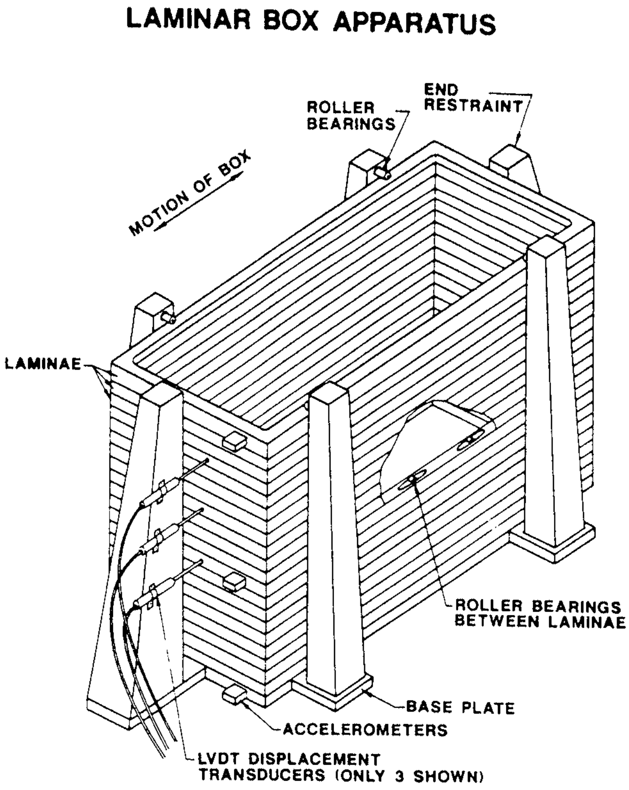

Figure 43 Laminar simple shear box for centrifuge experiment (After Hushmand et. al., 1988)

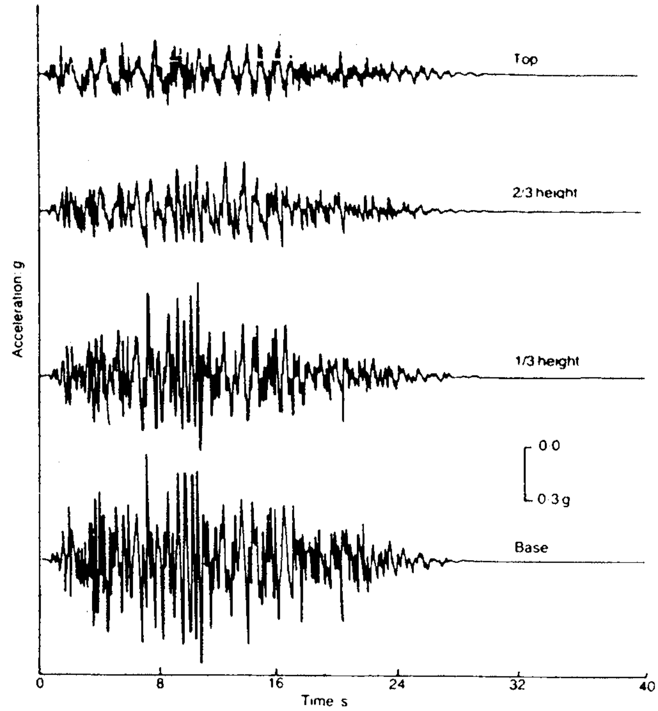

Figure 44 Recorded accelerations at various depths in centrifuge test (After Hushmand et. al., 1988)

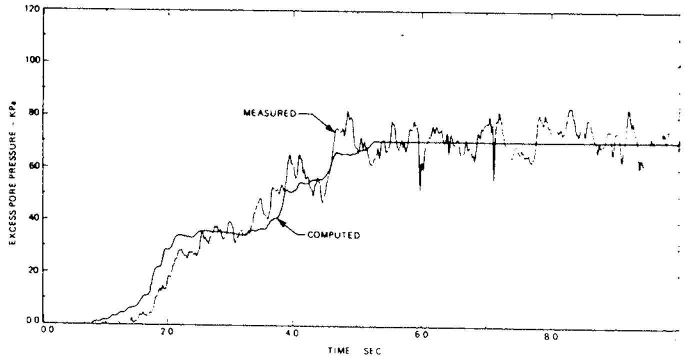

Figure 45 Measure pore pressure response versus computed response using DESRA (After Hushmand et. al., 1988)

These procedures and associated sensitivity analyses are described by Martin, et. al., (1981). By adopting this procedure, computed field pore pressure increments during earthquake loading are known to be compatible with the liquefaction strength curves with a reasonable degree of confidence.

The application of the DESRA computer program to effective stress earthquake response analyses and the assumptions described above to determine the appropriate physical soil constants, have been verified in a series of centrifuge tests conducted at the California Institute of Technology and described by Hushmand, et. al., (1987, 1988). The simulated earthquake response analyses were performed in a test container comprising a stack of rectangular sliding rings (as shown in Figure 43) containing a deposit of fine saturated sand. The tests were carried out at a centrifugal acceleration of 50g and the model subjected to input accelerations with a frequency content and duration corresponding to the 1940 El Centro earthquake scaled to a peak base acceleration of 0.55g. Recorded accelerations at various depths in the centrifuge test are shown in Figure 44. The measured pore pressure response at the 1/3 height is compared to the computed pore pressure response using the program DESRA in Figure 45. The comparison is relatively good, with initial liquefaction occurring after about five seconds of shaking. Note that the DESRA program cannot compute the pore pressure fluctuations arising from the cyclic dilation effects which occur at the larger cyclic strains following liquefaction.

Although the centrifuge provides a valuable means for verification of theoretical analyses and will continue to be used extensively for this purpose in the future, it is also desirable to undertake field instrumentation experiments where recordings of pore water pressure and site accelerations are made during actual earthquake events. Two such sites have been instrumented for this purpose in California. The sites have been chosen at locations where reasonably strong earthquakes occur fairly regularly. At these sites, piezometers have been located in sand layers susceptible to liquefaction. Figure 46 shows the instrumentation placed at an Imperial Valley site as described by Bennett, et. al., 1984. In utilizing recorded data from such installations, it is essential to have confidence that the piezometer installation procedures themselves have no influence on the recorded response. Anomalous response could occur because either the piezometers were not fully saturated or the installation procedure may have created anomalous permeability zones surrounding the piezometer.

4.5 Earthquake Induced Settlement of Saturated Sands

For saturated sand deposits, settlement will occur as the earthquake induced excess pore pressures dissipate. Because of the short duration of the ground shaking, the majority of dissipation will occur following the earthquake. The time required for complete settlement to occur can vary considerably depending on the characteristics of the soil and the length of the drainage path. Settlements may occur very quickly following an earthquake or in some cases take several days.

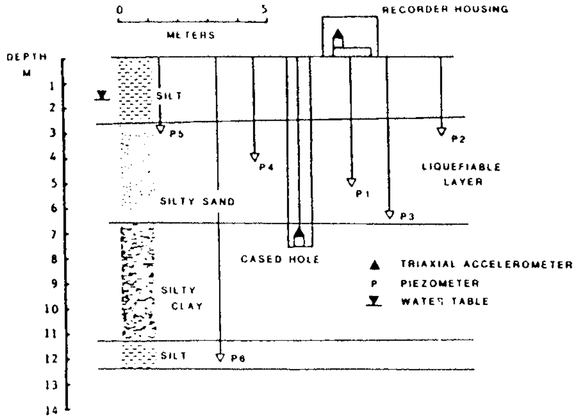

Figure 46 Instrumentation at Imperial Valley liquefaction test site (After Bennett et. al., 1984)

For excess pore water increases less than values required to induce initial liquefaction, the magnitude of settlement resulting from excess pore water dissipation approximately reflects the elastic reloading of the sand structure. That is, the reloading path retraces the original unload or rebound path as pore pressures increased. The magnitude of elastic recoverable deformation and hence settlement depends on the relative density, and level of effective confining stress. In most practical situations recoverable volumetric strains are less then 1%, and hence settlements could be expected to be less than 1% of the depth of sand effected.

For cases where liquefaction occurs and where earthquake induced shearing strains in sand deposits could be expected to be greater than 0.3%, larger volumetric strains may occur as a result of pore pressure dissipation, as described by Tokimatsu and Seed (1987). The larger cyclic strains reflecting "cyclic mobility," can be expected to induce a new or virgin intergranular structures in a sand, and hence volumetric strains induced by the reloading process can be expected to be larger than those for elastic reloading.

Approximate back calculation of volumetric strains from observed settlements in case histories, suggest volumetric strains in the range 1 to 2 percent. Tatsuoka, et. al. (1984) studied the volumetric strains resulting from dissipation of pore pressures following initial liquefaction in laboratory tests, and found that the amount of settlement was significantly influenced by the maximum shear strain induced during the test. However, post liquefaction cyclic shear strains induced during stress controlled liquefaction tests simulating equivalent earthquake loading may be significantly greater than those occurring in the field. Post-liquefaction cyclic shearing stresses occurring in the field, depend on inertial loading generated by site response, which may be significantly attenuated following initial liquefaction. Consequently, the evaluation of the earthquake induced settlement of saturated sands is prone to considerable error and evaluation methods are in need of further research.

4.6 General Observations

In a practical sense, our understanding of the mechanisms leading to the pore water pressure increases during earthquake loading of saturated sands are reasonably advanced, and the methods available for assessing liquefaction potential in the field using SPT or CPT based empirical procedures are generally adequate for design purposes. However, our understanding of post-liquefaction behavior is far from complete and requires further research. Such research should address needs related to post-liquefaction response analyses capable of predicting acceleration and displacement time histories required for liquefaction resistant design, evaluations of rates of pore water pressure dissipation and associated settlement and methods for assessing the influence of surface layers on improving ground stability. Improvements in these areas will require a better understanding of the governing constitutive relationships, improved analytical methods and centrifuge verification experiments.

5. RESPONSE OF EARTH STRUCTURES

5.1 Background

The evaluation of the dynamic response of earth structures such as earth dams, embankments and slopes, to earthquake ground shaking is a critical first step in the assessment of the stability or deformations of such structures arising from the earthquake loading. From the design standpoint, the importance of the dynamic response was not fully recognized until the early 1960's. Up until that point, the effects of earthquake loading were represented by an equivalent static seismic coefficient, the magnitude of which was independent of the height and stiffness of the earth structures being designed. Stability evaluations were performed using a pseudo static approach without recognition that the earthquake loads were time varying and of limited duration.

The state-of-the-practice now recognizes the importance of the dynamic response behavior. Although present state-of-the-art analysis techniques have the potential for coupling dynamic response evaluations and deformation computations, most design practice uses an uncoupled approach. That is, dynamic response analyses (usually equivalent linear analyses) are performed to determine the time history of lateral accelerations acting on the earth structure for a given design earthquake, and separate analyses performed to assess the deformations of the earth structure under such loading. In this section, an overview of methods for assessing dynamic response is presented. Stability and deformation assessments are addressed in a subsequent section.

5.2 Dynamic Response Analyses

The first dynamic response analysis for earth dams was developed by Monobe, et. al. (1936). The dam was assumed an infinitely long uniform elastic wedge which when subjected to horizontal ground motions deformed in horizontal shear only, with uniform shear stress distribution over horizontal planes. This is commonly referred to as a one-dimensional shear slice theory and is illustrated schematically in Figure 47. Natural periods of vibration and mode shapes result form the solution of the governing equilibrium equation. The fundamental period of the first mode $T_1 = 2.4h/v_s$ seconds, where h is the dam height and v_s is the shear wave velocity. Mode shapes for the first three modes are shown in Figure 47.

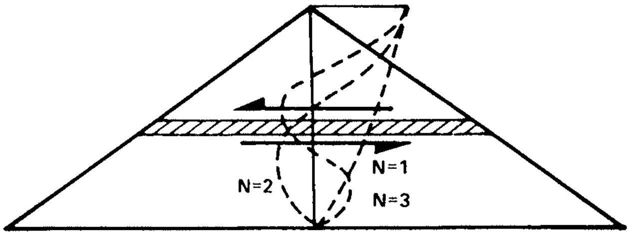

Figure 47 One dimensional shear strain theory

The one-dimensional shear slice theory was extended to uniform triangular wedges in rectangular canyons by Hatanaka (1955) and Ambraseys (1960). These two dimensional analyses are sometimes termed "shear beam analyses" and are reviewed by Martin (1965). The variation of fundamental period as a function of dam height and shear wave velocity for an infinitely long dam and a rectangular canyon case is illustrated in Figure 48. Extension of the shear beam theory to canyons of arbitrary shape has been described by Keightley (1964).

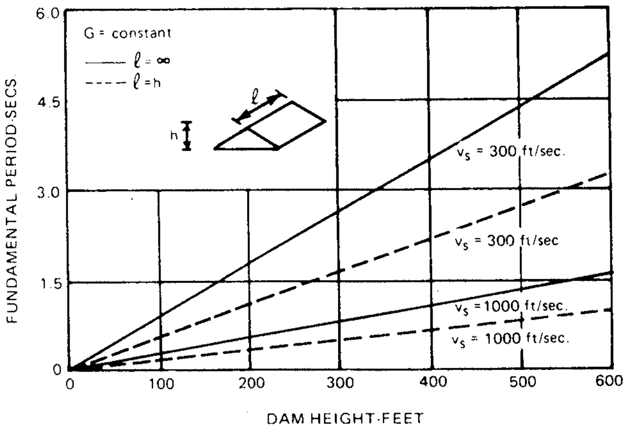

Figure 48 Fundamental period as a function of dam height and shear wave velocity (After Martin, 1965)

A number of verification studies of the two-dimensional "shear beam" elastic analyses have been reported in the literature. These studies utilized forced vibration tests on full scale earth dams to evaluate natural periods, mode shapes, and damping characteristics for a very low strain amplitudes. Measurements were then compared to computed natural periods and mode shapes by assuming an equivalent rectangular canyon shape and equivalent uniform shear wave velocity for the structure. Such studies are described by Keightley (1964), Martin (1965) and Abdel-Ghaffar and Scott (1979, 1981). The latter study documents an extensive series of tests on the Santa Felicia earth dam in Southern California. In all cases, the measured natural periods of vibration and mode shapes (along the dam crest) agreed reasonably well with the theory for the first few modes of vibration.

Although shear slice or shear beam theories are adequate for computing natural periods of vibration, the inherent assumptions place severe restrictions on the use of the theories to obtain stress distributions within a dam during an earthquake, and do not allow the response to vertical ground motions to be computed. From the results of field and model tests and theoretical considerations, Ishizaki and Hatakeyama (1962) concluded that the effects of horizontal and vertical compressive stresses on displacements should be also taken into account when a dam is subjected to horizontal base motion. Finite difference computations for sinusoidal base input motions showed significant expansion and contraction of elements across the horizontal section as waves propagated within the dam and that large tensile stresses could develop at the surface.

To investigate the deficiencies of shear slice theory, a two-dimensional elastic finite element approach was studied by Martin (1965). Mode shapes and natural frequencies of vibration for a 100-foot high infinitely long dam having a shear wave velocity of 1150 feet per second, illustrating results from a finite element analysis, are shown in Figure 49. The figures clearly show the vertical and rocking modes of vibration which can be mobilized during an earthquake. For the two shear modes, the horizontal displacements on the face of the dam are significantly less than the horizontal displacements along the centerline. The first mode natural frequency is 10% less than that computed by the shear slice theory. A comparison between mode shapes for the shear slice and finite element theories for the first two shear modes are shown in Figure 50. The reductions in the displacements amplitudes over the dam face shown by the finite element theory have been observed in experimental measurements on full size dams by Martin (1965) and Abdel-Ghaffar and Scott (1981).

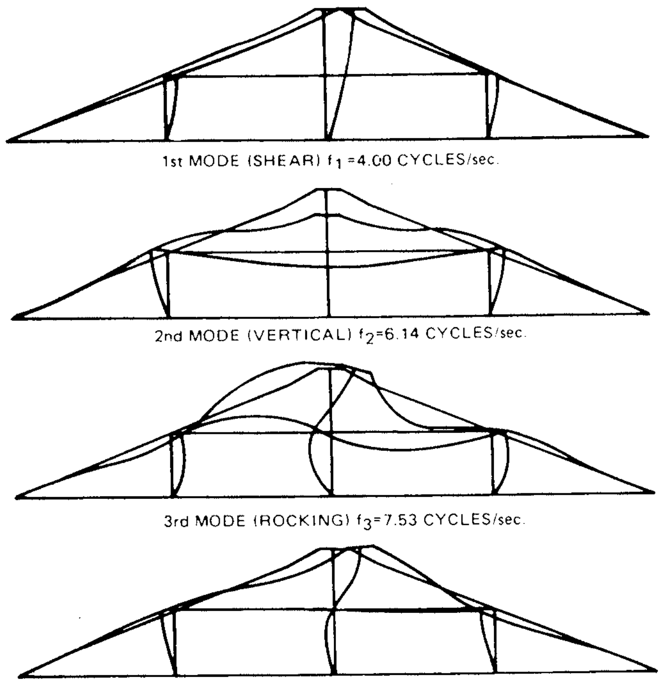

Figure 49 Fundamental mode shapes-elastic finite element theory (After Martin, 1965)

Reductions in accelerations near the face of a dam imply that average seismic coefficients computed over sliding masses at any instant of time would be greater when computed from shear slice theories as compared to finite element methods. The range of maximum seismic coefficients from shear slice analyses (normalized by maximum crest acceleration) as a function of the depth of sliding wedges for dams having natural periods ranging between 0.26 and 5.22 seconds, are presented in Figure 51. (Makdisi and Seed, 1978). Data from finite element analyses is also shown. Whereas the average seismic coefficients from shear slice analyses are generally greater than coefficients obtained from finite element analyses, for practical purposes, the average curve for all data has been recommended by Makdisi and Seed for seismic stability analyses.

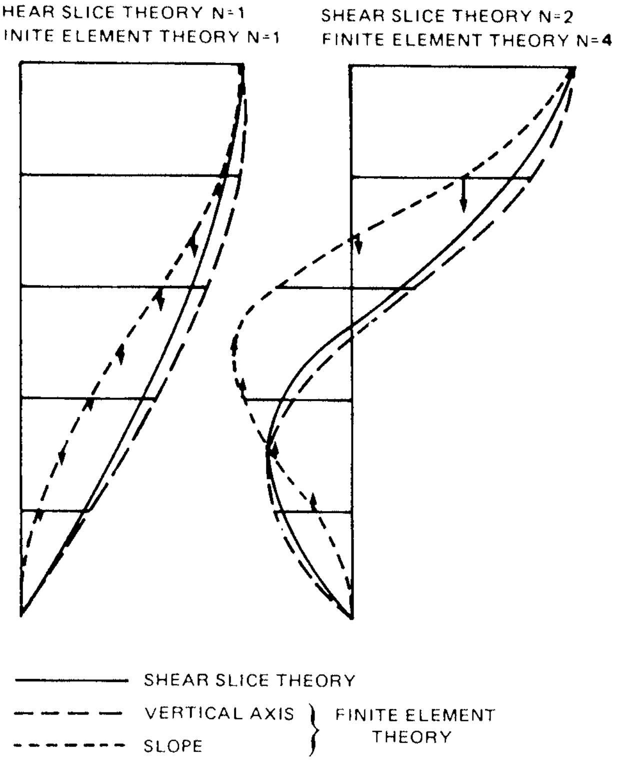

Figure 50 Comparison between mode shapes for shear slice and finite element theories (After Martin, 1965)

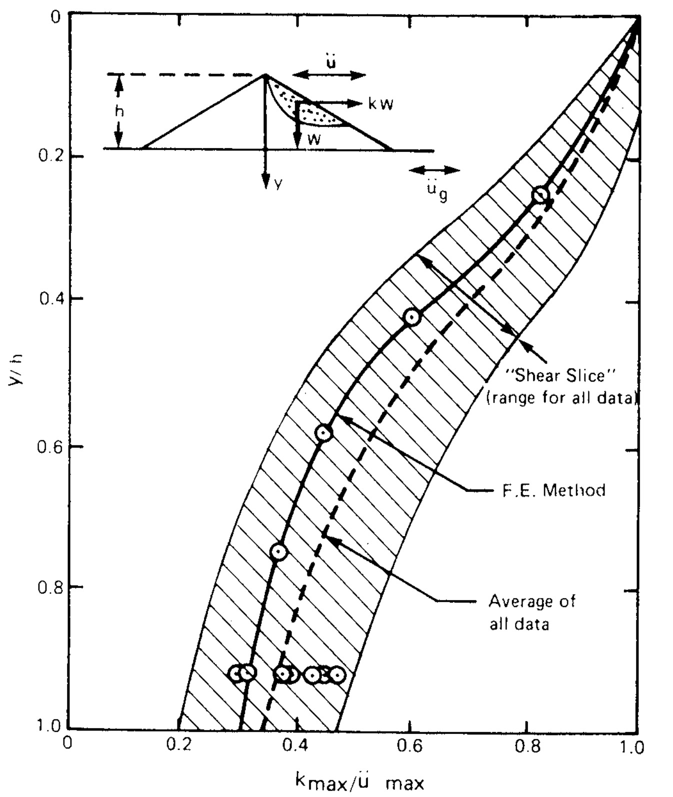

Figure 51 Comparison between average seismic coefficients for shear strain and finite element theories (After Makdisi and Seed, 1979)

The finite element approach also readily permits spatial and strain amplitude variations in shear modulus and damping to be incorporated in analyses. Field studies on Santa Felicia Dam reported by Abdel-Ghaffar and Scott (1979, 1981), have clearly shown that the high amplitudes of response generated by earthquakes result in lower shear modulus values and corresponding longer natural periods of vibration. The variations of shear moduli with strain amplitude measured from these field studies are similar to those reported in Section 2 of the paper.

To account for variations of shear modulus with strain amplitude in the form of equivalent linear response analyses, more versatile finite element analyses have been developed. The program QUAD-4 (Idriss, et. al., 1973) is perhaps the easiest and most widely used in practice. The program also allows for strain compatible damping ratios to be incorporated into each of the elements. The equations of motion are solved by direct integration in the time domain. Other equivalent linear finite element programs used for earth dam response studies include LUSH (Lysmer, et. al., 1974) and FLUSH (Lysmer, et. al., 1975). A comparative discussion of these programs and their applications has been given by Finn (1987).

The two-dimensional plane strain finite element analyses previously described are appropriate for dams with relatively large length to height ratios. However, for dams in steep walled or triangular canyons, three-dimensional response analyses are necessary. Makdisi, et. al., (1982) have performed equivalent linear finite element analyses for earth dams in triangular dams and compare natural frequencies of vibration of dams in rectangular and triangular canyons of different slopes with those for long dams where the assumption of two-dimensional plane strain response as valid. A three-dimensional version of the program LUSH was used for the study. A further study comparing 2-D and 3-D effects is reported by Mejia and Seed (1983).

The application of the LUSH program to evaluate the earthquake response of the Orville Dam in California is reported by Vrymoed (1981). The 750-foot high Orville Dam was subjected to seismic activity in 1975 where both bedrock and dam crest acceleration time histories were recorded. A two-dimensional plane strain analysis using the program LUSH was used to predict crest acceleration time histories from the recorded input accelerations at bed rock level. Adjustments in the natural period of 25% were made to account for three-dimensional effects. Computed versus observed accelerations at the crest are shown in Figure 52, and are seen to be in good agreement.

The QUAD-4 program may also be applied to slopes to evaluate accelerations time histories and dynamic stress distributions as described by Idriss (1968). A comparison between maximum horizontal shear stresses computed by finite element analyses and those computed by one-dimensional shear slice theory is shown in Figure 53 (Martin and Taylor, 1971). It may be seen that one-dimensional theory provides a reasonably good simulation in regions removed from the toe of the slope.

For most practical applications, the simplified shear slice or shear beam theories are adequate to evaluate natural periods of vibration of earth structures. However, for more detailed analyses where distributions of stress or acceleration time histories for specific regions of an earth structure are necessary, equivalent linear finite element analyses should be used. Although several advances have been made in the area of nonlinear dynamic response analyses for dams such as research reported by Prevost, et. al. (1985), Abdel-Ghaffar and Elgamal (1987), and Elgamal and Abdel-Ghaffar (1987), the elasto-plastic models used are complex and incorporate parameters not familiar to many practicing engineers. In addition, constitutive relationships and methods of analysis remain to be fully validated and the increased cost and complexity do not seem to result in corresponding improved predictions of dynamic response in the practical sense.

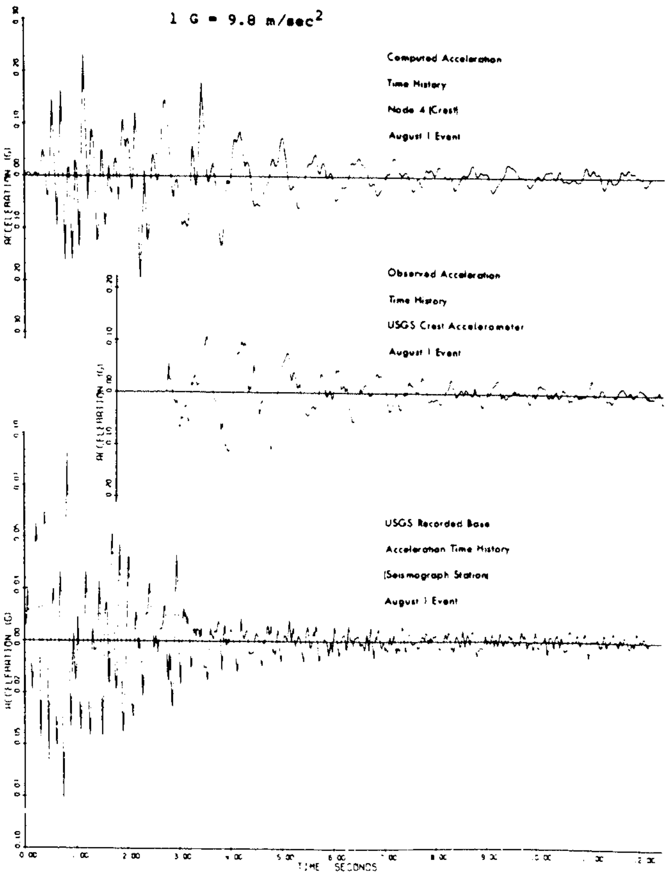

Figure 52 Oroville Dam acceleration records August 1, 1957 earthquake (After Vrymoed, 1981)

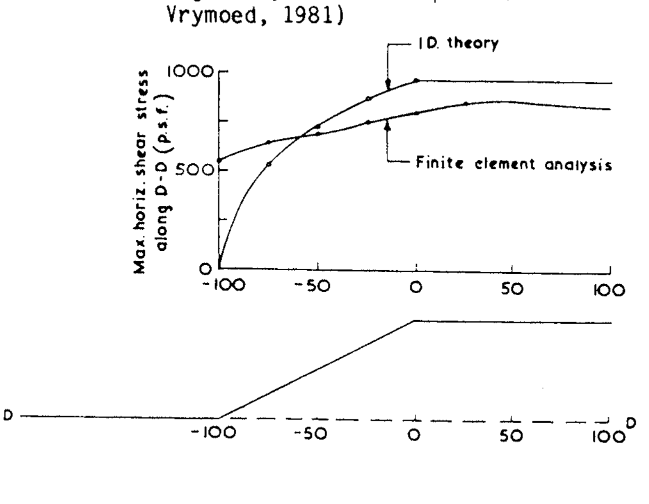

Figure 53 Comparison of maximum horizontal shear stress from one dimensional and two dimensional response analyses (After Martin and Taylor, 1971)

6.0 EMBANKMENT AND SLOPE STABILITY

6.1 Background

In the following section, procedures used for estimating earthquake induced deformations of limited extent are discussed, followed by a discussion of liquefaction induced instability arising in saturated cohesionless embankments or slopes.

6.2 Procedures For Estimating Earthquake Induced Deformations

Early approaches to evaluate the seismic stability of slopes (Seed and Martin, 1966) used a conventional pseudo-static method as shown in Figure 54, where the effects of an earthquake were represented by a lateral force kW acting on a potential sliding mass. The value of the seismic coefficient k was chosen somewhat arbitrarily and the slope judged unsafe if the factor of safety was less than 1.

However, slope displacements will only occur when the time varying lateral earthquake forces exceed the value required to cause yield on the sliding surface. During periods of yield, incremental displacements will occur leading to an accumulative slope displacement at the end of the earthquake, which may or may not be tolerable from a design standpoint.

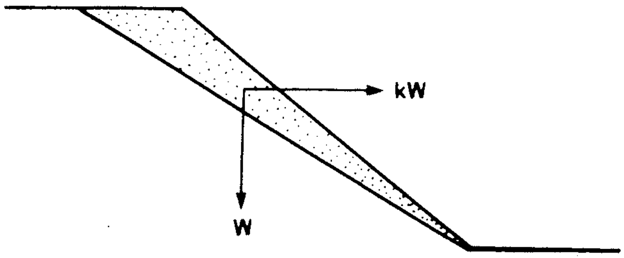

Figure 54 Pseudo static stability

Newmark (1965) proposed a simple method to assess earthquake stability in terms of slope displacement. Rigid plastic behavior was assumed such that displacements only occurred when accelerations exceeded a well defined yield acceleration. Downslope displacements of a sliding mass could then be computed by a simple double integration process as shown in Figure 55. Displacements are initiated when the time varying seismic coefficient exceeds the yield coefficient k_y. By integrating the velocity time histories, the accumulated downslope relative displacement may be computed. Adjustments can also be made for any reductions in yield acceleration which might occur as a result of displacement. Assuming the accelerations acting on the slope are equal to the ground accelerations, the magnitude of displacements become a function of the magnitude and time histories of ground acceleration. Maximum displacements for four earthquake records normalized to a maximum acceleration of 0.5g are shown plotted against values of the ratio of yield acceleration N to the maximum earthquake ground acceleration A in Figure 56. Note the rapid increases in displacement with reduction in the ratio of N/A. Shaking table tests on models of dry cohesionless slopes reported by Goodman and Seed (1966), verified this approach for the shallow earthquake induced slope failures characteristic of such slopes.

The concept proposed by Newmark was modified by Makdisi and Seed (1978) for earth dam analyses. A simplified procedure was developed for estimating earthquake induced deformations, which included the effects of elastic dynamic response on the magnitude of lateral accelerations acting on an assumed sliding mass. Time histories of lateral acceleration acting on assumed sliding masses were determined from simple one-dimensional or more complex finite response analyses. A range of embankment heights of varying natural periods subjected to a variety of earthquakes of different magnitudes were analyzed. The data showed that the fundamental period and the peak acceleration were the two parameters having the most influence on the calculated displacements. Average curves for normalized permanent displacements as a function of the ratio of yield seismic coefficient to maximum seismic coefficient are shown in Figure 57.

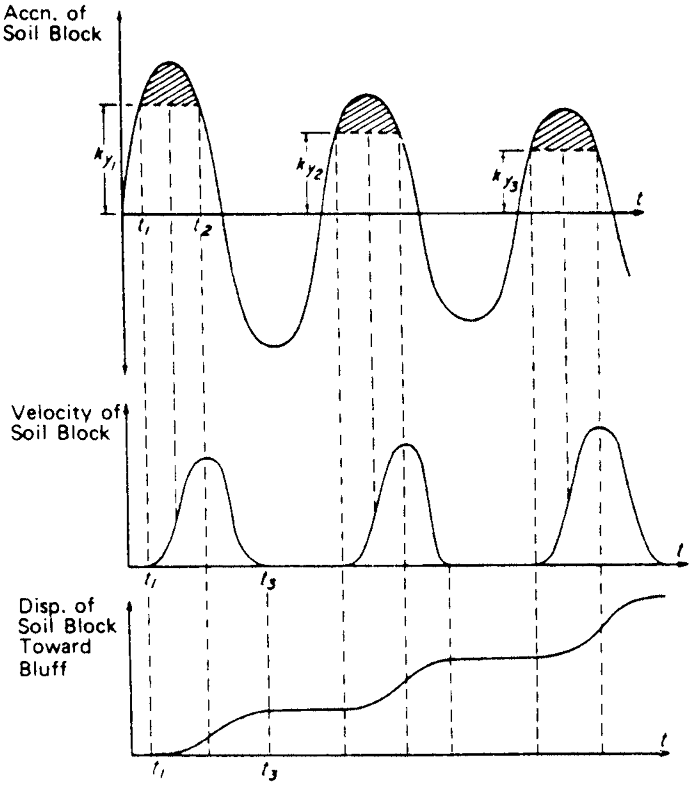

Figure 55 Integration of accelerograms to determine downslope displacements (After Goodman and Seed, 1966)

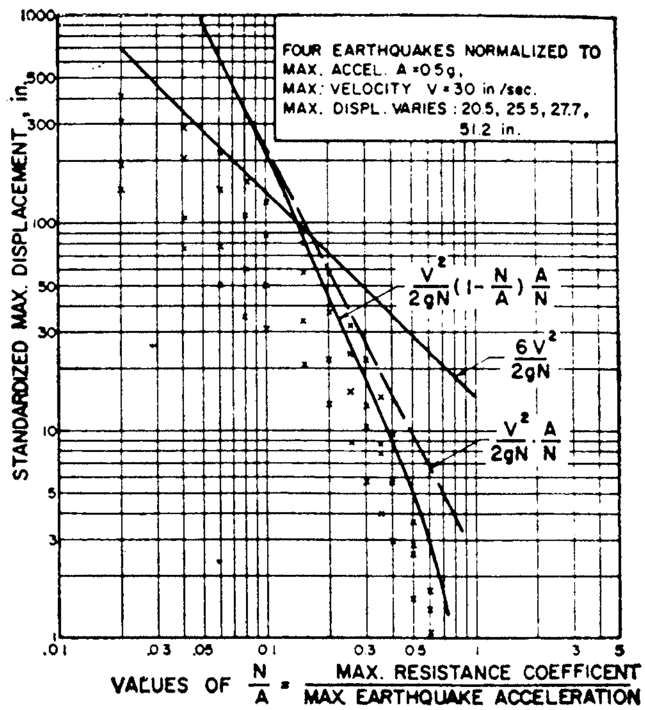

Figure 56 Standardized displacement for normalized earthquake - unsymmetrical resistance (After Newmark, 1965)

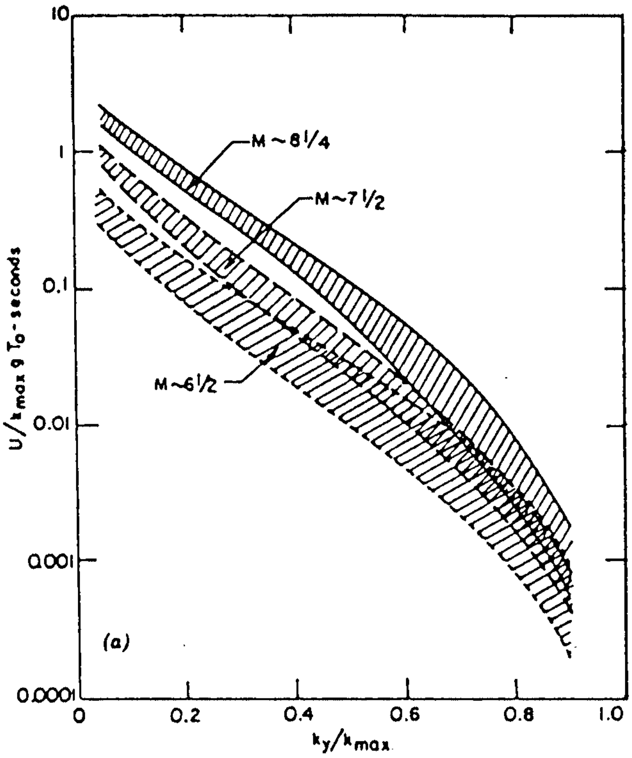

Figure 57 Variation of yield acceleration with normalized permanent displacement-summary of all data (After Makdisi and Seed, 1978)

Makdisi and Seed illustrate the application of the method by applying it to the 135-foot high Chabot dam subjected to a magnitude 8.25 earthquake having a maximum acceleration of 0.4g. This level of shaking is representative of the event which acted on the dam during the 1906 San Francisco Earthquake. The maximum crest acceleration was computed to be 0.57g and the first natural period equal to 0.99 seconds. Using static strength values, yield accelerations were calculated for a sliding mass extending over the full height of the dam as shown in Figure 58. Using the average curve shown in Figure 51, a value of $k_{max} = 0.2$ was computed. Depending on the strength parameters assumed, computed displacements ranged from 0.6 to 1.5 feet. These values are in reasonable accord with the observed performance of the dam in the 1906 event. Further representative examples of this procedure as applied to rock fill dams, are presented by Seed, et. al. (1985) and Tsai, et. al. (1985).

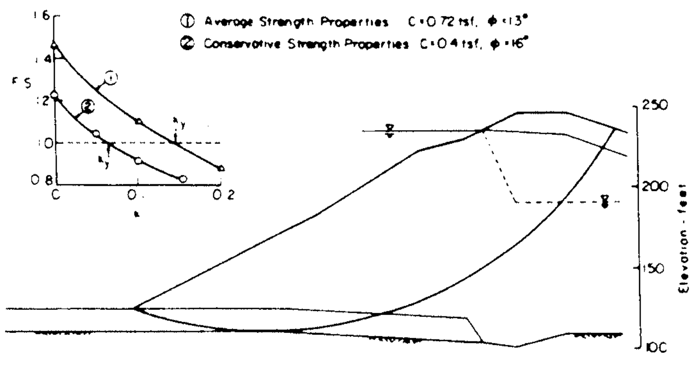

Figure 58 Yield acceleration values for slide mass extending through full height (After Makdisi and Seed, 1978)

An example of the application of the Newmark approach to a slope failure is presented by Idriss (1985). The 1964 Alaskan earthquake triggered a major horizontal block slide in downtown Anchorage commonly termed "the Fourth Avenue Slide." A cross section of the slide is illustrated in Figure 59. The large horizontal movements severely damaged an area about 1600 feet long and 900 wide and generated two grabens where vertical movements of up to 10 feet were recorded. Lateral movement of the soil mass between the bluff and the first graben was about 19 feet. Initiation of the sliding occurred about 1-1/2 to 2 minutes after the shaking began and did not continue after the shaking stopped. The horizontal failure zone was located in a slightly over-consolidated clay layer. Detailed back analyses of the failure which took into account reductions in strength of the clay layer with displacement, indicated displacements computed by the Newmark procedure were consistent with those observed.

Ishihara (1985) proposes the use of dynamic Mohr Coulomb failure envelopes, characterizing the stress levels where large residual strains occur. This approach allows the computation of a factor of safety against major seismic induced failure, using a conventional pseudo static approach.

In recent years there has been considerable research to develop improved nonlinear dynamic response analyses for earth dams. Such analyses enable the coupling of earthquake time history computaions with the prediction of nodal point permanent displacements in finite element analyses. A variety of constitutive relationships have been used in these studies, including kinematic hardening plasticity models and bounding surface plasticity theories. A commentary on the various constitutive models and their application to dynamic nonlinear analyses is provided by Finn (1988). Examples of the applications of these models to earth dams are given by Bureau, et. al. (1985),

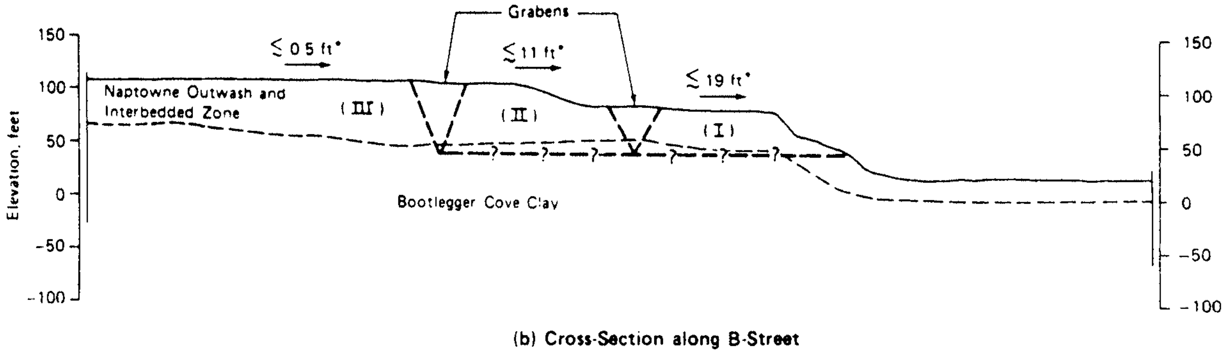

Figure 59 Cross section across Fourth Avenue Slide (After Idriss, 1985)

Other methods based on stress path dynamic testing in the laboratory, may also be used to compute earthquake induced permanent displacements of earth slopes and embankments. In these approaches, soil samples are first consolidated to in situ stress levels computed from either simplified or finite element methods of analyses. By performing dynamic response analyses of the earth structure, equivalent uniform cyclic stress histories may be obtained for the various soil elements chosen for study. Undrained cyclic tests are then performed on the consolidated samples and the residual permanent strain at the end of the test recorded. For more ductile soils such as clays, residual strains progressively increase as the levels of dynamic stress increase as shown schematically in Figure 60 (Ishihara, 1985). Estimates of insitu residual strain obtained in this manner are approximate, as the effects of interaction of soil elements on the total response is not accounted for. In the simplest form, such tests may be used as "proof" tests to evaluate whether residual strains are significant. Such a procedure has been suggested by Martin and Taylor (1971) for cohesive soil slopes. More complex evaluations such as those presented by Lee (1974), Serff, et. al. (1976), and Kuwano and Ishihara (1988), assimilate the residual strains or so called strain potentials for various elements, as softened stiffness values in subsequent static finite element analyses to evaluate overall earthquake induced displacement patterns in the earth structure.

Figure 60 also shows schematically, the residual strain behavior of so called "brittle" soils where residual strains become very large at critical dynamic stress levels. For the case of weathered residual soils which show such brittle behavior, Finn (1988), and Elgamal and Abdel-Ghaffar (1988). The end product of these analyses is in a form of a deformed finite element mesh illustrating permanent deformation profiles after the earthquake, as shown in Figure 61. The profile shown was obtained during an earthquake response study of a solid waste landfill performed by The Earth Technology Corporation. A kinematic strain hardening constitutive model was used in the analyses.

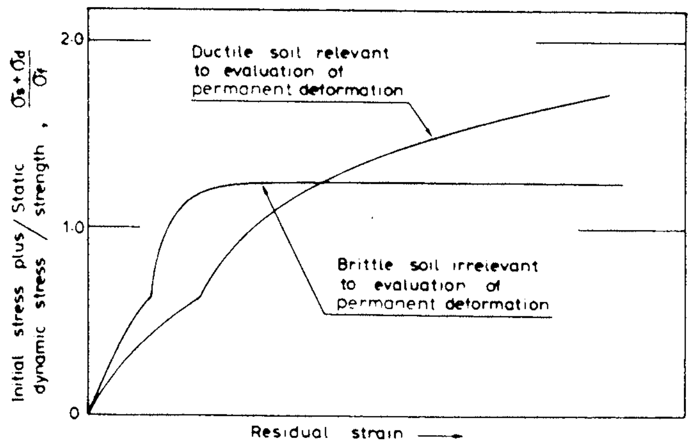

Figure 60 Residual strain behavior of ductile and brittle soils (After Ishihara, 1985)

A previously noted, elasto-plastic models are complex and in their present state of evolution cannot be regarded as the state-of-the-practice. However, with further research and simplification and further validation studies using the centrifuge, improvements are likely in the future.

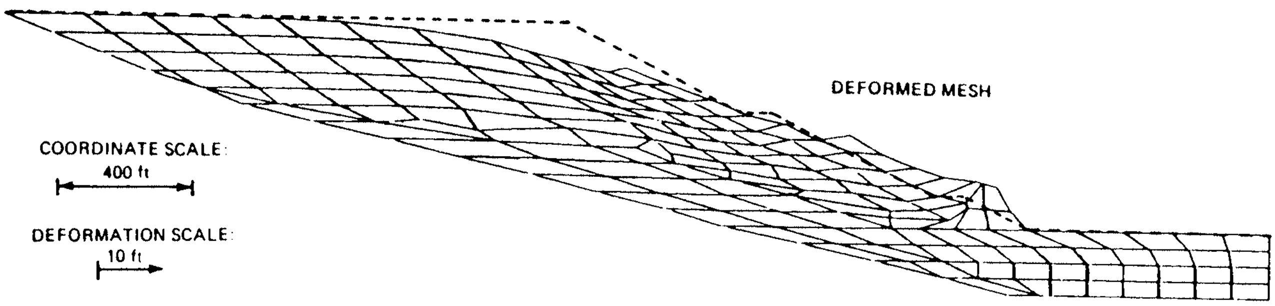

Figure 61 Example of coupled nonlinear finite element response/deformation analysis

6.3 Liquefaction Induced Instability

The potential for progressive pore water pressure increases during earthquake loading of saturated cohesionless earth structures, clearly raises concerns regarding the associated potential loss of soil stiffness and undrained strength. Although near zero effective stress conditions may develop during cyclic loading, considerable resistance to shear may still be mobilized during static undrained loading, due to the effects of dilation. This is illustrated by the curves shown in Figure 62 where undrained stress strain curves after cyclic loading are compared to those prior to cyclic loading. Figure 62a shows curves where zero static deviator stress is applied during a cyclic test while Figure 62b shows results from a test where an initial static deviator stress is superimposed during cyclic loading. After cyclic loading sands may still possess a substantial margin of strength following initial liquefaction to support any sustained loading.

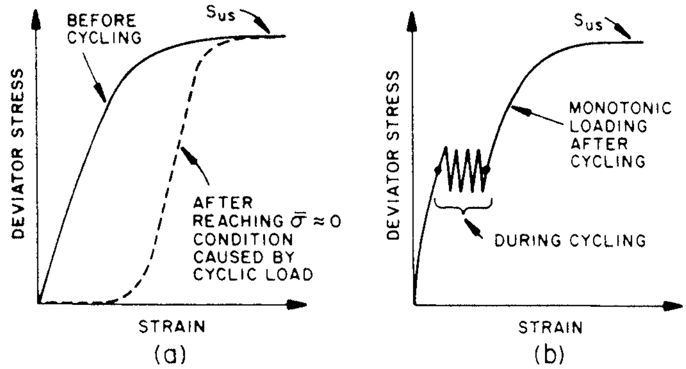

Figure 62 Undrained stress-strain curves for medium dense or dense sand. S_{us} denotes undrained steady-state shear strength (After U.S. National Research Council, 1985)

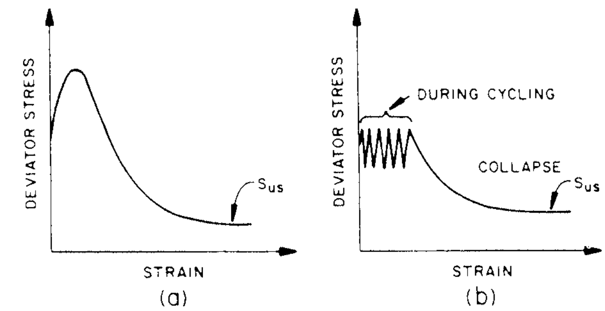

Figure 63 Undrained stress-strain curves for loose sand. S_{us} denotes undrained steady-state shear strength (After U.S. National Research Council, 1985)

The mobilization of residual strength following liquefaction may be understood through the concept of steady state strength or deformation (Poulos, 1981), where after a sufficiently large uni-directional deformation, the strength only becomes a function or void ratio and is independent of stress history or initial structure (compare constant volume deformation at large strains in drained strength tests on sands). The behavior shown in Figure 62 is typical of medium dense or dense sands under relatively low initial confining stress. For loose sands or if initial effective stresses are very large, the value of residual undrained strength can be less than the applied static stress. This form of stress-strain behavior is illustrated in Figure 63. If such materials are subjected to undrained cyclic loading in addition to a sustained deviator stress, they may collapse. In a field situation, such collapse may trigger a flow slide, often referred to as a liquefaction failure. On the other hand, for medium dense or dense saturated sands, whereas large strains (both cyclic and permanent) may result from cyclic loading due to high pore pressure buildup, high residual undrained strengths resist flow slide intiation.

For the case of horizontal or near horizontal saturated sand deposits, a condition of zero mean effective stress is reached at the point of initial liquefaction. However, for saturated sand slopes or embankments where large static shearing stresses act on planes of maximum cyclic shear stress and stress controlled boundary conditions prevail, progressive pore water pressure increases are limited by the failure surface as shown in Figure 64. Tests describing the effect of static shearing stress on pore pressure buildup are reported by Vaid and Finn (1979); Chang, et. al. (1983), and Ishibashi, et. al. (1985). The effects of initial static shearing stress increase the values of the cyclic stress ratios required to increase pore water pressures when compared to tests without initial static shear. This is illustrated in Figure 65 which shows schematically observed pore water pressure increases at various points in a model dam subjected to cyclic loading in a centrifuge. Point A shows greater pore pressure increases than points B, C, and D where higher initial static shearing stresses exist. For the latter points, the effects of cyclic dilation on pore pressure fluctuations are also more evident. Pore pressure increases in the vicinity of foundation systems may also be expected to be less than those occuring in the free field removed from the foundation, due to the effects of initial shear stress.

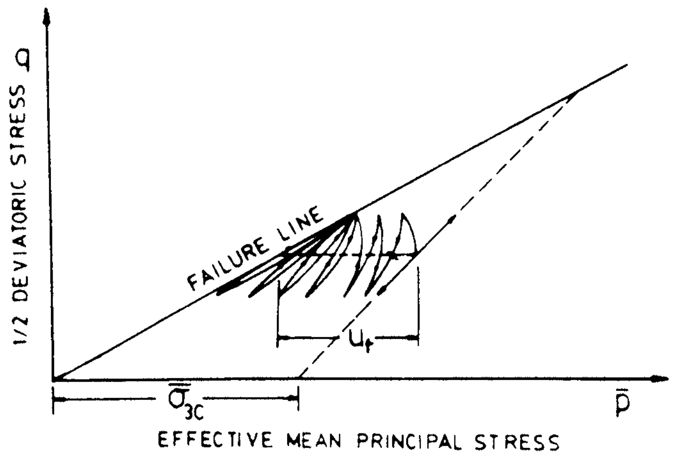

Figure 64 Effective stress path of anisotropically consolidated sample under cyclic loading (After Chang, et. al., 1983)

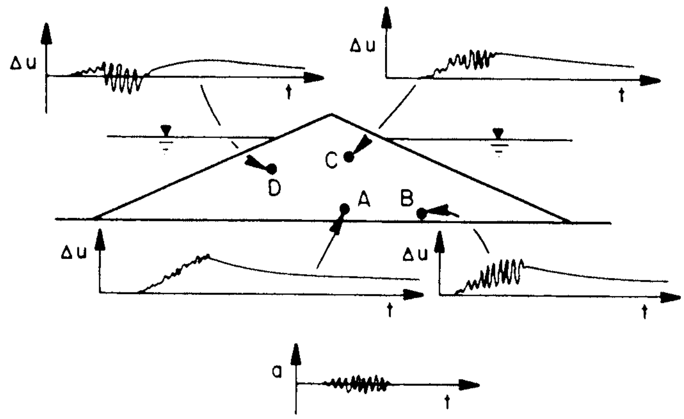

Figure 65 Pore pressure measured in a model dam shaken in a centrifuge (Adapted from Dean and Schofield, 1983)

The prediction of deformations during earthquake loading of saturated cohesionless earth structures is clearly a difficult problem due to the added complexity of time varying changes in effective stress. The computer program TARA-3 has been developed by Finn, et. al. (1986) to model the nonlinear effective stress response of 2-D structures. The application of the program for the analysis of dams is described by Finn (1988). The constitutive relationships used are an extension of those used in the DESRA program and provide the means for computing time histories of pore pressure increases and displacement response during earthquake loading. Validation experiments through centrifuge tests are also described by Finn (1988) with recorded and computed response showing encouraging similarity.

For slopes of earth structures or foundation systems comprising dense saturated sand, earthquake induced pore water pressure increases will either be small, or if significant, will only lead to small residual shearing deformation due to high residual undrained strengths. Although the prediction of associated deformations is difficult, the magnitude of deformations are likely to be tolerable. On the other hand for medium dense or loose sands where undrained shear strengths may be only slightly higher or even lower than the static driving shear stresses in a soil structure, the design problem becomes more significant. The determination of residual undrained strength following high pore pressure buildup or initial liquefaction hence becomes a key question in determining the potential for large deformations or flow slides in saturated earth structures.

Through a study of a number of liquefaction flow slide failures induced by earthquakes in the past, Seed (1987) back calculated estimates of residual strength as a function of SPT blow count. One such classical failure is that of the Sheffield Dam which failed near the end of an earthquake in Santa Barbara, California, in 1925. In effect, the entire embankment slid on a liquefied layer underlying the dam, being pushed downstream by the reservoir water pressure. The conditions at the time of failure are shown in Figure 66 (Seed, 1987). The calculations show that if liquefaction ocurred all along the base, the residual strength of the liquefied soil would be about 50 psf. Studies also showed that the foundation layer comprised silty sands having a relative density of about 40%, which would correspond to a SPT modified blow count N_1 = 6-8.

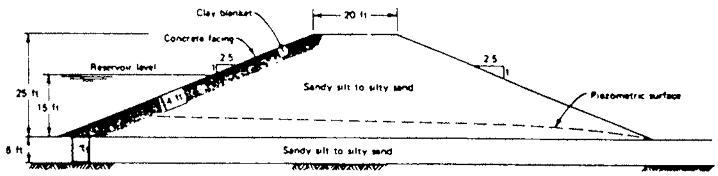

Figure 66 Cross section through Sheffield Dam at time of failure (After Seed et. al., 1969)

Possibly the most intensively studied case of a liquefaction type slide is that of the failure of the upstream slope of the lower San Fernando Dam following the San Fernando earthquake of 1971 (Seed, et. al., 1975). The final cross section of the dam after failure is shown in Figure 67. A cross section of the dam showing the approximate position of the sliding surface prior to failure is shown in Figure 68. As reported by Seed, field studies indicated that a zone of high residual pore water pressure extended over the greater part of the base of the upstream shell. As sliding occurred relatively slowly, (about one minute after the end of the earthquake shaking), the static forces tending to cause sliding were apparently just equal to the combination of the strength mobilized in the nonliquefied soil near the toe and the crest and the residual strength of the liquefied sand. Back calculations based on the above assumptions show the residual strength from liquefied sand at

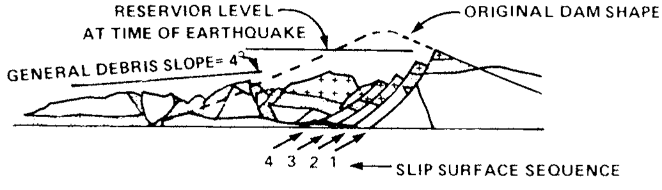

Figure 67 Final cross section of lower San Fernando dam after earthquake (After Seed et. al., 1973)

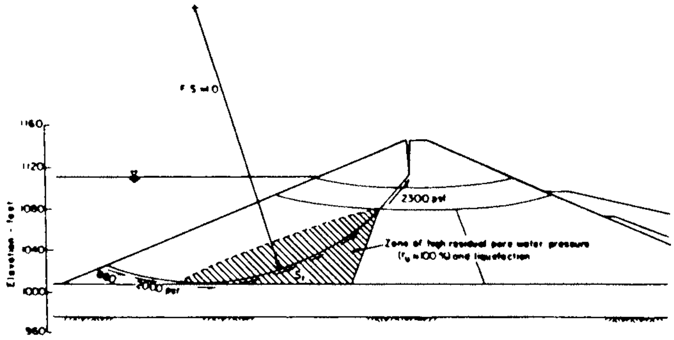

Figure 68 Cross section of lower San Fernando Dam at end of earthquake (After Seed, 1987)

the start of sliding was about 750 psf. Field tests indicated that the sand comprising the shells had a relative density of about 50% and a modified SPT blow count N_1 = 15 before the earthquake.

Back analysis of the above two case histories together with several others, resulted in a tentative relationship between undrained residual strengths and blow counts for sands as shown in Figure 69 (Seed, 1987). Whereas there is considerable scatter, the data provides a useful guide for engineering decisions concerning residual strengths.

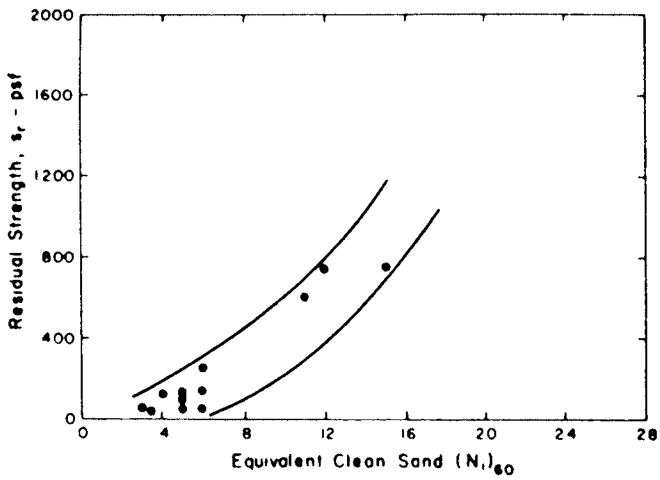

Figure 69 Tentative relationship between residual strength and blowcounts for sands (After Seed, 1987)

Residual undrained shearing strengths may also be determined from the results of consolidated undrained triaxial compression tests on undisturbed samples (Poulos, et. al., 1985). However, appropriate corrections need to be made for the effects of sample disturbance. Assuming that there is a unique relationship between the residual or steady-state strength and void ratio, and that the slope of the steady-state line is the same for reconstituted samples as it is for undisturbed samples, then a procedure for determining steady-state strengths of undisturbed soils for field void ratio conditions may be developed, as illustrated in Figure 70. However, as discussed by Seed (1987), there are concerns that the laboratory procedures which assume that the void ratio of a sand deposit after it liquefies is the same as that prior to liquefaction, may lead to unconservative values of residual strength. The rationale for the concern is that there may be a redistribution of water content during undrained cyclic loading in the field, due to the excess pore pressure gradients generated.

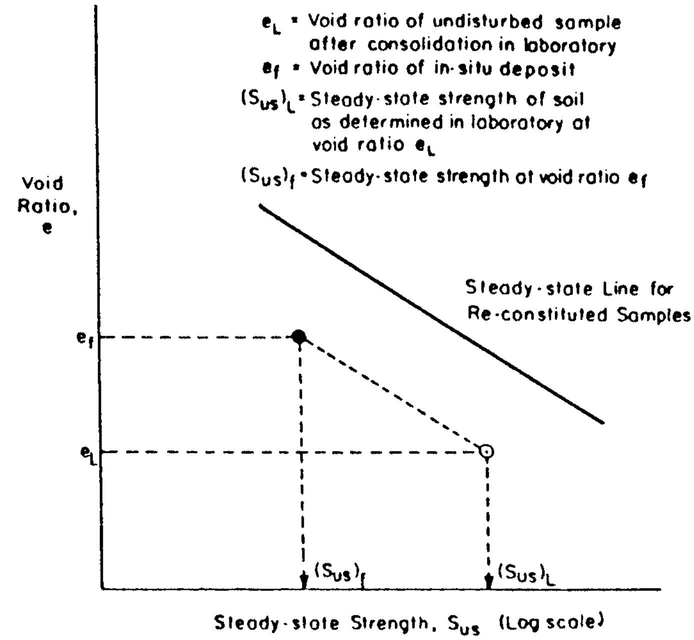

Figure 70 Procedure for determining steady-state strength for soil at field void ratio condition (After Poulos, et. al., 1985)

The shaking table tests on stratified sand layers described by Liu and Qiao (1984), show that even under undrained conditions, water may accumulate below an impervious zone and form a water interlayer as a result of water content distribution, as shown in Figure 71. The upward gradients resulting from excess pore pressure generated in saturated sand below a clay layer will naturally redistribute water vertically hence densifying sand in the lower part of the layer and loosening the sand in the upper part of the layer as shown schematically in Figure 72. In the extreme, a thin zone of water may accumulate at the sand-clay interface. Initial results from centrifuge model tests to examine the mechanisms causing flow failures reported by Arulanandan, et. al. (1988), tend to confirm that the above redistribution mechanisms are possible. Until the uncertainties related to water content redistribution are resolved it seems prudent to use past field performance data as a guide to assessing residual strengths, with data from laboratory tests complementing this where possible.

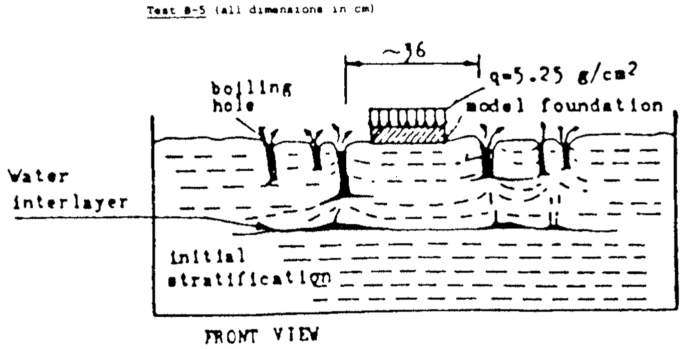

Figure 71 Results of shaking table test on deposit of stratified sand (After Liu and Qiao, 1984)

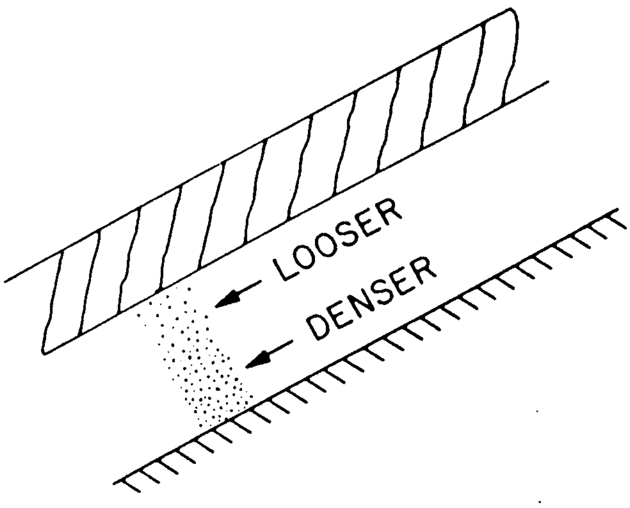

Figure 72 Example of potential situation for flow failure arising from rearrangement of soil into looser and denser zones

7.0 RETAINING STRUCTURES

7.1 Background

Retaining structures are highly susceptible to failure during strong earthquakes and failures have been documented in almost all earthquake damage reports. There are numerous accounts of movement or failure of bridge abutments due to seismic lateral earth pressures with movement causing distortion or even collapse of the bridge superstructure. Sheet pile bulkheads or quay walls associated with port and harbor facilities are particularly susceptible to damage due to liquefaction of backfills or foundation soils.

Analyses associated with conventional gravity or cantilever retaining structures supporting unsaturated or dry cohesionless backfills are first discussed followed by a review of problems associated with sheet pile bulkheads or quay walls supporting saturated sands.

7.2 Gravity or Cantilever Retaining Walls

For gravity or cantilever retaining walls which are able to yield laterally during an earthquake, the well established Mononobe-Okabe psuedo static approach is widely used to compute earth pressures induced by earthquakes. The method and associated design rules is described in detail by Seed and Whitman (1970). The method is an extension of the Coulomb sliding wedge theory and can take into account both horizontal and vertical inertia forces acting on the soil. The following assumptions are made:

1. The abutment is sufficiently free to move so that the soil strength will be mobilized. If the abutment is rigidly fixed and unable to move, soil forces will be much higher than those predicted by the Mononobe-Okabe analysis.

2. The backfill is cohesionless with a friction angle $\phi°$

3. The backfill is unsaturated so that liquefaction problems will not arise.

Representative wall structures showing associated forces during earthquake loading are shown in Figure 73. Equilibrium considerations of the active soil wedge behind the abutment as shown in Figure 74 lead to an expression for the force E_{AE} exerted on the abutment by the soil, namely $E_{AE} = 1/2\ \gamma H^2(1-k_v)\ K_{AE}$. The seismic active pressure coefficient K_{AE} is a function of K_h, K_v, ϕ, β, α, and i. Representative values of K_{AE} are shown as a function of the horizontal seismic coefficient k_h in Figure 75. The influence of different parameters on K_{AE} has been extensively analyzed by Seed and Whitman (1970) and Richards and Elms (1979). In spite of the simplicity, the accuracy of the approach has been substantiated by model tests (Seed and Whitman, 1970) and by back calculations from observed failures of flood channel walls (Clough and Fragaszy, 1977).

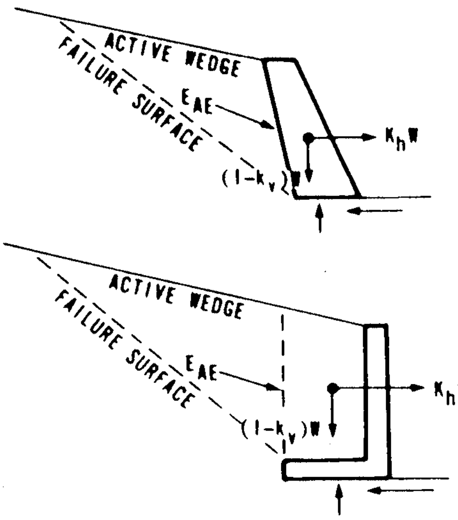

Figure 73 Coulomb sliding wedge theory

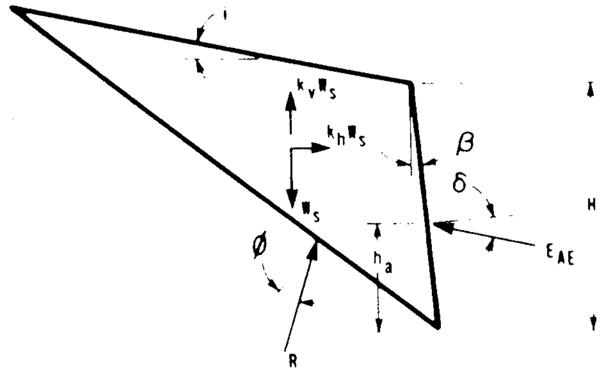

Figure 74 Limiting equilibrium force diagram for active wedge

Using values of lateral pressures computed from the Mononobe-Okabe approach, the seismic design of free standing retaining structures is customarily based on limiting equilibrium principles where the wall would be proportioned to remain stable, (that is, not displace laterally or tilt) for a selected value of k_h. Hence, the difficulty in its use (as for the case of pseudo static stability analyses of slopes) is in the choice of an appropriate value of k_h. If peak ground accelerations are used, the size of gravity or retaining structures to ensure a factor of safety >1, will often be excessively great. To provide a more economic structure, design for a small earthquake induced lateral displacement is preferable.

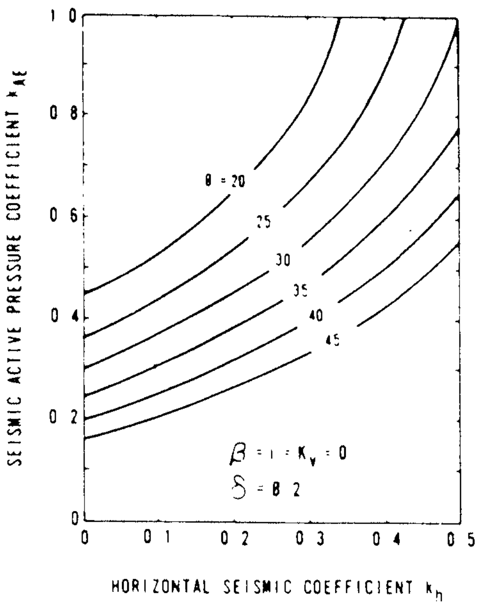

Figure 75 Effect of seismic coefficient on active earth pressure

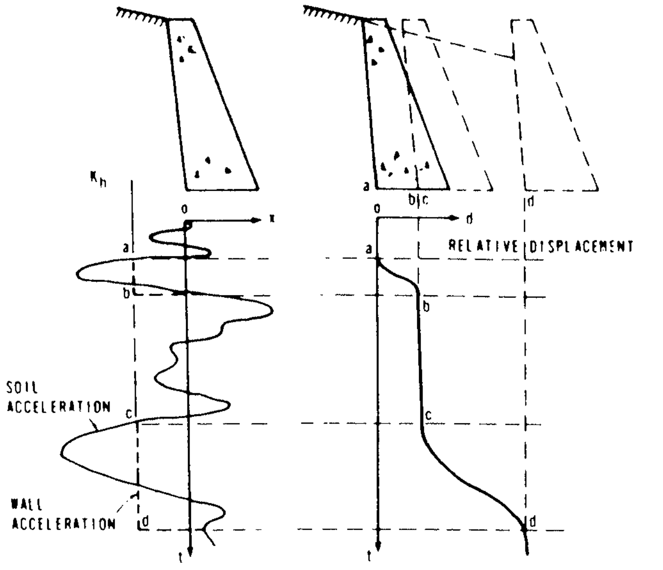

Figure 76 Idealized acceleration and relative displacement time histories of wall (After Richard and Elms, 1979)

An alternative design method based on a displacement approach, has been proposed by Richards and Elms (1979). Richard and Elms suggest an acceleration less than the expected peak ground acceleration be used for design, implying that relative displacement be allowed to occur between the wall and the backfill. The wall performance is considered satisfactory if the movement is less than a tolerable amount. For any earthquake record, the total relative displacement may be calculated using the sliding block method suggested by Newmark (1965). Figure 76 (Richards and Elms, 1979) shows how the relative displacement relates to the acceleration time history of the soil and wall. At a critical acceleration or yield coefficient, the wall is assumed to begin sliding. As previously discussed for slope stability problems, the accumulative relative displacements may be computed by a double integration process. Using the results obtained by Franklin and Chang (1977) for parametric evaluations utilizing Newmark's model, Richards and Elms (1975) suggest relative wall displacements D can be computed from the expression $D=0.087\ V^2\ (N/A)^{-4}/Ag$. Where V = maximum ground velocity, A = maximum ground acceleration coefficient, N = yield acceleration coefficient. By applying the above procedure to several simplified examples, Elms and Martin (1979) show that a value of $k_h = 0.5A$ would be adequate for most design purposes provided that allowance is made for an outward displacement of the abutment of the order of 10A inches.

The Richards and Elms model assumes the acceleration field is uniform within the backfill and equal to the ground acceleration. Nadim and Whitman (1983) present results of dynamic finite element analyses of retaining and backfill structures, which allow for amplification of ground motion in the backfill while computing permanent distortions. Results suggest that amplification of motion in the backfill can play an important role in displacement estimates in some cases.

7.3 Effects of Liquefaction

Werner and Hung (1982) report that by far the most significant source of earthquake induced damage to port and harbor facilities, has been the effects of pore pressure buildup in the loose to medium dense saturated cohesionless soils that prevail at such facilities. Damage arises from excessive backfill pressure applied to quay walls and bulkheads and to liquefaction induced foundation stability. Table 1 (Werner and Hung, 1982) illustrates examples of earthquake induced damage to port and harbor facilities in Japan during past earthquakes. Damage is characterized by significant sliding and tilting of bulkheads and quay walls arising from liquefaction. A specific example of damage to an anchored sheet pile bulkhead, is illustrated in Figure 77.

Where significant liquefaction potential is encountered at a site, remedial action conventionally calls for removal of the material and replacement with compacted fill, or insitu stabilization measures to prevent liquefaction such as densification by dynamic compaction. However, in some cases the expense of such measures can be prohibitive, particularly where dewatering is required to expedite removal of material or where extensive and deep dynamic compaction is required to alleviate the problem. In the case of bulkhead structures, an alternative approach is to design against potential liquefaction as illustrated in the example below.

Figure 78 shows the conceptual design of a liquefaction resistant bulkhead structure at a site where liquefaction potential exists to a depth of -25 feet. The existing level ground site has an elevation of zero feet with the water table at the surface. The site comprises layers of medium dense sands separated by clay layers. Based on extensive SPT and CPT data obtained at the site, insitu liquefaction strength curves for a magnitude 6.5 design earthquake were established and are shown as a function of SPT blow count in Figure 79. The idealized site profile is shown in Figure 80. Liquefaction evaluations using empirical blow count correlation charts, indicated that liquefaction could occur to a depth of -25 feet in the sand layers for a representative design earthquake event having a maximum peak acceleration of 0.25g.

The use of dynamic compaction at the site as a remedial measure was considered, but the costs found prohibitive in terms of the economics of the site development. The earthquake resistant design

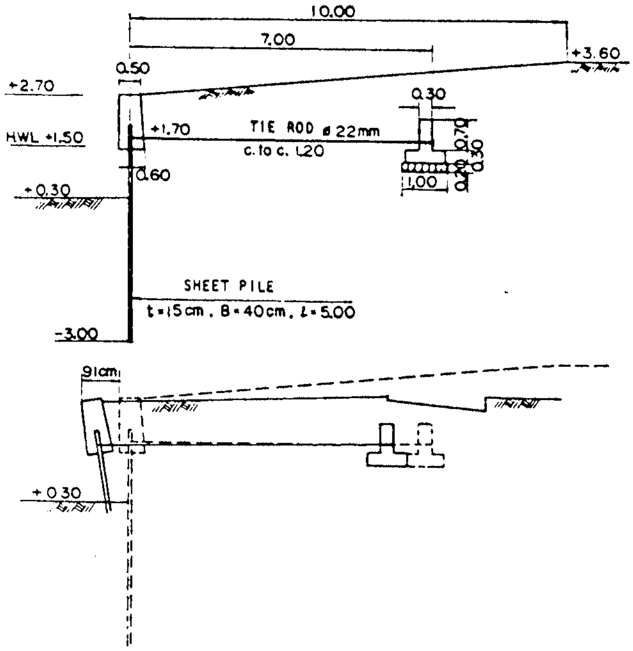

Figure 77 Damaged Quay wall at Hachinoe Port, 1968 Tokachi-Oki Earthquake (After Werner and Hung, 1982)

concept shown in Figure 78 was proposed as an alternative. This concept makes use of a relieving platform and a concrete slurry wall bulkhead structure. Battered piles provide support for the relieving platform and lateral resistance. Based on the liquefaction resistance curves shown in Figure 79, DESRA-2 effective stress response analyses were undertaken using an earthquake record representative of a magnitude 6.5 event with a peak ground acceleration of 0.25g. Maximum pore pressure increases for the various sand layers are shown in Figure 80. Initial liquefaction occurred in sand layers to a depth of -25 feet, as indicated by the empirical analyses. Assuming residual strengths were mobilized at the times when initial liquefaction occurred, post liquefaction inertial loading on the relieving platform was computed by continuing the response analyses. The lateral loads arising from the liquefied sand layers coupled with the post-liquefaction inertial loads could be successfully resisted by an economic battered pile and bulkhead system. The cost of the overall bulkhead structure was significantly less than the costs related to the dynamic compaction remedial measures.

In addition, the use of a drainage blanket between the existing ground and the superimposed 12-foot fill surcharge, provided a means for dissipation of

TABLE I
EARTHQUAKE INDUCED DAMAGE TO PORT AND HARBOR FACILITIES
(AFTER WERNER AND HUNG, 1982)

Earthquake			Port Location	Damage	
Location	Date	Magnitude		Description	Possible Cause(s)
Niigata, Japan	Jun 16, 1964	7.5	Niigata	Extensive damage due to liquefaction and sliding of soil strata. Summary of damage is as follows: Piers and landings: sliding (up to 5 m), submergence, and tilting Sheet-pile bulkheads: sliding (over 2 m), submergence, settlement (up to 1 m), and tilting. Extensive anchor failure Quay-walls: sea sliding (up to 3 m) settlement (up to 4 m) with extensive anchor failure and wall tilting	A,B,C
Tokachi-Oki, Japan	May 16, 1968	7.8	Hachinohe Aomori Hakodate	Steel sheet-pile bulkheads: outward sliding (0.9 m), tilting, and settlement, with anchor failure Gravity-type quay wall: sliding and settlement (0.4 m) Gravity-type breakwater: sliding (0.9 m) and pavement settlement (0.9 m) Steel sheet-pile bulkhead: seaward tilting (0.6 m) and apron settlement (0.3 m) Quay-wall: settlement (0.6 m) and sliding (0.4 m)	A A A,B
Nemuro-Hanto-Oki, Japan	Jun 17, 1973	7.4	Hanasaki Kiritappu	Gravity-type quay wall: sliding (1.2 m) and settlement (0.3 m) with corresponding apron settlement (1.2 m) Steel sheet-pile bulkhead: sliding (2 m) and anchor failure Steel sheet-pile bulkhead: relatively minor damage Gravity-type quay walls: relatively minor damage	A,B
Miyagi-Ken-Oki, Japan	Jun 12, 1978	7.4	Shiogama Ishinomaki Yuriage Sendai	Concrete gravity-type quay wall: outward tilting (0.6 m) and apron pavement settlement (0.4 m) Steel sheet-pile bulkheads: outward sliding (up to 1.2 m) and apron settlement (up to 1 m) Concrete block retaining wall: sliding, tilting, and cracking with corresponding pavement settlement (0.2 m) relative to wall Concrete block gravity quay wall and steel sheet-pile bulkhead: large horizontal displacements (up to 1.2 m) Steel sheet-pile bulkheads: cracking and settlement of apron and pavements	A,B

Legend

A: Excessive lateral pressure from backfill materials, in the absence of complete liquefaction, and possibly accompanied by reduction in water pressure on outside of wall
B: Liquefaction
C: Localized sliding
D: Massive submarine sliding
E: Vibrations of structure

excess pore pressures generated in underlying materials, hence, minimizing the potential for damage from cracking of the fill. The 12-foot fill layer provided a measure of protection to damage of surface structures as has been observed in case histories reported by Ishihara (1985). It is believed that with a full understanding of the liquefaction problem including post-liquefaction response, bulkhead structures can be successfully and economically designed to withstand the effects of liquefaction with minimum damage.

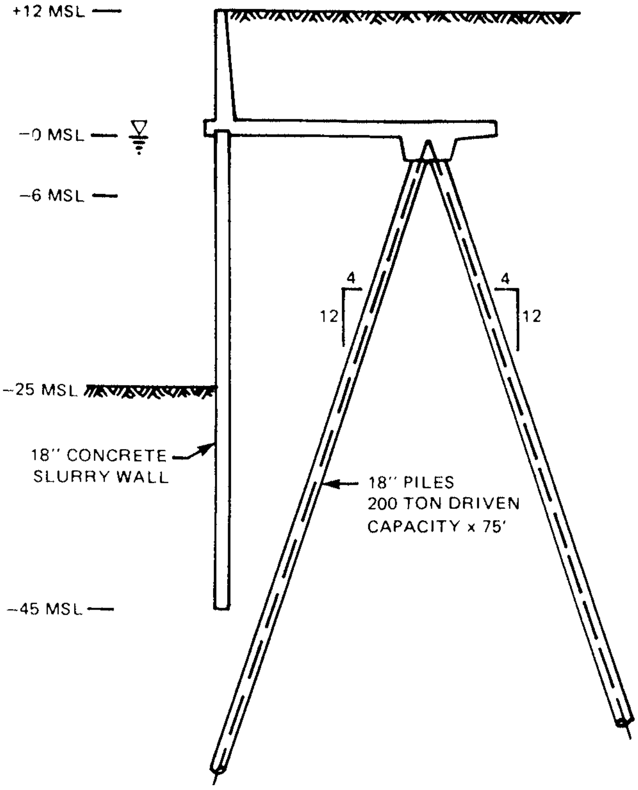

Figure 78 Bulkhead Design - Use of battered piles and relieving platform to resist liquefaction.

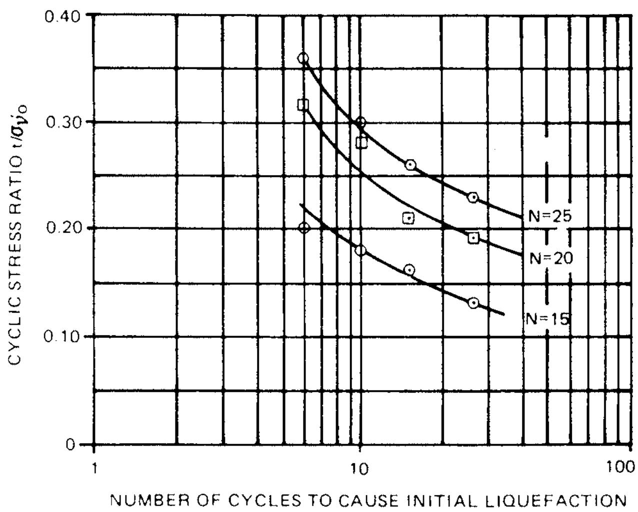

Figure 79 Liquefaction strength curves used for site response analysis

8.0 PILE FOUNDATIONS

8.1 Background

The 1964 Alaska earthquake highlighted the vulnerability of pile foundations to ground liquefaction.

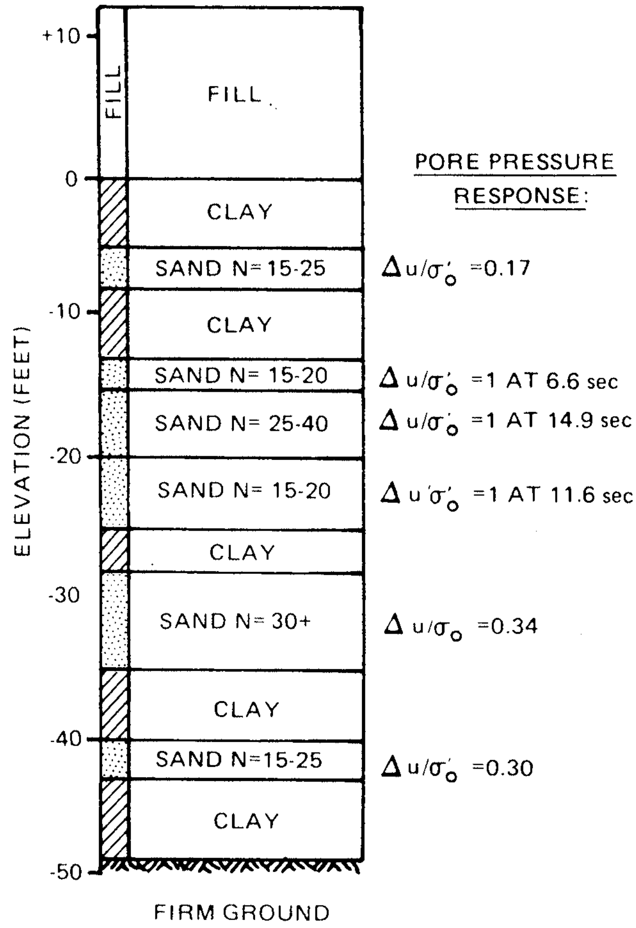

Figure 80 Soil strata and pore pressure response for bulkhead design.

Of the many bridges damaged by the earthquake in Alaska, the most severe damage occurred for those where piling was embedded in saturated loose to medium dense sands and silts having SPT blow counts of less than 20. Ross, et. al. (1973) provide a well documented account of the nature of the damage. For bridges founded on piling embedded in gravels and gravelly sands, in most cases little or only moderate damage occurred. This probably reflects the influence of rapid dissipation of earthquake induced pore water pressures due to the high permeability of such materials.

The interaction between a pile and the surrounding soils during an earthquake is illustrated schematically in Figure 81. Broadly speaking, the interaction may be viewed as having three zones. Near the surface (say to a depth of 5 pile diameters) the interaction is dominated by relative displacements between the pile and soil generated by inertial loading from the pile supported structure. At greater depths the pile deflections will follow the ground deformations because of the high relative stiffness of interaction elements. Between these two zones, a gradual transition from one kind of interaction to another will occur.

Degradation of the lateral soil support stiffness (p-y curves) may occur either from pore water pressure increases arising from the earthquake free field response, or from localized pore water pressure increases in the vicinity of the pile head generated by relative displacements caused by

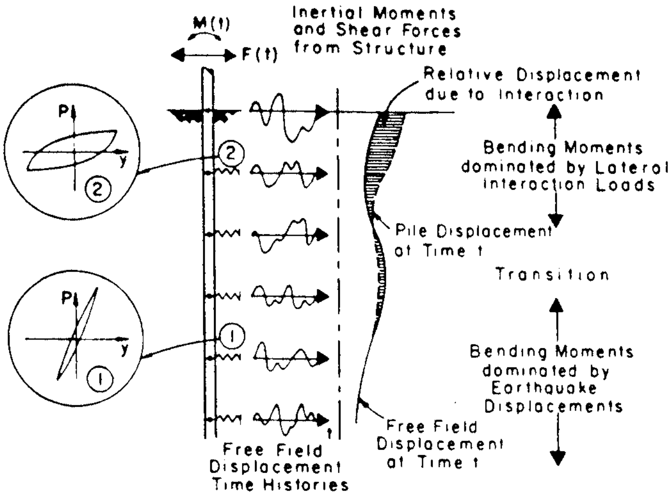

Figure 81 Soil pile interaction mechanisms during seismic loading (After Finn and Martin, 1979)

structural inertial loads. If liquefaction occurs, either total or partial loss of lateral stiffness support may occur depending on the density of the sand. From a design standpoint, it is clearly critical that the potential for and depth of liquefaction be identified prior to design.

Although many seismic design problems can be identified in relation to pile foundation design, the discussion below focuses on the liquefaction problem in view of its significance.

8.2 Analyses of Liquefaction Effects on Lateral Resistance

Simple analyses of the effects of liquefaction on the lateral stiffness of single piles may be made by assuming that liquefaction completely removes lateral support to a given depth. Finn and Martin (1979) describe analyses to show the influence of progressive lateral degradation of p-y curves on lateral stiffness, the results of which are summarized in Figure 82. The study considered a 100 foot long 4 foot diameter pile embedded in a saturated sand deposit subjected to earthquake loading. Time histories of free field pore pressure buildup were computed using the program DESRA-2. Liquefaction occurred to a depth of 10 feet after 10 seconds of shaking. p-y curves were assumed to degrade in proportion to the shear stiffness degradation which occurred as the effective stress reduced. Zero lateral support was assumed at the point of liquefaction. Figure 82 shows that the reductions in normalized stiffness for free and fixed piles calculated for a range of deflections, fall within a comparatively narrow band.

The significance of lateral stiffness degradation is further illustrated in Figure 83. The figure shows the distribution of deflection and bending moment (250 kip lateral load) for values of zero pore pressure increases (no degradation) and pore pressure increases after six seconds of shaking (significant degradation in p-y curves to a depth of about 20-feet). The effects of degradation are to increase the bending moments and deflections with maximum bending moments occurring at greater depths. The pile in effect is forced to deflect to greater depths to mobilize the needed lateral resistance.

The above free field analyses do not take into account the effects of local pore pressure increases at the pile head which may occur due to local interaction arising from structural inertial loads. This problem has been examined using a full scale experimental study of a single free headed instrumented vertical pile embedded in a saturated sand subjected to lateral sinusoidal dynamic loading. The test setup and results for this experimental project are described by Tsai, et. al. (1981) and Scott, et. al. (1982). During the study, liquefaction of the soil near the pile head occurred. Back calculations of measured data described by Ting (1987), enabled dynamic p-y curves to be computed at various depths below the ground surface. Figure 84 shows cyclic p-y curves for depths 1-6 pile diameters below the ground surface. Note the degraded dilatant response at a depth of 1 pile diameter below the ground surface, indicative of the high pore pressures generated in this region.

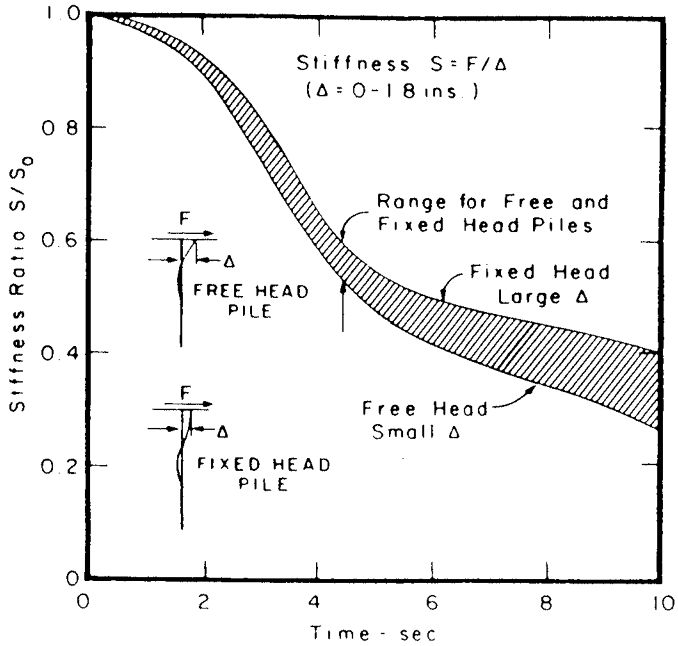

Figure 82 Lateral stiffness degradation with pore water pressure increase (After Finn and Martin, 1979)

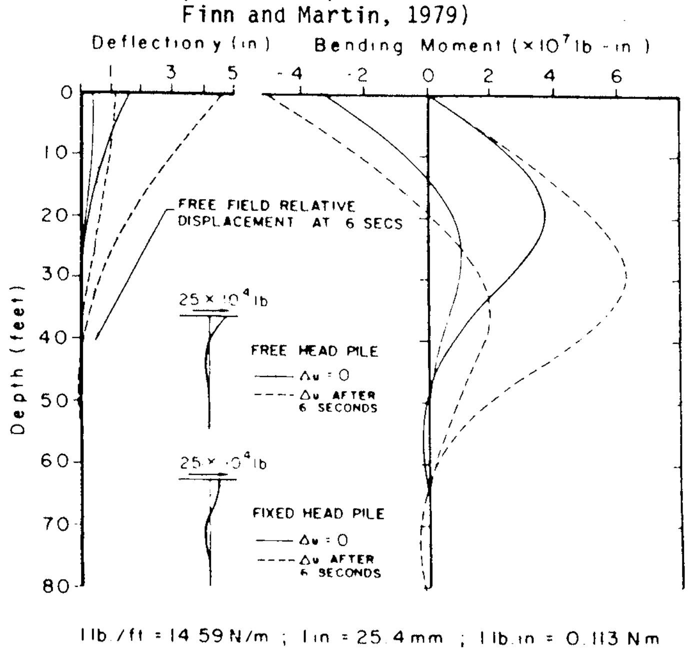

Figure 83 Effects of liquefaction on bending moments and deflections (After Finn and Martin, 1979)

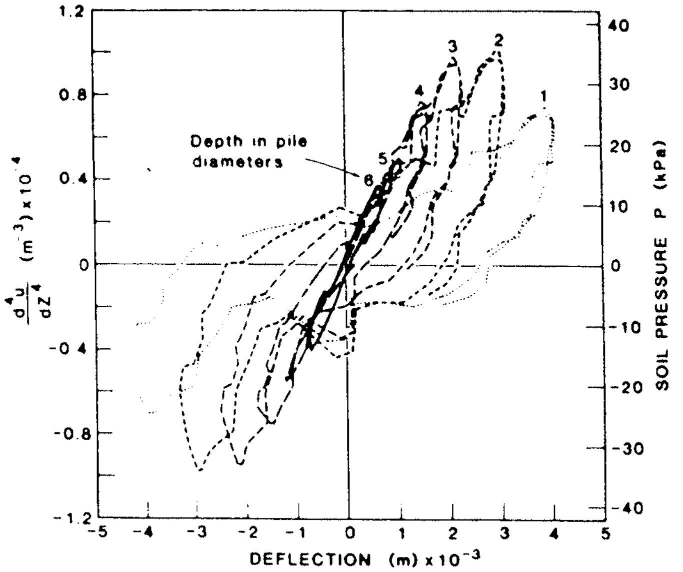

Figure 84 Cyclic p-y curves where liquefaction is induced by cyclic lateral loading (After Ting, 1987)

8.3 Bridge Pile Foundations

Although remedial measures to improve liquefaction resistance through replacement or insitu densification are always the first options to be explored, in many cases pile foundation design where the potential for liquefaction exists, can be accomplished reasonably economically providing ductile piles are founded at depths considerably below the zone of liquefaction.

Lam, et. al. (1987), present a study of a representative bridge pile foundation configuration founded in saturated sands, where the influence of liquefaction on foundation loads, stresses and deflections are examined. Figure 85 shows a schematic configuration of the bridge structure and foundation system studied. The structure represents a road overcrossing bridge in California. Bridge instrumentation indicated that the natural period of this bridge significantly increased during the 1979 Imperial Valley earthquake and it has been speculated that soil liquefaction was the underlying cause. The bridge is a 208-foot long two span reinforced concrete box girder supported at mid-span by a single column pier founded on a 5x5 timber pile group. The piles extend to a depth of 43.5 feet below the ground surface.

For the purpose of sensitivity analyses, the bridge was assumed to be supported by either battered (external piles) or vertical piles embedded in a idealized uniform sand deposit. The piles were assumed to have a fixed head condition at the pile cap. Two liquefaction scenarios were considered: (1) no liquefaction, and (2) liquefaction to a 20-foot depth.

The fundamental natural frequencies of the bridge structure for the cases examined are summarized in the table below.

Foundation Description	Liquefaction	Fundamental Frequency (Hz) Longitudinal	Transverse
Vertical	No	4.03	2.06
Battered	No	4.22	2.51
Vertical	Yes	0.72	0.70
Battered	Yes	1.68	1.64

It is seen that there is a dramatic reduction in the fundamental frequency when the soil liquefies which can significantly reduce the inertial loading on the structure. There is also a significant increase in the natural frequency when foundation piles are battered.

Prior to soil liquefaction, most of the earthquake induced longitudinal forces are absorbed by the abutments. After the soil liquefies (including abutment soils), all lateral loads are transferred to the central pier because of loss of resistance of abutment soils.

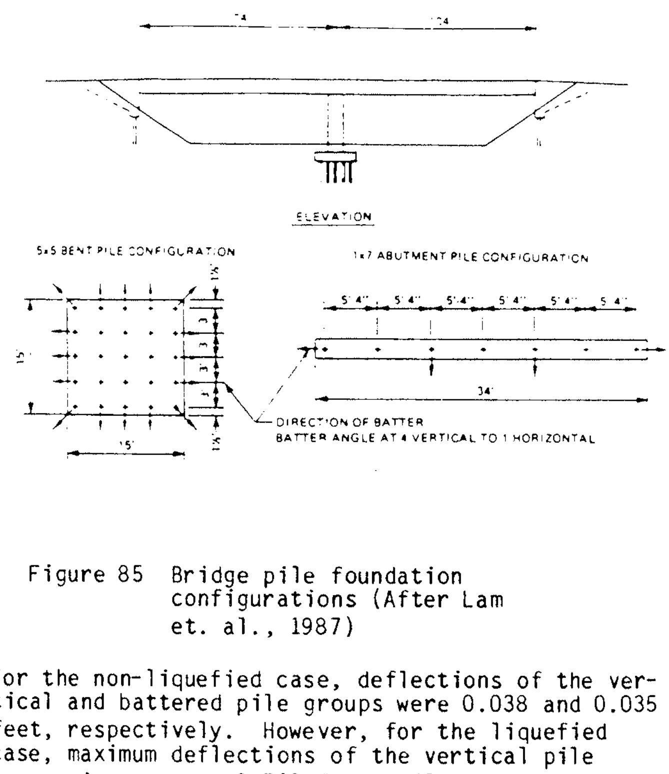

Figure 85 Bridge pile foundation configurations (After Lam et. al., 1987)

For the non-liquefied case, deflections of the vertical and battered pile groups were 0.038 and 0.035 feet, respectively. However, for the liquefied case, maximum deflections of the vertical pile groups increase to 0.548-feet while the battered pile group increase to only 0.177-feet. The effects of liquefaction on the bending moment distribution in the piles is shown in Figure 86. Bending moment diagrams for the non-liquefied case are typical for fixed head piles. For the liquefied case, moment is dominated by pile buckling in the case of vertical piles. The following summarizes the pertinent findings from the sensitivity studies:

1. Soil liquefaction reduces the foundation stiffness significantly and can lead to lower structural loads. However, lateral movements of pile foundations at ground level are increased, and allowable deflections can control design.

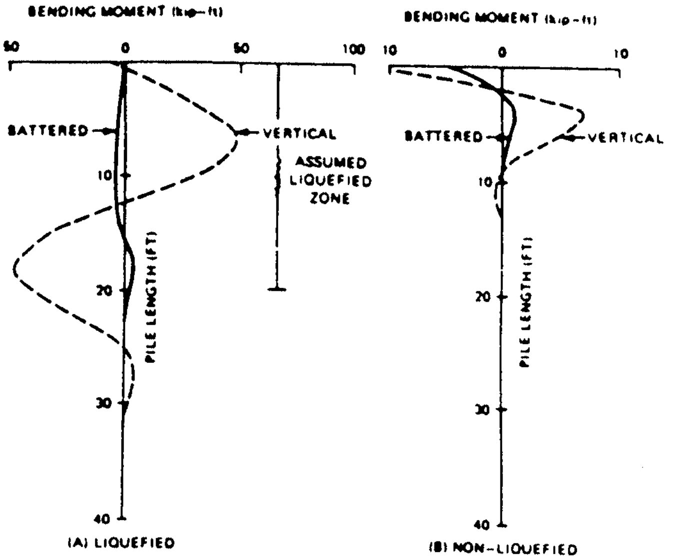

Figure 86 Effects of liquefaction on bending moment for vertical and battered piles (After Lam et. al., 1987)

2. For the seismic design of piles, it is not only important to design for sufficient axial capacity to support the dead weight but also to prevent excessive rotation of a pile group caused by the overturning moment induced by an earthquake.

3. A battered pile foundation can potentially reduce the lateral deflection deformation at ground level and should be considered when lateral deflections exceed design criteria.

4. Liquefaction can potentially reduce pile supported abutment stiffness significantly, leading to increased loads on pier foundations.

A more comprehensive review of foundation design procedures for highway bridges, including a number of parametric studies, has recently been reported by Lam and Martin (1986).

9.0 CONCLUSIONS

Over two decades have passed since the devastating earthquakes in Niigata and Alaska highlighted many of the geotechnical aspects of earthquake engineering. In this time, a considerable amount of knowledge has been accumulated both from the standpoint of research and design practice. However, despite the progress that has been made, more research and development is required and a greater transfer of technology from the research community to the practising profession is needed.

With respect to research in general, emphasis should be placed on two areas, namely,

1. Basic research to continue to improve our understanding of observed phenomena in the field and underlying constitutive relationships, and

2. Focused applied research aimed at transferring design tools to the practicing profession.

   Less research time should be spent on unnecessary refinement of analytical techniques, which is often done at the expense of technology transfer.

With respect to research needs, I see the following as important:

- Continued instrumentation of selected highly seismic sites and earth structures
- Increased emphasis on the study of soils other than clean sands, particularly silts
- Improved methods for evaluating the permanent deformations induced by prescribed earthquake shaking
- Increased emphasis on centrifuge model validation tests to provide insight into mechanisms of failure and for verifying analytical computer codes
- Increased emphasis on particulate constitutive modeling for cohesionless materials.

On the subject of technology transfer from research to design practice, I see the need for increased communication between practising engineers responsible for seismic design of major facilities and the research community. In this regard, there is a urgent need for:

- rationalizing existing analytical tools and research data to develop pragmatic design guides for prediction of earthquake displacements of earth structures, particularly deformations related to post liquefaction performance.
- continued development of improved seismic design guidelines for foundation systems both from the standpoint of assessing foundation stiffness for soil structure interaction analyses and evaluation of load distribution into foundation systems
- Increased transfer of technology at international levels, into design practice. Japan in particular, has major research programs underway. Cooperation at a professional level as well at a research level, would have significant benefits.

10.0 ACKNOWLEDGMENTS

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APPLICATION FOR MEMBERSHIP

of
New Zealand Geomechanics Society

A TECHNICAL GROUP OF THE INSTITUTION OF PROFESSIONAL ENGINEERS OF NEW ZEALAND

The Secretary
The Institution of Professional Engineers New Zealand
P.O. Box 12-241
WELLINGTON

I believe myself to be a proper person to be a member of the N.Z. Geomechanics Society and do hereby promise that, in the event of my admission, I will be governed by the Rules of the Society for the time being in force or as they may hereafter be amended and that I will promote the objects of the Society as far as may be in my power.

I hereby apply for membership of the N.Z. Geomechanics Society and supply the following details:

NAME: ______________________________
(to be set out in full in block letters, surname last)

PERMANENT ADDRESS: ______________________________
____

QUALIFICATIONS AND EXPERIENCE: ______________________________
____

NAME OF PRESENT EMPLOYER: ______________________________

NATURE OF DUTIES: ______________________________

Affiliation to International Societies: (All members are required to be affiliated to at least one Society, and applicants are to indicate below the Society/ies to which they wish to affiliate).

I wish to affiliate to:

International Society for Soil Mechanics for Foundation Engineering	(ISSMFE)	Yes/No	($11.00)
International Society for Rock Mechanics	(ISRM)	Yes/No	($12.00)
International Association of Engineering Geology	(IAEG)	Yes/No	($ 9.00)
		(with Bulletin	$15.00)

SIGNATURE OF APPLICANT: ______________________________

DATE: ______/ ______/ 19______

NB: Affiliation Fees are in addition to the basic Geomechanics Soiety membership fee of $24.00 which is reduced to $20.00 if member of IPENZ.

PLEASE DO NOT SEND FEES WITH THIS APPLICATION. AN ACCOUNT WILL BE SENT ON YOUR ACCEPTANCE INTO THE SOCIETY.

Nomination:

I ______________________________ being a financial member of the N.Z. Geomechanics Society hereby nominate ______________________________ for membership of the above Society.

Signed: ______________________________ Date: ______/ ______/ 19______

ORGANISATION DES REUNIONS

Pour chaque réunion un conférencier sera invité qui donnera une vue d'ensemble ou partielle du sujet (30 minutes). Ensuite un panel critiquera les exposés soumis (une heure environ). Le reste du temps de la réunion sera consacré à une "discussion à sujets fixés" d'environ une heure et une discussion spontanée d'environ une demi-heure. Les exposés soumis ne seront pas présentés oralement, mais les auteurs auront la possibilité de présenter leur exposé "sous forme d'affiche" au palais des congrès. Les conditions pour l'affichage et pour la discussion à sujets fixés vous seront données dans le deuxième Bulletin.

Dans les Colloques du mercredi un nombre limité d'exposés seront sélectionnés pour être présentés dans leur totalité par leurs auteurs; d'autres pourront être affichés.

Des comptes-rendus seront publiés avant le congrès et mis à la disposition avant et pendant le congrès . Le deuxième Bulletin contiendra une liste des résumés acceptés. La date limite de réception des exposés est février 1990. Les exposés seront jugés préalablement.

INFORMATION GENERALE

Langues
Les langues officielles de l'AIGI sont l'anglais et le français; les exposés et résumés peuvent être écrits dans une de ces deux langues. L'organisation prévoira une traduction simultanée lors des réunions.

Demande d'exposés
Les auteurs sont invités à présenter au Bureau d'Organisation du Congrès des résumés de moins de 200 mots, avec indication du sujet en question. Les auteurs des résumés acceptés recevront des instructions pour la préparation d'un exposé qui puisse être reprographié. Les résumés acceptés avant août 1989 peuvent être mentionnés dans le deuxième Bulletin.

Dates-Clés

Septembre 1989	Deuxième Bulletin
Novembre 1989	Date limite pour la présentation des résumés
Février 1990	Date limite pour la présentation des exposés
Mars 1990	Date limite d'inscription des exposants
Mai 1990	Date limite d'inscription des participants

Excursions et sorties
Le 8 août une excursion aura lieu qui visitera les travaux du plan Delta et la digue de barrage de l'Escaut oriental. Après le congrès il y aura de courtes excursions (de deux à sept jours) à des lieux intéressants du point de vue géotechnique en Europe occidentale. Nous espérons qu'il y aura entre autres la possibilité de visiter les travaux d'infrastructure en France qui se rapportent à la construction du tunnel sous la Manche et les travaux qui ont été entrepris en préparation des Jeux Olympiques d'hiver 1992 dans les Alpes françaises. Lors du congrès il y aura un programme complet de sorties et de divertissements pour les personnes accompagnantes.

Inscription
Tous les renseignements concernant les frais et la manière d'inscription, les excursions et les sorties seront donnés dans le deuxième Bulletin.

Si vous voulez être sûr de recevoir le deuxième Bulletin, vous êtes prié de retourner le coupon-réponse attaché. Si vous voulez recevoir une invitation personelle pour le congrès, veuillez vous adresser au Bureau d'Organisation du Congrès. Le Comité d'Organisation vous fera parvenir une telle invitation sans assumer aucune responsabilité financière.

Accès et accomodations KLM
L'organisation de voyage officielle pour le congrès est la KLM qui possède un service régulier entre Amsterdam et chaque aéroport de quelque importance dans le monde. A l'intérieur de l'Europe, Amsterdam est facilement accessible par le chemin de fer et par la route. Amsterdam a plus de 200 hôtels dans toutes les catégories; en termes internationaux les hôtels à Amsterdam sont relativement bon marché.

Exposition
Il y aura une grande exposition entres autres d'équipement de terrain et de laboratoire, de progiciel et de logiciel, d'équipement géophysique, des services et des systèmes.

Subventions et bourses
Un nombre de subventions et de bourses seront disponsibles pour aider des scientifiques individuels qui désirent participer au congrès, surtout pour les intéressés des pays en développement. De plus amples renseignements seront donnés dans le deuxième Bulletin.

IAEG/AIGI

Sixth International Congress of the International Association of Engineering Geology

Announcement

Premier Bulletin

25th Anniversary Engineering Geology

EN GEOL IAEG AIGI

25 annees de Geologie de l'Ingenieur

COMMITTEE OF RECOMMENDATION
COMITÉ DE RECOMMENDATION

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ORGANISING COMMITTEE/ COMITÉ D'ORGANISATION

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"25 YEARS OF ENGINEERING GEOLOGY"

The Netherlands National Group of the IAEG cordially invites you to the Sixth International Congress of the IAEG. This will mark the Silver Jubilee of the IAEG and is intended to give a broad review of developments in the total field of Engineering Geology.

SCIENTIFIC PROGRAMME: DATES AND THEMES

Monday, 6 August
Opening ceremony
'Engineering Geology of The Netherlands'

1 ENGINEERING GEOLOGICAL MAPPING AND SITE INVESTIGATION.
Including: engineering geological and environmental maps and plans; boring and sampling; laboratory and in situ testing and instrumentation; procedures, classification and interpretation for engineering design.

Tuesday, 7 August
2 REMOTE SENSING AND GEOPHYSICAL TECHNIQUES.
Including: aerial and terrestrial photography and photogrammetry; multispectral sensing and radar (including their applications in environmental and engineering geology); geophysics on land; geophysics overwater.

3 HYDRO-ENGINEERING GEOLOGY
Including: groundwater flow in dams, reservoirs, tunnels and excavations; reduction of groundwater flow by dewatering, injection and other methods; groundwater supply: methods and environmental consequences; groundwater pollution and waste disposal: environmental impact.

Wednesday, 8 August
Participants may either follow a technical excursion or attend symposia. Four symposia are proposed, each dealing in detail with a theme considered to be of great importance for Engineering Geology in 1990. The topics for the symposia are:
1 Computer use in Engineering Geology
2 Environmental protection, pollution and waste disposal
3 Coastal protection and erosion, including the engineering and environmental consequences of rises in sea level
4 Engineering Geology in the oil industry

4 SURFACE ENGINEERING GEOLOGY
Including: natural geotechnical hazards and the environment; foundations; stability of excavated slopes and embankments; infrastructure (roads, railways, pipelines, etc.)

5 UNDERGROUND ENGINEERING GEOLOGY.
Including: tunnels and shafts; large permanent underground openings; engineering and environmental problems caused by subsidence brought about by underground extraction of minerals, oil and gas; underground storage of energy, liquids and waste.

Friday, 10 August
6 ENGINEERING GEOLOGY OF LAND AND MARINE HYDRAULIC STRUCTURES.
Including: flood control and erosion protection: environmental impact; offshore structures and seabed stability; coastal protection and land reclamation; harbours, causeways and breakwaters.

7 CONSTRUCTION MATERIALS.
Including: exploration; production - methods and environmental impact; testing and classification; problem materials.

SESSION ORGANISATION

For each theme there will be an invited 'Key-note Speaker' who will present a review of the State of the Art for the whole or part of each theme (30 minutes). Panel reporters will then review the papers submitted (about one hour). The remainder of the session will be devoted to discussion, giving about one hour to 'registered discussion' and about one half hour to spontaneous discussion. Papers submitted will not be orally presented, but the author(s) will have the opportunity to present the paper in 'poster form' outside the congress hall. Requirements for posters and registered discussion will be given in the second announcement.

In the symposia a limited number of papers will be selected for complete oral presentation by their authors; others may be presented as posters.

Proceedings will be published before and be available at, the congress. The second announcement will contain a list of abstracts accepted. The deadline for receiving papers is February 1990. Papers will be refereed.

GENERAL INFORMATION

Languages
The official languages of the IAEG are English and French; papers and abstracts may be written in either language. Simultaneous translation will be provided at the Sessions.

Call for papers
Authors are invited to submit abstracts of less than 200 words, indicating the relevant theme, to the Congress Organisation Bureau. Authors of accepted abstracts will receive instructions for the preparation of a camera-ready paper. Abstracts accepted before August 1989 will be mentioned in the second announcement.

Key Dates

September 1989	Second Announcement
November 1989	Deadline for submission of abstracts
February 1990	Deadline for submission of papers
March 1990	Deadline for registration of exhibitors
May 1990	Deadline for registration of participants

Excursions and social programme
There will be an excursion to visit the Delta Works including the Eastern Scheldt storm surge barrier on August 8th. After the congress there will be short (two to seven days) excursions to sites of engineering geological interest in Western Europe. It is hoped that these will include a visit to the infrastructure works in France associated with the construction of the Channel Tunnel and the works being undertaken in preparation for the 1992 Winter Olympic Games in the French Alps.
There will be a full social programme for accompanying persons during the course of the congress.

Registration
Full information about registration costs and procedures, excursions and social programmes will be given in the second announcement.

Those who wish to be sure that they will receive the second announcement should return the attached reply card. Those who wish to receive a personal invitation to attend the congress should apply to the Congress Organisation Bureau. The Organising Committee will provide such an invitation without assuming any financial responsibility.

Access and accommodation KLM
The official carrier for the congress is KLM which flies between Amsterdam and every major airport in the world. Within Europe Amsterdam is easily reached by rail or road. Amsterdam has more than 200 hotels of all categories - in international terms Amsterdam hotels are comparatively cheap.

Exhibition
There will be an extensive exhibition including laboratory and surveying equipment, computer hardware and software, geophysical equipment, services and systems.

Grants and fellowships
A number of grants and fellowships will be awarded to help individual scientists, principally from developing countries, who wish to participate in the congress. Further details will be in the second announcement.

Addresses/Adresses

Venue / Arrivée — Congress Organisation Bureau / Bureau d'Organisation du Congrès

RAI	QLT/Congrex
International Congress Centre	Keizersgracht 782
Europaplein 8	1017 EC Amsterdam
1078 GZ Amsterdam	The Netherlands
The Netherlands	Telephone:
Telephone:	+31 (0)20-261372
+31 (0)20-5491212	Telefax:
Telefax:	+31 (0)20-259574
+31 (0)20-464469	Telex: 14527 congx nl
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Ville ____________________

Postal Code
Code Postal ____________________

Country
Pays ____________________

☐ I wish to receive the second congress announcement including registration– and hotel reservation forms.
Je veux recevoir le deuxième bulletin du congrès avec les formulaires d'inscription et de réservation d'hôtel.

☐ I intend to submit an abstract entitled:
J'ai l'intention de présenter un exposé intitulé:

____

____

☐ I am interested in attending a post-congress excursion
Je désire participer à une excursion après le congrès.

Please indicate your choice:
Veuillez marquer votre choix:

☐ Excursion in North - West Europe lasting 2 - 7 days
Excursion dans le nord-ouest de l'Europe de 2 à 7 jours
☐ French Alps/Alpes françaises
☐ Channel Tunnel/Tunnel de la Manche

Signature ____________________

Date ____________________

"25 ANNEES DE GEOLOGIE DE L'INGENIEUR"

Le Groupe National Néerlandais de l'AIGI a le plaisir de vous inviter au sixième Congrès International de l'AIGI. Ce congrès marquera le 25ème anniversaire de l'AIGI et vise à faire un large tour d'horizon des développements dans le domaine entier de la Géologie de l'Ingénieur

PROGRAMME SCIENTIFIQUE - DATES ET THEMES

Lundi, 6 août
Cérémonie d'ouverture
"La Géologie de l'Ingénieur aux Pays-Bas"

1 CARTOGRAPHIE ET ETUDE GEOTECHNIQUE DU TERRAIN

Comprenant: cartes et plans géotechniques et de l'environnement; forage et sondage; expérimentation et instrumentation en laboratoire et sur le terrain; procédures, classification et interprétation pour des plans techniques.

Mardi, 7 août

2 TELEMESURE ET TECHNIQUES GEOPHYSIQUES

Comprenant: photographie et photogrammétrie aériennes et terrestres; détections multispectrales et radar (avec leurs applications dans la géologie de l'environnement et dans la géologie de l'ingénieur); géophysique terrestre; géophysique marine.

3 GEOLOGIE HYDROTECHNIQUE

Comprenant: courant phréatique dans les digues, réservoirs, tunnels et excavations; réduction du courant phréatique par drainage, par injection et par d'autres méthodes; approvisionnement phréatique - méthodes et conséquences pour l'environnement; pollution de l'eau souterraine et traitement des déchets - influence sur l'environnement.

Mercredi, 8 août
Les participants peuvent soit prendre part à une excursion technique, soit assister à des Colloques.
Quatre Colloques sont prévus, qui tous traitent à fond un thème considéré comme ayant une importance primordiale pour la Géologie de l'Ingénieur en 1990. Les sujets des Colloques sont:

1 L'ordinateur dans la Géologie de l'Ingénieur
2 Protection de l'environnement, pollution et traitement des déchets
3 Protection littorale et érosion, y compris les conséquences techniques et les conséquences pour l'environnement de la hausse du niveau de la mer
4 Géologie de l'Ingénieur dans l'industrie pétrolière

Jeudi, 9 août

4 GEOLOGIE DE L'INGENIEUR DE LA SURFACE

Comprenant: dangers géotechniques naturels et l'environnement; fondations; stabilité des collines et des digues excavées; infrastructure (routes, chemins de fer, pipelines, etc.)

5 GEOLOGIE DE L'INGENIEUR SOUTERRAINE

Comprenant: tunnels et fosses; grands vides souterrains permanents; problèmes techniques et de l'environnement causés par la subsidence à la suite de l'extraction souterraine de mineraux, de pétrole et de gaz; stockage souterrain d'énergie, de liquides et de déchets.

Vendredi, 10 août

6 GEOLOGIE DE L'INGENIEUR DE STRUCTURES HYDRAULIQUES SUR TERRE ET SUR MER

Comprenant: contrôle de la marée et protection contre l'érosion - influences sur l'environnement; structures off shore et stabilité du fond de la mer; protection littorale et accroissement territoriale; ports, jetées et brise-lames.

7 MATERIAUX DE CONSTRUCTION

Comprenant: l'exploration; les méthodes de l'exploitation et les influences sur l'environnement; essais et classification; matériaux problématiques.

Please affix
stamp here

Sixth International IAEG Congress
QLT/CONGREX
Keizersgracht 782
1017 EC Amsterdam
The Netherlands

Organizing Committee of the First Iranian International Seminar on Soil Mechanics and Foundation Engineering.

Plan and Budget Organization
(Technical Research and Standards Bureau)

Tehran - IRAN
Post Cod : 15316
Dr. Beheshti Ave., Pakistan St.,
2nd Alley - No 7 - 4th Floor

IN THE NAME OF GOD

First Announcement

Call For Papers

First Iranian International Seminar On Soil Mechanics And Foundation Engineering

Tehran - Iran

November - 1990

Sponsored by :

Plan and Budget Organization
(Technical Research and Standards Bureau)

I- Object of the Seminar

The object of the seminar is to provide an opportunity for engineers and scientists working in the field of soil mechanics and foundation engineering to meet and present new ideas, achievements, and experiences.

There will be a special emphasis on practical applications and papers dealing with engineering practice in all its aspects such as:

- Site Investigation / Field and Laboratory Testing
- Soil Improvement and Stabilization
- Prediction and Analysis of Soil Deformation, Settlement and Displacement
- Special Problem Soils, as Desert Lands
- Retaining Structures, as Dams, Walls
- Prediction and Analysis of Dynamic Effects
- Offshore Exploration and Foundation
- Tunelling / Shaft Sinking / Buried Structures / Mining
- Numerical Models and Associated Parameters
- Groundwater Effects and Related Problems
- Subgrade and Pavement Materials
- Nonlinear Behavior of Materials in Three Dimensions

II- Official Language

The official languages of the seminar will be Persian (Farsi) and English

III- Submitting Papers

Parties interested in presenting papers are kindly requested to mail an abstract of not more than 300 words, type-written in English on international A4 size by January 31, 1990.

IV- Invitation to Exhibitors

In coordination with the seminar an exhibition will be arranged to display recent developments in soil mechanics and foundation engineering equipments and techniques. The Technical Exhibition will be held at the same site as the saminar

Cooporations interested in participating in the Exhibition should contact with the Organizing Commitee of the seminar.

V- Facilities

Seminar Organizers shall undertake all the expences of air travel to and from Iran, accomodation, and domestic tour of those whose papers are accepted for presentation by the Scientific Board of Seminar Organizers.

VI- Time Table

- Deadline for Abstracts
  January 31, 1990.
- Deadline for Full Papers
  April 30, 1990.

VII- Correspondence

Please send all correspondence regarding to seminar to :

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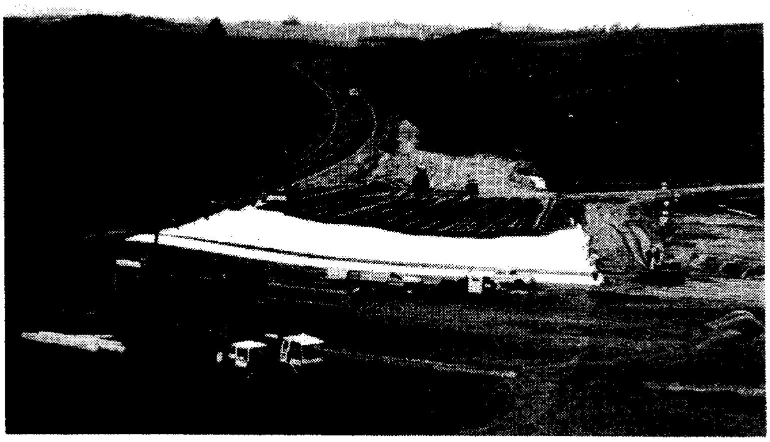

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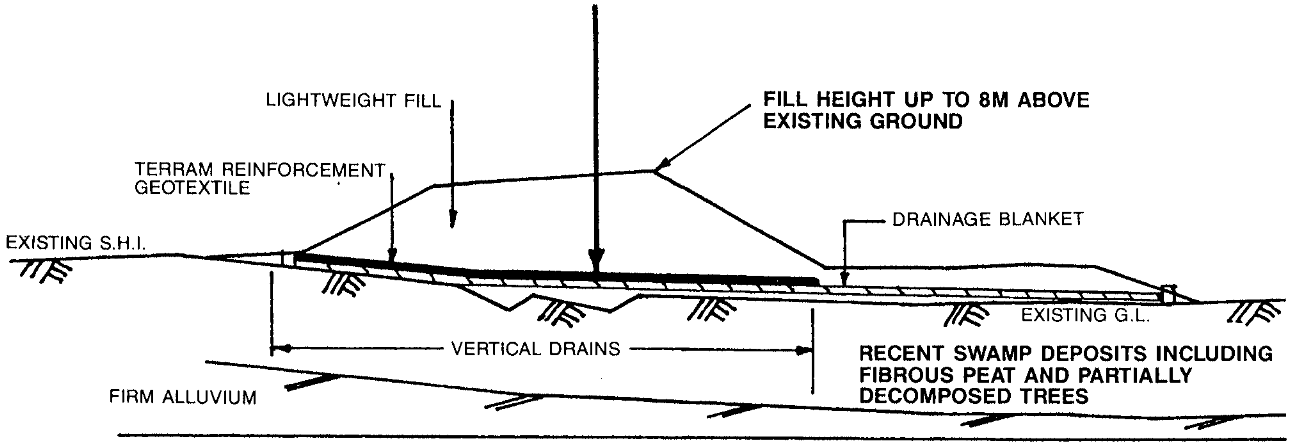

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Realignment of State Highway 1 at the Pokeno rail overbridge necessitated construction of an 8m high fill embankment over a soft swamp. TERRAM WB60/5 woven geotextile was used by Works Consultancy Services to provide a reinforcing layer in the base. The fabric has stabilised the embankment and has enabled project time constraints to be met. Instrumentation to monitor performance of the project was also supplied by Ground Engineering Ltd. This includes piezometers installed beneath the embankment and strain gauges fixed to the geotextile.

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