SEMINAR ON GEOTECHNICAL ASPECTS OF EARTHQUAKE ENGINEERING

University of Auckland – May, 1970

Over eighty civil engineers and scientists participated in the Seminar. The general interest was shown by the lively discussion which took place after every lecture, or panel discussion.

Speakers at about half the sessions were Visiting Lecturers. Mr. P. W. Taylor and Dr. G. R. Martin of the Civil Engineering department gave the remainder and have supplied the following resumé of their contributions :-

DYNAMIC PROPERTIES OF SOILS

Problems which require, for their solution, knowledge of dynamic properties of soils include (in addition to machine foundation vibration, not considered here) the estimation of the effects of soil layers in modifying surface motion; the design of foundations for earthquake-resistant structures; soil-structure interaction problems and the estimation of stability of earth dams and other slopes during earthquake. Liquefaction is considered separately.

For soil response problems (dealt with in "Surface Layer Modification of Earthquake Waves" later) small amplitude dynamic strains are appropriate (1% to 0.01% or less). It is found that soils are highly non-linear, i.e. the effective value of shear modulus is strongly dependent on amplitude. Thus both modulus and damping factor must be determined under a wide range of amplitudes. Results of strain-controlled dynamic triaxial tests on various local clays were given.

In strain-controlled tests, failure is impossible. For stability problems, then, a stress-controlled device is required. That developed at Auckland can provide a combined static plus sinusoidally-varying dynamic load to a sample in a triaxial cell. Various combinations of static and dynamic load can be applied and the number of cycles to failure determined in each case.

Some authors have presented approximate correlations between dynamic properties and conventional test results. Idriss and Seed, for example, relate damping factor for sands and clays to amplitude; shear modulus of clays to undrained shear strength and amplitude, and shear modulus of sands to confining pressure and strain amplitude. There are, however, wide deviations found for actual soils from these general relationships, as is shown by the tests done on local soils. Thus it appears preferable to measure dynamic properties than to infer them from other test results.

LIQUEFACTION OF SOILS

The basic mechanism of liquefaction is simple. When oscillatory loads are applied to a sand, it tends to 'densify' or decrease in volume. If saturated, the pore-water pressure will rise and the intergranular, or effective stress will decrease by an equal amount. As the strength of a sand is dependent on effective stress, its strength and hence its bearing capacity will be reduced. Effective stress may be reduced to zero when the material behaves as a fluid.

When liquefaction occurs during an earthquake, water flows out of cracks in the surface and sand boils occur; buried conduits and storage tanks float to the surface; structures sink and tilt. Liquefaction will take place wherever the necessary combination of initial conditions and vibration amplitudes is found. This may be near the surface or at a deeper level. If it occurs at some depth then an upward flow of water from the liquefied zone may cause quicksand conditions in the layers above. Thus, even though the layers near the surface may not undergo direct liquefaction, the hydrodynamic pressure from the liquefied zone may cause bearing capacity near the surface to be greatly reduced, or lost completely.

The use of load-bearing piles does not confer immunity. For the area of greatest damage of Niigata, structures on piled foundations showed only marginally better performance than those on spread footings. It was found that buildings on spread footings suffered damage when the penetration resistance (N) was below 15. When the various piled structures were compared (with piles 15 to 60 ft long) it was found that provided the penetration resistance was over 25 at the depth of the pile tips there was little or no settlement damage.

Analysis of landslides (e.g. Alaska, 1964) has shown that many of these are attributable to liquefaction in sand strata, or in thin sand lenses.

The density index (relative density) of a sand is the most important factor, the likelihood of liquefaction being much greater in loose materials than in dense. Other things being equal, the oscillatory stress required to cause liquefaction is roughly proportional to density index.

Not surprisingly, the greater the magnitude and number of repetitions of oscillatory loadings, the greater the possibility of liquefaction. High confining pressures tend to prevent liquefaction.

What is the range of materials for which liquefaction need be a serious consideration? Tests by Lee and Fitton show that fine or silty sands are most prone, and any materials whose 50% grain size (D_{50}) lies between 0.02 and 2 mm. can show liquefaction. A material with appreciable clay content does not.

In most cases it will be clear from a reasonably thorough site investigation whether or not a site may be subject to liquefaction in an earthquake. If the soils are clays, for example, there is no danger. If the foundation materials are fine, loose sands and the water table near the surface there is obviously a danger and some preventative measures should be taken. There will, however, be borderline cases which warrant closer investigation. Then density measurements should preferably be made by methods of greater accuracy than the standard penetration test; dynamic tests can be made in the laboratory; earthquake stresses in the upper layers can be assessed from dynamic analysis of the soil profile, for the design earthquake. This analysis can then provide a basis for deciding whether special treatment is necessary.

Generally deep foundation treatment to increase the density of the soil is the practical solution. If temporary groundwater lowering will enable excavation and re-compaction to be done, this may be the most economical method. Otherwise, if the job is large enough to justify importation of the special equipment, vibroflotation can successfully compact sands. This method has been used to a depth of over 60 ft.

For smaller works where cost rules out vibroflotation, compaction piles may be used. These are driven piles which compact the soil by displacement and by vibration during driving. They are not used for load bearing. Instead of precast concrete, sand piles are sometimes employed, where a steel casing with a detachable tip is driven, then, as the casing is withdrawn, additional sand is rammed in at the bottom.

Examples of the use in New Zealand of most of these methods were given.

SOIL-STRUCTURE INTERACTION AND FOUNDATION DESIGN

EFFECTS OF INTERACTION ON BUILDING RESPONSE

Current aseismic design practice for buildings generally assumes a rigid foundation condition, where base accelerations are assumed to be unaffected by the presence of the structure, and the effects of foundation compliance and associated soil-structure interaction are ignored. While these assumptions are probably realistic for structures founded on rock, for many situations where spread footings or raft foundations are sited on more compressible soils, or where piled foundations are used, the effects of interaction may have significant influence on the earthquake response. These effects can be grouped under three main headings :

1. Period changes - Horizontal or rocking compliance of the foundations will change the natural periods of vibration of the structure, as compared with those associated with the rigid base structure. Such changes in period may have a significant effect on the magnitude of dynamic forces acting on the structure.

2. Energy losses - Dynamic compliance of the foundations results in the propagation of stress waves away from the structure, and the associated loss in energy may have an effect in reducing the dynamic building response.

3. Base acceleration modification - Foundation compliance will result in accelerations at the base of the building which are different from those on the surface of the ground at distances remote from the building. As a result, response spectra computed from basement accelerations recorded during an earthquake may be more characteristic of the building response than the earthquake motion.

In general, analytical approaches to the problem can be separated into three main categories, depending on the degree of difficulty :

1. Pseudo-static approach - Foundation compliance is taken into account by use of static stiffness relationships with lateral

forces being computed from a design spectrum which takes into account the fundamental period of the structure.

2. Simplified dynamic approach - The dynamic response of the soil-structure system to earthquake loading is analysed using a lumped "mass-spring-dashpot" analogy of the foundation and surrounding soil.

3. Rigorous dynamic approach - Modern computer techniques, such as the finite element method, have made it possible to model an essentially continuous representation of the foundation-soil system, with earthquake accelerations being applied at a remote distance from the structure.

DESIGN OF FOUNDATIONS FOR EARTHQUAKE LOADING

Just as in the design of foundations for static loading, design for earthquake loading must avoid the likelihoody of shear failure and, at the same time, minimise the possibility of damaging differential settlements.

Bearing Capacity and Settlement

The problem of excessive differential settlement is only likely to arising with footings or piles founded in sands. Suitable methods of treatment were outlined in the section on 'Liquefaction'.

For spread footings on any soil, a load factor of three for static loading and of two for static plus earthquake loads usually results in a design which will not fail even in a severe earthquake. In the special case where a compensated or partly-compensated raft foundation is used, the net bearing pressure is reduced as a result of excavation. There is then little or no subsequent consolidation with its attendant increase in strength. Earthquake forces on the foundations are, however, not diminished at all and, particularly if the subsoil is a sensitive clay, shear failure during an earthquake is possible. A method of analysis of this situation was presented, based on results of a stress-controlled dynamic test.

Practically all building codes for seismic areas require foundations, whether pile caps or spread footings, to have interconnecting ties. This was found, many years ago, to be very effective in reducing earthquake damage. The use of shear walls has become widespread and foundations for these pose problems which were given special consideration.

Foundation Stiffness for Pseudo-Static Analysis

For use in structural analysis, it is desirable to know the stiffness of foundation elements against translational or rotational forces. It is illogical to carry out a careful structural analysis wherein the stiffness of every beam and column is carefully computed, while the foundation elements are considered to be completely rigid in some part of the analysis and, in another part, to have zero stiffness.

For spread footings, methods of computing the stiffness against vertical or horizontal translation and against rotation were given in terms of the elastic modulus of the foundation soil.

SURFACE LAYER MODIFICATION OF EARTHQUAKE WAVES

That the motion of the ground surface, where deep deposits of soils exist, is very different from the earthquake motion on rock outcrops, has been known in a general way for many years. This first came to the attention of engineers when it was observed that earthquake damage to structures was related in some way to the nature and thickness of the foundation materials overlying bedrock. As with most data concerning structural damage in earthquakes, the relationship was not simple, or clear. For every generalisation that was made, an exception could be found.

Observations demonstrate that the characteristics of vibrations measured on the surface depend markedly on subsoil conditions, but, in view of the fact that 'elastic' properties of soils - moduli and damping ratio - depend strongly on the amplitude of vibration these very small movements will not give, directly, information that is applicable in a major earthquake. Opportunities for observing major earthquakes are, perhaps fortunately, few and far between. Recourse must therefore be made to analytical techniques.

Two methods are in use: The multiple reflection theory developed by Kanai, Rosenblueth and others, and the 'lumped mass analysis' very successfully developed over recent years, largely at the University of California, Berkeley.

The soil profile is divided into a number of horizontal layers of infinite extent. As motion is considered uniform throughout the horizontal extent of any one layer we need only consider a column of soil of unit plan area. The mass is divided up and considered to be lumped at discrete points, the masses being separated by shear springs whose stiffness is simply the shear modulus (in suitable units) divided by the layer thickness. Viscous damping is introduced to allow for energy loss from material damping in the soil.

Thus there is an exact analogy with an elastic dynamic analysis of a framed structure with rigid beams and flexible columns which undergoes shearing distortions. This single-bay multi storey shear structure is one of the simplest for dynamic analysis. Just as we may obtain the motion of the roof of the structure for a given input base motion, so for our soil profile we can determine the surface motion for a given input bedrock motion.

A slight complication arises from the fact that moduli and damping ratios are not fixed quantities, as we might assume for a structural frame. Their magnitude depends on strain amplitudes in the soil. This is overcome by first analysing the problem with values of moduli and damping ratio appropriate for estimated amplitudes. The result gives closer estimates of amplitudes enabling more exact values of moduli and damping ratios to be used in a second analysis. The third run usually gives strain amplitudes closely compatible with the 'elastic' parameters used. Only in this iteration procedure, which is rapidly done on a digital computer, does the method differ from the dynamic analysis of structures.

From the 'computer model', for a given bedrock motion, the surface motion at a particular site is obtained and acceleration and veolcity spectra determined. These spectra may then be used as the basis of structural design.

The accurancy of the method is convincingly shown by examples where accelerograph records made during fairly strong earthquakes were compared with those predicted by the computer model. The method is now used in California for many high-rise building sites and all nuclear reactor sites. An example is given of the application of the method to an Auckland site where a motorway bridge, a third of a mile in length, is to be constructed.

Rational methods, then, are available to determine acceleration and velocity spectra, for a site, which can be used as a much more logical basis for structural design than those provided by a code which completely ignores the effects of local subsoil conditions.

A NEW VERSION OF THE CARACAS STORY

The earthquake of 29 July 1967 in Caracas, Venezuela, was of particular interest to engineers concerned with the design of earthquake- resistant structures because the occasion was one of the few in which buildings, designed to be earthquake-resistant, had been subjected to sizeable earthquake forces. Some of them collapsed. This caused alarming conclusions to be drawn by some engineers, concerning the adequacy of existing design codes, not only in Venezuela, but in other countries also.

The design code for Caracas was examined, both in its 'official' form, and in the form in which it was interpreted by some designers. These were compared with the provision for seismic forces made in current codes in California, Mexico City and in New Zealand, and also with a proposed Chilean code. The use of spectral velocity as a parameter of damage potential was examined.

One of the most interesting aspects of the Caracas earthquake is that, while some structures were extensively damaged, or collapsed completely, exactly similar structures in other parts of the city survived with negligible damage. The difference can be explained in terms of foundation conditions at the building locations. An analysis made by Professor H. B. Seed of the University of California, Berkeley, of building damage in relation to foundation condtions was outlined.

The formulation of a new code which incorporates, in a simple way, provision for the variety of foundation conditions in Caracas was described. This is, in effect, a sort of microzoning. It is based on Seed's analysis of the damage in the last earthquake and on computed estimates of effects of possible larger earthquakes.

It is not suggested that the type of code proposed for Caracas is necessarily suitable for other foundation conditions in other countries. The author believes, however, that some provision should be made, in the design of earthquake-resistant structures, for the effects of local foundation conditions.

DYNAMIC RESPONSE OF EARTH STRUCTURES TO EARTHQUAKES

The dynamic response characteristics of earth structures govern the magnitude and distribution of accelerations or stresses acting within the structure during an earthquake. As the magnitude and time history of dynamic stresses play an important role in the determination of soil strengths which can be mobilized during an earthquake, such analyses are an important part of the overall assessment of the seismic stability of earth structures.

The Response of Earth Dams to Earthquakes

Many existing design codes consider earth dams as rigid bodies for the purpose of aseismic design. However, it is now generally accepted that the behaviour of earth dams during earthquakes is governed primarily by their dynamic response characteristics, which are in turn controlled by the nature of the materials within the dam, and the height and geometric characteristics of the dam. As earth dams are large three dimensional structures constructed from inelastic and often non-homogeneous materials, dynamic response calculations necessitate many simplifying assumptions. In particular, most published solutions assume a two-dimensional structure (i.e. an infinitely long dam), and assume construction materials to be linear elastic. Initial dynamic response analyses considered earth dams to comprise a series of infinitely thin horizontal slices, interconnected by linear elastic shear springs and viscous dashpots, with shear stresses uniformly distributed over horizontal planes (cf. horizontal layer response theory). Hence the dam could deform in horizontal shear only. More recent analyses using the finite element technique have indicated the deficiencies of this assumption. In the finite element method the dam cross-section is replaced by a network of small triangular elements, the mass of each element being lumped at the nodal points. The method is particularly powerful, for it permits the effects of vertical ground motion and travelling wave ground motion to be taken into account, and also enables consideration of problems involving non-linear material properties, non-homogeneous dam sections, or dams on soil foundations.

The Response of Earth Banks to Earthquakes

Extensive failures of natural slopes have been a significant feature of many earthquakes. However it has only been in recent years with the development of the finite element method of analysis, that an assessment of the response of earth banks to earthquake ground motions has been possible. Analyses have shown that the response of earth banks may vary widely, depending on the nature of the soils involved, the slope geometry, and the depth of the foundation soils.

The Application of Dynamic Response Theories to Stability Considerations

Many generally accepted methods of assessing the dynamic stability of earth slopes and embankments during earthquakes, use a static seismic coefficient in conjunction with a conventional slope stability analysis. However, during an earthquake the lateral forces acting on a slope change in direction and magnitude many times. If the induced dynamic stresses are sufficiently high, permanent deformations of the slope may occur, the overall effect of the cyclic loading being a cumulative displacement of a section of the slope. Once the ground motions have ceased, no further deformation will occur unless the soil strength has been decreased significantly. As the resulting permanent displacement may not be of significant magnitude, it would appear that the criterion of a minimum factor of safety based on the limiting equilibrium principle could in many cases be too conservative. This had led to the concept of assessing earthquake stability in terms of deformations produced, with a design criterion based on a prescribed tolerable permanent displacement. The necessity for such a method, which involves the consideration of the entire time history of lateral forces acting on the slope or embankment, is further demonstrated by the experimental evidence which has shown that the strength mobilized by soils under dynamic loading is a function of both the magnitude and number of stress cycles. Dynamic response theories enable such time history dependence to be taken into account.

SLOPE STABILITY IN EARTHQUAKES

As many major slope failures have occurred during medium and strong earthquakes, the assessment of the seismic stability of slopes in earthquake-prone areas is of great concern to engineers. Prior to 1964, most design methods were based on a static analysis used in conjunction with an arbitrarily selected lateral force acting on the slope and soil strengths determined by conventional laboratory tests. However, the catastrophic slope failures which occurred during the Alaskan earthquake of 1964, resulted in a considerable reappraisal of such static design methods. In recent years significant progress has been made in developing new methods of laboratory testing to determine dynamic soil properties, improved techniques for analysing the dynamic response of embankments to earthquakes, and new concepts of aseismic design methods for earth slopes. Although research has yet to produce all the answers, the current state of knowledge at least provides an improved guide to engineering judgement in the assessment of the stability of earth slopes during earthquakes.

The problems of slope instability during earthquakes may be placed into three broad categories :

1. Slides in Dry Cohesionless Slopes

Observations of slides in such slopes during earthquakes and of dynamic model tests on small embankments constructed from dry sand, have indicated that the mechanism of failure is that of a shallow surface slide. As a yield criterion is relatively well defined, stability calculations for such slopes are particularly amenable to the concept of evaluating seismic stability in terms of the magnitude of slope displacements that occur during an earthquake. Model tests have indicated the promise of this method of approach. However, obviously such analyses are complex, and only a few simplified analyses of this type have been used in practice to date.

2. Slides due to Liquefaction

Flow slides result where liquefaction of a cohesionless soil zone extends to the free surface of the slope, and are characterized by extensive lateral movements. Such slides have occurred in many earthquakes, for example those at Valdez and Seward during the Alaskan earthquake of 1964.

Slides in earth banks may also be initiated by the liquefaction of thin horizontal sand seams. Such slides develop in the first instance by lateral movement of the slide mass, probably resulting from horizontal inertia forces generated by longitudinal waves propagating from the rear of the slide surface, shear wave propagation from below being cut off by the liquefied zone. The lateral movement is followed by the formation of a depression or 'graben' at the rear of the slide mass. The mechanism of failure was well illustrated by the Fourth Avenue and L Street slides, occurring in Anchorage during the Alaskan earthquake of 1964.

Liquefaction of numerous lenses of silt and fine sand contained within a deposit of marine clay, played a major role in the extensive landslides occurring in the Turnagain Heights area of Anchorage, during the Alaskan earthquake. The development of the slides was similar to those initiated by sand seams, except that the failure surface passes through clay between adjacent lenses.

3. Slides in Cohesive Soils

Relatively few slides in cobesive earth slopes which have occurred as a result of an earthquake, are reported in the literature. With little field evidence available which could form the basis of evaluating any proposed aseismic design method, the current state of the art is by no means fully developed.

The most promising approach to date is essentially a "stress path" technique, where soil samples are subjected in the laboratory to similar stress conditions that will develop on soil elements in the embankment during an earthquake. Observations of the resulting deformations are then assessed in terms of the embankment performance during the earthquake. However, such an assessment is particularly difficult for homogeneous embankments constructed of cohesive soils. The yield stress of such materials is considerably less than the strength, and hence large deformations approaching failure can occur under dynamic loading conditions even when the maximum applied stress is less than the static strength of the soil. Thus the overall displacement of the embankment during an earthquake will be due to permanent deformations occurring at many points within the section, and not within a small shear zone.

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