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Case Studies Through Material Modelling and Computation

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SYNOPSIS: This paper describes a number of case studies by using numerical procedures conducted by the author and his co-workers over a number of years. The case studies involve a wide range of static and dynamic stress-deformation, seepage and stability, and consolidation problems. The numerical procedures use simple linear and nonlinear elastic models, to advanced but simplified hierarchical plasticity based models for geologic materials and interfaces/joints. The evolution from the use of simple to advanced models is guided by the realization that it is essential to employ models that are capable of handling the complexities in geotechnical systems. In addition to use of the conventional and empirical methods, it is advisable to develop and utilize improved and simplified techniques based on basic principles of mechanics. This approach can allow the geotechnical engineer access to models and procedures towards improved and rational solutions for case studies and for practical applications.

INTRODUCTION

In conventional case studies in geotechnical engineering, the (field) observations are usually examined with the aid of empirical or simplified formulas, or theories to predict the observed behavior and to draw conclusions regarding the performance of the system, the adequacy of design methods used including their limitations, and need for future modifications. As the methods are highly simplified, the analysis performed is usually on a highly idealized system in terms of geometries and material properties that can render itself to simple calculations. Coupled with experience and intuition of the engineer, this approach can provide satisfactory solutions for many problems. However, since it does not allow for factors such as irregular geometries, nonlinear soil properties and complicated loadings, for many other problems, the conventional methods may not be appropriate for realistic solutions.

The notion that the uncertainties in material properties, geometry and loadings in geotechnical problems are high and hence, conventional methods are all that is required, and advanced (computational) methods may not be warranted,and may not be precise! This is because whether one uses a conventional method or an advanced modern method, the uncertainties are essentially the same. While, on the other hand, the modern methods are capable of easy analysis of the effects of uncertainties through parametric studies, and also capable to incorporation in the analysis itself, of newly developing models, e.g. for the material behavior. With this belief, it is considered useful and meaningful from a practical viewpoint of case studies to use modern (computer) methods with improved treatment of material response and other factors.

Scope

The scope of this paper includes:

1. A review of the author's work in case studies involving field measurements using computer (finite element, finite difference) methods for the following problems:

   (a) Static Stress-Deformation

   - (i) Axially and Laterally Loaded Piles - 1974, 1980
   - (ii) Group Piles - 1974, 1986
   - (iii) Tunnels - 1983
   - (iv) Retaining Walls - 1983, 1985
   - (v) Anchors - 1986

   (b) Seepage and Deformable Flow

   - (i) Seepage in River Banks - 1971, 1972, 1983
   - (ii) Consolidation of Layered Foundations - 1977
   - (iii) Seepage in Dams - 1980, 1983, 1986, 1987
   - (iv) Stress and Seepage in Dams - 1983

   (c) Dynamic and Earthquake Analysis

   - (i) Model Nuclear Power Plant - 1984

2. Consideration of mechanical behavior of geologic materials and interfaces and joints, starting from simple elastic and nonlinear elastic, to recently proposed new hierarchical and unified plasticity based approach by the author and co-workers. Here, the author has gone through a gradual realization that it is beneficial to think that a nonlinear elastic model, with its simple look, may be appropriate for some problems, only where it is applicable. However, for realistic simulation of the behavior of geologic materials, it is essential to develop improved models from the basic principles of mechanics. The author has found that such models with sound fundamentals need not be complicated if derived through a rational process of simplifications for practical

application. In fact, the hierarchical models [1-4] represent such an approach and involve equal or lesser number of material constants as compared to nonlinear elastic models, and at the same time, are capable of accounting for factors such as volume changes, stress paths, nonassociativeness, softening and anisotropy.

3. With the above viewpoint, in the following, are described a number of case studies, conducted by the author since 1970. Comments are offered on the capability, limitations and improvements in various material models in conjunction with computational methods.

CASE STUDIES

This paper would be too long if all the applications were described in details. Also, case studies involving field problems are related to other studies involving theoretical considerations and laboratory verifications. To overcome this, it is proposed to outline the case studies presented below and the related works in Tables 1, 2, and 3 for Static Stress-Deformation, Seepage and Deformable Flow and Dynamics and Earthquake, respectively. Some related case study topics that are not reviewed herein are also mentioned in these tables. The tables present statements of the problems, constitutive or material model(s) employed and other factors, numerical techniques, and special comments.

In the following, brief descriptions of only selected case studies involving field verifications are given with critical comments on the constitutive models and their gradual progression toward improved characterization and on the numerical techniques and improvements therein. Details of numerical analysis such as meshes are shown only for some problems, whereas for others only typical comparisons of computations and observations are included.

Static Stress-Deformation

Example 1 - Axially Loaded Piles: Figure 1 shows comparisons between predicted* and observed behavior for a typical axially loaded steel pipe pile, outer diameter = 41 cm., length = 16 m, [5] in sand tested in the field at the Arkansas Lock and Dam No. 4 (LD4) site [6]. Here, in the early stage of finite element applications, nonlinear elastic model using hyperbolic simulation [7] was used, which is considered essentially similar to the piecewise representation through data points used before [8] and the spline representation [9] in the sense that they are based on piecewise linear elastic approximation. The constants for these models are found from a set of triaxial test data with cylindrical specimens. The interface element used was a modified version of that with zero thickness as proposed in Ref. 10. A set of design charts (Fig. 11 in Ref. 5) were also prepared for finding bearing capabilities of piles in sands.

The results indicated that for monotonic loading, the finite element scheme with nonlinear elastic

*In most cases, predicted imply back predictions of observed response.

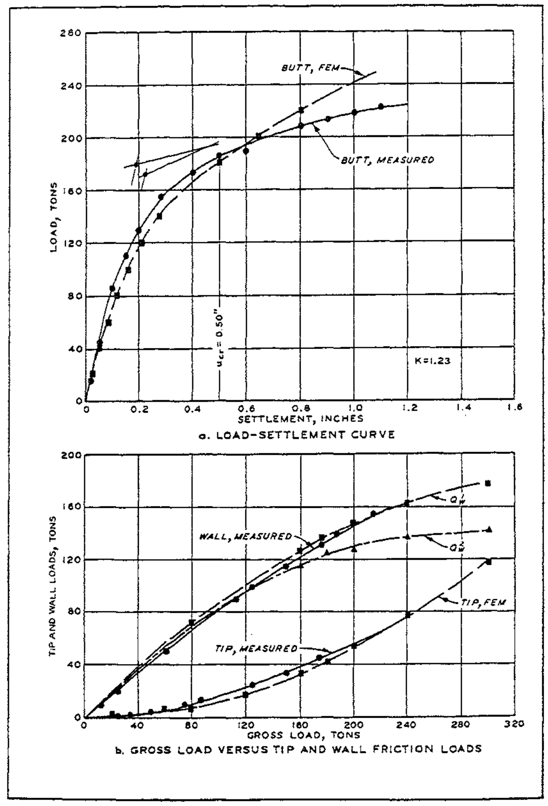


Fig. 1 Comparisons for Pile No. 10, LD4

models can provide satisfactory predictions of load displacement curves and bearing capacity for piles.

Although the results allow for nonlinear simulation of a set of the stress-strain curves, the above models can mainly allow for monotonic loading. They are deficient in terms of allowing for factors such as volume change, stress-path, unloading-reloading and nonassociative response. Moreover, these models cannot adequately represent unloading and reloading responses vital in many geotechnical problems. Hence, their use should be tempered with caution.

Example 2 - Pile Supported Lock: Figure 2 shows comparisons between predictions and observations of settlements of different points at various times during sequential construction for the stress-deformation behavior of pile supported Columbia Lock, on the Ouachita River near Columbia, Louisiana, Fig. 3 [11]. Here, the three-dimensional pile foundation system was idealized as structurally equivalent two-dimensional system.

The foundation soils consisted of cohesive backswamp deposits or cohesionless substratum deposits or both, beneath the east wall, and tertiary deposits interfacing with colluvium and substratum deposits beneath the west wall [12]. The stress-strain model used was nonlinear elastic, simulated through hyperbola. The interface model used was the same as in Example 1.

TABLE 1. Static Stress Deformation

1 Problem	2 Material Behavior	3 Other Factors	4 Numerical Procedure
1. Axially Loaded Footings	• Nonlinear Elastic - Data Points		2-D Finite Element
2. Axially Loaded Piles	• Nonlinear Elastic - Hyperbolic		2-D Finite Element
3. Laterally Loaded Structures	• Nonlinear Elastic - Ramberg-Osgood	• Construction Sequences	1-D Finite Element
4. Pile Groups	• Nonlinear Elastic - Hyperbolic	• Construction Sequences • Downdrag	2-D Simulation (of 3-D),Finite Element
5. Pile Groups	• Nonlinear Elastic - Hyperbolic	• New Thin-Layer Interface	3-D Finite Element
6. Tunnels	• Plasticity - Drucker-Prager	• Construction Sequences • Thin-Layer Joint	2-D Finite Element; Displacement, Hybrid Mixed
7. Retaining Walls	• Plasticity - von Mises	• Construction Sequences • Thin-Layer Interface • Flexible Structures	2-D Finite Element, Displacement, Hybrid, Mixed
8. Footings, Walls, Track Mechanics	Options for Non-Linear Elastic and Plasticity - von Mises, Drucker-Prager, Critical State, Cap	• Thin-Layer Interface • Flexible Structures	1-D, 2-D and 3-D Finite Element
9. Anchors	• Hierarchical Associative/ Nonassociative Plasticity	• Thin-Layer Interface • Interaction • Stress Relief	3-D Finite Element

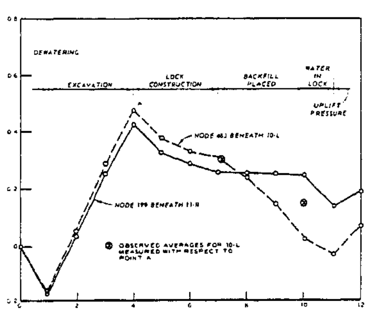


Fig. 2 Settlement Versus Construction Sequences at Typical Nodes, 199 and 483 (1 ft = 0.305 m)

Here the normal stiffnesses during compressive and tensile normal stresses is adopted arbitrarily to very high and very low values, respectively. The shear stiffness is simulated by using the nonlinear elastic, hyperbolic model.

The computer analysis with the nonlinear elastic model provide reasonable to satisfactory predictions of settlements and distribution of loads in the pile groups. They also provided a good prediction for the drag forces on the lock walls which compared well with the observed values [12].

Example 3 - Laterally Loaded Structures: A generalized one-dimensional finite element procedure with idealizations shown in Fig. 4 was used to predict field behavior of a laterally loaded (wooden) pile and a sheet pile retaining wall, Fig. 5, the latter involved (approximate) simulation of construction sequences [13].

The material behavior was simulated by using spring elements to represent translational and rotational components. The nonlinear response was simulated as nonlinear elastic using a Ramberg-Osgood type function, which contains the hyperbola as a special case.

TABLE 2. Seepage and Deformable Flow

1 Problem	2 Material Behavior	3 Other Factors	4 Numerical Procedure
1. Transient Seepage in River Banks	• Darcy's Law	• Steady/Free Surface • Stability	2-D Finite Difference
2. Seepage in Dams	• Darcy's Law	• Steady/Free Surface • Stability	2-D Finite Element 3-D Finite Element - Variable Mesh
3. Seepage in Dams	• Darcy's Law	• Steady/Free Surface • Stability	2-D Finite Element 3-D Finite Element - Residual Flow Procedure - Invariant Mesh
4. Stress Seepage and Stability of Dams	• Darcy's Law • Plasticity: von Mises, Drucker-Prager	• Steady/Free Surface • Construction Sequences	2-D Finite Element - Residual Flow Procedure, - Invariant Mesh
5. Consolidation	• Darcy's Law • Linear Elastic • Plasticity - Critical State	• Construction Sequences • Anisotropy	2-D Finite Element

TABLE 3. Dynamic and Earthquake

1 Problem	2 Material Behavior	3 Other Factors	4 Numerical Procedure
1. Model Nuclear Power Plant Structure in Field	• Plasticity: - Hierarchical and Cap	• Simulated Earthquake • Thin-Layer Interfaces	2-D Finite Element
2. Instrumented Pile Segments	Plasticity - Hierarchical, Anisotropic Hardening	• Thin-Layer Interfaces • Pore Water Pressure	2-D Finite Element

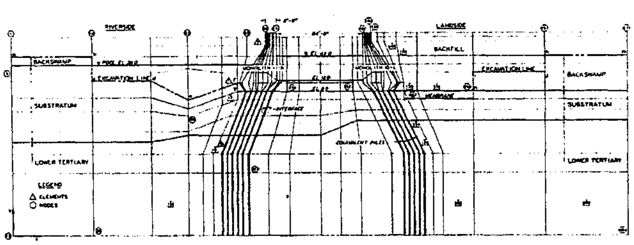


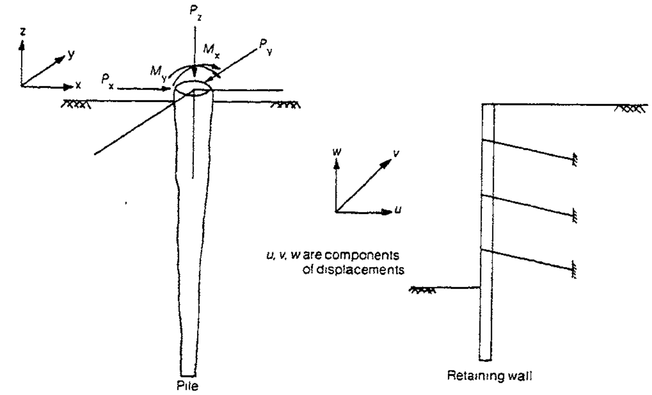
Fig. 3 Finite Element Mesh for Lock and Foundations (1 ft = 0.305 m)

Figure 6 shows comparisons for load-displacement response of the wooden pile tested in the field. Comparisons for the lateral displacements of the sheet pile for one- and two-dimensional predictions and observed response are shown in Fig. 7. This shows that the one-dimensional procedure can provide satisfactory predictions of the field behavior of some laterally loaded structures.

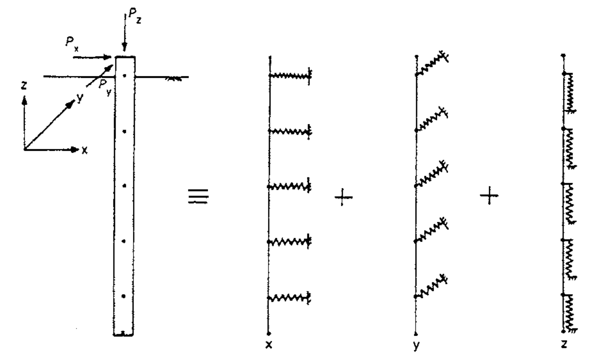
Example 4 - Braced Excavation: Field response of a braced wall for excavation tested in the field in Norway [14] was backpredicted by using displacement, hybrid and mixed finite element procedure [15, 16]. Details of the wall and the finite element mesh are shown in Figs. 8 and 9, respectively.

The construction sequences involving eight stages simulated are given below:

Stage 1:	Compute initial stresses, install wall and excavate to el. + 0.2m.
Stage 2:	Install first strut A, and excavate to el. -2.0m.
Stage 3:	Install struct B, and excavate to el. -3.0m.
Stage 4:	Install strut C, and excavate to el. -4.0m.
Stage 5:	Excavate to el. -5.0m.
Stage 6:	Install struct D, and excavate to el. -6.0m.
Stage 7:	Excavate to el. -7.0m.
Stage 8:	Install strut R, and excavate to el. -8.0m.



(a) Piles



(b) Idealization

Fig. 4 Axially and Laterally Loaded Structures and Idealization

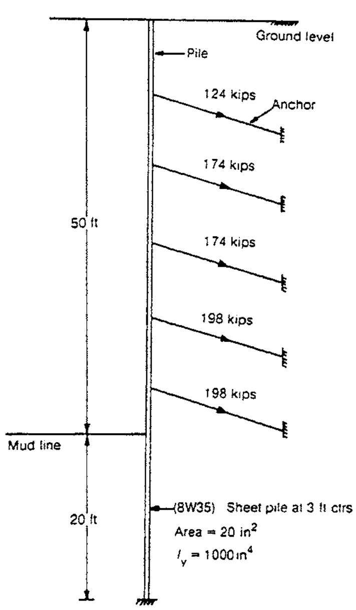


Fig. 5 Sheet Pile Wall

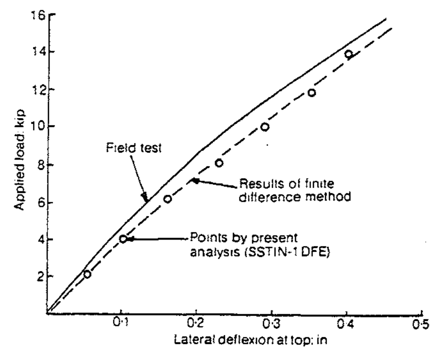


Fig. 6 Comparisons for Wooden Pile; Arkansas River, Lock and Dam No. 4

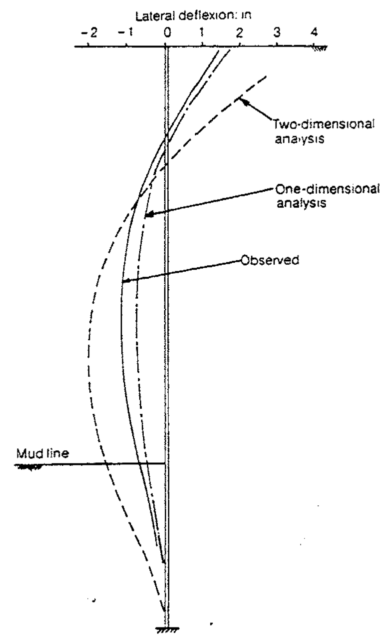


Fig. 7 Comparisons for Lateral Displacements of Sheet Pile Wall

The clayey soils were characterized by using an elastic-plastic model with von Mises yield criterion, while the wall and the struts were assumed to be linear elastic.

The new thin-layer element [17] was used to characterize the behavior of the interfaces.

Figures 10, 11 and 12 show typical comparisons between predictions and observations for wall deflections, heave and wall pressures, respectively.

It can be seen that overall the back predictions are satisfactory. It was found that the zero thickness element [10] adopted for soil-structure problems usually does not provide satisfactory predictions of interface stresses in flexible walls and situations where modes such as debonding other than slippage under compressive

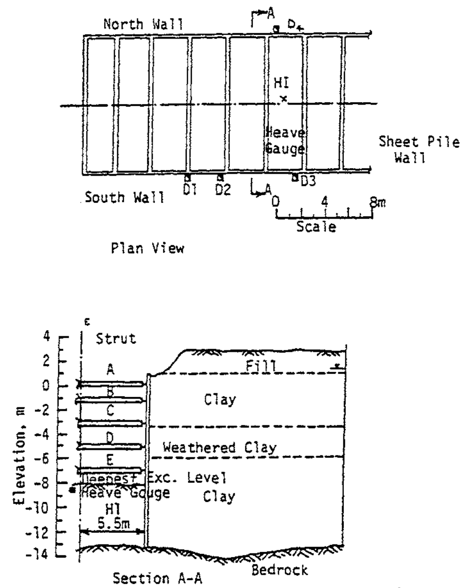


Fig. 8 Vaterland 1, Site and Soil Profile (14)

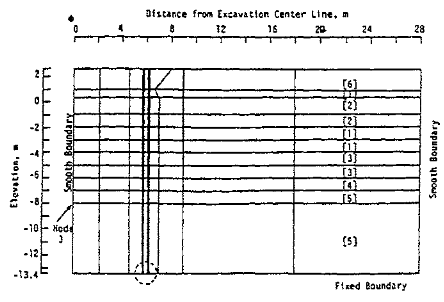


Fig. 9 Finite Element Mesh for Vaterland 1

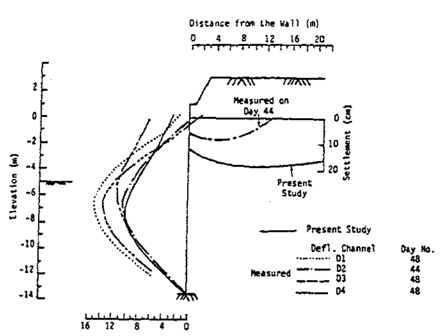


Fig. 10 Wall and Soil Deformations (Stage 5)

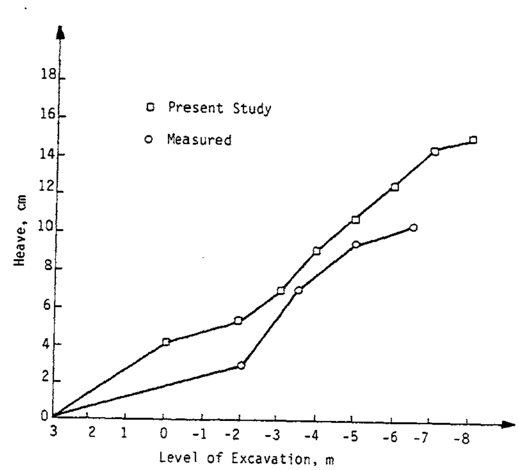


Fig. 11 Comparison of Heave at Node 3 (Figure 9)

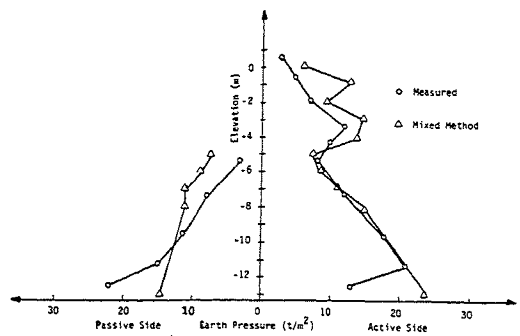


Fig. 12 Earth Pressures (Stage 5)

stresses. On the other hand, the new thin-layer element [17] provides improved predictions for the interface response and wall pressures, and also of various deformation modes. The von Mises plasticity model may be appropriate for essentially undrained response of clays. However, it is not capable of providing satisfactory predictions of volume changes, stress path dependence and dilative response.

Example 5 - Tunnels: The problem of an instrumented section of the tunnel in the Atlanta subway system [18] and the finite element mesh [19] are shown in Figs. 13 and 14, respectively. The construction sequences simulated are discussed in Ref. 19.

The rocks in the system were assumed to be linear elastic with the elastic moduli E and ν found from cylindrical and multiaxial tests [19]. The joints were simulated using the thin-layer element, and its properties were found from laboratory direct shear tests.

Figure 15 shows comparisons for displacements along an instrumented section; this and other comparisons [19] were satisfactory. However, for various reasons such as material modelling

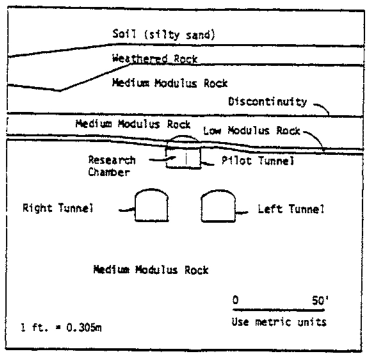


Fig. 13 Generalized Geologic Section Used for Analysis of Atlanta Subway Tunnels (18)

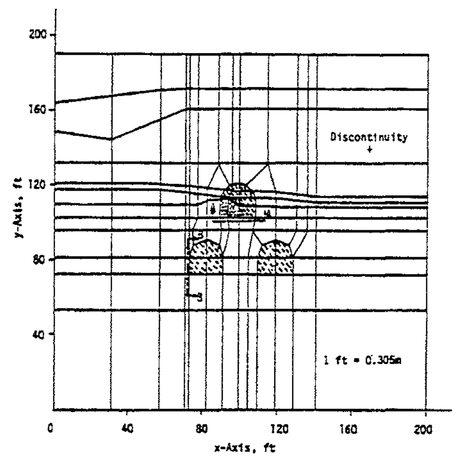


Fig. 14 Finite Element Mesh

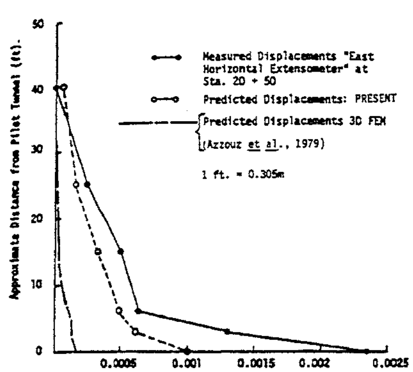


Fig. 15 Comparison Between Calculated and Observed Displacements at EE

and nearby blasting, the extensometer readings at the base of the test cavern were not predicted satisfactorily.

Example 6 - Anchors in Sand: In the next step towards improved material characterization, the new general yet simplified hierarchical plasticity based modelling approach [1-4] was used to study three-dimensional field behavior of grouted anchors in sand [20]. The interface response was simulated by using the thin-layer element, Fig. 16.

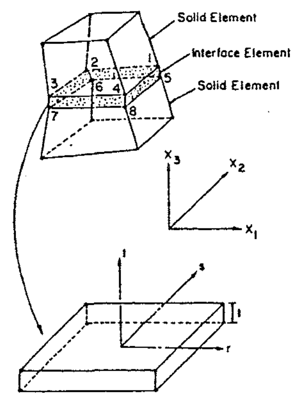


Fig. 16 Schematic of Solid and Interface Elements

Figure 17 shows details of the anchor-soil system tested in the field [21] and Fig. 18 shows details of the three-dimensional finite element mesh for the anchor-wall system.

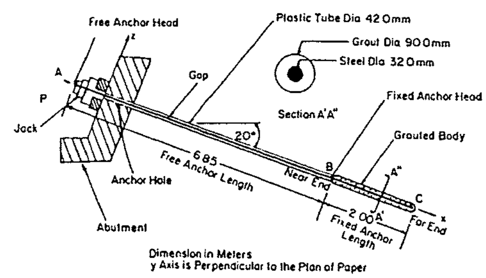


Fig. 17 Details of Components of Anchor

The loading was simulated incrementally as in the field. Figure 19 shows comparisons between predictions and observations for the load-displacement responses of the fixed anchor head, and Fig. 20 shows load distributions along the fixed (grouted) anchor length. Figure 21 shows distributions of normal and shear stresses in the interfaces between soil and anchor for linear and nonlinear analyses.

It can be seen that the finite element procedure with the hierarchical associative, isotropic hardening model and with the thin-layer element provides very good predictions of load displacement, and stress distribution responses as well

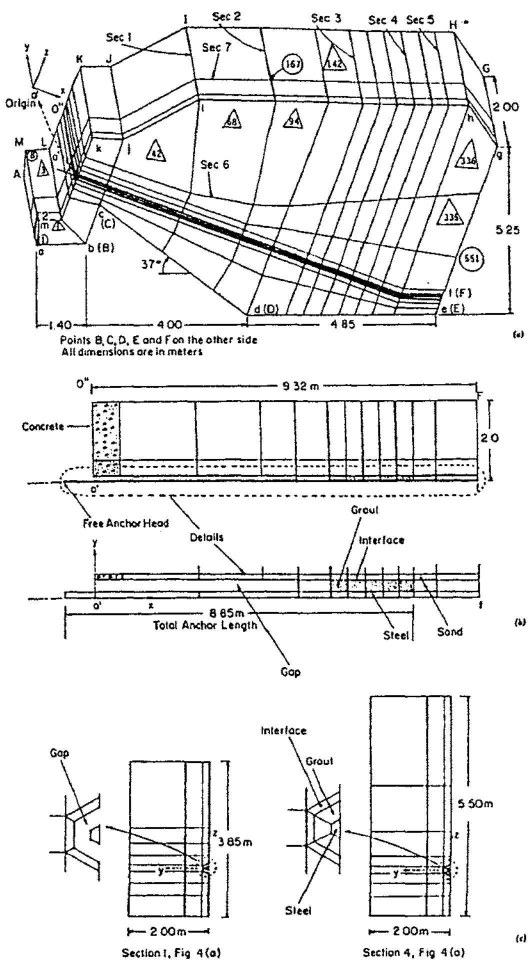


Fig. 18 Details of Finite Element Mesh: (a) Overall; (b) Along Grouted Anchor; and (c) Across Grouted Anchor

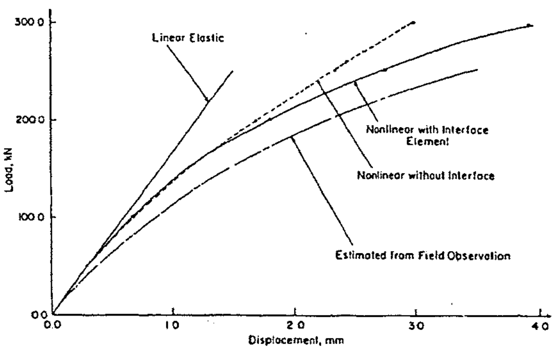


Fig. 19 Load-Displacement Curves at Fixed Anchor Head (Point B in Fig. 17)

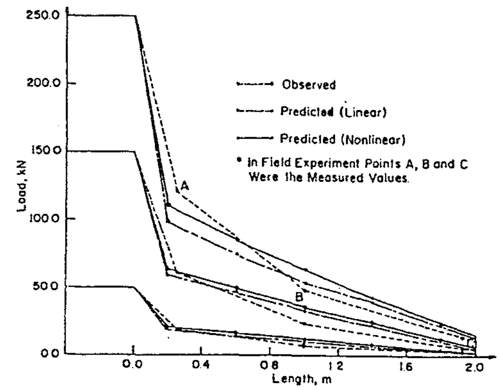


Fig. 20 Axial Load Distribution in Steel Along Fixed Anchor

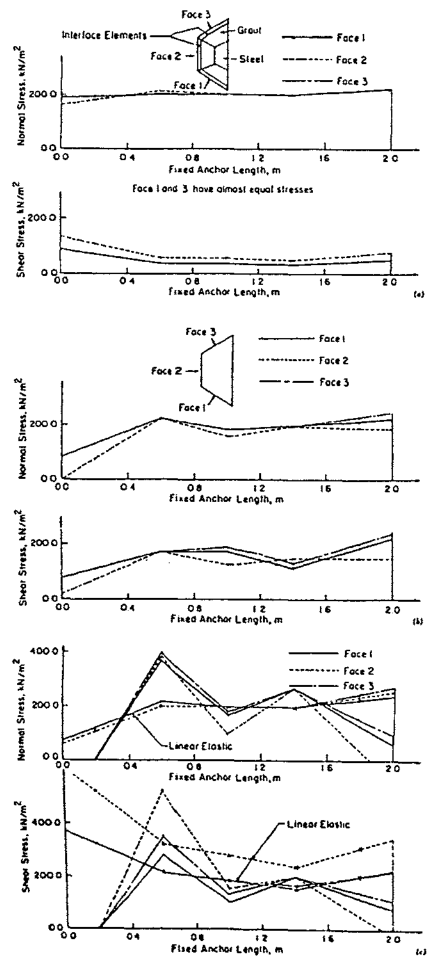


Fig. 21 Normal and Shear Stress Distributions in Interface Elements on Grouted Anchor: (a) At P = 50 kN; (b) At P = 150 kN; and (c) At P = 250 kN

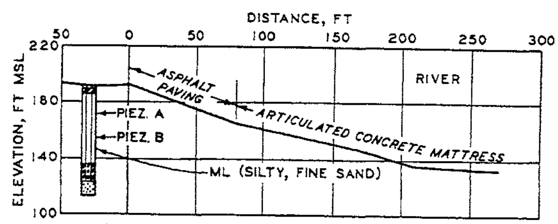
as the phenomenon of stress relief and arching at the ends of the anchor.

FLOW THROUGH (RIGID) MEDIA: SEEPAGE

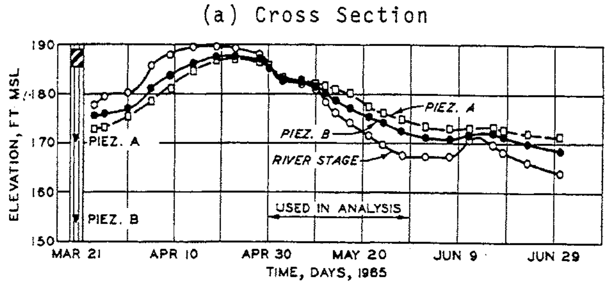
Steady and transient seepage, confined or unconfined (with free surface) is an important consideration in stable design of slopes, banks and dams. Although nonlinear constitutive laws describing relation between velocity and hydraulic gradient may be required for some problems, the linear Darcy's law is commonly employed in both conventional and computational procedures.

The finite difference and finite element procedures developed by the author and co-workers [22-29] have been applied for predictions of and verifications with respect to a number of analytical, laboratory and field problems. Here typical applications involving field problems and free surface flow are described. The techniques developed involve (a) variable mesh and (b) invariant mesh. The latter is based on a new method, called the Residual Flow Procedure (RFP), proposed by the author [22, 24, 26]. The RFP is mathematically different from methods proposed by other investigators [30] and has been found by Westbrook [31] to be equivalent to the recently proposed variational inequality methods for the flow problem. The RFP involving the invariant mesh is considered to be superior to the variable mesh procedure [23].

Example 7 - River Banks: The variable mesh finite element procedure [23] was used to back-predict transient development of free surfaces due to fluctuations (drawdown) in the Mississippi River Banks; typical instrumented cross section at Walnut Bend 6 with the boring log and fluctuations in the river stages are shown in Fig. 22.



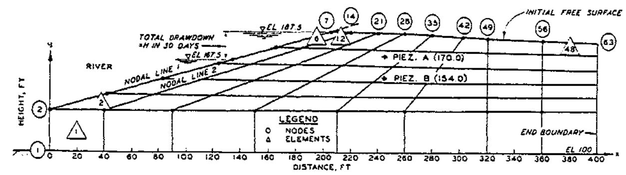
(a) Cross Section



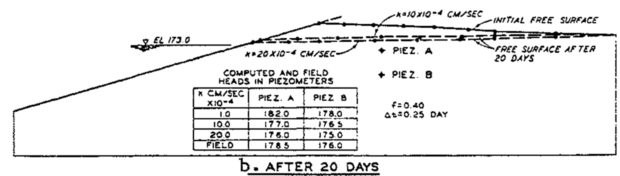
(b) Variation of Head

Fig. 22 Cross Section and Variation of Piezometer and River Heads at Walnut Bend 6

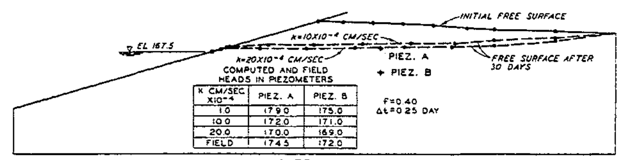
The finite element mesh and typical comparisons between back predictions at two time levels during the drawdown are shown in Fig. 23. The values of permeability k, porosity f and the time step Δt are also shown on Fig. 23. These results indicate that the numerical procedures provide very good predictions of the observed response.



a. Mesh



b. AFTER 20 DAYS



c. AFTER 30 DAYS

Fig. 23 Mesh and Comparisons Between Predictions and Observations

After a comprehensive series of comparisons between predictions and observations in the laboratory (using Hele-Shaw model) and the field behavior, design charts for stability analysis were also prepared [32].

Example 8 - Earth Dam: The field observations and material properties, variation of reservoir head with time and details of the Sherman Dam were provided by the U. S. Bureau of Reclamation [25], Fig. 24. The material in the dam was mostly clay, and the coefficient of permeabilities at various locations, Fig. 24(c), obtained from laboratory permeability and consolidation tests, were used to adopt an average value of k = 0.01 ft/year (0.03 m/yr).

The finite element mesh consisted of 408 nodes and 318 elements, Fig. 25. Comparisons between predictions and observations for computed head for typical piezometer locations are shown in Fig. 26. Despite various approximations such as the adoption of average permeability and assumption of fully saturated condition instead of possible partial saturation, the comparisons show good agreements.

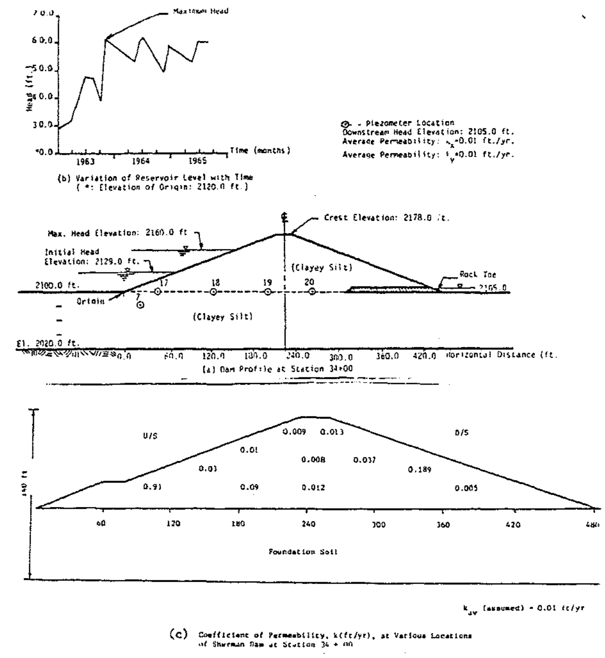


Fig. 24 Details of Sherman Dam

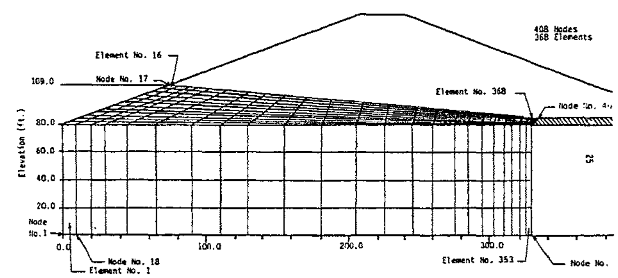


Fig. 25 Finite Element Mesh for the Sherman Dam

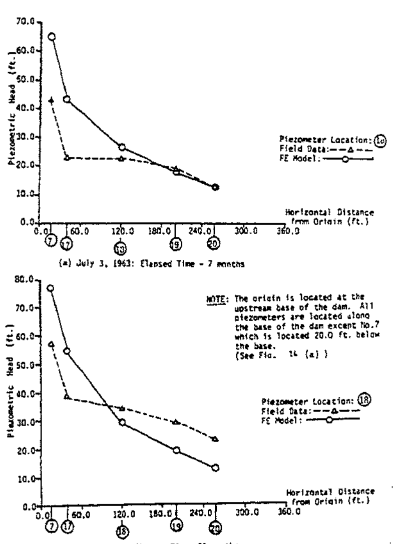


Fig. 26 Comparisons of Computed and Observed Heads

Example 9 - Combined Stress and Seepage in Dams: The assumption of rigid skeleton in conventional seepage may be too restrictive for certain field situations because, in general, soils in dams or slopes experience deformations during seepage. The general way of treating the problem is to use coupled (Biot's) theories for dynamic and static analysis of porous media. For practical analysis, however, it may often be appropriate to use the intermediate uncoupled approach. Here the nonlinear stress analysis is performed by superimposing on it known seepage forces caused by steady or transient seepage.

Applications of the uncoupled approach for back predictions of the field behavior of various dams have been presented in Ref. 27. Here the RFP is coupled with a nonlinear finite element with elastoplastic models for soils. The procedure also allows for sequential construction (embankment) of dams or banks with simultaneous transient change in head, and slope stability analysis.

The procedure possesses a number of advantages: For example, (a) the systematic approach for uncoupled analysis, (b) with RFP the same mesh is used for both stress and seepage analysis, (c) avoids necessity of assuming horizontal (transient) free surfaces in the region between upstream and the core of dam as was done in Ref. 33, and (d) can allow for partial saturation.

Figure 27 shows a cross section of the Oroville Dam [33] and transient locations of free surface due to the hydrograph showing variation of head with time in the reservoir. Figures 28 and 29 show comparisons between computed and observed horizontal movements for two sections, and observed movements of the core section, respectively. The back predictions show good correlation with observations.

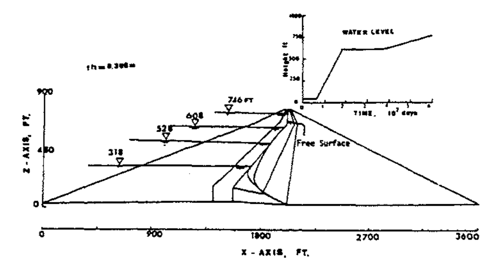


Fig. 27 Section of Oroville Dam, Hydrograph and Computed Locations of Free Surfaces During Reservoir Filling

Example 10 - Consolidation, Seepage in Deformable Soils: In order to allow for full coupling between flow and deformation, Biot's theory of flow through deformable media is often used [34]. Here both the displacements and pore water pressure are assumed to be unknowns in the finite element analysis.

Computations using a two-dimensional finite element procedure based on the Biot's theory were performed for a layered foundation involving clay deposits, Fig. 30 [35]; the finite element mesh is shown in Fig. 31.

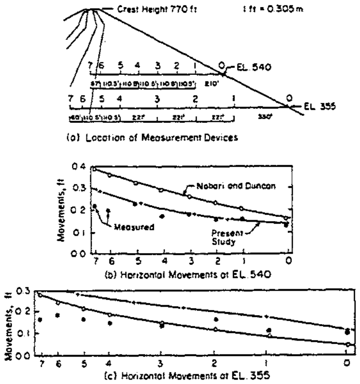


Fig. 28 Comparison of Computed and Observed Horizontal Movements for Two Sections

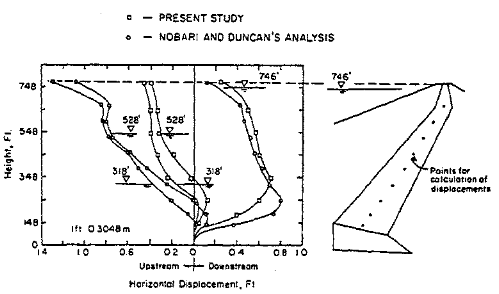


Fig. 29 Comparison of Computed Movements at a Section in Core

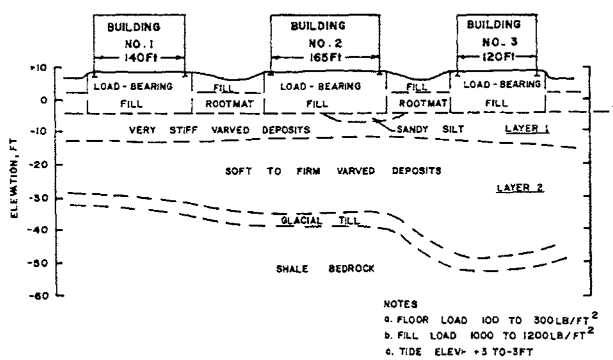


Fig. 30 Foundation in Varved Clay (from Reference 35)

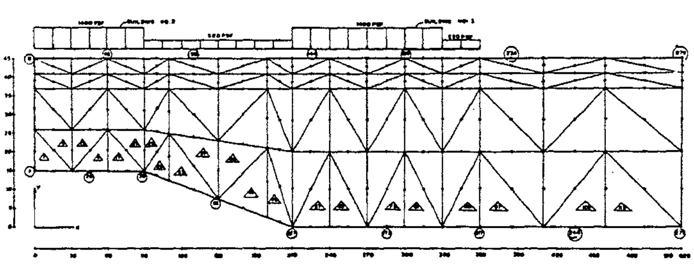


Fig. 31 Finite Element Mesh and Loading

It was found [35] that the settlement computations from the conventional one-dimensional Terzaghi theory were far too smaller than those observed in the field. Among the reasons for the discrepancy are the two-dimensional nature of the system, anisotropic characteristics of the varved clay and the history of loading. The finite element computations included effects of these three factors. In addition, parametric studies were performed in which the ratios of the horizontal to vertical permeabilities of the varved clay were varied. The computed settlements are compared with the observed values at typical locations in Fig. 32. It can be seen that the proposed procedure is capable of predicting the observed response, and that the computations with k_x/k_y = 10 showed the best correlation. This ratio is comparable to that found for many varved clays.

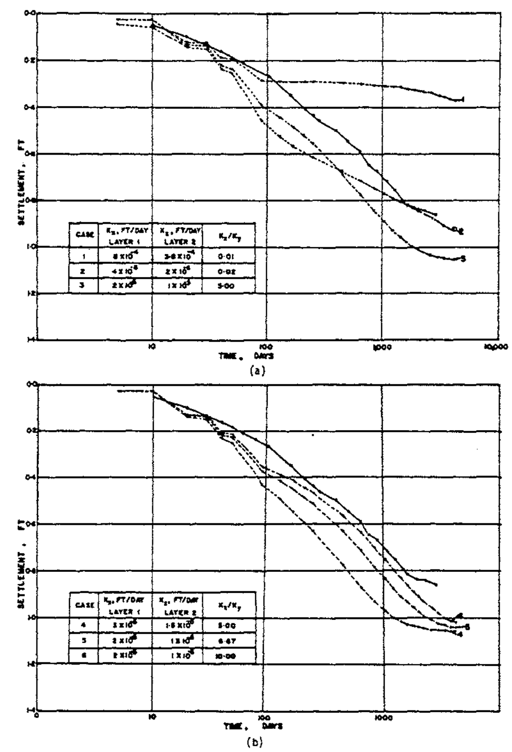


Fig. 32 Numerical Results and Comparison with Field Data for Variations in k_x/k_y

DYNAMIC ANALYSIS (36-42)

For dynamic nonlinear soil-structure interaction problems among other factors, it is necessary to consider the effects of relative motions at interfaces, nonlinear soil response including anisotropic hardening, and appropriate time integration schemes.

The problem of relative motions is handled by using the thin-layer element [17] and laboratory experiments using the cyclic multi-degree-of-freedom (CYMDOF) shear device [36] for determination of nonlinear elastic Ramberg-Osgood type [39] and elastoplastic hierarchical models [41].

The hierarchical model also allows for a general yet simplified model for anisotropic hardening due to cyclic loading [2, 4]. A procedure called Generalized Time Finite Element (GTFEM) is also proposed for improved time integration for nonlinear dynamics problems [42].

The author and co-workers [36-42] have performed comprehensive research on the above factors and applied the finite element procedure for comparisons with analytical solutions, and experimental (laboratory and field) observations.

Example 11 - Model Nuclear Power Plant Structure: A typical application for behavior of a model nuclear power plant structure SIMQUAKE II tested in the field [37, 43] is given below.

Figure 33 shows details of the SIMQUAKE II test structure, involving a 1/8 scale model of a nuclear power plant founded in a cohesionless soil [43]. The structure, interfaces and boundary of the soil island, Fig. 33, were instrumented with displacement, velocity, acceleration and pressure measuring devices. A blast type load was applied in two events at an interval of 1.2 seconds.

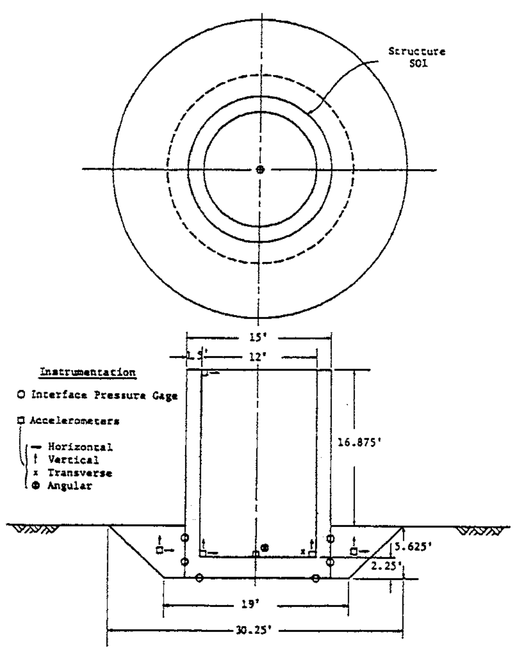


Fig. 33 1/8 Scale Model SIMQUAKE Structure (S01) Including Structural and Near-Field Instrumentation [43]

The interfaces, see mesh in Fig. 34, were characterized by using the Ramberg-Osgood type model and allowed for no slip, slip, debonding and rebonding motions, as well as control of interpenetration. The sand was characterized by using both the cap [43] and the δ_0-version of the hierarchical model [2]. The measured velocities on the boundaries of the soil island were integrated to obtain the displacement vs. time input.

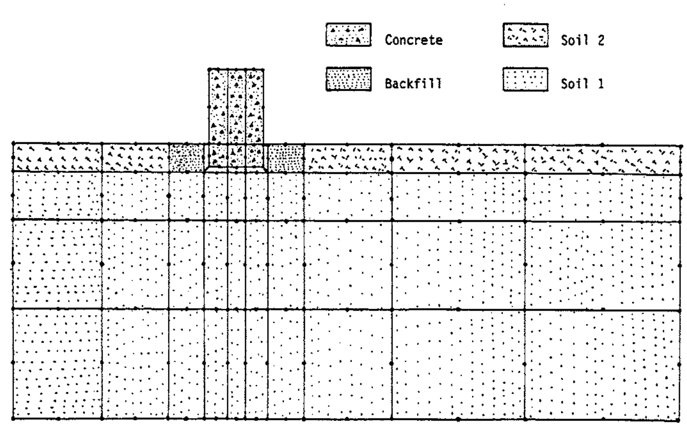


Fig. 34 Mesh Used in Simulation of Soil-Structure Interaction Due to SIMQUAKE II

Figures 35, 36 and 37 show comparisons between predictions and observations for typical horizontal and vertical velocities and contact pressures, respectively. It can be seen that overall the predictions show good comparisons with observations. The interface model assigns arbitrary high or low value for the normal stiffness during bonded and debonded states, respectively. This may be one of the reasons for the discrepancies. It is observed that for realistic simulation of interface response appropriate constitutive models for the normal response should be developed and used [40].

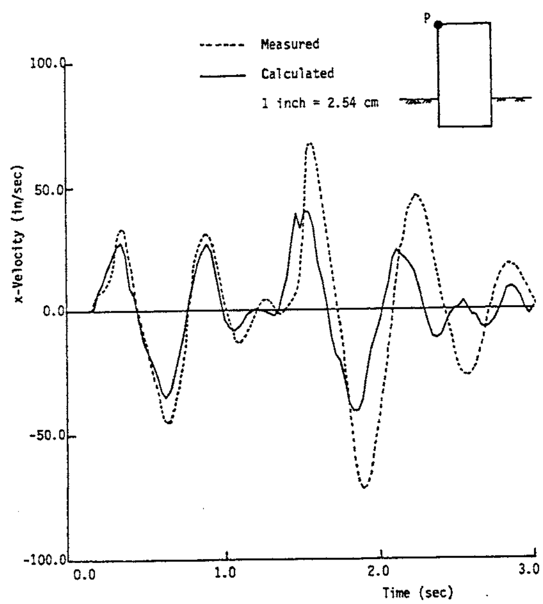


Fig. 35 Comparison of Computed and Measured Horizontal Velocity-Time History at Top of Structure, Point P

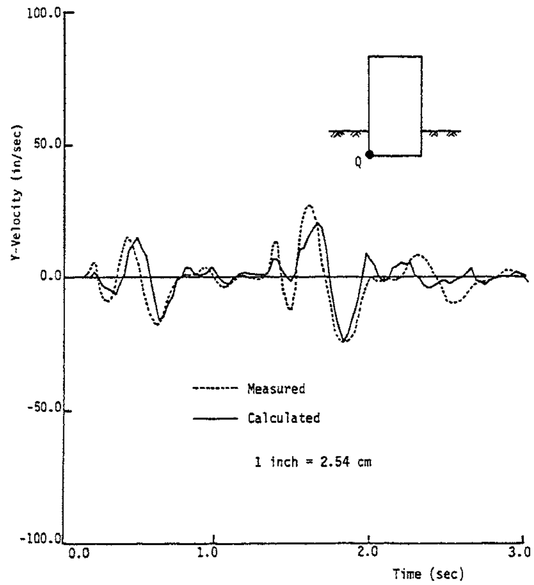


Fig. 36 Comparison of Computed and Measured Vertical Velocity-Time History at Upstream Corner of Structure, Point Q

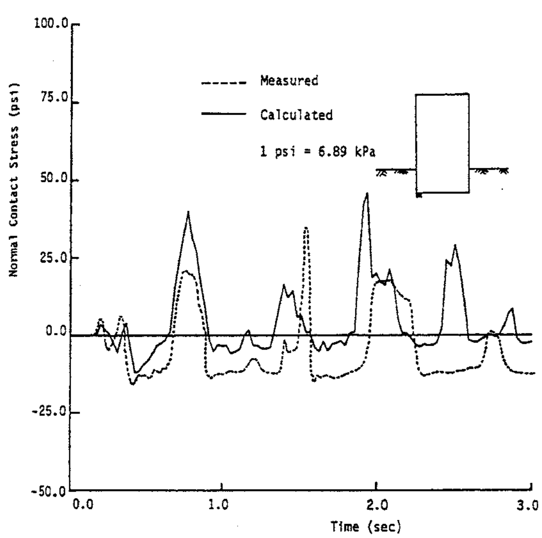


Fig. 37 Comparison of Calculated and Measured Normal Contact Stress-Time Histories Beneath Upstream Corner of Structure

CONCLUSIONS

Use of both conventional and modern solution procedures are important components for development of safe and economical schemes for design, analysis and performance evaluation of field structures. The 'art' of geotechnical engineering toward development of simplified and empirical procedures relies on intuition, experience and scientific thinking. The tradition of using conventional and empirical procedures for case studies is important in our heritage for design of geotechnical systems, and can provide satisfactory solutions for many problems; they need to be used, nurtured, and improved. At the same time, it is essential to continue vigorously to develop innovative and advanced procedures through a process of rational simplification starting with fundamental principles of mechanics and physics so as to reduce or eliminate a number of assumptions inherent in the conventional procedures. This is vital because many complex factors such as nonlinear response, loadings, geometries and environmental effects influence response of geotechnical problems.

This paper presents a summary of the personal experience of the author involving continuous modifications in thinking from use of conventional to advanced computer procedures. One of the main factors in this narrative has been constitutive models for geologic materials and discontinuities. Here the author has gone from use of linear elastic and piecewise linear elastic models about two decades ago to general models that can go beyond the capabilities of the models used in the past. In this growth, the objective of working towards 'simplified' models that can be applied easily in practice, starting from fundamentals, has been followed. The author can conclude that it is possible to develop as or more simplified models than linear and nonlinear elastic that can allow inclusion of many important effects towards more rational case studies of geotechnical problems.

Finally, the author believes that in order to remain competitive and advance into the next century, it is essential to improve our methods through scientific inquiry coupled with intuition and experience, in addition to using and improving on conventional empirical procedures.

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The results presented herein represent a part of the author's work over the last two decades. They include participation of a number of his students and colleagues, and support of a number of funding agencies.

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