RELOCATION AND REPAIR OF THE NATIONAL GEOTECHNICAL CENTRIFUGE

Final Report

to

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**Final Report:
Relocation and Repair of the National Geotechnical Centrifuge**

I. Introduction

In January of 1984, the large geotechnical centrifuge located at NASA Ames Research Center, was in the first stages of operational checkout when the main thrust bearing of the large D.C. drive motor failed. The failure analysis later showed that this failure was due to a faulty lubrication design in the bearing. In failing, the motor rotor dropped one inch and caused a short-circuit of power through the gear train and other points of contact as the electricity surged to ground.

This electrical current caused extensive damage by spot welding moving parts instantaneously, followed by ripping apart due to the momentum of the machine. The drive train and motor were so extensively damaged that the cost of repair was deemed prohibitive.

After many months of investigation and proposals for repair of the facility, it became evident that it would be far more advantageous to the engineering community to remove those components of the centrifuge that were undamaged to the Davis campus and replace the drive system completely. This proposal met with approval of NASA Ames Research Center, who offered and did pay for the physical transfer of centrifuge and of NSF, who accepted the government equipment as inventory to later transfer title to the University of California at Davis.

The large centrifuge had cost over 2 million dollars to build, and it would have been irresponsible to simply scrap it. Recognizing this fact, funds were solicited and received from various sources, and Beam Engineering Inc. was contracted to design and construct the centrifuge at its new location.

The University of California contributed a quarter of a million dollars and Tyndall Air Force Base, through Los Alamos National Laboratory, contributed \$140,000. There were funds also contributed by LANL, LLNL, US Navy and NSF. These funds permitted the digging of a pit, mounting the centrifuge on a foundation pad and bringing in the power to a new drive system.

The first stage in the phased development of the newly located centrifuge is nearing completion, which prompts the writing of this report. By the time that this report reaches the reader the first runs of the centrifuge will have been completed.

The present report describes the present capability of the centrifuge and the plans for upgrading as time goes on.

II. New Site and General Arrangement

The site chosen on the Davis campus of the University of California is about two miles from the campus center but still on University property which extends over 3000 acres in the central valley of California about 15 miles from Sacramento, the capitol city of California. This site has the advantages of 1) being adjacent to a facility with a complete mechanical workshop and plastics model shop, 2) being 500 feet from the University small craft airport, 3) having ample space for the formation of safety berms and entrance ramps, 4) being located in an area serviced directly by Pacific Gas and Electric, 12 kV power line and the University network, either of which can provide 1.5 megawatt at off-peak periods and 0.75 megawatt during peak periods for the centrifuge.

Many of the advantages of the NASA Ames Research Center site are replicated. Davis is 20 minutes from the Sacramento Metropolitan Airport that has many direct flights to major cities throughout the

country. It is fed by Interstate Highways 80 and 113, and the Southern Pacific Railroad with routes to Sacramento and East, North to Oregon and Washington and South-West to the San Francisco Bay Area and South.

Newly paved roads to the site make experiment model transport feasible without undue vibration. Space for model preparation within an adjacent Hydraulics Laboratory has been negotiated, however, making lengthy travel unnecessary in model building.

Pictures of Site

Front of Hydraulics Lab

Ground Breaking, 1986

Map of Local Area including centrifuge site - UC Davis

III. Phase I Development

In the interest of economy, the first phase in the development of the new facility at U.C. Davis was limited to a foundation and pedestal mounting of the centrifuge and the new drive system that was purchased from an oil well drilling equipment company in Oklahoma. The walls and roof have been postponed until phase II and future funding. A short stub wall was included and left with appropriate re-bar protruding to accept subsequent lifts of concrete wall.

In this state, irrespective of power, the centrifuge will be limited to 20-35 g centripetal acceleration, but is still capable of the full 300 g and 4 ton payload which will follow with the completion of the rotunda and obtaining 5 megawatts of power to additional motors. Room has been left for these additions and upgrading as time and funding permit. The payload capacity in size (42 sq. ft. in area) and in mass (4 tons) is unequalled by any geotechnical centrifuge in the U.S.

The 1000 slip ring stack was retrieved in the transfer although in the beginning only 125 electrical lines will be activated. Much of the instrumentation developed in the Centrifuge laboratory in Bainer Hall in the central campus will be made available for the large centrifuge. These include transducers for displacement, acceleration and pore water pressure and a high speed LeCroy digital data acquisition system. Closed circuit television will permit viewing of the experiment in flight and provisions are available for making still photography as well.

IV. Capacity

The capacity will be limited, as mentioned above, to 20-35 g, until funding can be obtained to enclose the centrifuge in a rotunda. In the

mean time, the centrifuge will be able to model prototypes up to 175 ft in height and 245 ft by 210 ft in plan.

The dead load capacity at these low g-levels increases to 34 tons although space limitations precludes reaching this mass, with soil alone. This ruggedness, however, permits the inclusion of rather heavy peripheral equipment such as an earthquake simulator without compromising strength.

A more reasonable mass within space limitations is 12 tons, which corresponds to 440 g-tons at 35 g.

V. Preliminary Test Programs

Several pilot studies were carried out by Bruce L. Kutter and by K. Arulanandan and their colleagues and students, that were funded by the grants from Tyndall AFB through Los Alamos National Laboratory. These experiments involved 1) the effects of nearby explosions on buried thin-walled containers and 2) the advection and dispersion of toxic waste water through soil. The research has been reported in publications that are appended to this report. It is anticipated that these types of tests will be carried out in the large centrifuge where the "box" effect of the small Schaevitz centrifuge will be minimized or eliminated.

On the explosive testing work one paper entitled "Centrifuge Modeling of the Effect of Blast Loading on Tunnels" with Kutter, O'Leary and Thompson was presented at the Second Symposium on the Interaction of Non-Nuclear Munitions with Structures in April of 1985. This paper has been followed by a paper submitted to the ASCE Journal of Geotechnical Engineering, entitled "Gravity Scaled Tests on Blast-Induced Soil-Structure Interaction" by Kutter, O'Leary, Thompson and Lather.

In this program centrifuge model tests were conducted on models of flexible shallow tunnels in dry sand subject to blast loading from nearby high energy explosives. Experiments were conducted with varying centrifugal accelerations (from 1 g to 97 g) on 2.5 to 5 cm diameter model tunnels made of thin brass and aluminum sheet. Buried, 1/2 gram and 1/16 gram PETN explosive charges were detonated at a distance of 0 to 2 diameters from the tunnels during centrifugation. It appears that for flexible tunnels which are exposed or nearly exposed by the crater, significant damage to the tunnel occurs during crater excavation. The intense flow which occurs in the soil near the crater appears to be the major cause of distortion of the tunnel and the shock wave itself is often less significant. The distance from the tunnel to the crater appears to be a more significant factor than the distance from the tunnel to the charge in causing damage to flexible tunnels. Since gravity influences the cratering process, and consequently influences the soil-structure interaction, it is important to conduct scale model tests of this type of event on a centrifuge. The centrifuge scaling laws were verified by conducting modeling of models tests using 1/97 and 1/48.5 scale tests. Cratering efficiency is also compared to that measured by other researchers, with reasonable agreement.

In the waste transport research one report was submitted to Tyndall Air Force Base entitled "Centrifuge Modeling of Transport Processes of Pollutant Travel Through Soils" by Arulanandan, Thompson, Meegoda, Muraleetharan, Yogachandran and Kutter.

In addition a paper with the same title and authors has been accepted by the ASCE Journal of Geotechnical Engineering.

In this work, modeling laws for pollutant transport in ground water are derived. Centrifuge test results show that advection and adsorption

can be modeled. To directly model dispersion, the Peclet number must be matched between model and prototype except when the Peclet number is less than one. Then the results are insensitive to Peclet number and it may be safely ignored. Scaling gravity is necessary in order to achieve the stress dependent soil properties in both model and prototype.

IV. Conclusion

The purposes for the funding from Los Alamos National Laboratory have been successfully accomplished, although belatedly owing to the number of interagency agreements that had to be handled. The Centrifuge has been proven to be a valuable tool in the study of soil interactive phenomena and useful to many DOP and DOE needs. The controversy regarding the necessity for centrifuge modeling in dynamic events that involve failure of buried structures should be resolved by the experimental results obtained by Kutter that compare 1 g test results to higher test results.

APPENDIX A

Centrifuge Modeling of the Effect of Blast Loading on Tunnels, Addendum to Proceedings of Second Symposium on the Interaction of Non-Nuclear Munitions with Structures

ADDENDUM TO

PROCEEDINGS
OF
SECOND SYMPOSIUM ON

**THE INTERACTION OF
NON-NUCLEAR MUNITIONS
WITH STRUCTURES**

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CENTRIFUGAL MODELING OF THE EFFECT OF BLAST LOADING ON TUNNELS

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ABSTRACT

Preliminary experimental results are presented from scale model tests under 1 g, 50 g and 100 g centrifugal accelerations to verify centrifuge scaling laws and to investigate the importance of gravity on blast induced soil structure interactions. The tests involved detonation of scaled charges near a buried tunnel in dry sand. It is expected that the importance of gravity increases as the range from the blast increases and as the level of blast induced ground accelerations approaches the acceleration due to gravity. These expectations are supported by experimental data. The initial pulses of high level blast induced tunnel accelerations appear to be less affected by gravity. A discussion of energy considerations suggests that the proportion of the total energy of an explosion that is spent by radiation of mechanical waves in the soil depends on the proportion of energy spent in the process of crater excavation. The energy of crater excavation for a given charge size is shown to be dependent on gravity.

INTRODUCTION

A proposed protective structure to resist blast loading consists of a buried 8' diameter reinforced concrete pipe with 8" thick walls. The prototype pipe is to be obtained in 8' long segments and then assembled in a trench with half the diameter below ground level and the other half covered with a soil berm. The ultimate goal of this study is to assess the survivability of these protective structures to withstand the blast of conventional weapons (say 200 to 2000 lb of TNT equivalent) in near miss conditions. Some full scale tests on such structures have been conducted and these are reported by Meyer (1984).

As stated by Meyer (1984) in the introduction to his report on the full scale test results "the high cost of conducting (full scale) tests generally limits the number of test events. As a result, the systems survivability cannot be assessed on the basis of (full scale) test data alone" (parentheses added). He goes on to say that analytical models are often used to fill the gaps in the test data.

In this paper it is suggested that centrifuge model tests can also be used to augment the field test data. Construction and testing of a centrifuge model is much less expensive than conducting a full scale test. Hence, many centrifuge tests can be conducted to carry out parametric studies and to optimize design parameters.

The research reported in this paper provides evidence to improve our confidence in the centrifuge modeling technique and emphasizes the importance of conducting scale model tests on a centrifuge. The objectives of this research can be stated concisely:

1. To conduct modeling of model experiments to verify the centrifuge scaling laws and to improve our faith in the test procedure; and,
2. To investigate the importance of gravitational acceleration on the response of underground structures to blast loading and hence determine whether it is necessary to conduct scale model tests on a centrifuge.

In order to minimize the cost of tests to validate design of underground structures to resist blast loading it is often desirable to carry out small scale tests prior to any full scale testing. An important question then arises: Do these tests need to be conducted on a centrifuge? In other words, are the blast induced stresses and ground accelerations significantly affected by gravity loading? If not, gravity scaling (i.e. centrifuge modeling) is unnecessary. This paper shows some preliminary results that suggest that accelerations transmitted to a buried structure may be affected by gravity loading. In addition, some theoretical and some qualitative arguments are presented to suggest that gravity effects will be significant in certain circumstances.

CENTRIFUGE TEST PROCEDURE

The goals of these tests were primarily to show the feasibility and desirability of conducting blast induced soil structure interaction model tests on a centrifuge. It is not necessary to precisely model an actual prototype to achieve these goals. However, it is desirable to show that the model tests do represent a prototype event that is approximately similar to a practical situation. For these reasons many of the complex geometric and material details that exist in the prototype were simplified or neglected. A larger centrifuge would be required to include all of these details in the model.

The tests were conducted in an aluminum box with an aluminum tunnel buried with its axis one diameter below a level surface of dense sand as shown in Figure 1. The wall thickness of the tunnels was selected to provide a similar elastic stiffness for the aluminum tube and the prototype concrete tunnel as follows.

According to the centrifuge scaling laws, stresses and strains are identical in model and prototype,

Figure 1. Centrifuge Model Details;
(dimensions given in prototype terms)

- A. 141 lb PETN charge
  - B. Aluminum tunnel
  - C. Model container
  - D. Accelerometer attached to tunnel
- a = 33 feet, b = 100 inches, c = 150 inches, d = 100 inches, e = 62.5 feet.

lengths are N times smaller in the model, accelerations are N times greater in the model than in the prototype and forces are N^2 times smaller in model than prototype. If we want to simulate the pipes stiffness in terms of its tendency to go "out of round" we can require

$$\delta_m = \frac{1}{N} \delta_p$$

where δ_m is the change in diameter of the model pipe and δ_p is the change in diameter of the prototype pipe. The change in diameter of a long thin walled tube subject to side loading is proportional to (Roark and Young, 1975):

$$\frac{W R^3 (1-\nu^2)}{E l h^3}$$

where W is the side force, R is the radius, ν is Poisson's ratio, E is Young's Modulus, l is the length of the tube and h is the thickness of the tube. For similarity we require $N \delta_m = \delta_p$, where the subscript m denotes model and p denotes prototype, that is:

$$N \frac{W_m R_m^3 (1-\nu_m^2)}{E_m l_m h_m^3} = \frac{W_p R_p^3 (1-\nu_p^2)}{E_p l_p h_p^3}$$

if we substitute $N R_m = R_p$, $N^2 W_m = W_p$, $\nu_m = \nu_p$, and $N l_m = l_p$ we obtain

$$h_m = (E_p / E_m)^{1/3} (h_p / N)$$

The model tunnels were made of aluminum since it would have been very difficult to produce reinforced concrete tubes small enough to test on a small centrifuge. Taking $E_m = 10,000$ psi (aluminum) and $E_p = 3,000$ psi (concrete) gives $h_m = 0.693 h_p / N$. According to this relation, the desired and actual model dimensions are summarized in Table 1.

Table 1. Dimensions of Model and Prototype Tunnels

Scale Factor (N)	Tunnel Diameter		Tunnel Wall Thickness		Young's Modulus (psi)
	ideal (in.)	actual (in.)	ideal (in.)	actual (in.)	
100	0.960	1.00	0.055	0.0625	10,000
50	1.92	2.00	0.110	0.125	10,000
prototype	--	96	--	8	3,000

As seen in Table 1, the diameter, wall thickness, and modulus of the model tunnels only approximately simulate the prototype structure, however, it should be emphasized that the 1/100 scale model is a precise 1/2 scale model of the 1/50 scale model.

Two rectangular model boxes were constructed from aluminum plate, one being exactly 1/2 scale of the other to insure that boundary effects would be similar in the model tests at different scale.

The aluminum model tunnels were instrumented as shown in Figure 1 with a high frequency accelerometer measuring the horizontal tunnel acceleration. After removing one side of the model container the container was set on the opposite side. The tunnels were set in place and Monterey 0/30 sand was pluviated in the direction of the axis of the pipe in order to make it possible to achieve a uniform density on all sides of the tunnel. The relative density of the models ranged between 95 and 100%. The finished sample was then placed on the centrifuge and the explosive charges were pushed into the sand to the same depth as the tunnel axis (one tunnel diameter) at a horizontal distance of 1.5 tunnel diameters from the tunnel axis. The charges were made of PETN and were detonated by exploding bridge wires. The charges, catalog items (RP-1 and RP-2) were purchased from Reynolds Industries Inc., San Ramon, CA. The RP-2 charges are 3.9 mm diameter, 5.1 mm long and contain 64 mg of PETN. The RP-1 charges were modified to have precisely 8 times the mass of the RP-2. The charges contain 512 mg of PETN, and are 7.6 mm in diameter and 9.5 mm long. Both charges were cased with a thin metal shell. The thickness of these shells could not be precisely scaled without major expense. The RP-1 had a 1.33 mm thick cylindrical brass shell, and the RP-2 had a 0.86 mm thick stainless steel shell.

After bringing the centrifuge up to the desired speed, the exploding bridge wire detonations were triggered remotely using an LSI 11/23 computer which began digitizing data from the test instrumentation at the same time that it initiated the firing sequence. The computer digitized data at a rate of 66,500 data points/second and only two or three channels of data were recorded. This rate of digitization is barely adequate for measurement of the high speed blast events, especially for monitoring the "short time" rapidly changing data.

RESULTS

To illustrate the reliability of the centrifuge scaling laws and the test procedures, "modeling of models" tests were carried out. "Modeling of Models" is the jargon for testing two models of different scale that represent

Figure 2. Crater profiles for five tests, 2 tests at 50 g, 1 test at 100 g, and 2 tests at 1 g. For the 1 g test, the crater actually uncovered part of the model tunnel. The lack of symmetry in the W-E profile may be due to Coriolis effects. The axis on the left is labelled with prototype dimensions.

the same prototype. Tests LB3 and LB4 had linear dimensions scaled to 1/50 of full scale and were tested at a centrifugal acceleration of 50 g while test LB5 was made to 1/100 scale and was tested at a centrifugal acceleration of 100 g. In addition to these tests, two additional tests, LB6 and LB7, were carried out at 1 g at 1/50 and 1/100 scale in order to investigate the effect of gravity on the response of the structure to blast loading.

Crater Dimensions

Figure 2 shows the results of measurements of crater profiles along two perpendicular cross sections through the deepest part of the craters. The E-W profile corresponds to the plane of rotation of the centrifuge (the centrifuge was spinning from W to E). The N-S profile corresponds to a plane parallel to the axis of the centrifuge. The dimensions given in Figure 2 are prototype dimensions. Excellent agreement is observed in terms of prototype crater dimensions for the centrifuge tests at 50 and 100 g. Also shown in this figure is the location of the crater for tests LB6 and LB7, which were conducted at 1 g. These results clearly indicate the effect of gravity on crater dimensions. Schmidt (1977) and Schmidt and Holsapple (1980) have obtained results indicating the effect of gravity on crater formation, so this is nothing new. However, it does illustrate the reproducibility and repeatability of the tests.

While the N-S crater cross-section appears very symmetrical, the W-E cross-section shows a marked asymmetry. While this may be partially attributable to the presence of the nearby buried tunnel, some asymmetry will be attributable to Coriolis effects.

Tunnel Accelerations

Figure 3 shows the results of measurements of horizontal acceleration of the tunnel at the point labelled

D in Figure 1 for various tests. In Figure 3a results from centrifuge modeling of models tests at 1/100 and 1/50 scale are shown. Production of this figure required scaling time according to $t_m = (\frac{1}{N})t_p$ and acceleration according to $a_m = Na_p$. In Figure 3 the scale for acceleration is changed after 15 ms by a factor of 10 to allow comparisons to be made for the later, lower level accelerations.

It is apparent that insufficient data points are available to completely define the shapes of the curves, however the peaks at about 7 ms and 17 ms seem to be reproduced in both tests in Figure 3a with reasonable accuracy. These results are not surprising, but they do give some confidence in the experimental technique. Next, tests were conducted using scale models without spinning the centrifuge in order to investigate the influence of gravity. Figure 3b compares results from two 1/100 scale tests, one tested at 1 g and the other at 100 g. While the peak acceleration at ~7 ms is approximately reproduced in both tests, the 1 g test appears to have a delayed peak. In the later (after 15 ms) lower level acceleration pulses, the agreement is very poor between these tests indicating the gravity may be important for these accelerations.

A similar comparison for 1/50 scale models is shown in Figure 3c. The influence of gravity appears to be less significant for these tests. Although the peak observed accelerations are similar for the 1 g test and the centrifuge test, the timing of the pulses appears different. In addition, the lower level spikes at 17 ms and 34 ms are quite different in the two tests.

Summary of Results

Modeling of Models experiments using the centrifuge scaling laws provide some confidence in the scaling laws and the experimental procedures. The effect of gravity on crater size is demonstrated. The effect

Figure 3. Tunnel acceleration versus time. All dimensions scaled to prototype terms for comparison. Best agreement is observed in (a) where proper modeling of models on the centrifuge is used. (b) and (c) show the effect of centrifuge acceleration on tests conducted at the same scale. After 15 ms prototype time, the acceleration scale is magnified by a factor of ten.

of gravity on tunnel accelerations appears to be more important for lower level accelerations, but some gravity influence may also exist for the short time high level acceleration pulses. Unfortunately, the speed of data acquisition was insufficient for clearly defining the acceleration-time histories. This means that more tests must be conducted before conclusive experimental proof can be obtained to illustrate the effect (or lack of the effect) of gravity on tunnel response to blast loading.

DISCUSSION

The influence of gravity on blast induced soil-structure interaction is a subject of debate and controversy. Gravity will affect the soil structure interaction in at least the three ways listed below:

1. Gravity loads themselves may be significant.
2. Self-weight induced confining pressures are caused by gravity and confining pressure in turn affects the mechanical properties of soil.
3. Gravity has a proven influence on crater formation and on the energy expended in crater excavation. The energy expended in crater excavation is energy that will not be available to cause damage to a buried structure.

Gravity Induced Loads

In static soil structure interaction problems gravity is often the primary loading force. In dynamic problems additional loads can arise from accelerations due to blast, earthquake or wave loading. The importance of gravity as a direct load will obviously depend on the relative magnitude of the dynamic accelerations, a , and the acceleration due to gravity g .

For blast loading, peak ground accelerations can be very large compared to gravity near the explosion, but ground acceleration rapidly decays with range from the source. It can then be stated that gravity becomes more important as range increases, and will become less important as the energy of the explosion increases. Additional experimental and theoretical work is required to define the borderline between cases where gravity loads are and those where gravity loads are not important.

Gravity Induced Confining Pressures

It is well known the soil properties are influenced by confining pressure. For example, for sand at a given density the shear strength is approximately proportional to the mean normal compressive stress, p' , the elastic shear moduli for low amplitude strains are approximately proportional to $(p')^{0.5}$, and the secant moduli for higher amplitude strains are proportional to $(p')^n$ where $0.5 \leq n \leq 1$.

The gravity induced confining pressures however, may be small compared to pressures generated by an explosion, so we expect gravity to become more significant for soil structure interaction problems when the structure is far away from the explosion. The distance at which gravity pressures become important still needs to be determined.

In the regions where gravity induced confining pressures are significant, pressure that can be transmitted by the soil to a buried structure will be limited by the strength of the soil. Vaughan et al. (1980) studied 1/3, 1/6 and 1/30 scale tests at 1 gravity in which he observed scale effects on the response of vertical shelters to blast loading. They suggested that these scale effects may be due to the effect of gravity on confining pressures and soil shear strength. They found that larger scale models suffered greater damage than small scale models.

In addition, the loads transmitted from the soil to the structure depend on the relative stiffness of the soil and the structure. For relatively flexible structures pressures felt by the structure will be reduced by arching and for relatively stiff structures, pressures felt by the structure may be greater than those in the free field due to stress concentration.

It is possible that advantage can be taken of arching to optimize the design of buried structures to resist blast loading. For example, a layer of relatively soft soil may be placed around the tunnel to isolate the structure from free field pressures.

In order to physically model a complex problem with different layers or isolation systems, the relative stiffness and strength of each material must be simulated. If gravity induced confining pressures are important, scale model tests involving geological materials must be conducted on a centrifuge.

Energy Considerations

The total energy of an explosion, E_T , is dissipated in several ways.

1. Heat energy, E_h
2. Crater excavation, E_e
3. Energy radiated into atmosphere, E_a
4. Energy radiated into the soil mass as shock or elasto-plastic waves, E_s
5. Other, E_o

Conservation of energy can be written as

$$E_T = E_h + E_e + E_a + E_s + E_o$$

At times after the explosion, the components of energy may be converted from one form of energy to another.

Of primary interest for the soil-structure interaction problem is the term E_s - the energy radiated through the soil. It is this energy that will cause a buried structure to accelerate or deform. The manner in which this energy is transmitted through the soil may depend on gravity in regions where the soil material strength and stiffness are important as discussed earlier. However, the initial shock waves that emanate from a blast are governed primarily by the magnitude of the pressure generated and by the material density. Material strength and stiffness are relatively unimportant in this regime.

Even if the transmission of shock waves is not affected by gravity, the portion of the total blast energy that is dissipated through the soil, E_s , may depend on gravity. If any of the terms on the right hand side of the above equation are influenced by gravity, there is a potential influence on E_s .

Croft (1977) suggests that the energy of crater excavation for impact craters can be approximated by

$$E_e = K_1 \rho g D^4$$

where K_1 is a constant and D is the crater diameter. Schmidt (1980) analyzed several impact cratering experiments with varying gravity and found for a given projectile density, mass and velocity and soil density:

$$D = K_2 g^B$$

where K_2 is a constant that depends on projectile density, mass and velocity and B is a constant with $-0.17 \geq B \geq -0.24$. Combining the above two equations gives:

$$E_e = K_3 g^{(1+4B)}$$

where K_3 is a constant which is a function of K_1 , K_2 and ρ . Taking $B = -0.20$ as an average of those observed by Schmidt gives:

$$E_e = K_3 g^{-0.20}$$

That is, if gravity is not scaled (e.g., by centrifuge modeling) the excavation energy in a 1/100 scale model will be in error by a factor of $100^{-0.20}$ or approximately a factor of 2.5. Alteration of the energy of crater excavation may alter the amount of energy that is available to radiate in the soil, E_s , which in turn will alter the energy that is available to cause damage to a buried structure.

CONCLUSIONS

Some preliminary experiments have been conducted to investigate the effect of gravity on blast loading on underground structures.

Due to inadequate data acquisition rates, measurements of tunnel acceleration during the initial blast pulses do not provide conclusive proof that gravity is important for this particular problem. However, the later lower level dynamic accelerations appear to be affected by gravity. Additional experiments are planned to help determine the importance of gravity on blast induced soil structure interaction.

It is expected that gravity will become more important as the range from the blast source increases, and as the relative size of the explosion decreases. In the experiments reported here, the explosive charge was very near to the tunnel (1 tunnel diameter) and this produced tunnel accelerations on the order of 100 g in prototype terms. For cases where lower blast induced accelerations occur, gravity will be more significant than it is for the experiments reported in this paper. It is important to note that blast induced accelerations on the order of even 1 g can cause damage to structures.

When the expected blast accelerations are on the same order as gravitational accelerations we expect gravity to play a more important role.

The total blast energy can be dissipated in several ways. For soil-structure interaction problems the energy radiated through the soil is of prime concern. Based on empirical relations found in the literature the energy absorbed by crater excavation E_e is shown to depend on gravity. If energy expended in crater excavation is not properly simulated in a physical model, we cannot expect to simulate the energy transmitted through the soil. By centrifuge modeling, both E_e and E_s can be accurately simulated in scale models.

Finally, additional work can be done on small centrifuges to study problems with relatively simple geometry to verify the validity of scaling laws and to determine the importance of centrifuge testing. To be sure, centrifuge modeling has potential to economically supplant some expensive full scale testing. However, much larger centrifuges are required in order to accurately model details. Larger centrifuges will be required before centrifuge modeling can be elevated from a role of verification of scaling laws or numerical codes to a direct role in engineering design.

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APPENDIX B

Gravity Scaled Tests on Blast-Induced Soil-Structure Interaction

GRAVITY SCALED TESTS ON BLAST-INDUCED SOIL-STRUCTURE INTERACTION

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INTRODUCTION

The response of buried structures to near-field, high intensity loading from conventional explosives is of critical concern to the U.S. Air Force. The feasibility and importance of using centrifuge model tests to study this problem is investigated in this paper.

Mechanisms. -- The detonation of explosives can damage buried structures by two principal mechanisms. First, the structure is loaded by a rapidly traveling shock wave (with soil pressures on the order of several kbar), which decays in sharpness and magnitude with distance from the charge. For hard, brittle structures such as concrete tunnels this shock may cause damage by cracking or spalling. For more flexible, ductile structures the shock pressures tend to arch around the structure; the ground displacement due to the shock may be absorbed in small elastic or ductile deformations of the structure.

The second mechanism of damage to buried structures occurs for structures in the zone of soil which undergoes large displacements in or surrounding the crater. As seen in the schematic cross section shown in Fig. 1 there are basically 3 zones in which the soil is permanently deformed to a significant degree. The zones of fallback and ejecta

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represent material that was thrown from the crater due to the explosion. The rupture zone and plastic zones involve large compressive and shear distortion of the soil. If a structure is intersected by the plastic zone, rupture zone, or fallback zone, the structure will be dragged along with the soil and/or distorted along with the shear and flow of the adjacent soil.

Modeling. -- Numerical modeling is valuable for analysis and design of structures to resist blast loading, but numerical models inherently involve assumptions regarding the constitutive relations for the material. In addition, large deformations and three-dimensional effects make numerical modeling difficult, and expensive. Experimental data is required to verify the numerical models.

Full-scale tests are costly and time consuming to perform. Scale model tests are less expensive and hence more experiments can be conducted within a given budget and time schedule. This permits repetition of experiments and confirmation of data.

Scale model tests in earth's gravity do not permit complete similarity of gravity-induced confining stresses. Since soil behavior is dependent on confining stresses, reduced scale tests in earth's gravity may yield erroneous results. Vaughan et al. (1980) conducted numerical and physical model tests at 1/3, 1/6, and 1/30 scale in earth's gravity in which they observed that large scale models of vertical shelters suffered more damage from blast loading, than small scale models. They suggested that these scale effects may be due to the lack of similarity of gravity-induced confining pressures.

Centrifuge modeling provides a means for conducting physical scale model tests with accurate simulation of gravity-induced confining pressures. Three dimensional effects, large deformations and real mate-

rial properties are naturally simulated. Centrifuge modeling can play several roles in geotechnical problems. A few of the possible applications of centrifuge modeling are: production of a data base for verification and development of numerical codes, parametric studies for design of geotechnical structures and basic research into the mechanisms of deformation and failure. Centrifuge modeling is also useful for ascertaining trends of behavior and for designing full-scale (prototype) tests of complex systems.

Importance of gravity. -- In several verbal discussions, the authors have found that many other researchers involved in the area of blast-induced soil-structure interaction believe that blast effects on structures are not influenced by gravity in a significant manner. Their view is that structural damage is due primarily to the shock wave, and that the stresses induced by the shock wave are so large compared to lithostatic stresses, that it is not necessary to model in situ lithostatic stresses in physical tests. In their view, gravity induced confining pressures are small enough to be insignificant, and therefore, there is no need scale the gravity forces in a model test. The shock wave loading in many cases may be a major cause of damage by spalling and brittle fracture, and in these cases the above argument is reasonable. In fact, some results are presented in this paper to show that the ground accelerations caused by the primary shock wave are not significantly influenced by centrifugation.

The shock loading is not the only cause of damage, however, for structures located in or near a crater. In this environment structures may undergo significant displacements and deformations after passage of the shock front. Crater excavation occurs on a much longer time scale than shock wave loading. During the cratering process, soil in the cra-

ter and in the surrounding rupture and plastic zones (see Fig. 1) undergoes large deformations. The flow of soil may impose significant drag forces on the tunnel, analogous to the blast wind forces which follow a shock wave after an explosion in air. It has been shown by others (Schmidt (1980) and Schmidt and Holsapple (1980)) that crater size is dependent on gravity. It is expected therefore, that the extent of structural damage caused during the cratering process involves gravity also.

The feasibility and desirability of using centrifuge modeling to study blast-induced soil-structure interaction problems such as this are investigated in this paper. Blast-induced soil-structure interaction has been studied in the past by full-scale tests, reduced scale tests in earth's gravity, and by numerical modeling. Recently, Nielsen (1983) used centrifuge modeling to study the effectiveness of burster slabs and Kutter et al. (1985) gave preliminary results from centrifuge experiments on blast-induced soil-tunnel interactions. A summary of centrifuge scaling laws is given by Schofield (1980) and is not repeated here.

To physically model an event which is influenced by gravity, gravity should be scaled in inverse proportion to length dimensions. The importance of the error caused by improper gravity scaling will depend on soil type, the distance between the charge and structure, the relative stiffness of the soil and the structure, and the scale factor. In many cases it may be adequate to test large scale models in earth's gravity (e.g. 1/2 or 1/3 scale) with insignificant errors. The errors can be significant, however, for model tests involving structures in the zones of soil distorted due to cratering and for tests at very small scale (e.g. 1/10 or 1/100).

Objectives. -- The purposes of this research were to:

- 1) Investigate the importance of scaling gravity in scale model tests of blast-induced soil-structure interaction events.
- 2) Verify the validity of scaling by conducting modeling of models tests.
- 3) Make observations of the mechanisms of blast-induced damage to buried tunnels.

EXPERIMENTAL PROGRAM

The tests were carried out on a Schaevitz, 1 meter (39 in.) radius, 10,000 g-lb geotechnical centrifuge at U.C. Davis. The model tests were performed in two separate aluminum boxes and with three types of tunnels, all using Monterey 0/30 dry sand. Two scaled charge sizes were used.

The larger of the boxes is made of 0.95 cm thick aluminum plate with the inside dimensions being 38 cm square by 25 cm deep. The small box is made of 0.48 cm thick aluminum plate with inside dimensions of 19 cm square and 12.5 cm deep. The depth of sand was 20 cm in the large box and 10 cm deep in the smaller box. The general test arrangements are shown in Figs. 2 and 3; model dimensions are summarized in Table 1.

Two sizes of tunnels were constructed from brass sheet. The large tunnels consist of 0.15 mm thick brass sheeting, 15.2 cm long and 5 cm in diameter. The smaller tunnels were designed to be half the size of the larger tunnels using 0.075 mm thick brass sheeting, 7.6 cm long and 2.5 cm in diameter. Wooden disks were glued inside both ends of the brass tubes to hold the ends in a circular shape. A small hole in the wooden disks allowed for venting any air pressure generated by tunnel distortion. The two previously described brass tunnels and boxes were

dimensioned according to centrifuge scaling laws to simulate the same prototype. The third type of tunnel was a 16 ounce aluminum beverage can 0.13 mm thick, 16.3 cm long and 6.5 cm in diameter.

The brass tunnels were placed in one of the two boxes designed for these experiments at depths of one tunnel diameter (from the axis of the tunnel to the soil surface). The beverage can tunnels were placed at various depths as shown in Fig. 2. The sand was pluviated in the direction of the axis of the tunnels in order to achieve a uniform sand density on all sides of the tunnels. This required placing a temporary lid on the soil surface and removing one side of the box. After completely filling the box by pluviation through the open side, the side is bolted on, the sample is placed upright, and the temporary lid is removed. The dry density of the sand was 1.7 g/cm^3 which corresponds to a relative density of 90-95%. Triaxial tests on the Monterey 0/30 sand by Abghari (1987) at this density showed that the peak friction angle gradually decreased from 55° to 42° as the confining pressure increased from 20 to 700 kPa.

The charges consist of PETN (pentaerythritol tetranitrate) and have about 150% the energy per unit mass (specific energy) of TNT (Kinney and Graham (1985)). The explosives were detonated by exploding bridge wires. The charges (catalog items RP-1 and RP-2) were purchased from Reynolds Industries Inc., San Ramon, CA. The RP-2 charges are 3.9 mm in diameter, 5.1 mm long and contain 64 mg of PETN. The RP-1 charges were modified to have precisely eight times the mass of the RP-2. The large charges contain 512 mg of PETN, are 7.6 mm in diameter, and are 9.5 mm long. PETN has a specific energy of approximately 5.44×10^{10} ergs/gram (Schmidt et al. (1986)). The charges were filled with PETN powder packed to an average density of approximately 1.1 g/cm^3 . The charges consist

of both high density PETN and low density PETN portions. Both charges were cased with a metal shell. The thickness of these shells could not be precisely scaled without major expense. The RP-1 had a 1.33 mm thick cylindrical brass shell, and the RP-2 had a 0.86 mm thick stainless steel shell.

"Modeling of Models" tests at two scales were performed on the large brass tunnel with 512 mg PETN charge in the large box at 48.5 times the acceleration of gravity (tests TK3 and RB3) and on the small brass tunnel with 64 mg PETN charges in the small box at 97 times the acceleration of gravity (tests TK4 and RB1-RB5).

Another series of tests was performed on a small brass tunnel with 64 mg PETN charge in the large box at 48.5 times the acceleration of gravity (tests RB6-RB13). These tests were performed in the large box to minimize boundary effects and with the explosive at various distances from the tunnel to study the influence of range on tunnel damage.

Another test series was performed at various g-levels with the aluminum beverage cans (LB12-LB17) in order to demonstrate a correlation between degree of damage and centrifugal acceleration. The above information is summarized in Table 1.

Two additional experiments were conducted to study the influence of gravity on ground accelerations. For these experiments the 512 mg charges were placed 5.1 cm deep in the middle of the large aluminum box. One accelerometer was placed 5.5 cm deep in the sand, directly facing the charge at a radial distance of 10 cm. Another accelerometer was placed directly below the charge at a distance of 7.7 cm from the charge. The accelerations were recorded using a high speed data acquisition system capable of sampling 4 channels at 1 Megasample per second.

Four additional tests (RB14-RB17) were conducted with no tunnel in order to provide additional data on cratering efficiency.

ANALYSIS OF RESULTS: TUNNEL DAMAGE

Gravity Effects. -- The influence of gravity on blast induced damage to nearby tunnels was shown by constructing several samples with identical geometry and materials (as shown in Fig. 2) and then testing them in various acceleration fields. In this test series, (test numbers LB12-17) three 16 oz. aluminum beverage cans were used as model tunnels. Tunnel 1 was approximately twice as far from the charge and at a different depth than tunnel 2, and tunnel 3 was directly beneath the charge. The closer tunnel (tunnel 2) was partially exposed by the crater in all tests while the more distant tunnel (tunnel 1) was just barely uncovered when detonations occurred in the lowest acceleration field (1g). Tunnel 3 did not experience significant damage and therefore no gravity effects could be observed for this tunnel.

Figure 4 shows a view of model LB14 (tested at 50g) and LB13 (tested at 1g), with all other factors identical. The influence of gravity on crater size is evident. Tunnel 2 is barely exposed in test LB14 and it is completely dislodged in test LB13. Tunnel 1 was barely exposed in test LB13 and remained completely covered in test LB14. Figure 5 shows a view of the deformation of the model tunnels.

By consideration of the Figs. 4 and 5 it is apparent that deformation/displacement patterns are influenced by gravity. In test LB13, the crater is larger than in LB14 and tunnel 2 is blown out of the sand and allowed to displace rather than deform. In test LB14, this tunnel is held firmly in place by the sand at the ends of the tunnel and the tunnel midsection is forced to deform rather than displace.

Data from these and other tests are given in Fig. 6a, which shows tunnel damage ($\Delta L/L$) versus gravity for tunnel 2. ΔL is the reduction in length of the can due to pulling in of the ends of the can during the blast, and L is the initial length of the can. There appears to be a straight line relation between damage ($\Delta L/L$) and gravity on the log-log plot, with damage increasing with gravity for this tunnel.

Figure 6b shows damage to tunnel 1, the more distant tunnel. For this tunnel it appears that damage decreases as gravity increases. For this tunnel a different index was used to quantify damage. The change in diameter ΔD divided by the initial diameter D is plotted versus gravity. It appears that the gravity effect is not very large for tests at elevated accelerations; for the tests at $1g$, however, the tunnel damage is significantly greater than in the other tests. The sudden drop in damage between $6g$ and $1g$ accelerations for tunnel 1 may be explained by the fact that the tunnel begins to be exposed when accelerations drop to about this level.

It is interesting to note the contrasting gravity effects for the two tunnels. For the nearby tunnel, damage increases with gravity, while for the more distant tunnel, damage decreases with gravity. This can be explained by the following hypothesis. If the tunnel is exposed by the crater in all gravity fields, damage decreases as gravity decreases, but if a tunnel is unexposed at high gravity and a reduction of gravity causes the tunnel to be exposed by the crater, damage may increase as gravity decreases.

During crater formation, flowing sand particles in the crater and in the rupture and plastic zones around the crater exert drag forces on

the structure if it is in or near the crater. These dynamic pressures are much smaller in magnitude than shock wave pressures, but they have a much longer duration and they may have more potential to cause large deformations or displacements.

For the flexible tunnels used in this study, the drag force appears to be the major cause of damage. The tunnels used were flexible enough to absorb the small displacements associated with the shock wave with little damage, but the more sustained pressures resulting from the drag dislocated and/or crushed tunnels that were near or in the crater. Hard tunnels may not be damaged by drag, but if they are very near the crater, they may be physically dislodged or displaced.

Neglecting proper gravity scaling may have different effects on damage to a model structure depending on the range. If the structure is outside the plastic zone in the prototype but inside the plastic zone in the model, the model will suffer more damage than the prototype. On the other hand, if the structure is inside the plastic zone for both model and prototype, the model will suffer less damage than the prototype. This is explained by a trade-off between damage and displacement: a large lithostatic pressure can hold a structure relatively firmly in place resulting in large drag forces as the soil flows around the structure. An improperly scaled (small) lithostatic pressure will allow the structure to move with the soil being accelerated out of the crater and this will reduce the structural damage.

Modeling of Models. -- In order to confirm modeling techniques and the scaling laws, a procedure called "modeling of models" is often carried out. This means that tests are conducted at different scales, with all tests modeling the same prototype. For this paper, modeling of model

experiments were carried out at 48.5 g and 97 g. The 1/48.5 scale tests were conducted with 5.1 cm diameter 0.15 mm wall thickness brass cylinders and the 1/97 scale tests were conducted with a 2.54 cm diameter, 0.075 mm wall thickness brass cylinders. Charge size, box size and charge distance were also scaled by a factor of two as explained in the procedure section of this paper.

The tests at both 1/48.5 and 1/97 scale represented approximately 2.5 m diameter tunnels with an 8 mm wall thickness buried with their axis 1 diameter below the ground surface. The distance from the charge to the tunnel axis was 1.5 tunnel diameters and the charge size in prototype terms was 58 kg of PETN in all modeling of models tests. The dimensions were not chosen to directly model a particular prototype. Within limits of experimental error on charge and tunnel placement the 1/97 scale tests can, however, be considered to be a 1/2 scale model of the 1/48.5 scale tests. Various measurements of the craters and deformed tunnels are listed in Table 2 with dimensions converted to prototype terms to facilitate the modeling of models comparison. Fig. 7 and Table 2 summarize various dimensions, crater size and tunnel damage for all five tests. The tunnel damage results in Table 2 are encouraging in that approximately the same damage was observed for all tests except TK4. The comparison is improved, however, if the errors in charge and tunnel placement are taken into account as shown in Fig. 8.

Initially it was thought that the damage could be correlated with the net distance from charge to tunnel. In the beverage can tests, however, it was found that tunnel 3 suffered much less damage than tunnel 1, even though it was at the same net distance from the charge. It is concluded that horizontal and vertical distances from the charge have different effects on tunnel damage. It was then discovered that

the distance between the tunnel and crater (the distance "b" in Table 2) was a more significant parameter in determining tunnel damage than the net distance to the charge.

Fig. 8 shows the normalized distance to crater, $\pi_{dc} = b(\rho/W)^{1/3}$ for two sets of tests versus tunnel damage $\Delta D/D$. Normalization by $(\rho/W)^{1/3}$ is done to obtain a dimensionless number. Fig. 8 shows that, as expected, tunnel damage is a strong function of π_{dc} .

The circular data points in Fig. 8 represent modeling of models tests. If the tunnels and charges were perfectly placed in the model tests all circles would have the same π_{dc} and a centrifuge modeler would expect the damage ($\Delta D/D$) to be identical for tests at different scale. Four of the tests (at $\Delta D/D \approx 10\%$) are grouped nicely with large and small models giving similar results. There is no apparent bias for large and small models to behave differently. The differences in tunnel damage appear to be largely accounted for by the distance, π_{dc} , not by undesirable scale effects.

Also shown in Fig. 8 is the relation between damage and π_{dc} for the small tunnel tested at 48.5 g (square data points). The damage-distance relation has a similar slope as for the modeling of models tests, but the range in π_{dc} is much greater. For the tests with π_{dc} negative, the tunnel was actually uncovered by the crater.

Ground Accelerations Due to a Shock Wave. -- In two experiments, accelerometers were buried in the soil as explained in the procedure section. Identical samples and instrumentation were used for each experiment, except one experiment was conducted at 48.5 g and the other was conducted at 1 g. Results from the accelerometer buried at a 5.5 cm depth,

at a 10 cm distance from the charge are shown in Fig. 9. The accelerometer recorded approximately a 50 μ s wide pulse with 15,000 g peak acceleration. This corresponds to a 2 ms pulse of 300 g acceleration in prototype terms (assuming a scale factor $N = 50$).

The pulses recorded in the 48.5 g test and the 1 g test were virtually identical. This result indicates that in this case the shock pulse is not significantly affected by scaling gravity.

ANALYSIS OF RESULTS: CRATER DIMENSIONS

A study of cratering efficiency is presented here to

1. Illustrate the suitability of the explosive charges used for modeling of models tests, that is, to show that the small charge is a good model of the large charge.
2. Give credence to the present results by comparing cratering efficiencies with the extensive results provided by Schmidt and his co-workers.
3. Determine the box size required to avoid adverse boundary effects on crater dimensions.

Fig. 10 shows crater profiles scaled to prototype dimensions, measured after several tests. The profiles are labeled to show what scale factor was used for the length dimension $l^* = (l_{\text{model}}/l_{\text{prototype}})$ and for gravity $g^* = (g_{\text{model}}/g_{\text{prototype}})$. For the tests where gravity, charges, and dimensions are scaled properly, the crater profiles agree closely. However, if gravity is not scaled (i.e., if $g^* = 1$) the craters are much larger. This figure clearly shows the importance of gravity on crater dimensions and demonstrates that the small charges produce similar craters as the large charges if gravity is properly scaled. Schmidt et al. (1986) and Croft (1977) and others have also shown the importance of gravity on cratering efficiency.

Fig. 11 shows the relation between dimensionless cratering efficiency, $\pi_1 = V\rho/W$ and gravity scaled yield, $\pi_2 = (g/Q_e)(W/\delta)^{1/3}$, where

V = apparent crater volume (the volume of the crater below the original ground surface)

ρ = soil density

W = charge mass

g = resultant of centrifuge acceleration and earth's gravity

Q_e = specific energy of explosive (energy per unit mass)

δ = average mass density of charge.

Schmidt et al. (1986) suggested the use of these particular dimensionless parameters (π_1 and π_2) and they produced a large amount of data for PETN and other explosives at various depths of burial, d . From their results a dashed line for $d/a=10$ is replotted in Fig. 11 for comparison with results from this study. The charge radius, a , was calculated according to $a = \sqrt[3]{3W/4\pi\delta}$ where W = charge mass and δ = average mass density of charge, 1.1 g/cm^3 . For the current study $a = 0.48 \text{ cm}$ and 0.24 cm for the large and small PETN charges respectively. The value of d/a was held constant at 10.6 for all tests reported in this paper.

The present results show a slightly greater cratering efficiency ($\pi_1 = V\rho/W$) than Schmidt et al. (9), which may be due to various factors. Perhaps most significant is the fact that their data were obtained for a different sand (Ottawa sand) at a greater density, $\rho = 1.8 \text{ g/cm}^3$.

Schmidt (7) conducted impact cratering experiments in Ottawa sand and found that reducing sand density from 1.8 to 1.7 g/cm^3 resulted in

roughly a 30% increase in cratering efficiency. The discrepancy between the present data (the solid line) and the Schmidt et al. (1986) line for $d/a = 10$ (the dashed line) in Fig. 11 is about 15 to 25%. It is quite likely that the discrepancy in crater dimensions is due, at least in part, to the difference in density of the Ottawa sand (1.8 g/cm^3) and the Monterey sand (1.7 g/cm^3). Considering this, the agreement between the results of Schmidt et al. and the present study is very good.

In Fig. 11 at $\pi_1 \approx 10^3$ and $\pi_2 \approx 6 \times 10^{-7}$ the bar represents a cluster of seven data points corresponding to modeling of models tests: 4 tests at 48.5 g with large box and large charge and 3 tests at 97 g with the small box and small charge. The small band for all tests verifies the scaling laws within the range of variation of parameters. The points with hollow circles represent tests in the large box with the large charge and various centrifuge accelerations. At low π_2 values, they fall below the criss-cross points which correspond to tests with a small charge (and hence a smaller crater) also in the large box.

The hollow circles at $\pi_2 \approx 1.4 \times 10^{-8}$ represent tests at 1 gravity and their crater profiles are similar to the ones labelled $g^* = 1$ in Fig. 10. The apparent crater diameters for these tests are about 80% of the width of the sand box and, hence, it is expected that the container could restrict the crater volume. The tendency for the tests with the large charge at low "gravity" to be lower than the star points in Fig. 11 are due to this boundary effect.

The fact that all tests at the higher π_2 values plot on this same line indicates that boundary effects are not significant when the crater diameter is less than about 50% of the box size. This supports the assertion by Schmidt (1980) that crater dimensions for impact craters were not affected if the crater diameter is less than about 60% of the

container diameter. The solid line in Fig. 11 is only based on data for which the crater diameter is less than half of the container size.

CONCLUSIONS

The cratering efficiency measured in the present study is 15 to 25% greater than that obtained by Schmidt et al. (1986). This difference can be explained by the difference in density and type of soil. Taking these differences into account, the results are in reasonable agreement with those obtained by other researchers (Schmidt 1980, Schmidt et al. 1980, 1986) in other laboratories. This indicates that the charges and test procedure used in this study produce repeatable, reasonable test data.

In addition, excellent agreement observed in modeling of models experiments ($g^* = 48.5$ and $g^* = 97$) indicate that the test procedure and scaling laws used are valid. Ideally, a larger range of scale factors should be used to verify the scaling laws, but to accomplish this, a larger centrifuge is required.

It appears that, at close range, the ground acceleration resulting from the shock wave caused by an explosion is not influenced by gravity-induced confining pressures to a significant degree. For structures located very near or actually in the crater, however, the shock wave is not the only potential source of damage.

Gravity-induced lithostatic pressures influence the size of the crater and the sizes of the plastic zones around the crater. If a soil in these zones distorts or flows it will impose drag forces as it moves around or into a buried structure. These forces can crush a flexible tunnel or displace any tunnel if it is not well anchored in soil outside the crater.

The above conclusions are based on experiments conducted with flexible tunnels buried in dry sand. The conclusions regarding the importance of gravity on tunnel damage is only proven for this situation.

Finally, most protective structures are usually made of relatively rigid reinforced concrete shells. The design of these shells will therefore be controlled in many cases by the shock wave pressures. There may be some benefit to considering the use of more flexible structures which can absorb the shock with very little damage, or even double walled structures with a thin metal shell to resist shock loading surrounding a more rigid, heavy concrete wall that can resist the lower magnitude but longer duration forces that occur after passage of a shock wave. If model tests are to be done to study such a scheme, gravity should be scaled.

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Appendix 2. - Notation

- a = charge radius
- b = distance from tunnel to crater
- d = depth of charge
- D = diameter of tunnel
- g = resultant of centrifugal acceleration and earth's gravity
- l = length
- L = length of tunnel
- Q_e = specific energy of explosive
- V = crater volume
- W = charge mass
- δ = density of charge
- π_1 = dimensionless cratering efficiency
- π_2 = dimensionless gravity scaled yield
- π_{dc} = dimensionless distance from tunnel to crater

superscripts

- * indicates the scale factor for the variable

TABLE 1

Test No(s)	Tunnel Type *	Centrifuge Acceleration g	Charge Size (mg)	H ** (cm)	r ** (cm)
TK3	BR(L)	48.5	512	20.3	5.0
TK4	BR(S)	97.0	64	10.2	2.5
RB3	BR(L)	48.5	512	20.3	5.0
RB4,5	BR(S)	97.0	64	10.2	2.5
RB6	BR(S)	48.5	64	20.3	2.5
RB7	BR(S)	48.5	64	20.3	0.0
RB8,9	BR(S)	48.5	64	20.3	5.0
RB10,12	BR(S)	48.5	64	20.3	1.25
RB11	BR(S)	48.5	64	20.3	2.5
RB13	BR(S)	19.4	64	20.3	2.5
RB14	none	1.0	64	20.3	---
RB15	none	48.5	64	10.2	---
RB16	none	24.2	512	20.3	---
RB17	none	4.2	64	20.3	---
LB12,14	AL	48.5	512	20.3	---
LB13,15	AL	1.0	512	20.3	see
LB16	AL	19.4	512	20.3	figure
LB17	AL	5.9	512	20.3	two
LB18	none		512	20.3	---

*Tunnel Types

Code	Material	Wall Thickness (cm)	Diameter (cm)	Length (cm)
BR(L)	Brass	0.015	5.0	15.2
BR(S)	Brass	0.0075	2.5	7.6
AL	Aluminum	0.013	6.45	16.3

** See Figure 3 for definition of H and r

TABLE 2. TUNNEL DAMAGE AND POSITION WITH RESPECT TO CRATER**

Test	g*	1. Crater Diameter (m)	2. Crater Depth (m)	3. r (m)	4. b (m)	5. Tunnel Depth (m)	Tunnel Damage ($\Delta D/D$)
TK3	48.5	10.9	1.94	2.70	0.87	1.27	.087
TK4	97	11.7	2.22	3.02	1.27	1.59	.019
RB3	48.5	11.1	2.06	2.86	0.64	1.11	0.099
RB4	97	11.4	1.91	2.38	0.95	1.58	.092
RB5	97	11.7	2.06	3.01	0.48	0.079	.122

** Refer to Figure 7 for definition of measurements. All distances converted to prototype terms.

Figure Captions

Fig. 1. - Schematic of crater and zones of distortion surrounding crater.

Fig. 2. - Experimental arrangement for using aluminum cans as model tunnels. All dimensions are given in centimeters (Note: 1 in = 2.54 cm).

Fig. 3. - Experimental arrangement for modeling of models tests and other tests with brass tunnels. Refer to Table 1 for particular values of r , H and D .

Fig. 4. - Crater/tunnel damage for two tests

- a) 1 g test (LB13)
- b) 48.5 g test (LB14)

Fig. 5. - Tunnel 2 damage observed in a 1 g test and 48.5 g tests. (LB12, 13 and 14)

Fig. 6. - Tunnel damage versus resultant of centrifuge acceleration and earth's gravity.

- a) For tunnel 2, damage increases with acceleration.
- b) For tunnel 1, damage reduces as acceleration increases.

Fig. 7. - Crater/tunnel distances after testing. See Table 2 for numerical values.

Fig. 8. - Tunnel damage versus normalized distance from the tunnel to the crater.

Fig. 9. - Comparison of blast induced ground accelerations measured a) on the centrifuge; and b) at 1 g. (Dimensions are not scaled to prototype values).

Fig. 10. - Crater profiles for five tests. The longest dashed lines correspond to 1 g tests. The short dashed and dotted lines correspond to modeling of models tests, during other tests.

Fig. 11. - Cratering efficiency vs. gravity scaled yield.

Fig. 1

Fig. 2.

Fig. 3.

LB12
50 G
CAN 2

LB14
50 G
CAN 2

LB13
1 G
CAN 2

Fig. 6a

Fig. 7

Fig. 8

Fig. 9.

--- $l^* = \frac{1}{48.5}, \quad g^* = 1$
 --- $l^* = \frac{1}{97}, \quad g^* = 97$
 ..... $l^* = \frac{1}{48.5}, \quad g^* = 48.5$

Fig. 10

Fig. 11

APPENDIX C

Centrifuge Modeling of Transport Processes of
Pollutant Travel Through Soils

Report to Tyndall Air Force Base

Centrifuge Modeling of Transport Processes of
Pollutant Travel Through Soils

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CENTRIFUGE MODELING OF TRANSPORT PROCESSES OF POLLUTANT TRAVEL THROUGH SOILS

INTRODUCTION

Movement of miscible liquid pollutants into soil and water bearing strata from spills, waste disposal ponds, and landfills is governed by the processes of advection, dispersion, adsorption and degradation. The first three of these processes are strongly affected by the character of the soil, particularly the grain size distribution, aggregation of particles, mineral composition, soluble salt type and amount, and the pore size distribution and shape. Adsorption and degradation depend on properties of the polluting compound as well as the character of mineral surfaces and possibly on soil organic matter. It follows that typical heterogeneous soils, having strata of differing characteristics, provide very complex media for the transport of pollutants. Prediction or evaluation of the movement of pollutants in such soils requires consideration of all the processes described above for the complex soil system.

Analyses of problems of this nature are currently being carried out using mathematical simulation studies (6,7). Design charts based on mathematical models were also proposed (10). The accuracy of these studies depends, among other factors, on the accuracy of the input parameters. For example, many investigators have concluded that values of longitudinal and transverse dispersivities in field systems are significantly larger than those values obtained in laboratory experiments on homogeneous materials or on materials with simple heterogeneities. The permeability of soils, which is another important input parameter in mathematical models, is well known to depend on the void ratio which is a function of the stress level of the soil. Hence the determination of input parameters such as permeability and diffusion coefficients in the laboratory for use in mathematical models must be such that they represent the in situ stress conditions which are very difficult to simulate in the laboratory.

Predictive methods, e.g., mathematical models, are used to indicate the major changes and the possible magnitude of the changes during the transport process, but these have been difficult to verify, due largely to lack of adequate data. Simulation costs are often high for the more complex models such as two or three dimensional models since the horizon of interest may span decades of real time. Detailed representation of geological structure, also possible in such models, adds to the difficulty of simulation of the transport of pollutants.

Certain of these difficulties may potentially be overcome by the use of centrifuge modeling. In particular, it should be feasible by such models to simulate the in situ stress conditions. In addition it is also possible to accelerate the transport processes that govern the fate of pollutants as they move through soils and aquifers. Testing of large centrifugal models using a large capacity centrifuge will enable inclusion of a large number of heterogeneities in the model. All these should greatly enhance general predictive capability and provide reliable data to verify and improve the capabilities of mathematical models now being used.

PHYSICAL TRANSPORT PROCESSES

The common starting point in the development of differential equation to describe the transport of solutes in porous media is to consider the flux of solute into and out of a fixed elemental volume within the flow domain. A conservation of mass statement of this flow domain is given by

$$\left[\begin{array}{l} \text{net rate change} \\ \text{of mass of solute} \\ \text{within the element} \end{array} \right] = \left[\begin{array}{l} \text{flux of solute} \\ \text{into the element} \end{array} \right] - \left[\begin{array}{l} \text{flux of} \\ \text{solute out} \\ \text{of the element} \end{array} \right] - \left[\begin{array}{l} \text{loss or gain of} \\ \text{solute mass due} \\ \text{to reactions} \end{array} \right]$$

The transport processes that control the flux into and out of the elemental volume are advection, and hydrodynamic dispersion. Loss or gain of solute mass from the solution is governed by the processes adsorption and degradation.

Advection is the component of solute movement attributed to transport by the flowing water. The rate of transport is equal to the interstitial water velocity, $\bar{v}$, where $\bar{v} = v/n$, v is the average velocity and n the porosity. There is a tendency, however, for the solute to spread out from the path that it is expected to follow according to the advective hydraulics of the flow system. This spreading phenomenon is called hydrodynamic dispersion. It causes dilution of the solute. It occurs because of mechanical mixing during fluid advection and because of molecular diffusion due to the thermal kinetic energy of the solute particles.

Hydrodynamic dispersion is comprised of diffusion and mechanical dispersion. Mechanical dispersion is entirely caused by the motion of the fluid and is basically governed by three mechanisms (4). The first occurs in individual pore channels because the molecules travel at different velocities at different points across the channel due to the drag exerted on the fluid by the roughness of the pore surfaces. The second process is caused by the difference in pore sizes along the flow paths followed by the water molecules. Because of the differences in surface area and roughness relative to the volume of water in individual pore channels, different pore channels have different bulk fluid velocities. The third process is related to the tortuosity, and branching of pore channels. Diffusion is the process whereby ionic or molecular constituents move under the influence of their kinetic activity in the direction of their concentration gradient. Diffusion occurs even in the absence of any bulk hydraulic movement of the solution. Hence it could be seen that at low velocities the diffusion will be dominant and as a result the hydrodynamic dispersion will be independent of velocity. As the velocity increases the mechanical dispersion takes a dominant role and thus the hydrodynamic dispersion will become dependent on the velocity.

Adsorption - desorption is the exchange of solute ions between the solution and the soil particle surfaces. This phenomenon is prominent in soils that contain an appreciable percentage of colloidal sized particles, i.e., clay particles. Degradation is the decay of solute ions or molecules due to chemical or nuclear reaction, such as decay of radioactive pollutants.

OBJECTIVES

Objectives of this study are 1) to derive the scaling laws governing the centrifugal modeling of the processes advection, hydrodynamic dispersion and adsorption, 2) to examine the validity of these scaling laws by performing "modeling of model" tests, 3) to examine the influence of gravity (i.e. self weight) on the transport processes, 4) to examine the influence of velocity on the hydrodynamic dispersion.

SCALING LAWS

Factors related to the processes advection, hydrodynamic dispersion and adsorption are as follows:

- a) Rate of change of concentration of the solute, $\partial C/\partial t$ or C/t for dimensional analysis ($ML^{-3}T^{-1}$)
- b) Concentration of the solute, C (ML^{-3})
- c) Interstitial velocity, $\bar{v}$ (LT^{-1})
- d) Coefficient of hydrodynamic dispersion, D (L^2T^{-1})
- e) Length along the flow, L (L)
- f) Mass of the solute adsorbed on a unit mass of the solid part of the porous medium, S (M of solute/ M of solids)
- g) Mass of the solid part of the porous medium associated with a representative volume of the porous medium, (M)

Considering C/t , C , $\bar{v}$, D , L and ($S.M$) as the parameters and using dimensional analysis, the following non-dimensional groups can be derived:

$$i) \frac{C \bar{v}}{(C/t)L}, \quad ii) \frac{D(C/t)}{C \bar{v}^2} \quad \text{and} \quad iii) (SM)(C/t)^3 (1/C)^4 (1/v)^3$$

Scaling of $\bar{v}$

$$\bar{v} = v/n \quad (1)$$

where v is the average velocity and n is the porosity,

$$v = \frac{k}{\gamma_f} \frac{\partial p}{\partial l} \quad (\text{Darcy's Law}) \quad (2)$$

where k is the coefficient of permeability, γ_f is the unit weight of the fluid and $\partial p / \partial l$ is the pressure gradient,

$$\frac{k}{\gamma_f} = \frac{K}{\mu} \quad (3)$$

where K is the absolute permeability and μ is the coefficient of dynamic viscosity of the pore fluid. K/μ is a function of soil type, porosity and the pore fluid if these variables are the same in both the model and the prototype then K/μ and hence k/γ_f remains the same in both the model and the prototype, i.e.,

$$\left(\frac{k}{\gamma_f}\right)_m = \left(\frac{k}{\gamma_f}\right)_p \quad (4)$$

where subscripts m and p stand for model and the prototype, respectively.

If the excess hydraulic pressure applied to the model and the prototype are the same and the model length is scaled N times smaller than the prototype then

$$\left(\frac{\partial p}{\partial l}\right)_m = N \left(\frac{\partial p}{\partial l}\right)_p \quad (5)$$

where Ng is the centrifugal acceleration.

Therefore,

$$v_m = N v_p \quad (6)$$

and

$$\bar{v}_m = N \bar{v}_p \quad (7)$$

Group (i) $\left[\frac{C \bar{v}}{(C/t)L} \right]$

If the concentration of the solute is the same in the model and the prototype then,

$$C_m = C_p \quad (8)$$

Also,

$$L_m = L_p / N \quad (9)$$

But,

$$\frac{C_p \bar{v}_p}{(C_p/t_p)L_p} = \frac{C_m \bar{v}_m}{(C_m/t_m)L_m} = \frac{C_p N \bar{v}_p}{(C_p/t_m)(L_p/N)} \quad (10)$$

Hence

$$t_p = N^2 t_m \quad (11)$$

Group (ii) $\left[\frac{D (C/t)}{C \bar{v}^2} \right]$

If the Peclet number ($\bar{v}d/D^*$, where d is the average particle diameter and D^* is the diffusion coefficient) is less than one then the dispersion coefficient (D) is independent of $\bar{v}$ (9). Generally for soils containing small particle sizes, such as clay size or for low flow rates (i.e., for small $\bar{v}$), Peclet number is less than one. If D is independent of velocity then,

$$D_m = D_p \quad (12)$$

also,

$$\frac{D_p (C_p/t_p)}{C_p \bar{v}_p^2} = \frac{D_m (C_m/t_m)}{C_m v_m^2} = \frac{D_p (C_p/t_m)}{C_p (N\bar{v}_p)^2} \quad (13)$$

Hence

$$t_p = N^2 t_m \quad (14)$$

Group (iii) [(SM) (C/t)³ (1/C⁴) (1/ $\bar{v}$ ³)]

If the soil type and the pollutant type and concentration are the same in the model and the prototype and equilibrium adsorption is reached in both the model and the prototype then,

$$S_m = S_p \quad (15)$$

If the porosity is the same in the model and the prototype then

$$\frac{M_m}{V_m} = \frac{M_p}{V_p} \quad (16)$$

where V_m and V_p are the corresponding representative volumes in the model and the prototype respectively.

But,

$$V_m = V_p N^3 \quad (17)$$

Therefore

$$M_m = M_p N^3 \quad (18)$$

But,

$$\begin{aligned} S_p M_p (C_p/t_p)^3 (1/C_p)^4 (1/\bar{v}_p)^3 &= S_m M_m (C_m/t_m)^3 (1/C_m)^4 (1/\bar{v}_m)^3 \\ &= S_p (M_p N^3) (C_p/t_m)^3 (1/C_p)^4 (1/N\bar{v}_p)^3 \end{aligned} \quad (19)$$

i.e.,

$$t_p^3 = N^6 t_m^3 \quad (20)$$

Hence

$$t_p = N^2 t_m \quad (21)$$

Therefore it can be concluded that if the coefficient of hydrodynamic dispersion is independent of the velocity then the prototype time taken for the transport of pollutants is N^2 times that of the centrifugal model because it was shown that in all three nondimensional groups $t_p = N^2 t_m$.

PHYSICAL ASPECTS OF MODELING

In addition to the similarity conditions established above for the transport processes, it is necessary to maintain the validity of Darcy's Law. This can be maintained if the Reynolds number $R_e = \frac{\bar{v}d}{\nu} \leq 1$

where

- $\bar{v}$ = interstitial velocity
- d = diameter of the soil grains
- ν = coefficient of kinematic viscosity.

For the condition where $R_e = 1$, $\nu/d = \bar{v}_{\max}$, and assuming a typical ν for water as $10^{-6} \text{ m}^2/\text{s}$, $\bar{v}_{\max} = \frac{10}{d} \text{ m/s}$ if d is in meters.

$$\bar{v} = kI/n$$

where

- k = coefficient of permeability (L/T)
- I = hydraulic gradient
- n = porosity

Therefore,

$$\bar{v}_{\text{model}} = \frac{kNI}{n}$$

For

$$\bar{v}_{\text{max}} = \bar{v}_{\text{model}}$$

$$N = \frac{10^{-6} n}{kdI}$$

The maximum permissible scale factor for a clay with $k = 10^{-9}$ m/s, $n = 0.5$, $I = 500$, and $d = 10^{-6}$ m is given by

$$N = \frac{(10^{-6})(0.5)}{(10^{-9})(10^{-6})(500)} = 1 \times 10^6$$

Since the scale factors generally used are much less than this value the requirement of $R_e \leq 1$ can be easily satisfied for clay samples. However, tests on soils containing larger particle sizes might violate this condition.

TESTING PROGRAM AND RESULTS

Tests were performed on five different types of soils. They were a medium sand, Monterey '0/30' sand, silica flour, 10% Monterey '0/30' + 90% Snowcal 50 and Snow Cal 50. Sand and silica flour samples were prepared in a 5 inch diameter plastic cylinder by pluviating and vibrating the soil into a 0.08N salt water solution. Fresh water was then allowed to seep through the models under an excess pressure head in the centrifuge.

Clay samples were prepared by making a slurry with a 0.06N lithium chloride solution and consolidating the slurry in a 6 inch plastic cylinder in the centrifuge at the corresponding g-levels. Clay layers were then leached with Ethyl alcohol by passing the alcohol through the clay layers under an excess pressure head in the centrifuge. Then a 0.08N sodium hexa-metaphosphate solution

was allowed to seep through the models under an excess pressure head in the centrifuge. Sodium ions will replace the lithium ions from the clay particle surfaces and will create an adsorption desorption process. Leaching with alcohol consists only of advection and hydrodynamic dispersion.

The change in concentration of the pore fluid at the mid depth of each model was monitored as a function of time utilizing a conductivity measuring system as shown in Fig. 1. The conductivity cell used in most of the tests is also shown in Fig. 1. The conductivity cell used in the testing of medium sand samples consisted of a porous filter in addition to the electrodes (1). This was removed in the later tests to avoid the lag in detecting the concentration changes (1). Pore pressure transducers were also used to monitor the excess pressure heads applied to the samples. Output from the conductivity cell and the pore pressure transducers were recorded on x-y plotters.

i) Modeling of model tests:

- a) Tests on a medium sand: Modeling of model tests were performed on a uniform medium sand with a mean grain diameter of 0.42 mm. Details of this tests can be found elsewhere (1).
- b) Tests on silica flour: Four silica-flour (mean grain diameter 0.125 mm) models varying in heights of 8.25, 6.19, 5.5 and 4.125 inches were tested in 25, 33.3, 37.5 and 50 g centrifugal accelerations respectively. Samples were prepared at a porosity of 0.344. The coefficient of permeability, under 1g conditions, corresponding to this void ratio is 3.05×10^{-3} cm/s. The prototype represented by these four models is given in Fig. 2. The ratio of concentration to initial concentration (C/C_0) at mid depth measured as a function of model time utilizing the conductivity cell for all these four models are given in Fig. 3. A plot of $\log_{10} t_m$ versus $\log_{10} N$ is given in Fig. 4, where t_m is the time taken for 50% reduction in concentration at

mid depth and N is the centrifugal acceleration. This result can be interpreted as $t_p = N^2 t_m$, where t_p is the prototype time. The relationship between C/C_0 at mid depth and the prototype time for these four models are given in Fig. 5. Prototype time was obtained using the relationship $t_p = N^2 t_m$. Since all four models gave the same prototype behavior it could be concluded that the scaling laws derived are valid. Hydrodynamic dispersion coefficients (D) for these tests will be examined subsequently.

- c) Tests on clays: Two centrifuge tests were carried out on 100% Snowcal 50 models of height 4 and 2 inches subjected to centrifugal accelerations of 50 g and 100 g respectively. Slurry was prepared as described before and samples were consolidated in the centrifuge in two layers with the conductivity cell and the pore pressure transducer placed at the center of the samples. Samples were subjected to an excess pressure head of 400 inches of water. Conductivity variations at mid depth with time were measured during seeping of sodium hexa-metaphosphate for both the samples and during leaching for the 100 g sample. The relationship between the ratio of concentration, C , to final concentration, C_f (C/C_f) and the prototype time for these tests are given in Fig. 6. Prototype times were obtained as discussed before. The curve denoted by "NO ADSORPTION" is the curve deduced from the measured variation of C/C_0 with time during leaching with alcohol. As seen from Fig. 6 adsorption retards the movement of the solute. Curves for 50g and 100g samples, which represents the same prototype behavior, seems to agree reasonably well. The maximum variation between these curves is about 70 days prototype time. This corresponds to 0.672 hours for the 50g test. This is small compared to the 12 hours (1250 days prototype time) duration of the test. Hence, it can be concluded that the scaling laws derived are valid. The hydrodynamic dispersion coefficient for these tests also will be examined subsequently.

(iii) Influence of self weight on the transport processes:

The conventional approach for the determination of breakthrough curves is to perform a longitudinal dispersion test of a tracer passing through a column of porous media. The breakthrough curve is used to calculate dispersion coefficient (D) and the average interstitial velocity ($\bar{v}$). The calculated values of D and $\bar{v}$ are then used to predict the breakthrough curve for any depth in the prototype. But in a lg test, prototype stress levels, i.e., stresses due to the prototype self weight cannot be simulated as in a centrifuge model.

To investigate the influence of selfweight on the breakthrough curves two sets of test were performed. One using silica flour samples and the other one using 100% Snowcal 50 samples. A lg test and a 37.5g test on silica flour and a lg and a 100g test on Snowcal 50 were compared. The height of the samples, excess pressure heads and the locations of the conductivity cell were identical in the lg and centrifuge models, i.e., 5.5 inch height and 17.5 inch water head for silica flour samples and 2 inch height and 400 inch water head for clay samples with conductivity cell at mid depth in all the cases. The lg clay sample was consolidated to the stress experienced by the mid depth of the centrifuge model and then allowed to swell back before testing. The results for the silica flour samples and the clay samples are given in figs. 7 and 8 respectively. It can be seen from these figures that the breakthrough curves are different with the time for 50% reduction in concentration for lg tests being less than that for the centrifuge tests. Also it can be seen that this effect is prominent in clays. This result is expected since the void ratio of clays and hence the permeability is more sensitive to the stress level. As the stress level increases the void ratio decreases. As the void ratio decreases the permeability also decreases, causing a shift in the breakthrough curves to the right. Part of the dif-

ference in the behavior of clay samples could be attributed to the void ratio increase during swelling of the lg sample, although this increase was small. Even if a uniform stress is applied to a lg clay sample during testing it will behave differently than a prototype layer or a centrifuge model which have linear variation of stresses with depth. Hence it is reasonable to conclude that a centrifuge model, behavior is more realistic than that of a lg model.

(iv) Influence of velocity on the hydrodynamic dispersion

To study the influence of velocity on the hydrodynamic dispersion two sets of tests were performed on Monterey '0/30' sand (mean grain diameter 0.45 mm). One set was performed on the centrifuge. In this set a sample of height 8.18 inches with the conductivity cell at the mid-depth was tested at 25g acceleration with varying excess pressure heads, thereby with varying average interstitial velocities ($\bar{v}$). Average interstitial velocities were calculated by dividing the depth of the conductivity cell by the time taken for 50% reduction in concentration. The relationship between C/C_0 and $\bar{v} \cdot t$ for these tests are shown in Fig. 9. $\bar{v} \cdot t$ was chosen as a parameter instead of the absolute time (t) in order to bring the 0.5 C/C_0 of all these tests to a point and to facilitate the comparison of the hydrodynamic dispersion. It can be seen from fig. 9 that as the velocity increases the spreading of the breakthrough curves also increases indicating an increase in the hydrodynamic dispersion with an increase in the velocity. However the differences are not very significant due to the small range of average velocities used in these tests.

Another set of tests were performed in lg acceleration. In this set 3 samples of heights 4, 8 and 16 inches with the conductivity cell at mid depths were subjected to average interstitial velocities of 0.014, 0.008 and 0.005 in/s respectively. The relationships between C/C_0 and $\bar{v}t/l$ for these

tests are given in fig. 10, where l is the depth of the conductivity cell. Here also $\bar{v} t/l$ was used as a parameter instead of t in order to bring the $0.5 C/C_0$ of all the tests to a point. It can also be seen from fig. 10 that as the velocity increases hydrodynamic dispersion also increases. The spreading of dispersion curves with increase in velocity is more prominent in lg tests than the centrifuge tests due to the bigger range of velocities used.

The hydrodynamic dispersion coefficients for all the tests were calculated using the following equation (5).

$$D = \frac{1}{4\pi} \frac{L \bar{v}}{\left[\frac{\partial(C/C_0)}{\partial(V/V_0)} \right]_{V/V_0 = 1}}^2$$

where

L = distance traveled by the tracer with a velocity of $\bar{v}$ in time t_{50} (time taken for 50% reduction in concentration).

and V/V_0 = pore volume. If there is no dispersion of any sort, the plot of C/C_0 versus V/V_0 is a step change from $C/C_0 = 0$ to $C/C_0 = 1$ at $V/V_0 = 1$ or vice-versa. If there is dispersion, $0.5C/C_0$ of the breakthrough curve will correspond to $V/V_0 = 1$.

$\left[\partial(C/C_0) / \partial(V/V_0) \right]_{V/V_0 = 1} = \text{slope of the breakthrough curve at } V/V_0 = 1 \text{ or } C/C_0 = 0.5.$

Although this equation is only valid for an ideal medium (5) it will give a fairly good estimate of the hydrodynamic dispersion coefficient. Calculated hydrodynamic dispersion coefficient for all the tests are given in Table 1. Also given in table 1 are the ratio of hydrodynamic dispersion coefficient to diffusion coefficient (D/D^*) and the Peclet number ($\bar{v}d/D^*$). The results of an additional test performed on a mixture of Monterey '0/30' sand and Snowcal 50 is given in Table 1. D^* of $1.5 \times 10^{-5} \text{ cm}^2/\text{s}$ was used for sand and silica flour and

D^* of 8.0×10^{-7} and 3.0×10^{-7} cm^2/s were used for sand/clay mixture and clay respectively. These values were based on the typical values for chemical species in geologic media (4). A plot of D/D^* versus Peclet number obtained from the present study is given in fig. 11 and it is compared with a similar plot obtained by a previous study (9) using only lg samples. It can be seen from fig. 11 that D/D^* is independent of velocity when the Peclet number is less than one, i.e., in the region where molecular diffusion dominates mechanical dispersion. Clay samples and a sand/clay mixture are shown to fall in this region and some fine silts can also be expected to fall in this region. Hence for the soils falling in this region the scaling laws derived previously are valid.

SUMMARY AND CONCLUSIONS

Samples prepared by the same method with same height and same excess pressure head tested in lg and in the centrifuge gave different breakthrough curves suggesting the importance of self weight, i.e., stresses due to gravity on the transport processes of pollutant travel through soils. This effect of self weight was shown to be more prominent in clays. Hence it can be concluded that centrifuge models, which represent the prototype stress distribution more closely, will yield more realistic data on pollutant migration through soils which in turn can be used to verify mathematical models. Adsorption in clay samples were successfully modeled in addition to advection and dispersion and it is shown to retard the transport processes. Centrifuge testing is also shown to accelerate the transport processes. Acceleration of transport processes can be achieved even in a one dimensional lg model by increasing the excess hydraulic pressure using air pressure. However, lg models cannot simulate prototype stress distribution or they cannot be used to simulate two or three dimensional flows where other gravity dominated effects such as a sloping water table, a phreatic surface, a boundary between saturated and unsaturated soils exists.

Samples tested with different average interstitial velocities ($\bar{v}$) showed that hydrodynamic dispersion increases as the velocity increases. Carbonell and Whitaker (3) also showed this phenomena theoretically using intrinsic phase average of the quantities involved. The relationship between the ratio of hydrodynamic dispersion coefficient to diffusion coefficient (D/D^*) and the Peclet number ($\bar{v}d/D^*$) obtained from the current study on centrifuge and lg samples was compared to that of a previous study on lg samples (9). A clay/sand mixture and clay samples tested in the current study were shown to fall in the region where D/D^* is independent of the Peclet number, i.e., velocity, obtained by the previous study (9). Scaling laws were derived assuming that the prototype and the model will have the same hydrodynamic dispersion coefficient (D), thereby assuming D is independent of velocity. Hence the scaling laws derived are only valid in the region where D/D^* is independent of the velocity, i.e., when Peclet number is less than one. However modeling of model tests performed on silica flour samples showed that scaling laws derived are valid (even with Peclet numbers greater than one), i.e., when all the model times were scaled by N^2 (obtained from the scaling laws) same prototype breakthrough curve was obtained. This might have been due to the fact that the range of Peclet numbers used in these tests was small (see fig. 11).

Validity of the scaling laws is necessary in order to model a particular prototype. Therefore only the prototypes where flow occurs at low Peclet numbers can be directly modeled in the centrifuge. This is a common situation in relatively impermeable soils.

Even if the validity of scaling laws exists it is impossible to simulate all the heterogeneities and the structure (such as aggregation, pore shape and pore size distribution) of the soil of a prototype. If one wants to model a prototype it is only feasible to do numerous tests with the heterogeneities and the structure of the prototype soil simulated as closely as possible and then to

adapt a statistical approach to analyze the data. In simulating large number of heterogeneities a large model and hence a large capacity centrifuge will be useful.

It should be noted that problems with precise simulation of prototype heterogeneities and soil structure are not much simpler for mathematical models, furthermore these models are not extensively verified.

In light of these difficulties the centrifuge can be used to produce data under repeatable laboratory controlled conditions where the physical processes of advection, dispersion and adsorption occur in heterogeneous two or three dimensional situations. These data can then be used to verify mathematical models. Once verified, the mathematical models could be applied to complex field situations with more confidence if site characterization to obtain all the heterogeneities and the structure of the soil can be made.

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Table 1

Soil	g-level/ Sample Ht(in)	Hydrodynamic Dispersion Coeff. (D) (cm ² /s)	Interstitial Velocity($\bar{v}$) (cm/sec)	Mean Partical diameter (d)(cm)	D/D*	$\frac{\bar{v}d}{D^*}$
Monterey '0/30'	1g/4	5.0×10^{-3}	0.036	0.0450	3.33×10^2	1.08×10^2
	1g/8	4.3×10^{-4}	0.020	0.0450	0.29×10^2	0.60×10^2
	1g/16	3.0×10^{-4}	0.013	0.0450	0.20×10^2	0.39×10^2
Snowcal 50	50g/4	3.7×10^{-7}	3.0×10^{-4}	2×10^{-4}	1.2	6.0×10^{-2}
	100g/2	2.2×10^{-7}	1.5×10^{-4}	2×10^{-4}	0.7	3.0×10^{-2}
Medium Sand (1)	53g/3.88	1.7×10^{-3}	0.086	0.0420	1.13×10^2	2.41×10^2
	40g/5.11	1.0×10^{-3}	0.064	0.0420	0.67×10^2	1.79×10^2
	33.3g/6.18	0.8×10^{-3}	0.054	0.0420	0.53×10^2	1.51×10^2
	25g/8.23	0.5×10^{-3}	0.041	0.0420	0.33×10^2	1.15×10^2
Silica Flour	50g/4.125	1.6×10^{-4}	0.031	0.0125	1.1×10^1	2.6×10^1
	37.5g/5.5	1.6×10^{-4}	0.023	0.0125	1.1×10^1	1.9×10^1
	33.3g/6.19	1.6×10^{-4}	0.016	0.0125	1.1×10^1	1.3×10^1
10% Monterey '0/30' + 90% Snowcal 50	100g/2	8×10^{-7}	2.51×10^{-4}	0.0468	1.0	0.9×10^{-1}

Fig. 1. Circuit diagram and the geometry of the conductivity cell for the conductivity measuring system.

Fig. 2. The prototype represented by the four silica flour models.

Fig. 3. The ratio of concentration to initial concentration (C/C_0) at middepth as a function of model time for silica flour models.

Fig. 4. Logarithmic model time for 50% reduction in concentration ratio at middepth vs. logarithmic q-level for silica flour models.

Fig. 5. The ratio of concentration to initial concentration (C/C_0) at middepth as a function of prototype time for silica flour models.

Fig. 6. The ratio of concentration to final concentration (C/C_f) at middenth as a function of prototype time for Snowcal 50 models.

Fig. 7. The ratio of concentration to initial concentration (C/C_0) at middepth as a function of model time for a silica flour sample with same excess pressure head tested at 1g and 37.5g.

Fig. 8. The ratio of concentration to final concentration (C/C_f) at middepth as a function of model time for Snowcal 50 samples with same excess pressure head tested at 1g and 100g.

Fig. 9. The ratio of concentration to initial concentration (C/C_0) at middepth as a function of $\bar{v}t$ (where $\bar{v}$ is the average interstitial velocity and t is the model time) for a Monterey '0/30' sand model tested at 25g with varying average interstitial velocities.

Fig. 10. The ratio of concentration to initial concentration (C/C_0) at middepth as a function of $\bar{v}t/l$ (where $\bar{v}$ is the average interstitial velocity, t is the model time and l is the middepth) for Monterey '0/30' sand models tested at 1g with varying sample thicknesses and with varying average interstitial velocities.

Fig. 11. Comparison of the relationship between D/D^* and the Peclet number obtained using Ig and centrifuge model tests.

APPENDIX D

Revised Edition of Appendix C

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Revised

CENTRIFUGE MODELING OF TRANSPORT PROCESSES FOR POLLUTANTS IN SOILS

By K. Arulanandan¹, M.ASCE, P.Y. Thompson², F.ASCE, B.L. Kutter³, AM.ASCE,
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Key Words

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Diffusion, Dispersion, Groundwater, Seepage, Water Pollution.

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CENTRIFUGE MODELING OF TRANSPORT PROCESSES FOR POLLUTANTS IN SOILS

By K. Arulanandan¹, M.ASCE, P.Y. Thompson², F.ASCE, B.L. Kutter³, AM.ASCE, N.J. Meegoda⁴, AM.ASCE, K.K. Muraleetharan⁵, SM.ASCE, and C. Yogachandran⁵

ABSTRACT

The physico-chemical processes of advection, dispersion, adsorption and degradation which control pollutant transport processes in ground water are described. Dimensional analysis is presented, and the scaling requirements for centrifuge modeling of these processes is developed. The validity of these scaling laws is examined by conducting "modeling of models" tests using several types of soils in idealized models of one-dimensional flow situations. The importance of scaling gravity (by using a centrifuge) is highlighted. The centrifuge permits simulation of prototype stress levels and hence makes it possible to obtain identical soil properties such as permeability in a small scale model as compared to a full scale soil deposit. Simulation of capillary effects and a phreatic surface is also possible in a centrifuge model, but not in a 1-g scale model. It is concluded that the scaling laws are valid for adsorption and advection in the reported model tests, but in coarse grained soils where the Peclet number is often greater than 1, the dispersion process cannot be directly scaled from model to prototype. Even

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if direct scaling is not possible, the centrifuge is a useful tool for providing data to test or verify numerical models, since full scale test data is not abundant and the full scale boundary and site conditions are poorly defined.

INTRODUCTION

Movement of miscible liquid pollutants into soil and water bearing strata from spills, waste disposal ponds, and landfills is governed by the processes of advection, dispersion, adsorption and degradation among others. The first three of these processes are strongly affected by the character of the soil, particularly the grain size distribution, aggregation of particles, mineral composition, soluble salt type and amount, and the pore size distribution and shape. Adsorption and degradation depend on properties of the polluting compound as well as the character of mineral surfaces and possibly on soil organic matter. It follows that typical heterogeneous soils, having strata of differing characteristics, provide very complex media for the transport of pollutants. Prediction or evaluation of the movement of pollutants in such soils requires consideration of all the processes described above for the complex soil system.

Analyses of problems of this nature are currently being carried out using mathematical simulation studies (Javandel, et al. 1984 and Orlob 1984). Design charts based on mathematical models were also proposed (Rowe and Booker 1985). The accuracy of these studies depends, among other factors, on the accuracy of the input parameters. For example, many investigators have concluded that values of longitudinal and transverse dispersivities in field systems are significantly larger than those values obtained in laboratory experiments on homogeneous materials

or on materials with simple heterogeneities. The permeability of soils, which is another important input parameter in mathematical models, is well known to depend on the void ratio which is a function of the stress level of the soil. Hence the determination of input parameters such as permeability and diffusion coefficients in the laboratory for use in mathematical models must be such that they represent the in situ stress conditions which are very difficult to simulate in the laboratory.

Predictive methods, e.g., mathematical models, are used to indicate the major changes and the possible magnitude of the changes during the transport process, but these have been difficult to verify, due largely to lack of adequate data. Simulation costs are often high for the more complex models such as two or three dimensional models since the horizon of interest may span decades of real time. Detailed representation of geological structure, also possible in such models, adds to the difficulty of simulation of the transport of pollutants.

Certain of these difficulties may potentially be overcome by the use of centrifuge modeling. In particular, it should be feasible by such models to simulate the in situ stress conditions. In addition it is also possible to accelerate the transport processes that govern the fate of pollutants as they move through soils and aquifers. Testing of large centrifugal models using a large capacity centrifuge will enable inclusion of a large number of heterogeneities in the model. All these should greatly enhance general predictive capability and provide reliable data to verify and improve the capabilities of mathematical models now being used.

The common starting point in the development of differential equation to describe the transport of solutes in porous media is to consider the flux of solute into and out of a fixed elemental volume within the

flow domain. A conservation of mass statement of this flow domain is given by

$$\left[\begin{array}{l} \text{net rate} \\ \text{change of mass} \\ \text{of solute} \\ \text{within the} \\ \text{element} \end{array} \right] = \left[\begin{array}{l} \text{flux of solute} \\ \text{into the element} \end{array} \right] - \left[\begin{array}{l} \text{flux of solute} \\ \text{out of the} \\ \text{element} \end{array} \right] - \left[\begin{array}{l} \text{loss or gain} \\ \text{of solute} \\ \text{mass due to} \\ \text{reactions} \end{array} \right]$$

The transport processes that control the flux into and out of the elemental volume are advection, and hydrodynamic dispersion. Loss or gain of solute mass from the solution is governed by the processes adsorption and degradation.

Advection is the component of solute movement attributed to transport by the flowing water. The rate of transport is equal to the interstitial flow velocity, $\bar{v}$, where $\bar{v} = v/n$, v is the average "Darcy" velocity and n the porosity. There is a tendency, however, for the solute to spread out from the path that it is expected to follow according to the advective hydraulics of the flow system. This spreading phenomenon, called hydrodynamic dispersion, causes dilution of the solute.

Hydrodynamic dispersion is comprised of diffusion and mechanical dispersion. Mechanical dispersion is entirely caused by the motion of the fluid and is basically governed by three mechanisms (Freeze and Cherry 1979). The first occurs in individual pore channels because the molecules travel at different velocities at different points across the channel due to the drag exerted on the fluid by the roughness of the pore surfaces. The second process is caused by the difference in pore sizes along the flow paths followed by the water molecules. Because of the differences in surface area and roughness relative to the volume of water in individual pore channels, different pore channels have different bulk fluid velocities. The third process is related to the

tortuosity, and branching of pore channels. Diffusion is the process whereby ionic or molecular constituents move under the influence of their kinetic activity in the direction of their concentration gradient. Diffusion occurs even in the absence of any bulk hydraulic movement of the solution. Hence it could be seen that at low velocities the diffusion will be dominant and as a result the hydrodynamic dispersion will be independent of velocity. As the velocity increases the mechanical dispersion takes a dominant role and thus the hydrodynamic dispersion will become dependent on the velocity.

Adsorption - desorption is the exchange of solute ions between the solution and the soil particle surfaces. This phenomenon is prominent in soils that contain an appreciable percentage of colloidal sized particles, i.e., clay particles. Degradation is the decay of solute ions or molecules due to chemical or nuclear reaction, such as decay of radioactive pollutants.

Objectives of this study are 1) to derive the scaling laws governing the centrifugal modeling of the processes advection, hydrodynamic dispersion and adsorption, 2) to examine the validity of these scaling laws by performing "modeling of model" tests, 3) to examine the influence of gravity (i.e. self weight) on the transport processes, 4) to examine the influence of velocity on the hydrodynamic dispersion.

CENTRIFUGE SCALING LAWS

First consider that for the sake of simplicity and accuracy it is desired to construct a model with the same material properties as those in the prototype (for example permeability, strength, density and moduli).

The properties of soils are highly dependent on confining pressures. For example the void ratio, e , of soil is a function of confining stress

and the Kozeny-Carmen equation suggests that permeability is proportional to $e^3/(1+e)$. And, the Mohr-Coulomb failure criteria suggests that shear strength is proportional to the effective normal stress on a shear plane.

To account for the stress dependence of material properties stresses must be similar in model and prototype.

Therefore, the condition of identical stress is imposed on the model:

$$\frac{\sigma_{\text{model}}}{\sigma_{\text{prototype}}} = \sigma^* = 1 \quad (1)$$

(the asterisk on a quantity indicates the scale factor for that quantity)
Since stress has units of force/Area, the scale factor for length and force must obey the equation

$$\frac{F^*}{(\ell^*)^2} = \sigma^* = 1 \quad (2)$$

Let the scale factor for length be

$$\ell^* = \frac{\ell_{\text{model}}}{\ell_{\text{prototype}}} = 1/N \quad (3)$$

and it follows that

$$F^* = (\ell^*)^2 = N^{-2} \quad (4)$$

Now, since the model and prototype are composed of identical material, the scale factor for density is

$$\rho^* = 1 \quad (5)$$

The scale factor for mass is calculated by

$$\rho^* = \frac{m^*}{(l^*)^3} = 1 \quad (6)$$

$$m^* = (l^*)^3 = N^{-3} \quad (7)$$

and the scaling factor for gravity follows from Newton's law of motion for a body at rest in a gravitational field

$$F^* = m^* g^* \quad (8)$$

or

$$g^* = \frac{F^*}{m^*} = \frac{N^{-2}}{N^{-3}} = N \quad (9)$$

Therefore, if identical material is used in model and prototype, and length dimensions are reduced N times, the gravity should be increased N times in order to preserve identical stresses in model and prototype. The centrifuge is used to provide an increased acceleration field to simulate this required increase in gravitational force on the model. Thus the strong stress dependence of soil properties is accounted for in centrifuge model tests.

Scaling of Interstitial Flow Velocity $\bar{v}$

$$\bar{v} = v/n \quad (10)$$

Where v is the average "Darcy" velocity and n is the porosity.

$$v = \frac{k}{\gamma_f} \left(\frac{\partial p}{\partial l} + F \right) \quad (\text{Generalized Darcy's Law}) \quad (11)$$

Where k is the coefficient of permeability, γ_f is the unit weight of the fluid, $\partial p / \partial l$ is the pressure gradient, F is the body force per unit vol-

ume on the pore fluid in the direction of flow, and

$$k/\gamma_f = \frac{K}{\mu} \quad (12)$$

where K is the absolute permeability and μ is the coefficient of dynamic viscosity of the pore fluid. K/μ is a function of soil type, porosity and the pore fluid. Assuming these variables are the same in model and prototype and assuming body force on pore fluid is only due to gravitational field, the scale factor $\bar{v}^*$ is derived as follows

$$F = \gamma_f \quad (13)$$

$$\gamma_f^* = N \quad (14)$$

$$\left(\frac{K}{\mu}\right)^* = (k/\gamma_f)^* = 1 \quad (15)$$

$$p^* = \sigma^* = 1 \quad (16)$$

$$l^* = 1/N \quad (17)$$

$$v^* = (k/\gamma_f)^* \cdot \frac{p^*}{l^*} = (k/\gamma_f)^* \cdot \gamma_f^* \quad (18)$$

$$\bar{v}^* = N. \quad (19)$$

Scaling Laws for Solute Transport

The concentration of a solute in the pore water is a function of many other variables in addition to the properties of the soil particle matrix. A comprehensive list of these variables for a problem where advection, dispersion and adsorption occur is

$$C = f(\mu, D_m, S, \bar{v}, T_f, \rho_f, g, l, l_p, t, \text{soil properties}) \quad (20)$$

where

$$C = \text{concentration of solute in pore water} \quad (M/L^3)$$

$$\mu = \text{dynamic fluid viscosity} \quad (M/(LT))$$

D_m = coefficient of molecular diffusion	(L^2/T)
S = mass of adsorbed solute/unit volume	(M/L^3)
$\bar{v}$ = interstitial flow velocity	(L/T)
T_f = surface tension for fluid particle interface	(M/T^2)
ρ_f = density of fluid	(M/L^3)
g = acceleration due to gravity	(L/T^2)
l = characteristic macroscopic length (e.g. sample height)	(L)
l_p = characteristic microscopic length (e.g. particle size)	(L)
t = time	(T)

Dimensional Analysis

Since the soil bulk material properties will be identical in model and prototype the following dimensional analysis will not include these parameters. From the total of 10 remaining independent variables and one dependent variable, C , a total of $11-3 = 8$ linearly independent dimensionless groups can be derived. One possible set of dimensionless groups is listed below.

$$\begin{aligned}
 \pi_1 &= C/\rho_f && \text{(concentration number)} \\
 \pi_2 &= \frac{\rho_f \bar{v} l_p}{\mu} = Re && \text{(Reynold's Number)} \\
 \pi_3 &= \frac{\bar{v} t}{l} && \text{(advection number)} \\
 \pi_4 &= \frac{D_m t}{l^2} && \text{(diffusion number)} \\
 \pi_5 &= \frac{\rho_f g l l_p}{T_f} && \text{(capillary effects number)} \\
 \pi_6 &= S/\rho_f && \text{(adsorption number)} \\
 \pi_7 &= g t^2 / l && \text{(dynamic effects number)} \\
 \pi_8 &= \frac{\bar{v} l_p}{D_m} = Pe && \text{(Peclet Number)}
 \end{aligned}$$

$$\pi_1 = C/\rho_f$$

π_1 is considered to be the dependent dimensionless group and it can be written as a function of the other seven groups.

$$\pi_1 = f(\pi_2, \pi_3, \dots, \pi_8) \quad (22)$$

In order to scale the pollutant concentration in a model test, each independent variable that is a member of an important dimensionless group must be properly scaled. The importance of each of the independent dimensionless groups, and the corresponding scaling requirements for the independent variables that appear in the dimensionless groups is discussed below.

$$\pi_2 = \rho_f \bar{v} l / \mu$$

π_2 , more well known as the Reynolds number must be kept below the critical value ($Re < 1$) for flow in porous media (Bear 1972) to insure the validity of Darcy's Law. It will be shown later that the $Re < 1$ for the experiments reported in this paper.

$$\pi_3 = \frac{\bar{v} t}{l}$$

$$\text{If } \pi_3^* = \frac{\bar{v}^* t^*}{l^*} = 1 \quad (23)$$

$$t^* = \frac{l^*}{\bar{v}^*} \quad (24)$$

But $l^* = 1/N$ and $\bar{v}^* = N$ therefore, time must scale by $t^* = N^{-2}$.

$$\pi_4 = \frac{D_m t}{l^2}$$

The coefficient of molecular diffusion, D_m , is known to depend on the void ratio and hence the state of stress. D_m will be identical if identical materials are used and identical stresses are obtained in the prototype and the centrifuge model: $D_m = 1$. To maintain:

$$\pi_4^* = \frac{D_m^* t^*}{l^{*2}} = 1 \quad (25)$$

time must scale by

$$t^* = \frac{l^{*2}}{D_m^*} = \frac{(1/N)^2}{1} = N^{-2} \quad (26)$$

which is in agreement with the time scale factor derived based on π_3 .

$$\pi_5 = \frac{\rho_f g l l_p}{T_f}$$

This dimensionless group is proportional to the ratio of sample height over the height of capillary rise. π_5 must be identical in model and prototype if capillary effects such as the height of capillary rise or flow above the ground water table are to be simulated in a model test. Using identical materials insures that $\rho_f^* = T_f^* = l_p^* = 1$. The condition for simulation of capillary effects is

$$\pi_5^* = \frac{\rho_f^* g^* l^* l_p^*}{T_f^*} = 1 \quad (27)$$

$$\text{or } g^* = \frac{1}{l^*} = N \quad (28)$$

This is consistent with the scaling laws derived earlier based on imposing the condition of identical stress (equation 9).

Flow of water in saturated and unsaturated soil is caused by gradients in total potential. The total potential is the sum of gravitational potential, potential due to capillary tension, and osmotic potential. For soils near or below a water table the total potential of the pore water is dominated by the gravitational potential. Most problems of pollution of water supplies involve (at some stage) flow in and near aquifers which are below the ground water table. Therefore, gravity should be properly scaled in most model tests to simulate pollution of water supplies.

At significant distances above a ground water table, evaporation or infiltration may cause spacially abrupt and relatively rapid changes in total potential. In order to simulate the associated high suction gradients, the boundary conditions which cause them (eg. rapid infiltration and evaporation) must be simulated. This introduces additional complexities to the modeling process.

The potential gradient due to capillary or osmotic suction can be many times larger than gravitational potential gradients at certain points in a soil profile. In certain problems (such as contamination of unsaturated soil) the gravitational gradient may never be large compared to the suction gradients. In these problems the centrifuge may be unnecessary. This statement must be qualified, because, even though the gravitational gradients are small, they are consistent in direction and they may act over a large depth of soil. The net flow from the top to the bottom of a soil deposit may be influenced by gravity while on a more localized scale, the flow is caused by more erratic suction gradients.

$$\pi_6 = S/\rho_f$$

The dimensionless group, π_6 and the concentration of adsorbed solute, S , will be identical in model and prototype if identical materials are used and equilibrium adsorption is achieved. Equilibrium adsorption is achieved if S is only a function of solute concentration, C , in the pore fluid, with no additional time dependence. This is often the case for flow through uniform soils of all types. If there is a small pocket of relatively impermeable soil, it may take time for solute ions to find their adsorption sites in the interior of the pocket, and equilibrium may not be achieved.

Furthermore, for flow of certain chemicals through some types of soils (eg., any solute through sand) adsorption is not an important factor in the transport process and π_6 can be ignored.

$$\pi_7 = \frac{gt^2}{l}$$

This group represents the relative sizes of gravitational accelerations and the ratio of characteristic sample length over characteristic time squared. This group will only be significant in dynamic problems where inertia forces are significant. For example, to simulate explosions or earthquakes on a centrifuge, π_7 must be identical in model and prototype. If $\pi_7 = 1$ is enforced, this results in

$$t^* = \sqrt{\frac{l^*}{g^*}} = \sqrt{\frac{N^{-1}}{N}} = \frac{1}{N} \quad (29)$$

and the time scale factor is in conflict with those derived based on similarity of π_3 and π_4 . For laminar flow problems in porous media where inertia forces are unimportant, however, π_7 can be ignored.

$$\pi_8 = \frac{\bar{v} l_p}{D_m}$$

π_8 , more well known as the Peclet Number (Pe), represents the relative importance of mechanical dispersion and molecular diffusion. It is not possible to maintain π_8 constant if flow velocities are scaled and identical materials (with $\rho_p^* = D_m^* = 1$) are used in model and prototype. It has been established however that the hydrodynamic dispersion coefficient (D) is a function of flow velocity if $Pe > 1$ (Pfannkuch 1962). For $Pe < 1$ D is independent of flow velocity (Pfannkuch 1962), and dispersion can be modeled even without insuring similarity of π_8 . This is often the case in practice especially for flow through fine grained soils where v is relatively small.

In many cases dispersion need not be simulated. Often the most important mechanism of the transport processes is advection, which can be simulated even if $Pe < 1$ condition is violated. In addition, since the dispersion coefficient becomes larger in model than in the prototype (since $\bar{v}^* = N$), the transport due to dispersion is faster in the model than it should be for similarity. This error is often on the conservative side if we are concerned with transport of pollutants. For example, the time taken for small concentrations of a pollutant to reach a water supply well from a waste disposal pond will be underestimated.

PHYSICAL ASPECTS OF MODELING

In addition to the similarity conditions established above for the transport processes, it is necessary to maintain the validity of Darcy's Law. This can be maintained if the Reynolds number $Re = \frac{\bar{v}d}{\nu} \leq 1$

where

- $\bar{v}$ = interstitial velocity
- d = diameter of the soil grains
- ν = coefficient of kinematic viscosity.

For the condition where $Re = 1$, $\nu/d = \bar{v}_{\max}$, and assuming a typical for water as $10^{-6} \text{ m}^2/\text{s}$, $\bar{v}_{\max} = 10^{-6}/d \text{ m}^2/\text{s}$ is the maximum permissible flow velocity.

From Darcy's law for flow in a 1 g prototype:

$$\bar{v} = \frac{k}{\gamma_f n} \frac{\partial p}{\partial l} \quad (30)$$

The velocity of flow in a centrifuge model is N times greater than that in the 1 g prototype,

$$\bar{v}_{\text{model}} = N \left(\frac{k}{\gamma_f n} \frac{\partial p}{\partial l} \right)_{\text{prototype}} \quad (31)$$

For the limiting case when $Re = 1$ in the model

$$\bar{v}_{\max} = \bar{v}_{\text{model}} \quad (32)$$

$$N_{\max} = \frac{10^{-6} \frac{\text{m}^2}{\text{s}}}{d} \cdot \frac{n}{k} \cdot \frac{\gamma_f}{(\partial p / \partial l)} \quad (33)$$

For example, the maximum permissible scale factor for a fine sand with $k = 10^{-4} \text{ m/s}$, $n = 0.5$, $d = 10^{-4} \text{ m}$ and assuming a prototype hydraulic gradient of $\gamma_f / (\partial p / \partial l) = 1$ is given by

$$N_{\max} = \frac{10^{-6} \frac{\text{m}^2}{\text{s}}}{10^{-4} \text{ m}} \cdot \frac{0.5}{10^{-4} \frac{\text{m}}{\text{s}}} \cdot 1 = 50 \quad (34)$$

For tests on coarse sand, or medium sand with large hydraulic gradients, this condition might be violated. However, for clayey soils $N_{\max}$ is

very large. For the tests reported in this study the largest Re used was 0.36.

TESTING PROGRAM AND RESULTS

Tests were performed on five different types of soils under 1 g conditions and using a 1m radius Schaevitz Centrifuge. They were a medium sand, Monterey '0/30' sand, silica flour, a mixture of 10% Monterey 0/30 with 90% Snowcal 50 and pure Snowcal 50. Sand and silica flour samples were prepared in a 0.127 m diameter plastic cylinders by pluviating and vibrating the soil into a 0.08N salt water solution. The change in concentration was monitored as fresh water was allowed to seep through the models under an excess pressure head in the centrifuge.

Clay samples were prepared by making a slurry with a 0.06N lithium chloride solution and consolidating the slurry in a 0.152 m (6 inch) diameter plastic cylinders at the appropriate centrifuge acceleration to produce a normally consolidated clay layer. The consolidation process was monitored using pore pressure transducers. The clay layers were then leached with Ethyl alcohol in the centrifuge, a process which does not involve adsorption. Then, a 0.08N sodium hexa-metaphosphate solution was allowed to seep through the models under an excess pressure head in the centrifuge. This process involves replacement of the lithium ions by sodium ions from the clay particle surfaces, an adsorption-desorption process.

The change in concentration of the pore fluid at the mid depth of each model was monitored as a function of time utilizing a conductivity measuring system as shown in Fig. 1. The conductivity cell used in most of the tests is also shown in Fig. 1. The conductivity cell used in the testing of medium sand samples consisted of a porous filter in addition to the electrodes (Arulanandan et al. 1985). The porous filter was

removed in the later tests to avoid the lag in detecting the concentration changes (Arulanandan et al. 1985). In all tests the cylinders containing samples were placed inside another cylinder with larger diameter and the difference in fluid level between these two cylinders was maintained by sending fluid into the inner cylinder continuously and allowing the fluid to overflow at fixed heights of both the inner and outer cylinders. The difference in fluid level between the cylinders is such that this difference in fluid level will produce the required excess pressure head at a given g level. In addition to maintaining the difference in fluid level, pore pressure transducers were used to monitor the excess pressure heads applied to the samples. Output from the conductivity cell and the pore pressure transducers were recorded on x-y plotters. Pore Pressures were measured to confirm that they were consistent with the intended fluid levels.

Modeling of model tests:

- a) Tests on a medium sand: Modeling of model tests were performed on a uniform medium sand with a mean grain diameter of 0.42 mm. Details of this tests can be found elsewhere (Arulanandan et al. 1985).
- b) Tests on silica flour: Four silica-flour (mean grain diameter 0.125 mm) models varying in heights of 0.210, 0.157, 0.140 and 0.105 meters were tested in 25, 33.3, 37.5 and 50 g centrifugal accelerations respectively. The water levels were also scaled so that water pressures were identical at corresponding points in all models. Samples were prepared at a porosity of 0.344. The coefficient of permeability, under 1g conditions, corresponding to this void ratio is 3.05×10^{-3} cm/s. The prototype represented by these four models is given in Fig. 2. The ratio of concentration to initial

concentration (C/C_0) at mid depth measured as a function of model time utilizing the conductivity cell for all these four models are given in Fig. 3. A plot of $\log_{10} t_m$ versus $\log_{10} N$ is given in Fig. 4, where t_m is the time taken for 50% reduction in concentration at mid depth and N is the model scale factor. This result verifies the $t^* = 1/N^2$ scaling law derived earlier. The relationship between C/C_0 at mid depth and the prototype time for these four models are given in Fig. 5. Prototype time was obtained using the relationship $t_p = N^2 t_m$, where subscripts p and m stand for prototype and model respectively. Since all four models gave the same prototype behavior it could be concluded that the scaling laws derived appear to be valid. Hydrodynamic dispersion coefficients (D) for these tests will be examined subsequently.

- c) Tests on clays: Two centrifuge tests were carried out on 100% Snowcal 50 models of height 0.102 m and 0.051 m subjected to centrifugal accelerations of 50 g and 100 g respectively. Slurry was prepared as described before and samples were consolidated in the centrifuge in two layers with the conductivity cell and the pore pressure transducer placed at the center of the samples. Samples were subjected to an excess pressure head of 10.16 m of water in prototype terms. Conductivity variations at mid depth with time were measured during seeping of sodium hexa-metaphosphate for both the samples and during leaching for the 100 g sample. The relationship between the ratio of concentration, C , to final concentration, C_f and the prototype time for these tests are given in Fig. 6. Prototype times were obtained as discussed before. The curve denoted by "NO ADSORPTION" is the curve deduced from the measured variation of C/C_0 with time during leaching with alcohol. As seen

from Fig. 6 adsorption retards the movement of the solute. Curves for 50g and 100g samples, which represent the same prototype behavior, seem to agree reasonably well. The maximum variation between these curves is about 70 days prototype time. This corresponds to 0.672 hours for the 50g test. This is small compared to the 12 hours (1250 days prototype time) duration of the test. Hence, it can be concluded that the scaling laws derived are supported. The hydrodynamic dispersion coefficient for these tests also will be examined subsequently.

Influence of self weight on the transport processes:

The conventional approach for the determination of breakthrough curves is to perform a longitudinal dispersion test of a tracer passing through a column of porous media. The breakthrough curve is used to calculate dispersion coefficient (D) and the average interstitial velocity ($\bar{v}$). The calculated values of D and $\bar{v}$ are then used to predict the breakthrough curve for any depth in the prototype. But in a lg test, prototype stress distributions, i.e., stresses due to the prototype self weight cannot be simulated as in a centrifuge model.

To investigate the influence of selfweight on the breakthrough curves two sets of test were performed. One using silica flour samples and the other one using 100% Snowcal 50 samples. A lg test and a 37.5g test on silica flour and a lg and a 100g test on Snowcal 50 were compared. The height of the samples, excess pressures and the locations of the conductivity cell were identical in the lg and centrifuge models, i.e., 0.140 m soil height and an excess water pressure of 4.4 kPa water for silica flour samples and 0.05 m soil height and an excess water pressure of 99.7 kPa for clay samples with conductivity cell at mid

depth in all the cases. The excess pressure for 1 g testing of silica flour samples was applied using static water column and the excess pressure for 1 g testing of clay samples was applied using air pressure. The 1g clay sample was consolidated to the stress experienced at mid depth of the centrifuge model and then allowed to swell back before testing. The results for the silica flour samples and the clay samples are given in figs. 7 and 8 respectively. It can be seen from these figures that the breakthrough curves are different with the time for 50% reduction in concentration for 1g tests being less than that for the centrifuge tests. Also it can be seen that this effect is prominent in clays. This result is expected since the void ratio of clays and hence the permeability is more sensitive to the stress level. As the stress level increases the void ratio decreases. As the void ratio decreases the permeability also decreases, causing a shift in the breakthrough curves to the right. Part of the difference in the behavior of clay samples could be attributed to the void ratio increase during swelling of the 1g sample, although this increase was small. Even if a uniform stress is applied to a 1g clay sample during testing it will behave differently than a prototype layer or a centrifuge model which have linear variation of stresses with depth. Hence it is reasonable to conclude that a centrifuge model, behavior is more realistic than that of a 1g model.

Influence of velocity on the hydrodynamic dispersion:

To study the influence of velocity on the hydrodynamic dispersion two sets of tests were performed on Monterey '0/30' sand (mean grain diameter 0.45 mm). One set was performed on the centrifuge. In this set a sample of height 0.208 m with the conductivity cell at the mid-

depth was tested at 25g acceleration with varying excess pressure heads, thereby with varying average interstitial velocities ($\bar{v}$). Average interstitial velocities were calculated by dividing the depth of the conductivity cell by the time taken for 50% reduction in concentration. The relationship between C/C_0 and $\bar{v}t$ for these tests are shown in Fig. 9. $\bar{v}t$ was chosen as a parameter instead of the absolute time (t) in order to bring the 0.5 C/C_0 of all these tests to a point and to facilitate the comparison of the hydrodynamic dispersion. It can be seen from fig. 9 that as the velocity increases the spreading of the breakthrough curves also increases indicating an increase in the hydrodynamic dispersion with an increase in the velocity. However the differences are not very significant due to the small range of average velocities used in these tests.

Another set of tests were performed in 1g acceleration. In this set 3 samples of heights 0.102 m, 0.203 m and 0.406 m with the conductivity cell at mid depths were subjected to average interstitial velocities of 0.36×10^{-3} m/s, 0.20×10^{-3} m/s and 0.13×10^{-3} m/s respectively. The relationships between C/C_0 and $\bar{v}t/\ell$ for these tests are given in fig. 10, where ℓ is the depth of the conductivity cell. Here also $\bar{v}t/\ell$ was used as a parameter instead of t in order to bring the 0.5 C/C_0 of all the tests to a point. It can also be seen from fig. 10 that as the velocity increases hydrodynamic dispersion also increases. The spreading of dispersion curves with increase in velocity is more prominent in 1g tests than the centrifuge tests due to the bigger range of velocities used.

The hydrodynamic dispersion coefficients for all the tests were calculated using the following equation (Greenkorn 1983)

$$D = \frac{1}{4} \frac{L\bar{v}}{\left[\frac{\partial(C/C_0)}{\partial(V/V_0)} \right]_{V/V_0 = 1}^2} \quad (35)$$

where L = distance traveled by the tracer with a velocity of v in time t_{50} (time taken for 50% reduction in concentration)

and V/V_0 = pore volume.

If there is no dispersion of any sort, the plot of C/C_0 versus V/V_0 is a step change from $C/C_0 = 0$ to $C/C_0 = 1$ at $V/V_0 = 1$ or vice-versa. If there is dispersion, 0.5 C/C_0 of the breakthrough curve will correspond to $V/V_0 = 1$.

$[\partial(C/C_0)/\partial(V/V_0)]_{V/V_0 = 1}$ = slope of the breakthrough curve at $V/V_0 = 1$ or $C/C_0 = 0.5$.

Although this equation is only valid for an ideal medium (Greenkorn 1983) it will give a fairly good estimate of the hydrodynamic dispersion coefficient. The calculated hydrodynamic dispersion coefficients for all the tests are given in Table 1. Also given in Table 1 is the ratio of hydrodynamic dispersion coefficient to molecular diffusion coefficient (D/D_m) and the Peclet number ($\bar{v}l_p/D_m$). The results of an additional test performed on a mixture of Monterey '0/30' sand and Snowcal 50 is given in Table 1. D_m of 1.5×10^{-5} cm²/s was used for sand and silica flour and D_m of 8.0×10^{-7} and 3.0×10^{-7} cm²/s were used for sand/clay mixture and clay respectively. These values were based on the typical values for chemical species in geologic media (Freeze and Cherry 1979). A plot of D/D_m versus Peclet number obtained from the present study is given in Fig. 11 and it is compared with a similar plot obtained by a previous study (Pfannkuch 1962) using only lg samples.

It can be seen from Fig. 11 that D/D_m is independent of velocity when the Peclet number is less than one, i.e., in the region where molecular diffusion dominates mechanical dispersion. Clay samples and a sand/clay mixture are shown to fall in this region and some fine silts can also be expected to fall in this region. Hence for the soils falling in this region the process of dispersion can be modeled in a centrifuge.

SUMMARY AND CONCLUSIONS

Dimensional analysis was carried out to determine the scaling laws for flow of pollutants in groundwater. Conditions for satisfying the scaling laws and justifications for violating unimportant scaling laws were discussed. Samples prepared by the same method with same height and same excess pressure head tested in lg and in the centrifuge gave different breakthrough curves indicating that stresses due to gravity affect the soil permeability and retard pollutant travel through soils. The effect of self weight on permeability was shown to be more prominent in clays. It can be concluded that centrifuge models, which represent the prototype stress distribution more closely, will yield more realistic data on pollutant migration through soils which in turn can be used to verify mathematical models.

Centrifuge testing is also shown to accelerate the transport processes. Acceleration of transport processes can also be achieved in a one dimensional lg saturated soil model by increasing the excess hydraulic pressure using air pressure. However, lg models cannot simulate the prototype stress distribution or a zone of capillary rise above a phreatic surface and they cannot be used to simulate two or three dimensional flows where other gravity dominated effects such as a sloping water table.

In unsaturated soils at some distance above a water table, suction gradients may be much larger than gravity gradients, suggesting that scaling gravity may not be important. If the transport process is confined to an unsaturated soil, this argument may have some validity. Most water pollution problems, however, involve flow at some stage in an aquifer near or below a water table, where gradients of total potential are governed by gravity. Even if only part of the transport process is governed by gravity, it is important to scale gravity.

In order to include high suction gradients in a model, the conditions which cause them such rapid infiltration or evaporation must be simulated. This would add additional complexities to the model tests, which have not been discussed.

In addition to dispersion and advection, adsorption in relatively uniform clay samples was successfully modeled. Adsorption is shown to retard the transport process. There will be some difficulty in modeling adsorption if equilibrium adsorption is not achieved. Non-uniform soils with pockets of impermeable clay may be particularly slow in achieving equilibrium and in these cases it may be difficult to model adsorption.

Samples tested with different average interstitial velocities ($\bar{v}$) showed that hydrodynamic dispersion increases as the velocity increases. Carbonell and Whitaker (1983) also showed this phenomena theoretically using intrinsic phase average of the quantities involved. The relationship between the ratio of hydrodynamic dispersion coefficient to molecular diffusion coefficient (D/D_m) and the Peclet number ($\bar{v}l_p/D_m$) obtained from the current study on centrifuge and lg samples was compared to that of a previous study on lg samples (Pfannkuch 1962). A clay/sand mixture and clay samples tested in the current study were shown to fall in the region where D/D_m is independent of the Peclet number, i.e., velocity,

obtained by the previous study (Pfannkuch 1962). The process of dispersion can be directly scaled only if D/D_m is independent of the velocity, i.e., if the Peclet number is less than one in model and prototype. This appears to be a common situation in relatively impermeable soils.

Even if the validity of scaling laws exists it is impossible to simulate all the heterogeneities and the structure (such as aggregation, pore shape and pore size distribution) of the soil of a prototype. If one wants to model a prototype it is only feasible to do numerous tests with the heterogeneities and the structure of the prototype soil simulated as closely as possible and then to adapt a statistical approach to analyze the data. In simulating a large number of heterogeneities a large model and hence a large capacity centrifuge will be useful.

It should be noted that problems with precise simulation of prototype heterogeneities and soil structure are not much simpler for mathematical models, furthermore these models are not extensively verified.

In light of these difficulties the centrifuge can be used to produce data under repeatable laboratory controlled conditions where the physical processes of advection, dispersion and adsorption occur in heterogeneous two or three dimensional situations. Irrespective of the validity of the scaling laws, these data can be used to verify mathematical models. Once verified, the mathematical models could be applied to complex field situations with more confidence.

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APPENDIX I: REFERENCES

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Table 1

Soil	g-level/ Sample Ht(in)	Hydrodynamic Dispersion Coeff. (D) (cm ² /s)	Interstitial Velocity(v) (cm/sec)	Mean Partical diameter ($\bar{l}_p$)(cm)	D/D _m	$\frac{\bar{v} \bar{l}_p}{D_m}$
Monterey '0/30'	1g/4	5.0×10^{-3}	0.036	0.0450	3.33×10^2	1.08×10^2
	1g/8	4.3×10^{-4}	0.020	0.0450	0.29×10^2	0.60×10^2
	1g/16	3.0×10^{-4}	0.013	0.0450	0.20×10^2	0.39×10^2
Snowcal 50	50g/4	3.7×10^{-7}	3.0×10^{-4}	2×10^{-4}	1.2	6.0×10^{-2}
	100g/2	2.2×10^{-7}	1.5×10^{-4}	2×10^{-4}	0.7	3.0×10^{-2}
Medium Sand (1)	53g/3.88	1.7×10^{-3}	0.086	0.0420	1.13×10^2	2.41×10^2
	40g/5.11	1.0×10^{-3}	0.064	0.0420	0.67×10^2	1.79×10^2
	33.3g/6.18	0.8×10^{-3}	0.054	0.0420	0.53×10^2	1.51×10^2
	25g/8.23	0.5×10^{-3}	0.041	0.0420	0.33×10^2	1.15×10^2
Silica Flour	50g/4.125	1.6×10^{-4}	0.031	0.0125	1.1×10^1	2.6×10^1
	37.5g/5.5	1.6×10^{-4}	0.023	0.0125	1.1×10^1	1.9×10^1
	33.3g/6.19	1.6×10^{-4}	0.016	0.0125	1.1×10^1	1.3×10^1
10% Monterey '0/30' + 90% Snowcal 50	100g/2	8×10^{-7}	2.51×10^{-4}	0.0468	1.0	0.9×10^{-1}

NOTE: 1 in. = 0.0254 m

APPENDIX II: NOTATIONS

- C = concentration of solute in pore water
- D = coefficient of hydrodynamic dispersion
- D_m = coefficient of molecular diffusion
- d = particle diameter
- K = absolute permeability
- k = coefficient of permeability
- l = characteristic macroscopic length
- l_p = characteristic microscopic length
- n = porosity
- Pe = Peclet number
- Re = Reynolds number
- S = mass of adsorbed solute/unit volume
- T_f = surface tension for fluid particle interface
- v = average "Darcy" velocity
- $\bar{v}$ = average interstitial velocity
- γ_f = unit weight of fluid
- ρ_f = density of fluid
- μ = coefficient of dynamic viscosity
- ν = coefficient of kinematic viscosity
- $\pi_1 - \pi_8$ = dimensionless groups

Figure Captions

- Fig. 1. Circuit diagram and the geometry of the conductivity cell (lower right corner) for the conductivity measuring system (1 in. = 0.0254 m).
- Fig. 2. The prototype represented by the four silica flour models (1 in. = 0.0254 m).
- Fig. 3. The ratio of concentration to initial concentration (C/C_0) at mid-depth as a function of model time for silica flour models.
- Fig. 4. Logarithm of time for 50% reduction in concentration ratio at mid-depth vs. logarithm of scale factor for silica flour centrifuge models.
- Fig. 5. The ratio of concentration to initial concentration (C/C_0) at mid-depth as a function of prototype time for silica flour models.
- Fig. 6. The ratio of concentration to final concentration (C/C_f) at mid-depth as a function of prototype time for Snowcal 50 models.
- Fig. 7. The ratio of concentration to initial concentration (C/C_0) at mid-depth as a function of model time for a silica flour sample with same excess pressure head tested at 1g and 37.5g.
- Fig. 8. The ratio of concentration to final concentration (C/C_f) at mid-depth as a function of model time for Snowcal 50 samples with the same excess pressure head tested at 1g and 100g.
- Fig. 9. The ratio of concentration to initial concentration (C/C_0) at mid-depth as a function of $\bar{v}t$ (where $\bar{v}$ is the average interstitial velocity and t is the model time) for a Monterey '0/30' sand model tested at 25g with varying average interstitial velocities (1 in = 0.0254 m).
- Fig. 10. The ratio of concentration (C/C_0) at mid-depth as a function of $\bar{v}t/l$ (where $\bar{v}$ is the average interstitial velocity, t is the model time and l is the mid-depth) for Monterey '0/30' sand models tested at 1g with varying sample thicknesses and with varying average interstitial velocities (1 in. = 0.0254 m).
- Fig. 11. Comparison of the relationship between D/D_m and the Peclet number obtained using 1g and centrifuge model tests.

Figure 1

Fig 2

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Figure 4

Figure 5

Figure 6

Figure 7

Figure 8

Figure 1

Figure 16