Estimation of Displacements of Rockfill Dams Due to Seismic Shaking

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SYNOPSIS A computer code for estimation of displacement of sliding surfaces during dynamic loading is discussed. The procedure allows selective incorporation of the effects of horizontal and vertical inertial forces, excess pore pressure development, hydrodynamic forces, shear strength reduction, and dilation. The relative effect of these parameters on the computed displacements can be readily determined for alternative design layouts or material behavior assumptions.

INTRODUCTION

The current consensus of professional opinion with regard to the dynamic performance of well designed and constructed clay core rockfill dams appears to be that only limited permanent deformation would occur even under strong earthquake shaking. This opinion is supported by a limited number of case histories for rockfill dams, by a considerably greater number of case histories for earth dams, and by analytical formulations which predict small permanent displacements.

Two of the most commonly used analytical methods for estimating permanent displacements of embankment dams may be referred to as (1) the cyclic shear strain procedure and (2) the sliding block procedure. The latter procedure is considered in this paper.

A rigorous calculation of permanent displacement by the sliding block procedure requires consideration of the time history of the earthquake and requires definition of such items as the geometry of the slip plane(s), the behavior of the slide mass, and the strength of the materials. Because of the complexity of including the acceleration time history, the uncertainty associated with the definition of the analysis parameters, and because the displacements are thought to be small, approximate or simplified methods (Newmark, 1965, Makdisi and Seed, 1977) have been developed to provide the dam designer with an estimate of the amount of displacement to expect under dynamic loading. Despite the relatively high degree of confidence and widespread utilization of the simplified procedures, it is clearly evident that these procedures are intended for noncritical structures and are not a substitute for a detailed individualized analysis for critical structures especially when some of their design parameters go beyond the limit of existing experience. The studies for a rockfill dam at the Auburn damsite in California require such an individualized investigation and toward this end, this study examined in detail the means for analysis of the displacement contribution from the sliding of blocks, wedges, or slices of material.

POTENTIAL DISPLACEMENT MODES FOR A ROCKFILL DAM UNDER DYNAMIC LOADING

In general a central clay core rockfill dam consists of upstream and downstream shells of rock and one or more filter zones bounding an inclined or centered clay core. If it is considered that these materials may develop increased pore pressures and have reduced strength during dynamic loading but that drastic loss of strength (liquefaction) does not take place, then the potential modes of deformation that could contribute to lowering of the crest would be:

1. Flattening of the slopes due to raveling and due to sliding of shallow slices of material on planar or nearly planar surfaces.

2. Sliding of a surface or surfaces of a deep seated nature of generalized shape that may or may not involve the central core material.

3. Generalized deformation or settlement without the formation of specific extensive planes of sliding.

Only the method for analysis of displacement associated with the first two modes is considered in this paper.

APPLICATION OF THE NEWMARK CONCEPT FOR DISPLACEMENT CALCULATION DURING DYNAMIC LOADING

If it can be assumed that:

1. Slide planes (of arbitrary shape) exist or develop within the mass or near the surface of the mass during the time of earthquake shaking, and

2. That the material above the slide plane can be considered to behave as a rigid body;

Then, one may use the limit equilibrium method of analysis and the concept of computing an incremental permanent displacement along the sliding plane each time the earthquake loading causes the driving forces along the sliding plane to exceed the resisting forces.

These assumptions appear to be well founded based on the form of sliding of natural slopes and embankment dams under dynamic loading, but since few significant displacements due to dynamic loading have occurred on any rockfill dams, the pattern on these structures has not been observed. Nevertheless, the assumption that this mode of displacement is possible appears to be a reasonable assumption.

A procedure of computing displacement of the slide mass whenever the calculated resistance is exceeded by the calculated driving force is a flexible procedure in that it can account for the time history variation of horizontal and vertical inertial loads, pore pressure, hydrodynamic forces, shear strength, or any other factor which may affect the resistance of the sliding plane or the forces that it must resist.

In order to apply the concept to a displacement calculation, two steps are involved. First, one must track the time history of ground motion that is relevant to the stability of the mass being analyzed (along with other relevant parameters if desired) in order to determine the instant in time when relative displacement (permanent) begins between the sliding mass and the body of the dam. The second step, which allows the quantitative estimate of displacement to be made, requires computation of the time history of the difference between the input acceleration and the resistance to motion along the slide plane which can be represented as an acceleration. Numerical integration of these values allows determination of the time when relative velocity between the mass and the structure is zero and a second integration allows determination of the relative permanent displacement that occurs during the time interval when movement took place.

Although the concept was introduced as general in nature, the common application of the method [Sarma, 1975; Makdisi and Seed, 1977] is based on determination of a constant value of "yield acceleration" which "is defined as that average acceleration producing a horizontal inertia force on a potential sliding mass so as to produce a factor of safety of unity and thus cause it to experience permanent displacements" [Makdisi and Seed, 1977]. Recognition has been given by various authors to the fact that the least value of yield acceleration is at an angle other than horizontal but its influence in estimating the amount of displacement was not considered significant.

In the generalized procedure development below the time history of the available resisting force (a time history of the "yield acceleration") is accounted for as well as the time history of the earthquake loading. Use of a time history of resistance and inclusion of additional parameters in defining this resistance in the computation of permanent displacement is the principal departure from the existing procedures. Several investigators have reported that there is only a small influence on the initial value of "yield acceleration" (i.e., the acceleration level when motion is started) by including the effect of a general orientation of the input acceleration. However, the change in resisting capacity (yield acceleration) once the input acceleration has exceeded the initial "yield acceleration" is not negligible. The generalized procedure of using a time history of resisting capacity allows for the incorporation of pore pressure increases and strength changes as a function of the input acceleration (e.g., the number of equivalent cycles exceeding a specified level) and the displacement history (the cumulative displacement can be monitored and appropriate changes in strength or pore pressure can be made according to predetermined criteria). Although our state of knowledge with respect to dynamic pore pressure, strength effects, and other factors is limited at present, the general formulation allows for examining the potential effects of these variations acting alone or in concert over the ranges of behavior currently considered rational.

PROCEDURE FORMULATION

The procedure presented estimates the relative displacement of a specific failure mass given the applicable time history of both horizontal and vertical accelerations. The critical surfaces for a structure must be found by the examination of many failure surfaces and the total displacement determined by integration of the results from these surfaces.

STATIC STRENGTH ANALYSIS

A fundamental concept of this method is that of excess available resistance, which is the amount of shear strength remaining after the forces tending to cause instability are accounted for. The excess available resistance is calculated, for a given failure surface, by performing a limit equilibrium analysis to determine the amount of shear strength mobilized (D) and the total available resistance (R). The factor of safety (FS) is the quotient of the total available resistance and the mobilized resistance.

$$FS = R/D \qquad (1)$$

The excess available resistance (Ra) is the difference of the total and the mobilized resistance.

$$Ra = R-D \qquad (2)$$

It is important to note that cohesion, internal friction angles, static fluid pressures, and surface loads have all been accounted for in the excess available resistance calculation.

ADDITION OF EARTHQUAKE ACCELERATIONS

The procedure for analyzing the problem of additional earthquake accelerations as illustrated on figure 1 is applicable to surfaces of general shape. However, for nonplanar surfaces a rigorous solution of the time history of the resisting capacity requires repeated limit equilibrium analyses over the range of combinations of horizontal and vertical accelerations expected [Chugh, in press]. Such information may be generated prior to application of the time history of acceleration to the failure mass. The rigorous solution is at present considered to be reserved for application at final design stage. For most problems the time history of available resistance may be estimated. The approximations to include the effect of nonplanar failure surfaces and variable materials are discussed below. The basic assumption required for these approximations is that the general pattern of the force distribution existing under the static loading exists during the cyclic loading.

NONPLANAR FAILURE SURFACES

For planar surfaces, R acts along the failure plane opposite the direction of motion. To accommodate nonplanar surfaces an effective angle of sliding (α_e) is calculated as the weighted average of the angles of sliding of each individual slice. The use of the normal forces (N) from the limit equilibirium analysis describes the distribution of the side forces on the slices prior to the addition of dynamic loads.

$$\alpha_e = \frac{\sum_{1=i}^{n} N_i \alpha_i}{\sum_{1=i}^{n} N_i} \quad \text{where } n = \text{number of slices} \qquad (3)$$

The resistance (Ra) is assumed to act at an inclination of α_e, opposite the direction of motion.

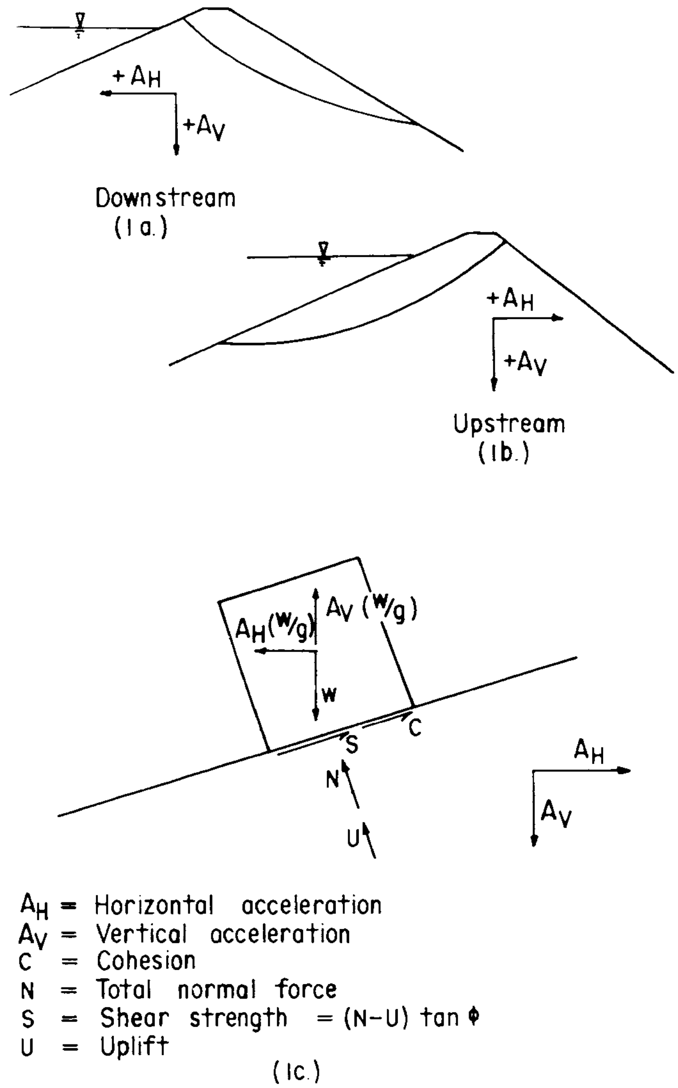


FIGURE I

VARIABLE MATERIALS

When more than one material is present along the failure surface, an effective angle of internal friction (ϕ_e) is calculated using a weighted average of the normal forces and internal friction angles of each slice. The use of the normal forces (N) allows the effect of the side forces on the slices to be accounted for. This effective friction angle is applicable only for the differential loads (poststatic analysis) that are applied to the structure.

$$\phi_e = \tan^{-1} \frac{\sum_{1=1}^{n} N_i \tan\phi_i}{\sum_{1=1}^{n} N_i} \qquad (4)$$

IMPENDING MOTION

Impending motion occurs when the forces caused by the earthquake accelerations combine to negate excess available resistance. The resultant force (D_H), in the direction of motion, caused by a horizontal acceleration (A_H) equals the sum of the increased inertial driving force and decreased resisting force. Accelerations are reported as a ratio of the acceleration of gravity.

$$D_H(T) = W\, A_H(T) \cos\alpha_e + W\, A_H(T) \sin\alpha_e \tan\phi_e \quad (5)$$

Similarly, the resultant force (D_V), in the direction of motion, caused by a vertical acceleration (A_V) is the sum of the reduced driving force and the reduced resisting force.

$$D_V(T) = W\, A_V(T)\,(-\sin\alpha_e) + W\, A_V(T) \cos\alpha_e \tan\phi_e \quad (6)$$

Hence, impending motion, at time T_o, occurs when the resultant forces created by the horizontal and vertical accelerations are equal and opposite to the excess available resistance.

$$R_a = W\, A_H(T_o)\,(\cos\alpha_e + \sin\alpha_e \tan\phi_e) + W\, A_V(T_o)\,(-\sin\alpha_e + \cos\alpha_e \tan\phi_e) \quad (7)$$

Because acceleration time histories are given in digitized, piecewise linear form (usually 0.01 second) interpolation is necessary to find the time (T_o) of impending motion. Knowing the time of impending motion the horizontal and vertical acceleration pair which causes impending motion $[A_H(T_o), A_V(T_o)]$ is calculated.

If a more critical acceleration pair exists at the next instant in time (T_1) then an <u>additional reduction in resisting capacity</u>

$$[A_H(T_1) - A_H(T_o)]\, W \sin\alpha_e \tan\phi_e + [A_V(T_1) - A_V(T_o)]\, W \cos\alpha_e \tan\phi_e$$

must be accounted for as well as an incremental change in driving force

$$[A_H(T_1) - A_H(T_o)]\, W \cos\alpha_e \tan\phi_e + [A_V(T_1) - A_V(T_o)]\, W\,(-\sin\alpha_e),$$

these two effects combine to represent the unbalanced force in the direction of motion and cause displacement of the failure mass. This unbalanced force, which continues to change with time, may be expressed as an acceleration that represents the relative acceleration between the main body of the embankment and the sliding mass.

CALCULATION OF DISPLACEMENTS

The amount of displacement may be calculated by double integration of the relative acceleration. Using Newton's second law of motion, the relative acceleration of the failure mass (A) may be calculated as:

$$A(T) = [A_H(T) - A_H(T_o)]\, g\,(\cos\alpha_e + \sin\alpha_e \tan\phi_e) + [A_V(T) - A_V(T_o)]\, g\,(-\sin\alpha_e + \cos\alpha_e \tan\phi_e) \quad (8)$$

The relative acceleration is calculated for each time step and a linear variation in acceleration between the time steps is assumed. Thus the equation for relative acceleration between any two time steps as t varies from 0 to ΔT is:

$$A(t) = A(T) + \frac{A(T) + A(T+\Delta T)}{\Delta T}\, t \quad (9)$$

where T = the total time lapsed from the beginning of the acceleration time histories
ΔT = the uniform interval between time steps
t = the time lapsed since the previous time step $0 \leq t \leq \Delta T$

Integration of the relative acceleration between two time steps gives the change in relative velocity for that time interval. The equation for relative velocity (V) between two time steps is:

$$V(t) = V(T) + \int_0^t A(t)\, dt \quad (10)$$

The relative velocity at the end of the interval when ($t = \Delta T$) is:

$$V(T+\Delta T) = V(T) + [A(T) + A(T+\Delta T)]\, \frac{\Delta T}{2} \quad (11)$$

Integration of equation (11) gives the change of displacement for the interval. The equation for displacement (U) between two time steps is:

$$U(t) = U(T) + \int_0^t V(t)\, dt$$

The relative displacement at the end of the interval is calculated using the limits 0 and ΔT and is:

$$U(T+\Delta T) = U(T) + V(T)\Delta T + [2A(T) + A(T+\Delta T)]\, \frac{\Delta T^2}{6} \quad (12)$$

SEPARATION OF FAILURE MASS

If, during the earthquake, the forces acting on the failure mass combine to separate the failure mass from the rest of the embankment, the resistance of the failure mass to sliding becomes zero. At separation equation (8) is no longer valid since it would continue to reduce the resistance. The correct equation is:

$$A(T) = A_H\, g\, (\cos \alpha_e) + A_V\, g\, (-\sin \alpha_e) + g\, (\sin \alpha_e) \tag{13}$$

END OF DISPLACEMENT

The relative displacement ceases when the relative velocity becomes zero. The time of the end of displacement is calculated by monitoring the relative velocity at each time step and interpolating back to the time of end, using equation (10), when the velocity drops below zero.

Setting equation (10) equal to zero and rearranging gives:

$$\frac{1}{2\Delta T}\,(A(T+\Delta T) - A(T))\, t^2 + A(T)\, t + V(T) = 0$$

The quadratic formula provides the solution:

$$t = \frac{-A(T) \pm \sqrt{A(T)^2 - \frac{2}{T}\,[A(T+\Delta T) - A(T)]\, V(T)}}{\frac{1}{\Delta T}\,[A(T+\Delta T) - A(T)]} \tag{14}$$

The solution which lies in the interval $(0,\Delta T)$ is determined and the time at which motion stops is T + t.

The cycle of computing displacements is repeated until every time step of the acceleration time histories is checked. The total relative displacement is the summation of the displacement for each cycle.

INCORPORATION OF CHANGE IN PORE PRESSURE OR SHEAR STRENGTH

If, during the course of the shaking, the pore pressure or strength values change, their effects may be incorporated into equations (7) and (8) by accounting for the increase or decrease in resistance similar to the way change in resistance due to the accelerations was handled.

In practice this adjustment can be made by obtaining the material properties and pore pressures each as a function of cyclic loading and shear displacement. For the pore pressure adjustment, the input earthquake motion can be tracked and when an appropriate number of equivalent cycles of loading has been reached an additional term representing the change in resistance due to the change in excess pore pressures (ΔU) is added to equations (7) and (8). The equations become:

$$Ra = WA_H(T)\,(\cos \alpha_e + \sin \alpha_e \tan\phi_e) + WA_V(T)\,(-\sin \alpha_e + \cos \alpha_e \tan\phi_e) + \Delta U \tan\phi_e \tag{7a}$$

$$A(T) = [A_H(T) - A_H(T_0)]\,(\cos \alpha_e + \sin \alpha_e \tan\phi_e) + [A_V(T) - A_V(T_0)]\,(-\sin \alpha_e + \cos \alpha_e \tan\phi_e) + [\Delta U(T) - \Delta U(T_0)]\tan\phi_e / W \tag{8a}$$

The method of estimating pore pressure increase incorporated in the analysis is similar to that used by Banerjee, Seed, and Chan (1979) in their analysis of Oroville Dam.

The displacement along the shear surface is also tracked and when a specified amount of displacement occurred the strength properties and possibly the pore pressure (for dilatant material) may be modified. If desired, the number of displacement events may also be tracked in order to alter strength properties. In any event, these changes would be incorporated into equations (7) and (8), if required, following the appropriate input earthquake loading cycle.

INCORPORATION OF THE EFFECT OF HYDRODYNAMIC FORCES ON DISPLACEMENTS

The method of determining hydrodynamic forces is based upon electric analogy tray experiments performed by Zangar (1952) available in the Water and Power Resources Service (formerly the U.S. Bureau of Reclamation) publication Design of Small Dams.

The change in horizontal pressure (Pe) due to earthquake loading is determined by Zangar's equation. The variation of pressure with depth is not linear and the total horizontal force (He) due to the pressure change is given by Zanger as:

$$He = 0.726\ Pe \times (\text{depth of the failure surface below the water level}) \tag{15}$$

The vertical force (Ve) due to the earthquake loading is calculated by changing the weight of the water (Ww) above the failure surface by the appropriate vertical acceleration. Cv is a coefficient which accounts for the nonrigid response of the reservoir.

$$Ve = Cv\ Ww\ A_V(T) \tag{16}$$

A positive horizontal and/or vertical acceleration tends to separate the dam from the reservoir, thus creating further instability. The reduced (or increased for negative accelerations) water forces may be considered as additional forces applied to the failure mass since only the resultant force along the plane of sliding is required.

Considering the horizontal and vertical loadings separately, the change in resistance is analogous to the change caused directly by the earthquake accelerations and may be handled in the same manner.

Equation (7) for impending motion becomes:

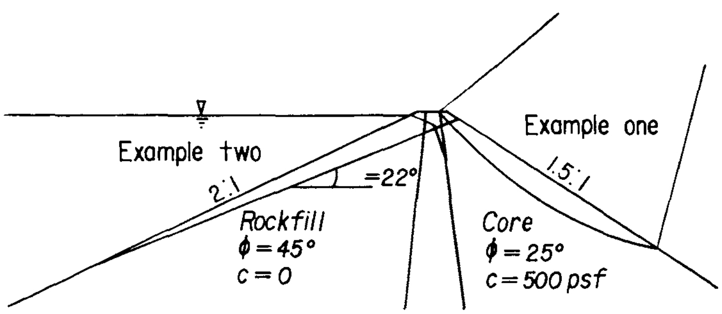
$$Ra = [WA_H(T) + He(T)]\ (\cos\alpha_e + \sin\alpha_e \tan\phi_e) + [WA_V(T) + Ve(T)]\ (-\sin\alpha_e + \cos\alpha_e \tan\phi_e) \quad (7b)$$

Equation (8) for relative acceleration becomes:

$$A(T) = [A_H(T) - A_H(T_o)]\ g\ (\cos\alpha_e + \sin\alpha_e \tan\phi_e) + [He(T) - He(T_o)]\ g/W\ (\cos\alpha_e + \sin\alpha_e \tan\phi_e) + [A_V(T) - A_V(T_o)]\ g\ (-\sin\alpha_e + \cos\alpha_e \tan\phi_e) + [Ve(T) - Ve(To)]\ g/w\ (-\sin\alpha_e + \cos\alpha_e \tan\phi_e) \quad (8b)$$

TYPICAL RESULTS

The method presented has been adapted to the computer. Several example problems have been analyzed and the results are summarized in figure 2. Example output produced by the program is shown in table 1.



EXAMPLE DAM

(2a.)

EXAMPLE RESULT SUMMARY								
Example Number		Horizontal Acceleration	Vertical Acceleration	Excess Pore Pressure	Hydro-dynamic Forces	Shear Strength Reduction	Dilation	Displacement (feet)
1	a	✓						0.22
	b	✓	✓					0.24
2	a	✓						0.26
	b	✓	✓					0.31
	c	✓	✓	✓				0.76
	d	✓	✓	✓	✓			2.36
	e	✓	✓	✓	✓	✓		2.44
	f	✓	✓	✓	✓	✓	✓	1.91

(2 b.)

FIGURE 2

The first problem (figure 2a) is a circular failure surface, extending approximately 80 feet in to the shell of the downstream 1.5H to 1.0V slope of a 700-foot-high dam. No pore pressures nor hydrodynamic forces acted upon the failure mass. The second problem (figure 2a) is a planar failure surface on the same cross section, cutting through the central clay core and upstream 2.0H to 1.0V slope. Excess pore pressures, hydrodynamic forces, strength reduction, and dilation were each considered in turn. Excess pore pressures were allowed to develop in the shell to equal the effective confining pressure in 10 equivalent uniform cycles. The shear strength parameter Ø was reduced 10° after 1.0 foot of displacement took place. The shell material was allowed to dilate, hence reduce the excess pore pressures to zero upon 0.5 foot of movement. The acceleration time histories for both examples one and two are representative of a near field magnitude 6.5 earthquake.

The displacement results are presented to illustrate the effect of including various options only. They do not represent the final estimate of displacements for two reasons: (1) the input acceleration has not been adjusted for the response of the structure and (2) the time history of displacements estimated on postulated surfaces below the surface shown must be examined and appropriately combined.

CONCLUSION

The procedure developed allows for a great degree of flexibility in the use of the rigid sliding mass approach for estimation of displacements. Additional studies to compare the displacements computed by this procedure to (1) laboratory models, (2) calculations by other procedures, and (3) case histories are in progress. Because the procedure uses similar programs and input information as static stability analyses, dynamic evaluations of design schemes can be made readily.

DESCRIPTION OF SAMPLE OUTPUT

Sample output showing one cycle of displacement is shown in table 1. Columns (1) and (2) show the time in seconds and the equivalent number of uniform cycles (ENUC). The earthquake-induced accelerations are shown in columns (3) and (4). Columns (4) through (8) show the computed relative acceleration, velicity, displacement, and sum of displacements. The resultant force due to the pore pressures and hydrodynamic forces, as a ratio of the excess available static resistance, are shown in the last two columns.

Table 1
Sample Output

(1) T (s)	(2) ENUC	(3) $A_H(T)$ (g)	(4) $A_V(T)$ (g)	(5) A(T) (g)	(6) V(T) (ft/s)	(7) U(T) (ft)	(8) ΣU (ft)	(9) PP(T)/Ra*	(10) H(T)/Ra*
5.3284	5.588	0.0967	0.0261	0.0000	0.0000	0.0000	0.6168	0.1649	0.3478
5.3300	5.720	0.1312	0.0036	0.0333	0.0008	0.0000	0.6168	0.1669	0.3479
5.3400	5.720	0.3905	-0.0089	0.5368	0.0925	0.0003	0.6171	0.1669	0.9581
5.3500	5.720	0.4285	-0.0256	0.5861	0.2732	0.0021	0.6189	0.1669	0.9896
5.3600	5.720	0.1793	-0.1725	-0.1601	0.3417	0.0054	0.6222	0.1669	-0.2171
5.3700	5.720	0.1067	-0.1551	-0.2783	0.2712	0.0085	0.6253	0.1669	-0.3340
5.3800	5.720	0.0858	-0.1404	-0.2963	0.1787	0.0107	0.6276	0.1669	-0.3297
5.3900	5.720	0.1473	-0.1269	-0.1496	0.1069	0.0121	0.6290	0.1669	-0.1207
5.4000	5.720	0.2993	-0.0468	0.2899	0.1295	0.0132	0.6300	0.1669	0.5783
5.4100	5.720	0.1127	0.0060	-0.0001	0.1761	0.0148	0.6316	0.1669	0.3101
5.4200	6.220	-0.0892	-0.0034	-0.4226	0.1081	0.0163	0.6332	0.1729	-0.2404
5.4300	6.220	-0.0116	0.0816	-0.1252	0.0200	0.0169	0.6337	0.1729	0.2889
5.4358	6.220	-0.0094	0.1013	-0.0883	0.0000	0.0169	0.6338	0.1729	0.3712

* PP(T) = resultant force due to excess pore pressures.
H(T) = resultant force due to hydrodynamic forces.

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