Seismic Response of Earth Dams on Liquefiable Foundations and Compaction Remediation

by

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ABSTRACT

Earthquake induced liquefaction is a major concern for earth dam safety in seismically active regions of the world. Many liquefaction induced embankment failures or near-failures have been reported around the world during various earthquakes. Such embankment damages were particularly destructive when the underlying saturated granular soils liquefied.

Through a series of twelve separate geotechnical centrifuge modeling tests, seismic behavior of a zoned earth dam with saturated sandy soil foundation was studied under moderate earthquake conditions. Soil response during and after shaking was monitored by many miniature accelerometers, pore pressure transducers, and displacement gauges placed throughout the soil model. The effect on the seismic behavior of the dam of different parameters such as the thickness, location, and depth of liquefiable layer is studied. This paper describes some of these tests and briefly presents the preliminary results. Valuable insights into the dynamic behavior of the employed embankment-foundation systems are provided. Currently, such testing results offer a valuable alternative to actual full-scale dynamic response, which is virtually non-existent.

KEYWORDS: Compaction; dams; liquefaction; remediation; seismic response

1. INTRODUCTION

The U.S. Army Corps of Engineers (COE) has responsibility for over 600 dams with a large number, over 1/3, being embankment dams with most of these lying in highly seismic areas of the country. The vast majority was constructed in the 1940's and 1950's when earthquake engineering was in its infancy and seismic hazards were neither recognized nor understood, leaving some inadequate for a seismic event. It is estimated that there are many other such seismically inadequate embankment dams around U.S. under the responsibility of federal, state and local governments, as well as the private sector.

Some of these embankments are founded on liquefiable soils, in many cases, necessitate-ing the development of appropriate remediation countermeasures (Marcuson et al. 1996). However, it is not feasible to remediate all of these structures due to the considerable cost involved. A determination of how much damage such structures could tolerate and still be able to perform their primary function could alleviate the need to remediate many dams. In cases that require remediation, understanding the deformations and dynamic response mechanisms of such dam/foundation systems would enhance our ability to design remedial procedures in a more effective and economical way. The potential for economic savings through better understanding of the involved mechanisms can be enormous considering the high cost of remedial treatment and the volume of dams that may require treatments in the coming years. Foundation remedial projects of Sardis Dam in Northern

Mississippi and Mormon Island Dam north of Sacramento, California both owned by the COE are only two recent cases, each of them costing over $30 million. Casitas Dam, Ventura-California owned by U.S. Bureau of Reclamation is another earth dam that is currently receiving a remedial treatment

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against foundation liquefaction with a cost of over $20 million. A similar $15 million upgrade is under way at the Bureaus' Bradbury Dam in the Santa Barbara-California area. Many other dams are currently under evaluation for liquefaction-induced hazards mainly by several federal and state governmental agencies.

It has been shown that the centrifuge dynamic model testing technique can play a major role in understanding the dynamic behavior of various earth structure-soil systems including earth dams (Arulanandan and Scott 1993; Adalier et al. 1998). Centrifuge testing is the most practical, most economical, and the only method for properly investigating and verifying earthquake induced equivalent-prototype behavior in soil, which behaves non-linearly and is stress-state dependent. Moreover, this centrifuge experimental model response also provides a basis for calibration of design and computational modeling procedures currently being performed by the ERDC.

In a series of 12 separate highly instrumented centrifuge model tests, seismic behavior of a zoned earth dam with saturated sandy soil foundation (Fig. 1) was studied under moderate earthquake conditions. The effect on the seismic behavior of the dam of different parameters such as the thickness, width, and depth of liquefiable layer is studied. This paper describes some of these tests and briefly presents the preliminary results. Valuable insights into the dynamic behavior of the employed embankment-foundation systems are provided. In addition to the pore pressure, acceleration, and model surface displacement measurements that are commonly done in most centrifuge tests, very detailed and extensive deformation mapping was performed based on pre- and post-shake meshes of markers throughout the body of the models. Such earthquake-induced deformations and damage is the key to making well-informed seismic safety and remediation decisions for embankment dams. Practice has demonstrated that remediation measures based on displacement criteria are much more cost-effective than those based on the factor of safety approach (Finn 2000). This study is believed to significantly expand and enhance our earthquake case history database regarding the earth dam on liquefiable foundation problem with more completely known and defined conditions and earthquake responses. Such an earthquake response database will provide: (1) a basis for modification and improvement of current methodology and assumptions, (2) realistic data for validation and improvement of numerical procedures (a current ongoing research), and (3) definitions of the physical processes and mechanisms involved in the liquefaction process and resultant effects on soil-embankment behavior.

2. CENTRIFUGE MODELING EQUIPMENT

The tests were done using the RPI's 100g-ton geotechnical centrifuge. It has an in-flight radius of 3.0 m and can test a payload up to 1 ton at 100g or 0.5 ton at 200g (i.e., 100 g-ton). More details about this centrifuge can be found on the WEB at http://www.rpi.edu/~dobryr/centrifuge/.

The main principle in centrifuge modeling is that a 1/N scale model subject to a gravitational acceleration of Ng (g is acceleration of gravity) will feel the same stress as the prototype. Then, stress-strain relationships at all equivalent points in the model and prototype will be the same if the same soil is employed and the behavior of the model will mimic the behavior of the prototype. Consequently, with the help of scaling laws (Schofield 1981) measurements in centrifuge tests under closely controlled conditions can be related directly to an equivalent full-scale prototype. The centrifuge modeling technique allows soil liquefaction tests to be performed at a conveniently reduced scale, and provides data applicable to full-scale problems.

In all tests, an in-flight shaker was employed to impart the model base shaking. Designed and built at Rensselaer, the centrifuge shaker is an electro-hydraulic, servo-controlled device with dual actuators (Laak et al. 1998). Total maximum force capability of the simulator is 80 kN. Input motion is imparted (in-flight) in a direction parallel to the centrifuge axis of the RPI shaker.

Special miniature accelerometers, pore pressure transducers, and displacement gages were used as sensors. A PC-based data acquisition system was used to register the data coming from the sensors. During flight, three closed-circuit television cameras monitor the centrifuge, the soil model plan, and the soil model side through a window in the model container's wall. A rigid model container with inner dimensions of 0.88 (l) x 0.37 (w) x 0.36 (h) meters was used. A transparent side window allowed for observation of the side of the model. More detailed information about the geotechnical centrifuge modeling instrumentation used in RPI is given by Adalier (1996).

3. MODEL CONSTRUCTION AND TESTING PROCEDURE

Nevada-120 sand was used in all tests. This is a fine (d_{50} = 0.15mm), uniform, sub-round, clean sand. Extensive data about the monotonic and cyclic response characteristics of this soil has been documented by Arulmoli et al. (1992) and available on the WEB at http://geoinfo.usc.edu/gees/velacs/. The embankment core was made out of kaolin clay compacted at around 32% water content (2% to wet side of optimum). It had a dry unit weight of 13.8 kN/m^3 and unconfined shear strength (Su) of 18-20 kPa. Soil model was build by air pluviation (Adalier 1996). Pluviation was interrupted periodically to place instrumentation. Thin bands of colored Nevada sand were placed at the interface of each horizontal foundation layer. Thin spaghetti sticks were then inserted vertically (driven in a steel tube casing), at predetermined positions. When softened by the pore fluid these sticks acted as inclinometers and made it possible to measure the internal deformations during model dissection. The models were saturated under vacuum. Metulose solution of 50 times water viscosity was used as the foundation soil pore fluid. Considering the fact that the tests were conducted at a 100g gravitational acceleration field, and in view of the scaling laws applicable to centrifuge experiments, about a two times more permeable foundation sand (relative to 1-g water permeability) was simulated (still a fine sand permeability). Embankment shells and reservoir fluid was water rather than a viscous fluid to simulate a coarser embankment material (as mainly found in field cases).

The tests were conducted in a 100g gravitational field. The centrifuge was brought to 100g very gradually so as to allow pore pressures to buildup almost statically without causing any potential instability problems. Self-weight compression of the model was monitored in each test and found to be insignificant compared to those dynamically-induced during the subsequent shaking events. All models were subjected to the same sinusoidal base horizontal acceleration of 30-cycles, 0.2g magnitude, and 1.5 Hz dominant frequency (prototype).

After the test was complete, the specimen was dissected carefully, digital photographs were taken, and a detailed visual observation was conducted of the deformation of each model. The profiles of the models at dissected sections were mapped very carefully (transformed on a thin plexiglass plate) and then digitized into computer graphics. By this method, very detailed pre- and post-test mid-section profile meshes of each model was obtained.

4. TESTING PROGRAM

A total of 12 dynamic tests were performed on 12 different soil models. At a 100g gravitational acceleration field, the models depicted in Fig. 1 (a setup of a typical model) simulated a prototype earth dam of 10 m in height, 39.5 m in width, sitting on 9 m thick fine sand foundation deposit. Table 1 gives the summary of the conducted centrifuge tests. As seen in Table 1, mainly the effects of liquefiable layer I) thickness, ii) depth, and iii) location on the dynamic performance of the dam-foundation system were studied. In Series IV, the effects of longer earthquake shaking (40 cycles) and of a clay interlayer at 2.5-3.5 m depth on both downstream and upstream sides of the dam was evaluated. Full sets of horizontal accelerations, pore pressures, and deformations at different locations throughout the foundation-dam model were obtained for further analysis and interpretation. Additionally, these data are being studied numerically and used as a database for calibration and verification of several different FE codes or numerical schemes. Due to space limitation, only selected response records of Series I tests will be presented in the following sections and preliminary results will be briefly discussed. All of the test results are presented and discussed in prototype units, unless otherwise stated.

5. SERIES-I TESTS RESULTS - EFFECT OF LIQUEFIABLE LAYER THICKNESS

5.1 Model 1: LLL-LLL: Entire Foundation at D_r = 35%

Figure 2 depicts model response at selected transducer locations and post-test deformed shape of the model mid-cross-section (after dissection). The simulated earthquake caused very large deformations both in embankment and in the foundation. A huge embankment crest settlement of 2.4 m was measured. Most of this settlement took place uniformly during the course of the base excitation. Both sides of the embankment slumped and moved laterally away from the centerline on the excess pore pressure (EPP) softened foundation. The migration of underlying foundation soil towards the free field was largely responsible for the observed embankment slump. Movements as large as 4 m were observed near the toe areas. In general, the deformations on the downstream side were somewhat bigger than those of upstream side. This is mainly attributed to the higher initial static shear stresses in the downstream foundation. The lateral deformation in the foundation soil was found to attain its maximum near ground surface, and to decrease with depth. This deformation may be associated with an average accumulated normal lateral tensile strain of about 20% along the embankment base. Indeed, this tensile strain was clearly manifested in the form of stretching and slumping of the embankment body.

At the upstream toe-foundation, the acceleration response gradually decreased within 3-4 cycles of base excitation, reflecting the associated loss of soil stiffness and strength due to induced high EPP. However, after a few initial cycles, a very peculiar behavior of large asymmetric acceleration spikes started to appear. This asymmetric spiky acceleration response is associated with cyclic-mobility down-slope shear deformations (Elgamal et al. 1996). Notice that the direction of these spikes are in opposite directions in upstream (a3) and downstream (a10) sides, as the lateral movement, and the initial static lateral shear stresses, are actually in opposite directions. It is noteworthy that the core/crest accelerations were significantly attenuated relative to the base input. As will be discussed later, the crest motions were largely affected by the embankment and foundation sandy soil state of strength during shaking event.

At P3 and P12, the soil built up EPP corresponding to initial liquefaction. The buildup at the upstream side was somewhat faster. Due to very large shear strains induced in the foundation and associated dilation effect the foundation EPP (P6) did not reach initial liquefaction values. The estimated excess pore pressure ratio at P6 is around 0.7.

5.2 Model 2: LLD-LLD: Top 3m Densified Foundation

In this model the top 3 m of the entire foundation layer was densified to D_r= 70%. In practice, various ground treatment strategies can be used to mitigate the liquefaction hazards. The most common improvement techniques have been densification, soil replacement, and cementation. Among these, compaction or densification has been the most popular method for embankment foundation remediation projects. This model may be considered simulating a case where top 3 m of the foundation soil densified as a countermeasure or this kind of layering exist in nature.

Figure 3 depicts model response at selected transducer locations and the post-test deformed shape. Both the embankment and the foundation deformations were much reduced when compared with the previous model. Embankment crest settlement was 1.3 m (i.e., about half of that of Model 1). The pattern of deformations was considerably different than that observed in the first model, as the maximum shear deformations were shifted downwards to the 6-m loose layer. From the deformed mesh one may infer that the 3-m top dense soil plus the embankment basically translated sideways (without internally going into significant shear deformations) on the EPP softened 6-m loose underlying layer. As in the first model, the deformations on the downstream side were slightly bigger than those of upstream side. The average accumulated normal lateral tensile strain along the embankment base was about 6.5% (compared to 20% in the Model 1). The accelerations at foundation mid-depth (a3 and a10) were both somewhat attenuated relative to base input. Also the asymmetric spiky response phenomenon was much less significant (compared to Model 1) as the lateral deformations were reduced. Clay core crest accelerations, although attenuated relative to base input, were higher than those of Model 1. This is attributed to overall stiffer and stronger foundation material surrounding the core. Both P3 and P12 at the mid-depth (in loose layer) measured EPP corresponding to initial liquefaction. P6 under the embankment also measured EPP values corresponding to initial liquefaction. Contrary to the Model 1 case, reduced overall lateral spreading and dilation effects helped the foundation soil to build up higher EPP.

5.3 Model 3: LDD-LDD: Top 6m Densified Foundation

In this model the top 6 m of the entire foundation layer was densified to D_r= 70%. Figure 4 depicts the Model 3 response at selected transducer locations and post-test deformed shape of the model cross-section. As seen, embankment crest settlement was a little less than the one measured in Model 2 at about 0.8 m. Likewise the embankment and foundation internal deformations were further reduced by the increase in densified layer thickness from 3 to 6 meters. The maximum shear deformations were observed in the base 3-m loose layer. As in the previous two cases, the downstream deformations were somewhat larger. The average accumulated normal lateral tensile strain along the embankment base was about 4.5% (compared to 20% in Model 1).

The acceleration response of a3 and a10 were significantly stronger than those observed in Model 1 and 2, as these transducers were in dense soil. Asymmetric acceleration behavior was less significant. Clay core crest accelerations were significantly larger than the ones measured during Model 1 and 2 tests. Despite the fact that P3 and P12 were located in the dense zone, they measured EPP corresponding to initial liquefaction. Their position being close to the underlying loose layer probably helped the EPP buildup at these zones.

5.4 Model 4: DDD-DDD: All Foundation Densified

In this model, the entire foundation layer had a D_r= 70%. It is noted that, in this case, a calibration error resulted in an increase of about 20% in the peak input shaking accelerations. Yet, the embankment and foundation deformations were reduced drastically (Fig. 5). The pattern of internal deformations were somewhat different than those observed in Model 2 and 3 tests, as they were more uniformly distributed through the model height in this case. The crest settled about 0.8 m uniformly during shaking. The average accumulated normal lateral tensile strain along the embankment base was about 4.0%. The foundation deformations were largely lateral with essentially negligible vertical component, as the dense soil did not contract under cyclic shear.

The EPP buildup was overall somewhat slower; however the dissipations were faster (related to the smaller bulk modulus of the dense material) than those observed in the other model tests. Despite the high EPP, due to dilative characteristics of the initially dense soil, stiff response prevailed throughout the foundation (as suggested by the strong accelera-tions). It should be noted that dense sand, even liquefied, does not deform excessively like loose sand due to its high residual shear strength and dilative behavior arresting large strain increments. Dilative acceleration spikes were apparent at records a3 and a10, helped by the higher base input motion and higher soil density. The relatively high overall foundation sandy soil stiffness increased the effective confinement effect on the clay core resulting in relatively strong core-crest accelerations. It is interesting to note that, the densified foundation greatly reduced the earthquake induced deformations but at the same time increased the embankment accelerations (due to the ability of dense sand in transmitting shear stress through dilative behavior). In this respect, for the dams with sensitive appurtenant structures one may consider Model 3 countermeasure case as a viable option.

6. OVERALL RESPONSE

Figure 6 depicts the measured normalized (relative to Model 1) dam crest settlements, average dam base lateral tensile strains, maximum foundation settlements, and normalized (relative to base input motion) crest acceleration arias intensities for the four models. Densifications to all depths are all found to reduce the embankment and foundation deformations by a range of 50% to 90%. However, the gain in deformation parameters has diminished after 6 m deep densification. The biggest gain was achieved from 3 m densification. Deeper densifications (i.e., 6 and 9 m) further reduced the deformations however the improvement was at a diminishing rate. On the other hand, stiffer foundations (denser material) have resulted in stronger embankment accelerations. The variation of dam settlement and acceleration (for the cases that the crest accelerations may be important, e.g., dams with various superstructures or auxiliary systems) with densification depth (Fig. 6) suggest it may be necessary to optimize the treatment depth to reduce the dam settlement and lateral spread to an acceptable level while at the same time ensuring that the dam accelerations are tolerable.

7. SUMMARY AND CONCLUSIONS

Through a series of highly instrumented dynamic centrifuge tests the effects of loose foundation layer i) thickness, ii) location, and iii) depth on the dynamic behavior of a zoned earth dam with liquefiable foundations were investigated. Results of some of these tests were briefly presented herein. The study provided many valuable insights into the dynamic behavior of earth dams sitting on alluvial soils subjected to moderate earthquake shaking. Moreover, these test results are providing a very valuable database for the calibration and verification of several numerical analysis codes and procedures modeling the dynamic behavior of liquefiable foundation-earth dam systems.

Test results suggest that there may be an optimum depth of densification treatment beneath an earth dam beyond which the reduction of the earthquake-induced deformations is relatively minor. The tests results also indicate that relatively small and isolated zones (e.g., at depth) of loose material within a densified volume of soil may not impair the overall effectiveness of treatment and do not necessarily result in damaging displacements. This suggests that the remedial designs should be based on displacement criteria rather than on the factor of safety against liquefaction. The difficulty in such cases, however, is in determining the acceptable size and distribution of such zones. Further centrifuge modeling and/or calibrated numerical parametric studies in this respect will be very useful. As shown by this program, the results of such tests would be extremely effective in the development of verified design guidelines, as well as in calibration of computational procedures.

8. ACKNOWLEDGEMENTS

The authors would like to acknowledge the Corps of Engineers, Headquarters for their support of this research.

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Table 1. Centrifuge testing program.

Centrifuge Test Series	**Model Configuration** 1DS-2DS-3DS—1US-2US-3US D: 70% D_r; L: 35% D_r
Series I- Effect of liquefiable layer ***thickness***	L-L-L—L-L-L L-L-D—L-L-D L-D-D—L-D-D D-D-D—D-D-D
Series II- Effect of liquefiable layer ***depth***	D-D-L—D-D-L D-L-D—D-L-D L-D-D—L-D-D
Series III- Effect of liquefiable layer ***location***	D-D-L—D-D-D D-L-D—D-D-D D-D-D—D-D-L D-D-D—D-L-D
Series IV- Clay Interlayer and Large Earthquake	L-L-Clay-D—L-L-Clay-D D-L-D—D-L-D

D_r : Relative Density; DS : Downstream; US: Upstream

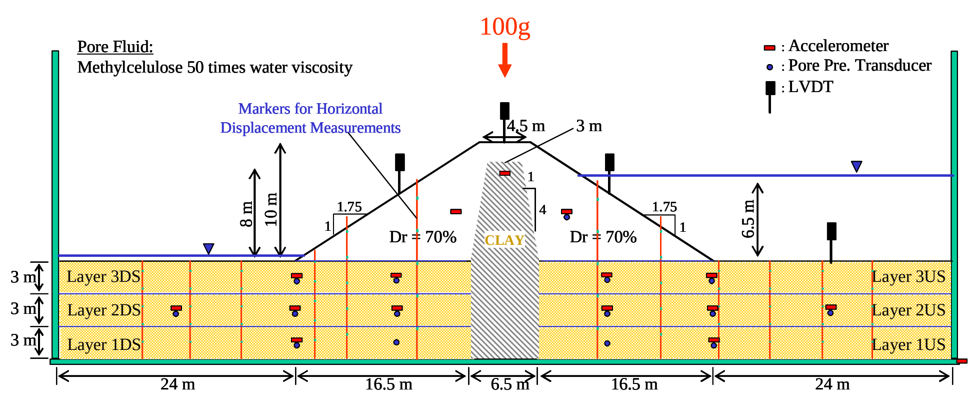

Fig.1 Schematic of a typical dam-foundation model.

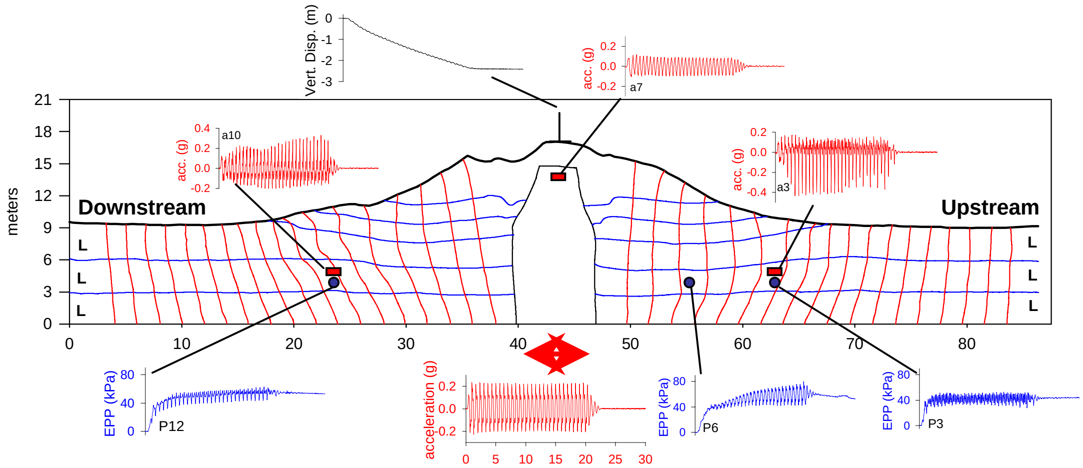

Figure 2. Model 1 selected transducer data and post-shake mid-cross-section.

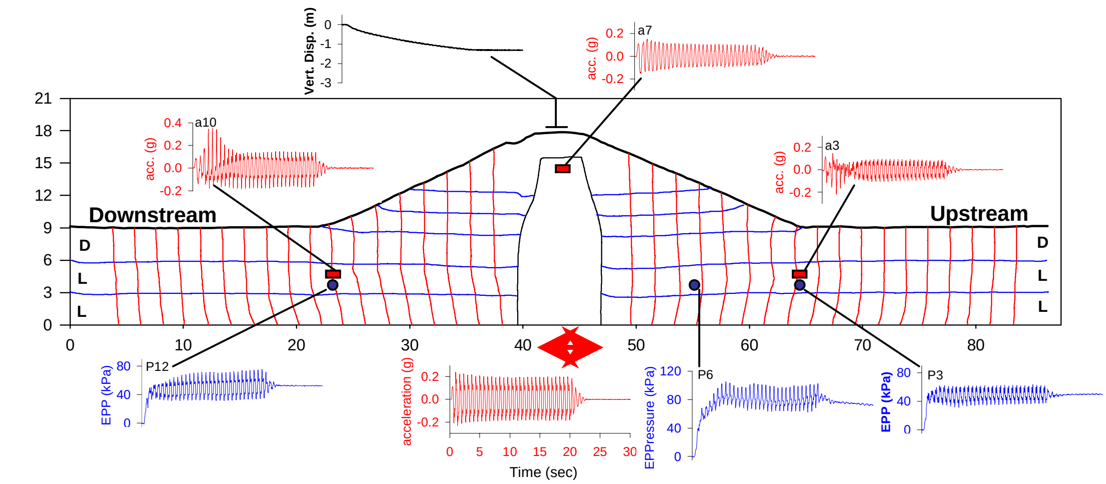

Figure 3. Model 2 selected transducer data and post-shake mid-cross-section.

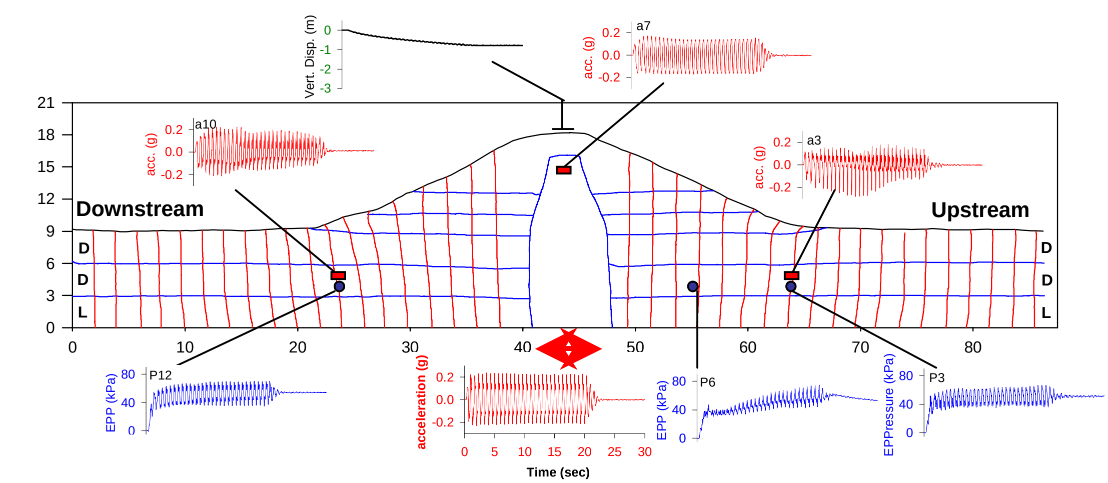

Figure 4. Model 3 selected transducer data and post-shake mid-cross-section.

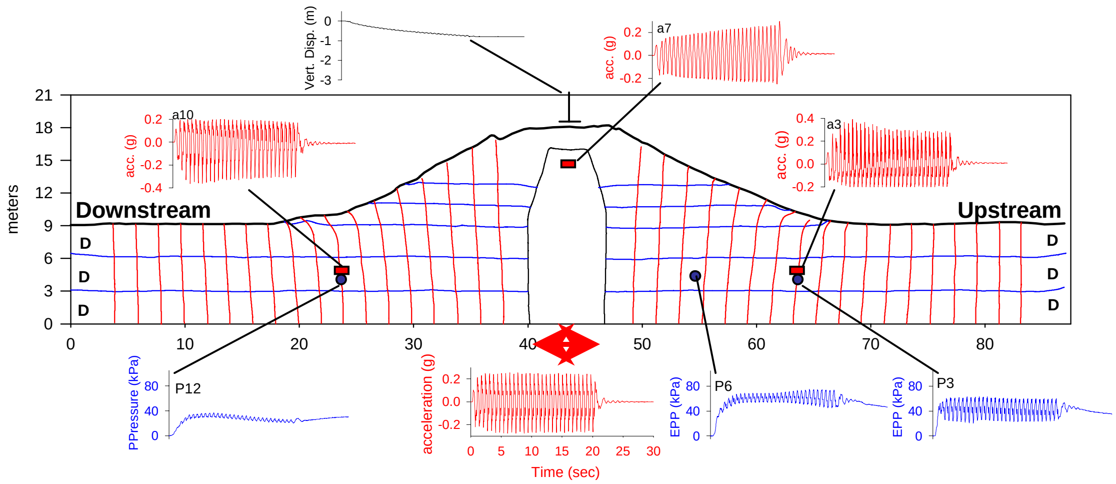

Figure 5. Model 4 selected transducer data and post-shake mid-cross-section.

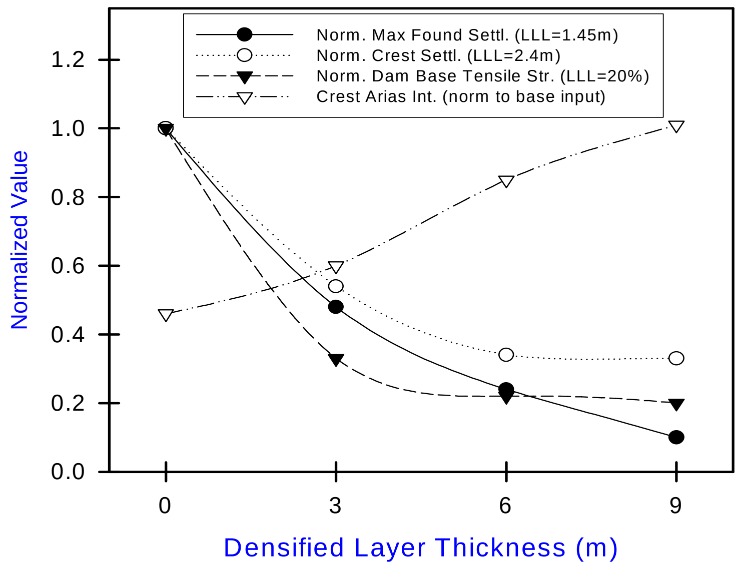

Figure 6. Some key dynamic performance parameters for the Models 1-4.