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Dynamic centrifuge tests of concrete dam

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SUMMARY

Dynamic tests of a concrete gravity dam are, for the first time, performed inside a centrifuge. Details of the experimental procedure, data interpretation, and results are presented. It is shown (in conjunction with a parallel paper) that these tests cannot only provide a direct assessment of certain aspects of dam safety, but more importantly provide a data base for possible non-linear finite element code validation. Copyright © 2005 John Wiley & Sons, Ltd.

KEY WORDS: dams; earthquake; centrifuge

1. INTRODUCTION

There are over 78 000 dams in the United States. Of these approximately 10 000 are considered to have ‘high-hazard’ potential meaning their failure could result in loss of life or severe property damage [1]. This particular concern was preceded by the National Dam Safety Program Act [2] which stipulated that a national program of inspection of dams for the purpose of protecting human life and property should be undertaken as soon as practicable. The program should determine whether a dam constitutes a danger to human life or property, and shall take into consideration the possibility that the dam might be endangered by cracking. Hence, one of the program objectives is to ensure that new and existing dams are safe through the development of technologically and economically feasible programs and procedures for national dam safety hazard reduction.

Given this public awareness on dam safety, there will be a greater reliance on computational models to efficiently assess dam integrity when subjected to seismic loads. However, numerical codes are often difficult to validate with actual structural failures, and hence laboratory model testing is essential to provide a data base for possible validation. Laboratory testing of structures where gravity plays a dominant role (such as in gravity dams) can only be performed in centrifuges.

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Centrifuge tests as a tool to provide a database of experimental soil results from which to calibrate/validate finite element codes is not new in geotechnical engineering. Most notable example was the verification of liquefaction analysis by centrifuge studies (VELACS) program where centrifuge models were selected to supply boundary value problems with well-defined boundary conditions and soil properties for ‘before the event’ predictions using a variety of numerical codes [3]. However, such data base is certainly not yet available for concrete gravity dams.

It is notorious that a linear elastic dynamic analysis will grossly over-estimate the stresses as it does not take into account the stress redistribution caused by joint openings, and cracking. Hence a serious dam seismic investigation must be a non-linear one. Indeed, a U.S. Army Corps of Engineers [4] states that: The analysis based on non-linear material behaviour represents the greatest possible refinement and it produces the most accurate results. However, it is also the most complex and the most costly. It requires time–history ground motion input, direct integration solution, a large main frame computer, specialized computer programs, and a considerable amount of computer time. As such, it is the last recourse in the attribute refining process. The non-linear analysis should only be undertaken under the guidance of an expert in the field of fracture mechanics and finite element methods.

The premises of this 1995 document remain acutely true, with the notable exception that main frame computers must be used (3D non-linear dynamic analysis can now be performed on personal ones). Yet it is nearly impossible to assess how accurate those non-linear analyses are. Surely, most of them begin with an analysis of Koyna or Morrow point dams and compare their results with the ones of other researchers, occasionally there is a comparison with shaking table tests, such as those performed by the Bureau of Reclamation [5]. However, for all practical purposes, no two non-linear analysis will yield the same results. There are two major reasons for this discrepancy. The first one is at the modelling level. There are so many factors to be accounted for in a truly and completely non-linear analysis, that few analysis focus on the same model assumptions. Some will ignore foundation joints, other will ignore the foundation mass, other may not account for dynamic uplift; hence given the range of modelling variable there is a wide spectrum of possible modelling assumptions. The second source of difference is a more subtle one. Given the same modelling assumptions different analysis may make different assumptions for a given model. Joints for instance, even if they were the only source of non-linearity in an analysis, they can be represented by a variety of models ranging from the simplest variation of Goodman’s element [6] to the most sophisticated ones based on non-linear fracture mechanics [7]. Finally, in the context of a dam analysis, the non-linearity stems almost exclusively from joint openings and closing, and from crack nucleation and propagation.

Hence, if it is practically impossible to assess a non-linear analysis by mere comparison with others (other than comparing orders of magnitudes), and (fortunately) there are very few seismic induced concrete dam failures to use as a benchmark. Furthermore, there is great risk in continuously comparing (or worst yet calibrating) new analysis with previous ones which may have been performed by ‘reputable’ codes. As it is very much possible that the first analysis may have some hidden errors associated with certain deficiencies and the new ones would look at it as the standard by which its accuracy must be evaluated. This will result in a perpetuation of an error, not unlike a numerical instability.

Accordingly, one must resort to an evaluation of a non-linear model through the analysis of an equally complex laboratory tests. Laboratory dam tests have for the most part restricted themselves to 2D shaking table ones. Yet, and specially for gravity, a laboratory test to be minimally exact and accurate must satisfy all the laws of similitude, at least the most relevant ones. For gravity dams, the most essential one is the scaling of gravity which must be inversely proportional to the reduction in size from prototype to model to keep the stresses equal. Hence, one can either perform tests at $1g$ but with a concrete with an artificial density (and may thus violate other important scaling factors), or perform tests inside a centrifuge.

Not surprisingly, very few centrifuge tests of concrete dams were undertaken. To the best of our knowledge, only three set of experiments have thus far been undertaken. The first one, within the context of an Electric Power Research Institute (EPRI) funded project, the third author tested gravity dams impounded with water, and then assessed a fracture mechanics based analysis with the experimental results [8, 9]. Subsequent tests were performed at ISMES [10] on a smaller specimen with water trapped inside a rubber container on the upstream face, and most recently, through a work funded by the Tokyo Electric Power Service Company (TEPSCO) the third author performed additional centrifuge tests on dam models with uplift measurements [11].

Furthermore, if there has been only three sets of static centrifuge tests of concrete dams, there has never been a dynamic one as this requires a very sophisticated, complex and expensive equipment (a shaking table mounted on a centrifuge capable of applying a dynamic excitation during a fraction of a second).

Hence, this paper presents result of a concrete gravity dams excited by a shaking table mounted inside a centrifuge. In order to simulate cracking, relatively low concrete strength had to be selected in this otherwise very complex test.

Finally, it should be emphasized that the objective of these tests is not to assess the safety of one particular structure, but rather to perform a test of a generic dam which is as representative as possible of this type of gravity dams (common in Japan). The test results in turn should be used to assess the capability of non-linear finite element programs to capture the observed response.

2. CENTRIFUGE TESTING

As reported by Ko [12], centrifuge modelling had been proposed for testing bridge models as early as 1869. However, the concept of using a centrifuge to replicate the effects of body forces in scaled earth structure models was not realized until circa 1930 when, independently, Bucky *et al.* [13] in the United States, and Pokrovskii and Flodorov [14] in the USSR, began using centrifuges for this application. This pioneering work paved the way for numerous other research studies, and currently there are numerous geotechnical centrifuges around the world.

Due to the complete absence of additional centrifuge tests on concrete structures since the earlier work of the third author, the reader is referred to Reference [9] for a detailed literature survey and a basic description of the physics of centrifuge tests.

With regard to the specific work reported in this article, a few additional considerations are warranted. As for soils, one is confronted with two possibilities: (1) the model is composed of exactly the same concrete as in the prototype; or (2) a different material is used. In either case, one can model a model (i.e. testing same prototype at two different scale models).

If one were to accept, the first approach, then Table I summarizes the major relevant scaling parameters inside a centrifuge.

Table I. Various commonly used scale factors in centrifuge testing [15].

Quantity	Prototype	Model at λ–g
Length	1	$1/\lambda$
Time-dynamic events	1	$1/\lambda$
Velocity	1	1
Acceleration	1	λ
Mass	1	$1/\lambda^3$
Force	1	$1/\lambda^2$
Energy	1	$1/\lambda^3$
Fracture energy	1	$1/\lambda$
Pressure	1	1
Stress	1	1
Strain	1	1
Density	1	1
Frequency	1	λ

As new ground is being broken in the area of concrete testing in a centrifuge, there are a three major concerns which should be addressed.

First, whereas in geotechnical applications one can relatively easily use the same soil for both prototype and model, in concrete this is clearly not possible. The model can only have smaller maximum size aggregates (MSA) than in the dam. Such a problem is not new, and the practice of ‘wet-sieving’ in core tests (process by which aggregates larger than the appropriate size for testing) is common, codified by ASTM [16] and used to determine from laboratory tests elastic properties for dam concrete with very large aggregates (up to 15 cm).

The second major concern is that whereas strength (stress) and strain scale as 1 to 1, fracture energy G_F will scale as $1/\lambda$. Hence, in theory the fracture energy (a material property) should be λ times smaller in the centrifuge model than in the dam model. *a priori*, this would call for a special concrete mix which should have the same Young’s modulus, and tensile strength as the prototype concrete, but a much smaller fracture energy. It should be noted, that a similar problem has already been raised by Palmer and Rice [17] in the context of slope stability problems.

However, closer look at the problem will immediately reveal that by having already reduced the aggregate size by a factor of λ, (tantamount to ‘wet-sieving’), we have effectively reduced the size of the fracture process zone (along which bridging aggregates can transmit cohesive stresses), and thus we have also reduced the fracture energy [18]. Furthermore, while we have scaled down the aggregate sizes (and thus the fracture energy), there is no certainty that the fracture energy is properly scaled unless fracture energy tests of dam concrete are conducted. This is practically impossible, and just as we accept strengths based on ‘wet-sieved’ specimens, we should accept the $1/\lambda$ fracture energy scaling (albeit with some reservations).

In summary, it can be reasonably assumed that the centrifuge concrete is identical to the prototype concrete to the extent this is practically possible, and as accepted in other similar tests codified by ASTM.

The third and last concern is size effect. It is by now well accepted that concrete cracking is size-effect dependent, whereas soil may not be. Saouma and Barton [19] have shown that centrifuge test results should be adjusted for size effect before proper extrapolation is done (through a process which remains much in dispute in the concrete community).

3. CENTRIFUGE DESCRIPTION

The centrifuge used in the experiments has a 700*g*-ton capacity, Figure 1. The arm radius is 7.01 m for static tests, and 6.86 m for dynamic ones. The shaking table platform is

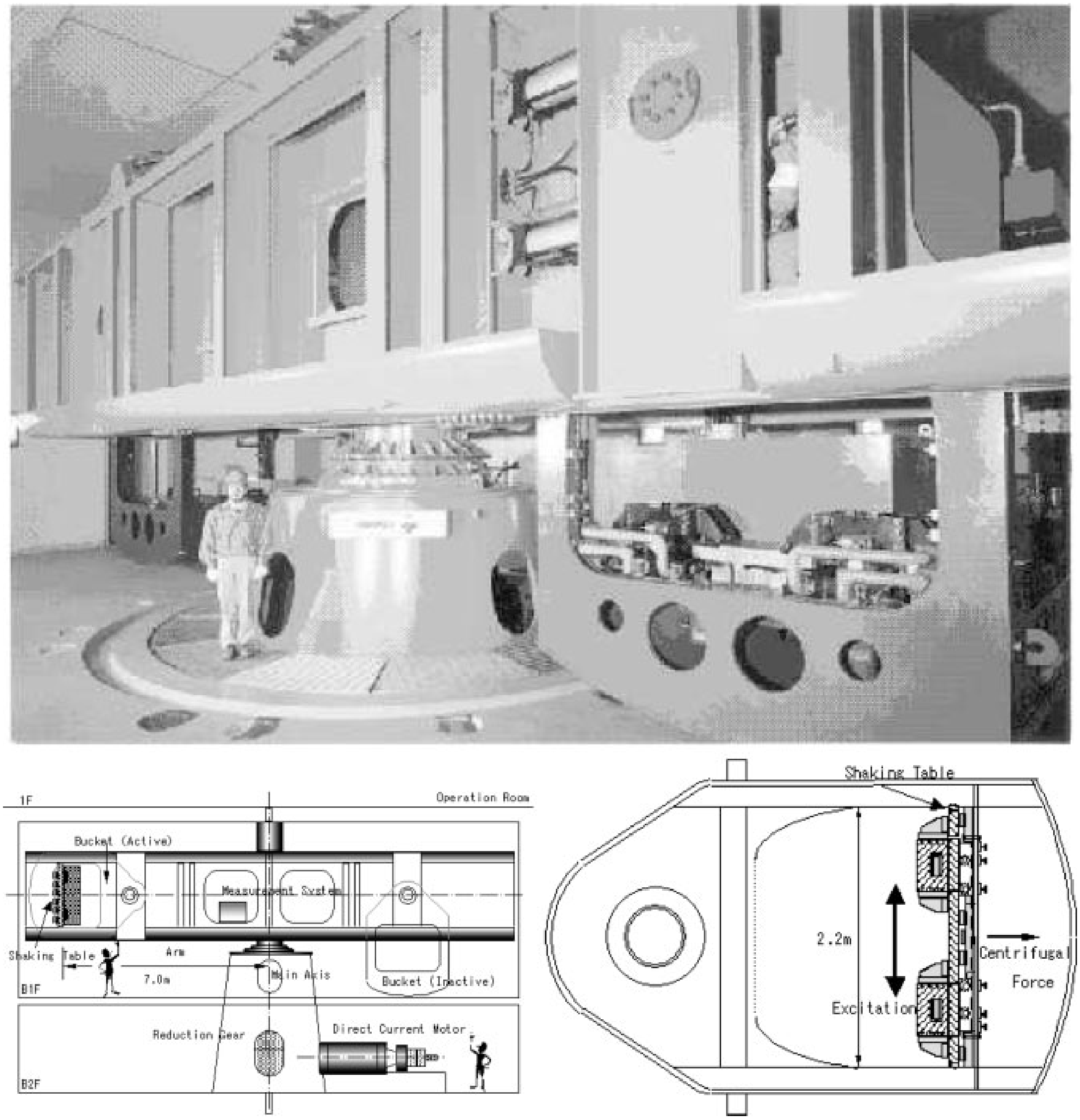


Figure 1. Obayashi's centrifuge used in reported tests.

2.2×1.07 m, and can subject the model to a maximum dynamic acceleration of $50g$ with a maximum shaking force of 120 tonf, and a maximum frequency of 200 Hz.

Dam specimens were installed inside a specially built $40 \times 150 \times 85$ (w × l × h) steel container. In order to maximize elastic wave dissipation, the sides of the container to be impounded were plastered with a special flexible steel grid selected on the basis of previous tests on embankment dams.

4. TEST NO. 1

4.1. Specimen design

Given the extensive and expensive preparation of a dam model, for a test which lasts about 0.1 s, great care was exercised in planning the test. The initial constraint on the model design were the platform size, the shaking table maximum frequency, and the need to simulate crack propagation. Hence, numerous non-linear time history analyses were performed on various geometries, elastic moduli, initial crack location and orientations, and support conditions with the code Merlin [20]. Natural frequencies, crest accelerations, crack propagation and potentials for crack nucleation were thoroughly examined. In the end, a $\frac{1}{30}$ model test of a 18 m high prototype dam (without foundation), with $E = 15\,000$ MPa, and an excitation frequency of 167 Hz was selected, Figure 2. The presence of heavy reinforcement inside a shallow 10 cm high 'foundation' to properly secure the dam into the shaking table must be noted.

The model without reservoir had a natural frequency of 464 Hz, and with the reservoir (based on Westergaard's added mass) 332 Hz.

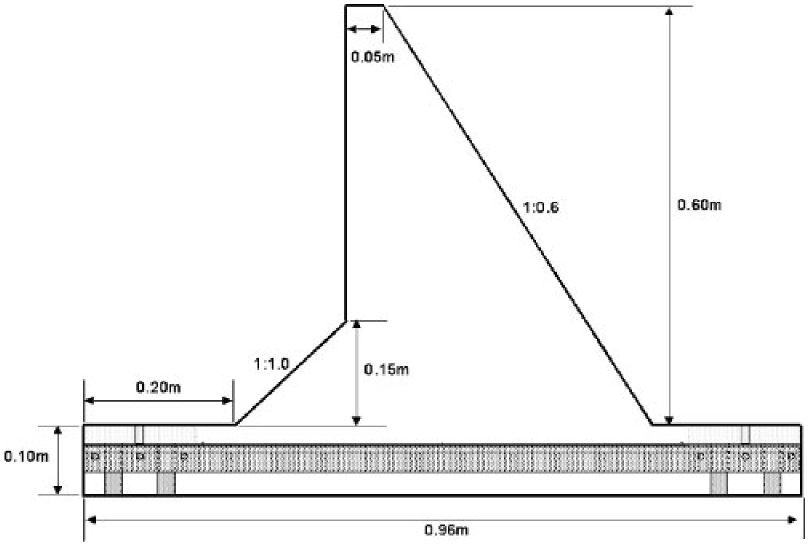


Figure 2. Dam model section retained in Test Series 1.

Table II. Elastic properties of concrete.

Days	f_c' (MPa)	E (GPa)	f_t' (MPa)	ν	G_F (N/m)
Preliminary test (C.T. 20°C)					
7	5.5	3.9	0.7	—	51.3
14	7.0	10.2	0.9	0.18	51.8
21	9.6	12.0	1.1	0.22	77.5
28	12.5	13.2	1.4	0.18	99.3
Test # 1 (C.T. 12°C)					
14	7.9	10.4	0.9	0.17	—
20	9.3	10.6	1.0	0.18	—
Test # 2 (C.T. 7°C)					
14	6.1	7.2	0.7	0.17	—
20	7.7	7.4	0.8	0.15	—

4.2. Mix design

Mix design, should ideally yield compressive strength and elastic modulus close to the one of the prototype. However, since the primary objective of the test was to trigger failure (cracking) of the model through a harmonic excitation, additional constraint were imposed. Beginning with the maximum frequency of the shaking table of 200 Hz, it was decided to excite the dam at 80% this frequency or 167 Hz ($T = 0.006$ s). Through preliminary calculations it was determined that normal concrete strength (say about 30 MPa), would have resulted in a model too rigid to crack by the shaking table, and hence the concrete strength was lowered to a relatively low value. Hence, a preliminary target value of 15 GPa for the Young's Modulus and 10 MPa for the compressive strength was set. Using fine aggregate from an actual dam site (maximum size set to 5 mm which correspond to $\frac{1}{30}$ of 150 mm used in actual dam), three sets of mixes were tested and evaluated, Table II (where C.T. stands for curing temperature). Because of the lower curing temperature (specimens were cured inside an external tent), the target value for E was then dropped to 10 GPa. Finally, the retained mix had a water to cement ratio of 70%, 6.5% air content (water, cement, and sand: 250, 357, and 1562 kg/m^3, respectively), and appropriate additives for water reduction and air content increase. Compressive strength, elastic modulus, and Poisson ratio were tested at 7, 14, 21 and 28 days on 100×200 mm cylinders, tensile strength on 150×200 mm cylinders, and fracture energy G_F on $100 \times 100 \times 400$ mm prisms.

4.3. Loading

Specimens in the first test series (a second one was performed to correct pitfalls encountered during the first originally planned) were loaded in the following sequence. First the centrifuge was ramped up to 30g in 15 min, then a series of excitations (10 000, 20 000, 24 000, 28 000, 32 000, 36 000 and 10 000 gal) all at 167 Hz were applied. There was a 30 min of constant 3000 gal vertical acceleration, and no horizontal ones between each series of excitation (total test duration was approximately 4 h). This provided time for the evaluation of the transfer functions, and visual inspection (through the CCD camera) of the dam integrity. Finally, the entire test series was preceded and concluded by white noise (sine-sweep) excitations at 3000 gal to identify possible damage. Each excitation series consisted of five cycles to linearly ramp up the model followed by five cycles of constant peak harmonic excitation. In the first test series the dam was not impounded.

4.4. Instrumentation

Instrumentation for test series one consisted of the measurements of, Figure 3:

Accelerometers: Three placed on the crest, one at the base, and two on the supporting platform, each recording the accelerations in the three principal directions.
Displacements: Two recorded the crest and base displacement through laser instruments with an accuracy of 3 μm, a range of ±15 mm, and sampling each 1024 μs.
Strain gages: Eleven were extensively used on both the upstream and downstream face of the dam.
Crack opening displacements: were also recorded using a specially designed instrument (cantilever type) with an accuracy of 0.4 μm, and a range of $\simeq 2$ mm.
Crack gages: Three were placed on one side of the dam along a potential crack trajectory previously determined from the finite element simulation.
CCD camera: Three were used to monitor the upstream and downstream faces of the specimen during test.

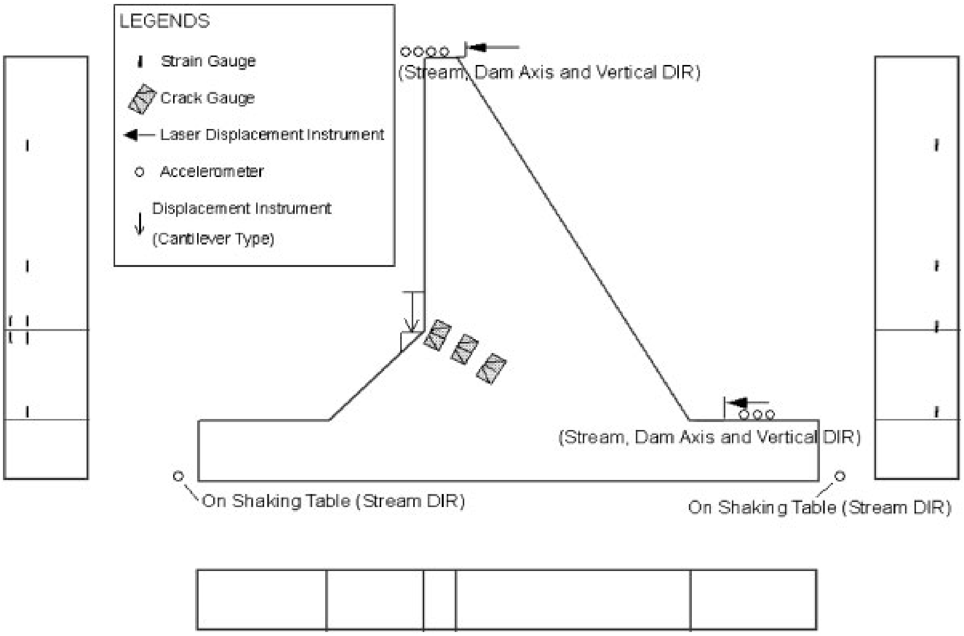


Figure 3. Instrumentation layout in Test Series 1.

Table III. Target and recorded ‘ground’ accelerations (m/s^2).

Series	Target	Recorded
W1	30	24
S1	100	145
S2	200	295
S3	240	325
S4	280	400
S5	320	464
S6	360	423
S7	100	127
W2	30	39

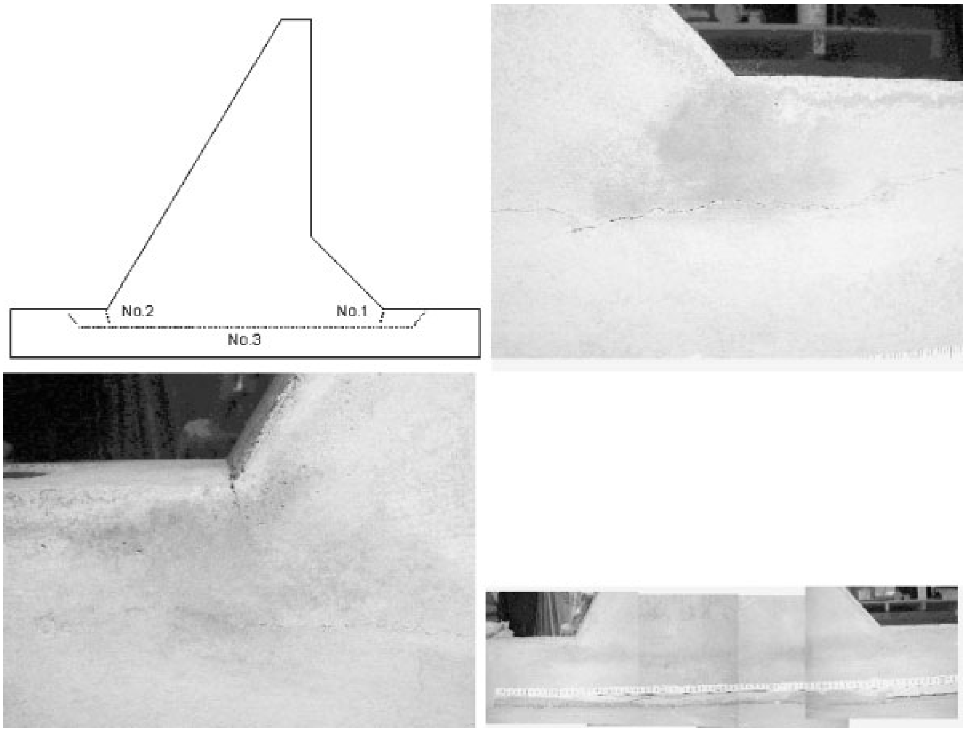


Figure 4. Cracks in Test 1.

4.5. *Test results*

Before test results are presented, Table III illustrates the discrepancies between the target accelerations and the recorded one (on the foundation).

Due to the experimental complexity, some discrepancy between target and recorded acceleration were anticipated, it was felt that in this preliminary test it was unusually high. This was subsequently explained upon specimen removal, and the realization that unanticipated foundation cracks had developed thus distorting some of the experimental results, Figure 4.

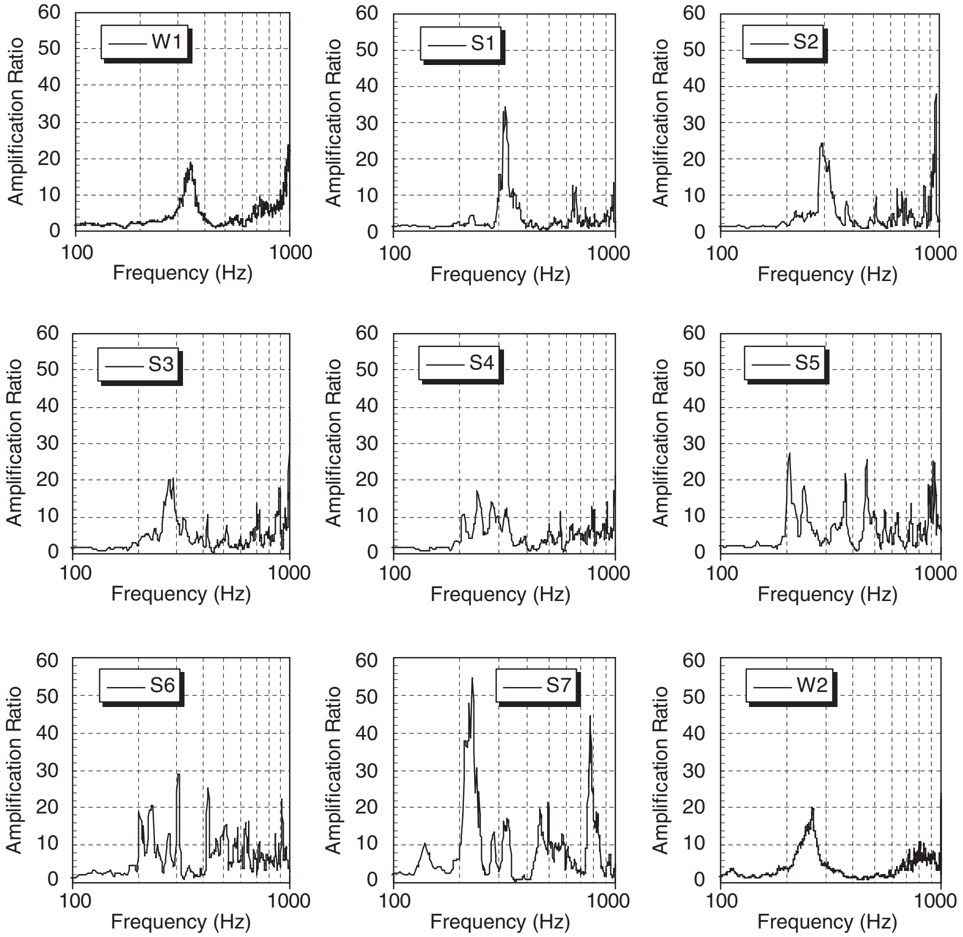


Figure 5. Transfer functions between base and crest along the stream direction for Test Series 1.

Whereas cracks often occur at the dam–foundation line, their inclusion in the centrifuge model would have included yet another major complication to the test.

It was determined that this longitudinal crack in the foundation was caused by the heavy reinforcement of the foundation which was deemed necessary to bolt the model to the steel plate base itself bolted to the shaking table.

Results were primarily assessed through the evaluation of transfer functions between base and crest along the three directions. The most important set, along the stream direction, is shown in Figure 5.

Two major observations can be made. First there is a net left shift of the first mode suggesting a stiffness decrease which was later attributed to the unanticipated crack formation. Also we observe a gradual overall decrease of the amplitudes accompanied by an increase in non-negligible spikes. Again this is an indication of a complex response caused by extensive internal cracking.

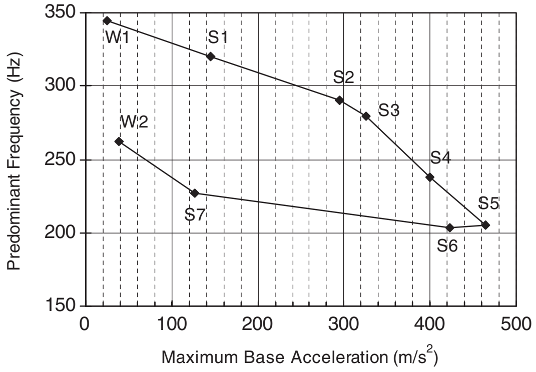


Figure 6. Variation of first modes frequencies of transfer functions in terms of maximum foundation acceleration.

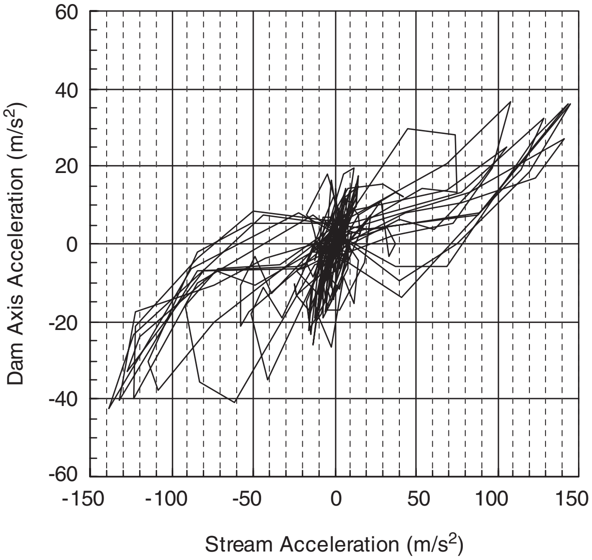


Figure 7. Stream and dam axis base accelerations.

Figure 6 in turn shows the variation of the first mode frequency from each of the nine sets of transfer functions in terms of the maximum foundation acceleration.

Figure 7 shows the stream and dam axis base accelerations in Test 1. In an ideal case, the acceleration along the dam axis should have been zero, yet from this figure we observe that the net shaking table acceleration imposed on the dam is skewed. This is attributed to possible phase lags between the four shaking table actuators.

Finally Figure 8 shows the ratio of crest over foundation accelerations in terms of maximum foundation acceleration in the stream direction.

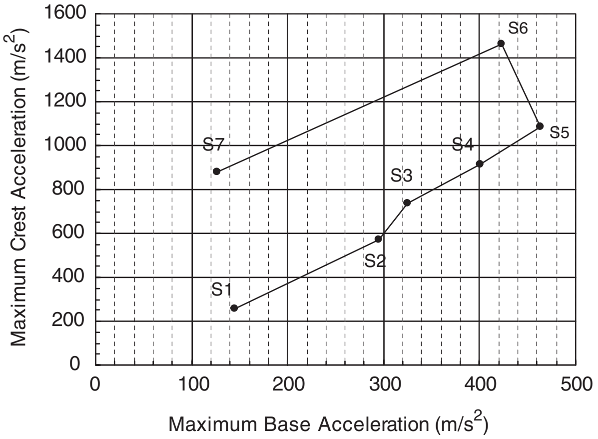


Figure 8. Ratio of crest over foundation accelerations in terms of maximum foundation acceleration.

Strain gages did not exceed 100 micro-strain (tensile strain). The gages measuring the crack mouth opening displacement gave very small values, and thus there was no indication that the initial notch extended as a crack through the specimen.

5. TEST SERIES 2

5.1. Description

Following this first test, it was apparent that major modifications were to be made. To avoid the parasitic foundation crack (which greatly perturbed the test/analysis correlation), its depth and width were increased from 10 to 15 cm and from 13 to 38.5 cm, respectively, Figure 9. In addition, to simulate crack propagation, a 1 cm notch was cut at the reentry corner on the upstream face. Furthermore, this next test was to assess the effect of water structure interaction and hydrodynamic forces (which can be approximated at best with the limitations imposed by the container size and material. Hence, the specimen container was retrofitted to accommodate a container which will be impounded by the water once the proper g level is reached. Whereas the instrumentation remained essentially the same, additional strain gage (on one side) and crack gages (on the other side) were placed along the anticipated crack trajectory determined through pre-analysis with Merlin (A TEPSCO/EPRI funded computer program for the dynamic non-linear analysis of dams [20]). In addition, two pressure transducers were used to monitor the hydrostatic pressure.

Finally, the loading procedure was identical to the first test with the notable exception of 'white-noise' excitation following each harmonic excitation (as opposed to only two in the previous test). Anticipating failure at around 300 m/s^2, the first four harmonic aimed at maximizing test data in the pre-cracking dry range. This was followed by four harmonic on the wet (with hydrostatic pressure applied through a rubber membrane, i.e. no internal uplift) one, Table IV.

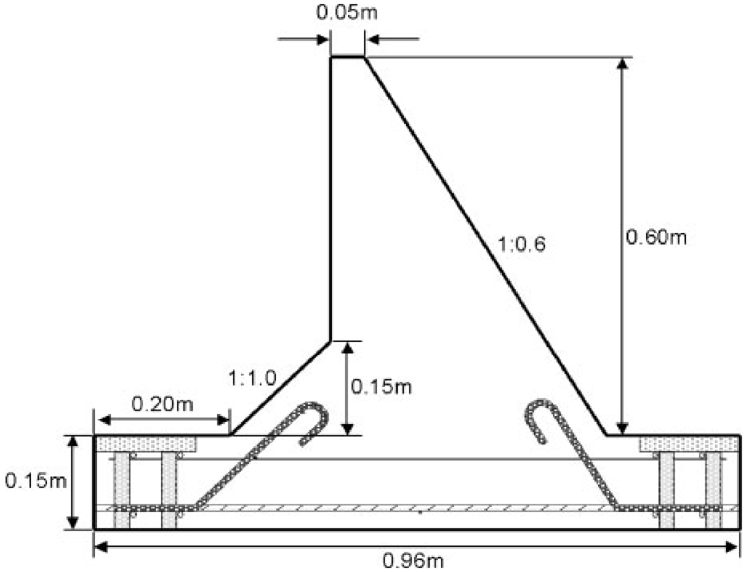


Figure 9. Dam model section retained in Test Series 2.

5.2. Test results

As in Test 1, the specimen was spun to $30g$ over a 15 min period, and then the dam was impounded at a rate of 1.57cm/min. When the water reached an elevation 5cm below the crest, the measured hydrostatic pressure was 154 kPa whereas the theoretical one was 162 kPa. This discrepancy may have been caused by the finite radius of the centrifuge, and the non-uniform g-field inside the container (not accounted for in our calculation).

Prior to the test, it was speculated that damage (possibly leading to either failure or test interruption) could be caused by: (a) Separation of the model from the base plate connected to the shaking table; (b) Cracking of either the foundation (as in the first test) or along the dam at the re-entry corner on the upstream face; or (c) Microcracking along the surface.

Following the test, two sets of transfer functions were closely examined: (1) those from base to crest, Figure 10; and (2) those associated with the harmonic excitation, Figure 11.

Close observation of those two figures reveal that on the one hand the transfer function at W4 is sharper than the one of W5, and on the other hand there is a subtle degradation from S1 to S5. This can be attributed to the formation of surface micro-cracks.

Cracking at S6 is confirmed by Figure 12which shows the strain gage readings along the potential crack. In particular, we note that ST19 was damaged (crossed by the crack) at S6, resulting in the negative offsets at S11 (wet) subsequently.

Estimating a cracking strain of about $100\,\mu\varepsilon$, cracking was detected by ST-20 at about 0.1 s. during the application of S6.

The peak intensities of the transfer function associated with first mode in terms of foundation acceleration are shown in Figure 14.

Table IV. Excitation history during Test 2.

Test		Target m/s^2	Recorded m/s^2
Dry			
White	W1	—	5
Harmonic	S1	50	41
White	W2	—	5
Harmonic	S2	100	84
White	W3	—	7
Harmonic	S3	200	184
White	W4	—	7
Harmonic	S4	300	251
White	W5	—	8
Harmonic	S5	400	336
Crack occurrence			
White	W6	—	7
Harmonic	S6	450	450
White	W7	—	8
Wet			
White	W8	—	7
Harmonic	S8	40	50
White	W9	—	6
Harmonic	S9	120	86
White	W10	—	6
Harmonic	S10	200	139
White	W11	—	6
Harmonic	S11	400	360
White	W12	—	7

There was an indication of noticeable cracking, as indicated by the strategically positioned strain and crack gages, around S5. On the other hand, because of the membrane, no CMOD measurements were made. As with the first test, the bulk of the data interpretation is through analysis of the transfer functions.

Two major cracks have developed, one (anticipated) at the sharp corner of the upstream face, and the other at the intersection of the dam with the foundation on the downstream side (which progression was undoubtedly restrained by the internal reinforcement), Figure 13.

Again, there was an asymmetry in the crest acceleration, Figure 15shows the stream and lateral acceleration. Thus, despite the foundation stiffening, the parasitic lateral acceleration persisted in Test 1 (Figure 7).

The maximum crest (stream) acceleration in terms of the maximum base acceleration is shown in Figure 16.

Finally, there was indication that there was a separation of the foundation with respect to the base plate.

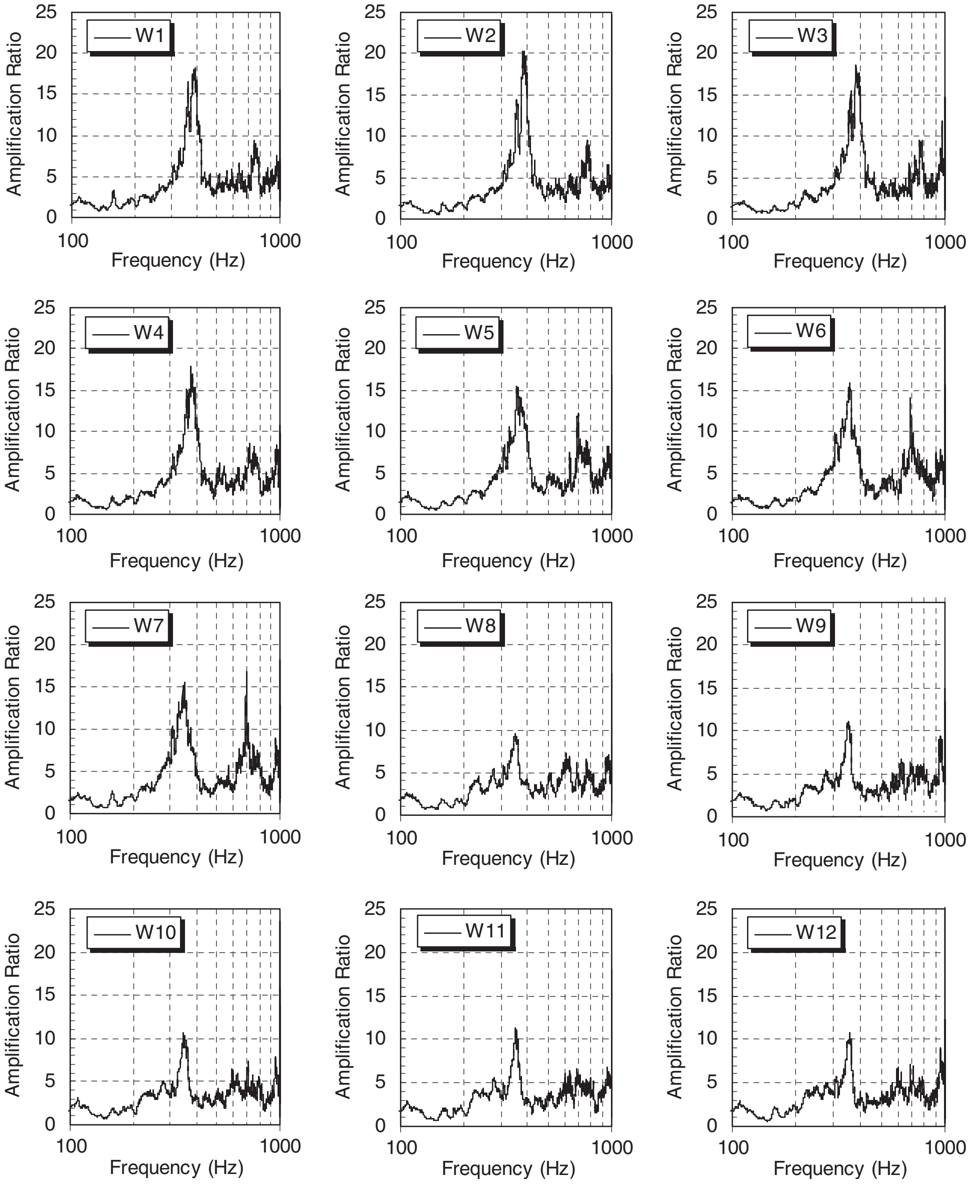


Figure 10. Transfer functions from base to crest in Test 2.

6. CONCLUSIONS

To the best of the authors' knowledge, this is the first shaking table test of a concrete dam mounted on a centrifuge. The complexity of this test limited us to only two tests.

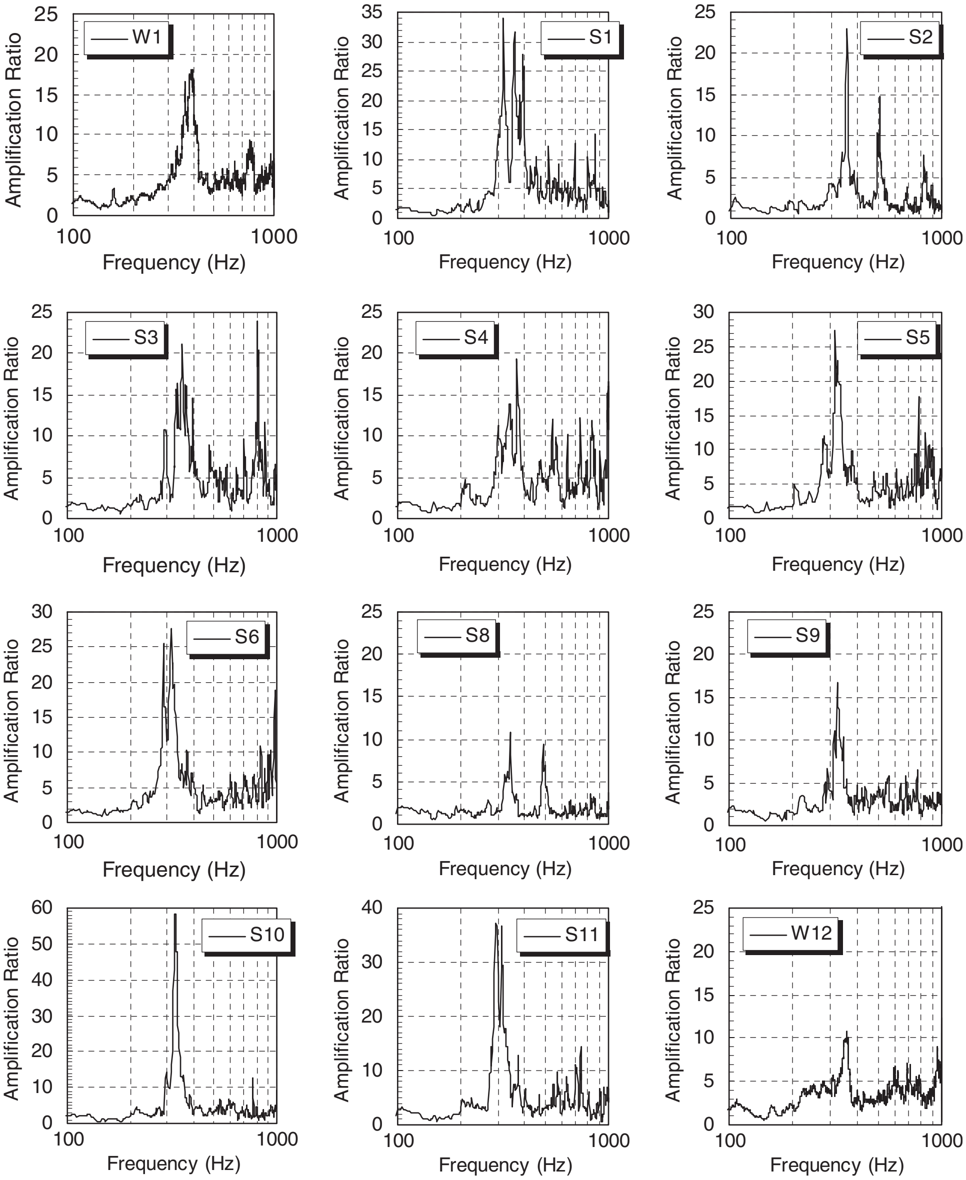
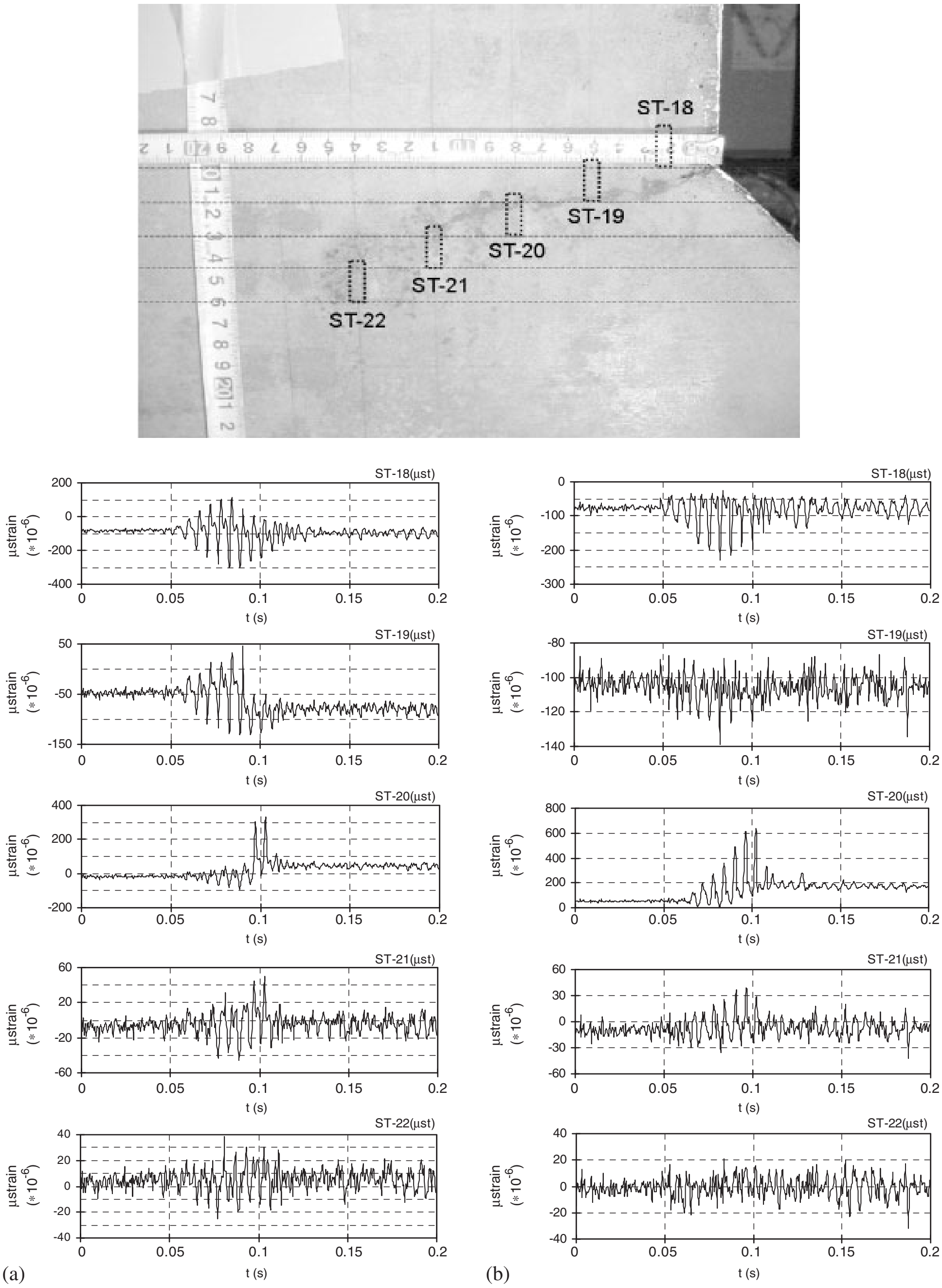


Figure 11. Transfer functions associated with the harmonic excitations in Test 2.

An 18 m dam, representative of Japanese design, was modelled and tested on a shaking table mounted on a centrifuge. Dimensions were driven by centrifuge and shaking table capabilities, as well as by the data acquisition system (smallest sampling rate 5 ms). Hence, model (of an 18 m high prototype) with 400 Hz natural frequency was subjected to 30*g* of centrifugal acceleration and excited by harmonic waves at 167 Hz.



(a) (b)

Figure 12. Strain temporal distribution during (a) S6 (dry); and S11 (wet).

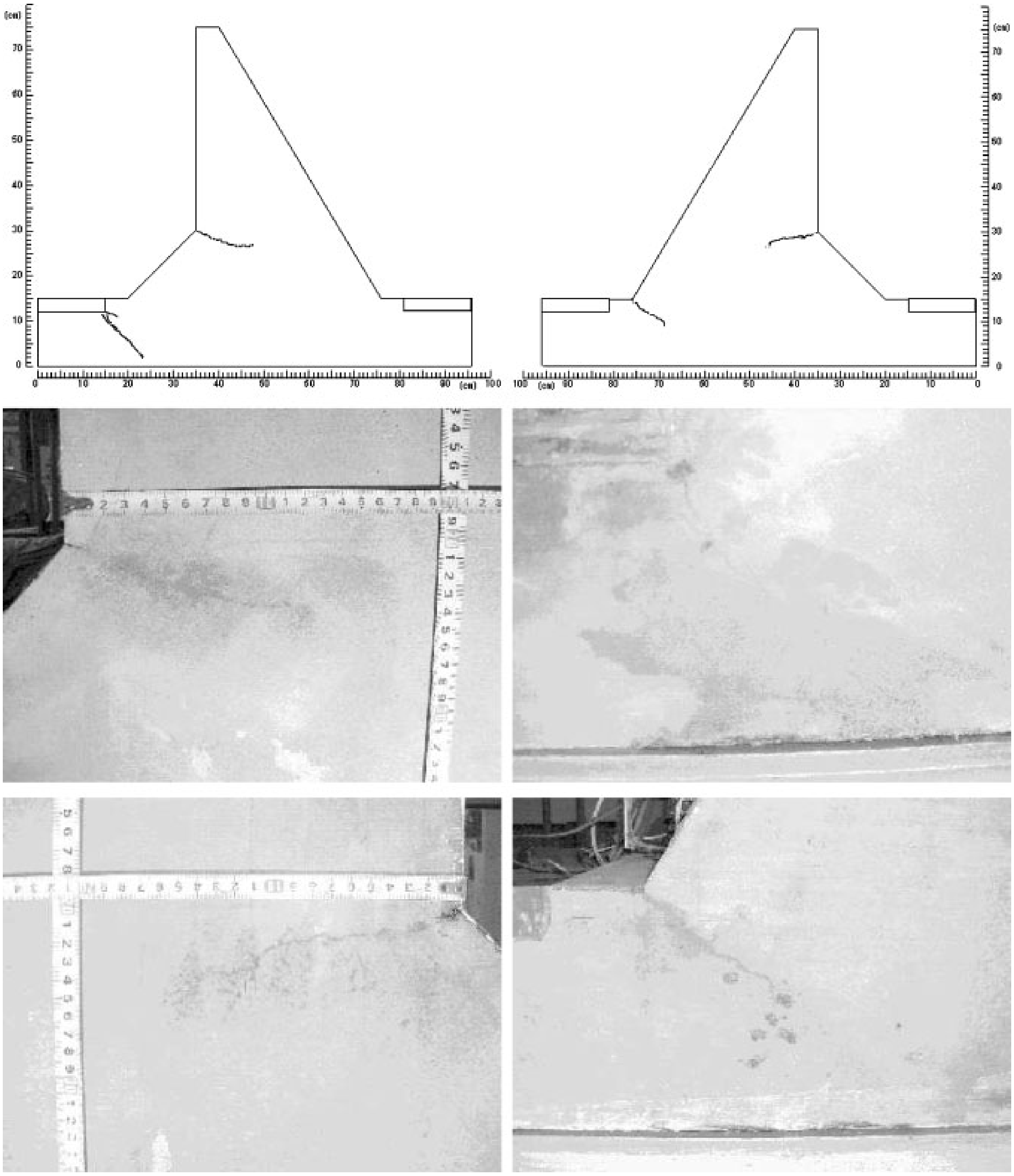


Figure 13. Crack pattern in Test 2.

Two tests were performed, a dry and a wet one, and in each case transfer function analysis of both the excitation and of 'white noise' proved essential in understanding the pseudo-seismically induced damage/cracks.

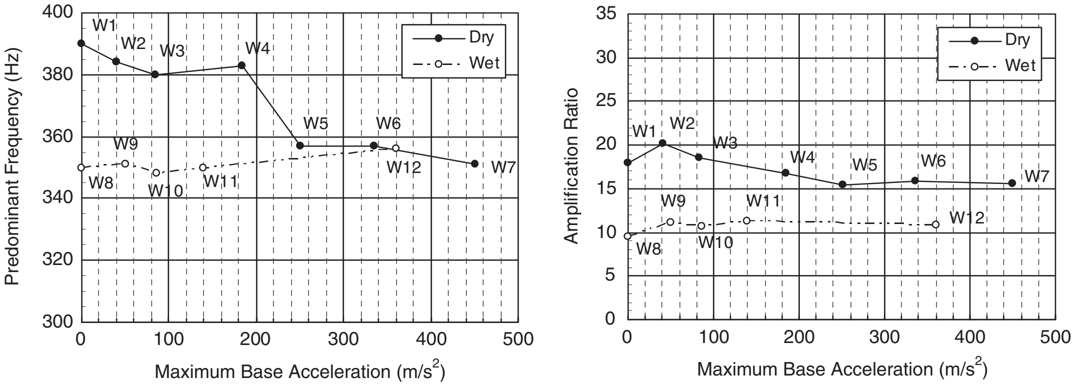


Figure 14. Peak intensities of transfer function's first mode in terms of foundation maximum acceleration in Test 2.

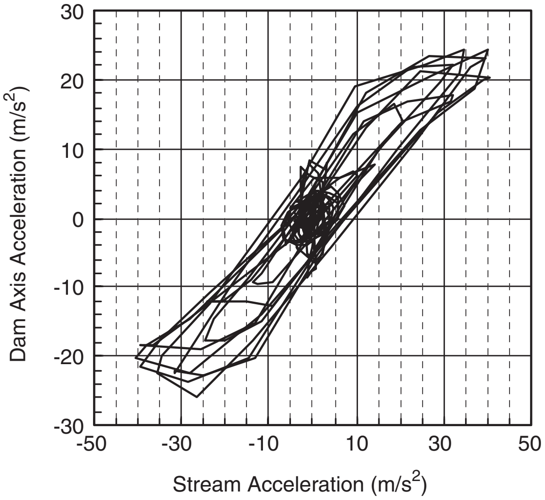


Figure 15. Stream and dam axis base accelerations in Test 2.

These preliminary tests have provided valuable data to validate the non-linear (caused by crack propagation) dynamic capabilities of our (and possibly other) finite element code (Merlin).

It is unfortunate that both tests had to be interrupted by secondary (not representatives of actual dams) types of failures: cracking or separation of the base. Nevertheless, this is a work in progress which seeks to: (1) achieve realistic centrifuge modelling of concrete dams subjected to seismic excitation; (2) Improve the understanding of damage which can

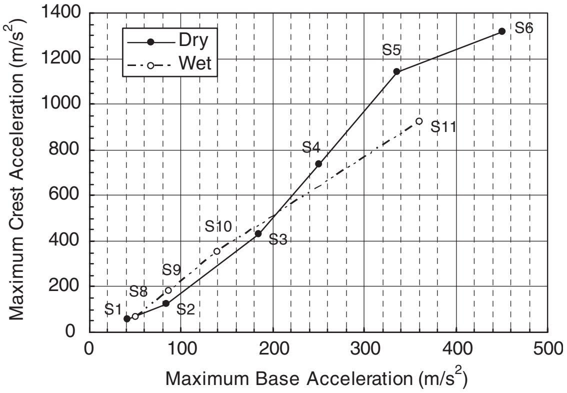


Figure 16. Crest stream acceleration in terms of base acceleration for dry and wet specimen.

be induced on a dam by a major earthquake; and (3) Provide a data base of meaningful test results to validate finite element codes.

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