11th European Conference on Earthquake Engineering © 1998 Balkema, Rotterdam, ISBN 90 5410 982 3

Invited lecture: Seismic testing of structures

E.C. Carvalho
Laboratorio Nacional de Engenharia Civil, Lisbon, Portugal

Keywords: Shaking table tests, pseudo-dynamic testing, cyclic tests, similitude laws

Although numerical methods and computer capabilities for the simulation of structures under seismic excitation have increased dramatically in the last years, experimental methods are still an irreplaceable tool in earthquake engineering. Hence, the paper aims at providing a general and comparative description of the different testing techniques currently used. Essentially it addresses static cyclic testing, pseudo-dynamic tests and shaking table tests.

Static cyclic tests are simple to perform and do not require very complex testing equipment. Tests can be done on complete structures, normally at reduced scales, or on parts of those structures (subassemblages) which can be built in full scale. Control of the test is made in terms of displacements imposed at certain points of the specimen. Those displacements should be representative of a seismic response but, since they have to be established "a priori", the actual interaction between the seismic excitation and the (nonlinear) response of the structure (or subassemblage) is not in fact reproduced. This is the major drawback of these of tests, together with the difficulty in defining the appropriate boundary conditions in the case of subassemblages which normally should vary with the progression of the test. Anyway, static cyclic tests are still a valuable tool, particularly for the validation of analytical models at the element level.

Pseudo-dynamic tests are also essentially static tests with imposed displacements but, the use of a control algorithm for the definition of those displacements as the test progresses, enables a much more exact approximation to the real seismic response. The principle beneath this technique is to combine an analytical simulation of the dynamic response of the structure, solved step by step by a numerical integration scheme, with the experimental measurement of the (nonlinear) restoring forces in each integration step. Hence, inertia forces, viscous damping forces and the seismic excitation are modeled analytically, whereas the restoring relations (normally a major source of uncertainty, particularly in the nonlinear range) are measured on the specimen along the test and its effects on the seismic response is thereby actually considered. The method allows an "elongation" of the testing time (in comparison with a real dynamic response) and thus, even for large structures, the power requirements for the actuators is relatively small. On the other hand, pseudo-dynamic testing is most suitable for structures with discretized masses and less for distributed mass systems since the former allows the use of a limited number of control degrees of freedom (not greater than the number of actuators). Also, the nature of the method allows the introduction of the substructuring concept and thus making possible the testing of only the part of the whole structure where nonlinear response is expected to occur.

Shaking tables permit the performance of real dynamic tests on structures and are available with different number of components of the motion that may be applied (ranging from one to six DOF). Naturally this sort of test is, in principle, the most suitable to reproduce accurately the seismic response of structures. However, the necessarily limited power available to the actuators, together with force and geometrical constraints in the table itself, require in most cases the use of reduced scale specimens with the inherent complexities in terms of fulfillment of similitude laws and the construction of small specimens. Additionally, complex control algorithms are needed for the correct simulation of the target motion, represented either by a specific accelerogram or a response spectrum. Shaking table tests do not lend easily to sub-structuring, as pseudo-dynamic tests do, but, on the other hand, do not impose specific restrictions in terms of the mass distribution in the structure, being, by this, particularly suitable for continuous systems or for multi-component responses.

Seismic testing of structures

E.C. Carvalho
Laboratorio Nacional de Engenharia Civil, Lisbon, Portugal

Keywords: similitude laws, shaking table tests, pseudodynamic testing, cyclic tests

ABSTRACT: Although numerical methods and computer capabilities for the simulation of structures under seismic excitation have increased dramatically in the last years, experimental methods are still an irreplaceable tool in earthquake engineering. Hence, the paper aims at providing a general and comparative description of the different testing techniques currently used. Essentially it addresses static cyclic testing, pseudodynamic tests and shaking table tests.

1 INTRODUCTION

Although numerical methods and computer capabilities for the simulation of the behaviour of structures under seismic excitation have increased dramatically in the last years, experimental methods are still an irreplaceable tool in earthquake engineering. Hence, the paper aims at providing a general and comparative description of the different testing techniques currently used. Essentially it addresses static cyclic testing, pseudodynamic tests and shaking table tests.

In terms of the type of specimens under testing, a distinction can be made between the situations in which a complete structure, either at full or reduced scale, is used and the situations in which only a structural element or a small set of elements, usually denoted as subassemblages, is tested.

Testing of complete structures is normally used to enable the understanding of the global behaviour of a construction and to capture the interplay of the response of its different components. It corresponds to quite expensive tests but, in principle, provides quite realistic information on the expected response of the specific structure under testing. By the nature of these tests and the usually high degree of redundancy of complete structures, the comparison of the experimental results with analytical simulations can only be made in terms of global variables as for instance storey displacements or accelerations and external forces or base reactions. Measurements of internal forces are normally quite difficult, among other reasons, because normally the corresponding load cells would disrupt or significantly modify the structural behaviour and also due to the large number of those load cells that would be required to enable the complete identification of the internal action effects pattern.

On the contrary, tests on single structural elements or subassemblages are much less expensive but can only provide information of what could be called local nature. They are mostly suitable for the calibration and validation of analytical models of the elements that can be incorporated into a computer code with which the simulation of the response a complete structure may be achieved. Hence, tests on elements or subassemblages are normally conducted on a large series of specimens so that the effect of different variables (for instance, geometrical proportions, mechanical properties or sequence of loading) can be checked and considered in the above referred validation of analytical models.

2 STATIC TESTING

Static cyclic tests are simple to perform and do not require very complex testing equipment. Tests can be done on complete structures, normally at reduced scales, but in most cases they are carried out on individual elements or small subassemblages which can be built in full scale or, more precisely, with dimensions similar of those of real structures.

Considering those dimensions, the construction of the specimens does not pose any difficulty and the use of exactly the same materials as used in real structures is almost always possible. This is the case, for instance, for the concrete mix and reinforcing bars in reinforced concrete specimens, for the steel profiles, plates and bolts in steel specimens and for the ceramic bricks or concrete blocks in masonry specimens, for which commercially available ordinary materials can be used.

In the opposite direction, in what concerns the obtaining of the specimens, when dealing with subassemblages with some degree of redundancy difficulties are found in defining the appropriate boundary conditions. In fact, those boundary conditions should normally vary with the progression of the test due to the different nonlinear response of the different parts of the specimen. Obviously, if the tests, in spite of its inherent limitations, are intended to minimally capture and simulate the response of the subassemblage within a global structure, those boundary conditions should be established in a way such that the distribution of the internal action effects within the subassemblage follows, at least in a "loose" way, the corresponding distribution in the global structure.

For instance, for framed structures, a very usual assumption is to consider that, under lateral loads, counter-flexure sections are found approximately at mid span of beams and at mid height of columns in between storeys. This assumption supports the very common testing layout for beam-column subassemblages in which a cross shaped specimens are tested with hinges at those mid span and mid height sections. For columns the assumption is quite reasonable but for the beams it deserves some remarks.

In fact, in a building structure beams are subjected to vertical loads and thus the corresponding bending moments and shear forces are combined with those resulting from the lateral (seismic) action. This results in that the assumption of the existence of a fixed or almost fixed point of counter-flexure at mid span of beams is not really true. To the contrary, counter-flexure points change position largely within the span as the lateral load progresses. In typical situations beams in framed structures under vertical load alone, present two counter-flexure sections (sections with zero moment) in the vicinity of the supports and when a lateral load is imposed these two sections are moved sideways leading to the disappearance of one of those points and to the approaching of the other one to the mid span of the beam. Thus the assumption of zero moment at mid span is only approximately valid for large lateral loads and is very unrealistic at other testing steps. Furthermore if beam sections are unsymmetrical (as is mostly the case in reinforced concrete structures) an additional motive for the counter-flexure point not to occur at mid span is introduced.

The consequences of this is that such testing arrangement does not realistically prescribe the shear span ratio of the beams throughout the whole range of application of the lateral action. Hence, if the shear span ratio is an important variable determining the response (as is the case in reinforced concrete elements for a certain range of the shear span ratio values) test results of this kind may be somewhat misleading.

It should be noticed that these remarks are not intended to discredit the testing of subassemblages but rather to stress the need in such cases to carefully analyze the situation so that the chosen boundary conditions (or load application pattern) are as realistic as possible in terms of the most important variables conditioning the seismic response of the specific case.

Just to illustrate what was just said, reference is made to two tests of subassemblages in which the drawbacks described above were somehow compensated.

The first refers to the test of an external beam column subassemblage tested by Carvalho (1979) at 1:2,5 scale, in which, to account to the presence of the vertical load on the beam a constant vertical force was applied in the vicinity of the beam support, at the section where zero moment would occur under vertical load alone. Thus the reproduction of shear span ratio at the critical (yielding) section of the beam was much better throughout the testing sequence, including the reproduction of the clear distinction that occurs in reality when negative or positive moments are applied (i.e. negative moments occur in conjunction with high shear creating a low shear span situation whereas positive moments occur in conjunction with low shear creating a high shear span situation of almost pure bending). Such testing arrangement also counterbalanced the effect of the unsymmetrical reinforcement arrangement of the beam (resulting from the design process that accounted to the combination of vertical and lateral loads) resulting in an global force-displacement loops almost symmetrical. It should be noticed that in a similar specimen of the same testing program but without the consideration of the vertical force in the beam, the global response was clearly unsymmetrical and pinched in one of the loading directions (due to the unsymmetrical beam reinforcement).

The other example refers to the test by Qi and Pantazopolou (1991) of a statically indeterminate subassemblage at scale 1:4 representing the lower storey of a two bay multi-storey reinforced concrete framed structure. With this arrangement, boundary conditions had only to be established for the columns, thus avoiding the difficulties referred above in what concerns beams. Columns were rigidly fixed at its bottom to the strong floor of the testing hall and were considered hinged at the mid height of the second floor, where lateral displacements were imposed. Columns were loaded axially and weights were placed on the slab (the beam had a T shaped section to simulate the slab contribution) to install in the beam a realistic initial internal force distribution.

The test was carried out up to a very high 8,6% inter-storey drift and this testing arrangement was able to reproduce realistically a number of phenomena that, if the beams were not reproduced entirely, could not be captured. In fact, in spite of the unsymmetrical beam cross section (not only in terms of reinforcement but in terms of its T shaped configuration) the overall response was essentially symmetrical. Additionally, the different behaviour of the internal and external beam ends in terms of the bond deterioration and bar slippage was evident as well as the interaction between the inelastic expansion of the beam and the lateral restraint provided by the columns. In fact this was one the mechanisms (the other being the relative strength of the columns including the effect that the variation of the axial load due to the overturning moment had thereupon) that influenced the clear redistribution of the shear force among the three columns. Such redistribution was apparent (and measurable since there were multifunctional load cells under each column enabling the evaluation of the internal forces in the specimen) as the applied inter-storey drifts were increased and upon reversing of the direction of loading.

For seismic testing purposes static tests are normally controlled in terms of a displacement or a set of displacements imposed at certain points of the specimen. In case of more than one displacement being controlled the usual assumption is to use a proportional displacement pattern but even in such case the control systems required for such purpose are very simple since they only have to ensure the following of a displacement history prescribed "a priori".

Whereas this simplifies the running of the test itself and the testing requirements (in hardware and software) it poses some difficulties in what concerns the establishment of the more appropriate displacement sequence. In very generic terms it may be stated that those displacements should be representative of the response of the structural element (or subassemblage) under testing to a seismic excitation and in that context the essence is the occurrence of large cycles of alternate loading.

No best solution can be presented to the problem of selecting the sequence of displacements for testing which has to be decided taking into account essentially the purpose of the tests. Several alternatives for displacement sequences in static seismic tests are possible and referred to in the literature. Just for illustrating purposes it is referred that those sequences normally revolve around the following aspects: a) use of the yielding or ultimate displacement as the reference (or normalizing displacement); b) application of progressively increasing displacements or application of repetitions of displacement cycles with the same amplitude; c) mixed application of cycles with small and large amplitudes or cycles with partial unloading or reloading.

In spite of its limitations static cyclic tests are still a valuable tool (and reasonable in terms of "cost-benefit") in earthquake engineering, particularly for the validation of analytical models at the element level or for the verification of the response of new materials. Its major drawback is that in does not really reproduce the dynamic response of structures under seismic excitation and the interaction between such excitation and response due to the variation of the structural characteristics or state of damage of the elements. To improve this situation the so-called pseudodynamic tests were devised.

3 PSEUDODYNAMIC TESTING

Pseudodynamic tests are essentially static tests with imposed displacements but, the use of a control algorithm for the definition of those displacements as the test progresses, enables a much more exact approximation to the real seismic response. Although the difference with respect to static tests occurs essentially only in the control aspects, the much more realistic approximation to the real seismic response that this method provides makes it naturally suitable for the testing of very large structures and hence the facilities where the system is implemented are in general of very large size.

In Europe, the biggest installation able to perform pseudodynamic tests is the ELSA reaction wall facility at the Joint Research Center of the European Union in Ispra (Italy), Donea et al. (1996). The reaction wall is 21m wide by 16m high and the corresponding strong floor is also 21 m wide by 25m long. The mechanical characteristics are also impressive with a 200 MNm bending capacity and 20 MN horizontal shear capacity for the reaction wall and 240 MNm bending capacity for the strong floor. Fixing points are placed in a square mesh of 1m and each point is able to anchor a force of up to 500 kN. The actuators have capacities of 0,5 to 1,0 MN with strokes ranging from 0,25 to 1,0 m. The ELSA facility has a fully digital servo control system for the control of the displacements which contributes to a very accurate testing procedure and enabling the use of various alternative algorithms for the step by step integration of the equations of motion.

The method was originally developed in Japan and the principle beneath this technique is to combine an analytical simulation of the dynamic response of the structure, solved step by step by a numerical integration scheme, with the experimental measurement of the (nonlinear) restoring forces in each integration step. Hence, inertia forces, damping forces and the seismic excitation are modeled analytically, whereas the restoring relations (normally a major source of uncertainty, particularly in the nonlinear range) are measured on the specimen along the test and its effects on the seismic response are thereby actually considered. The process automatically accounts for the hysteretic damping due to inelastic deformation and damage of the structural elements.

To simulate the earthquake response of the structure an earthquake ground acceleration time history is selected to represent the excitation and is input into the computer running the control algorithm. For the structure itself, existing in reality, a discretization must be considered and the corresponding degrees of freedom (usually horizontal displacements) are those to be controlled along the test. The displacements at those degrees of freedom, where, additionally, the mass of the structure as to be considered lumped, are calculated for a succession of small time steps with a step-by-step integration algorithm of the equations of motion. These displacements are then applied to the test structure by the servo-controlled actuators. At each step, load cells measure the force that the actuators apply in order to achieve the required displacement pattern and those structural restoring forces are input into the computer for the next time step calculation.

The equations of motion could, in principle, be established for any number of degrees of freedom but in reality the number of points to be controlled must be relatively reduced and correspond to the number of actuators applied to the structure. The limitation of such number derives not only from the number of actuators effectively available but also from practical testing considerations since too many attachments to the structure could disturb and modify its behaviour. Hence, the pseudodynamic testing technique is most suitable for structures where mass is concentrated in certain locations as, for instance, building structures where mass is concentrated at floor levels and the slabs may be considered rigid in-plan so that the lateral displacements may be represented by a single degree of freedom. The same applies to bridges where most of the mass occurs at the deck level and the structural discretization may be established by the horizontal displacements at the top of each pier. To the contrary, in masonry structures where mass is more distributed, pseudodynamic testing has some limitations particularly for the simulation of out of plane response of the walls where clearly an almost uniformly mass distribution exists and the attachment of actuators in several discretization points transversely to the wall without significantly disturbing the response is a major practical difficulty.

Since the inertia forces are accounted for analytically (and not actually generated during the test) there is no need to perform the test in real time or, in other words, dynamically (hence the denomination "pseudodynamic"). Consequently this allows a significant "elongation" of the testing time which typically ranges in the order of a couple of hours, in comparison with a real seismic response that takes, also typically, not more than 30 seconds. Associated with such elongation of the time factor, even for large structures, the power requirements for the actuators is relatively small and not much different from the requirements for static testing.

On the other hand, because the method relies on a step by step resolution of the equations of motion it is most sensitive to the effects of experimental errors. In fact the effects of such errors are much more severe in pseudodynamic testing than in other testing methods because the errors tend to propagate in the sequence of numerical calculations. Incorrect application of the desired displacements at each testing step or incorrect measurement of those displacements or of the forces actually applied by the actuators result in erroneous initial conditions at the beginning of the next step resulting in a possible very fast propagation of the error and consequently in unreliable test results even when individual errors at each time step are small.

Measurement and control errors tend to have a cumulative effect and in some cases these have been seen to dominate the response, Negro and Magonette (1998a). In particular, systematic errors can induce unchecked error growth due to the spurious energy introduced in the system at higher frequencies that unrealistically excite the higher vibration modes of the structure.

Akin to this matter of error propagation and accuracy of the results is the choice of the integration algorithms. No detailed discussion of this matter shall be made herein but reference is made to a recent comparative paper by Negro & Magonette (1998b) or to older pioneer studies by Shing and Mahin (1984) and Thewalt and Mahin (1987). Anyway a few basic remarks are presented.

The central difference method was the first to be successfully applied in pseudodynamic testing. It has the advantage of being of the explicit scheme type and it proved to be robust and simple to implement. However, when the stability requirements become more stringent, for instance when testing stiff structures or structures with many degrees of freedom, the method is not anymore satisfactory. To overcome these difficulties and also to counteract the perverse effects of experimental errors, explicit schemes with some sort of numerical energy dissipation to damp out spurious responses of higher modes were developed and applied to pseudodynamic testing. With appropriate selection of parameters Newmark methods of this type may become unconditionally stable and sufficiently accurate.

The use of implicit integration methods has also been attempted but they embody a major conceptual contradiction in the fact that it requires knowledge of the expected acceleration at the end of the time step and thus iteration within the time step is required. This should be avoided in pseudodynamic testing because of the risk of displacement overshooting within the iteration process which would cause the need for unloading and this may significantly affect the response of the structure. To avoid such iteration, an Operator Splitting method has been proposed by Nakashima et al (1993) and developed by Combescure and Pegon (1994) for implementation at the ELSA facility. The main idea is to split the response into a sum of an elastic part and a nonlinear one. This method has the stability characteristics of the α-Newmark method, remaining implicit for the elastic part of the response but does not require iterations since it is explicit for the nonlinear part. Under these circumstances, this method appears to be the best choice whenever the stability requirements would make the explicit central difference method not viable. In fact, it is claimed that the method is unconditionally stable, is able (by appropriate selection of α) to introduce some damping of higher frequencies (thus being able to limit the accumulation of errors) and keeps the numerical simplicity of an explicit scheme.

The hybrid character of the pseudodynamic testing concept with a part of the dynamic problem represented analytically (inertia forces, viscous damping and the base excitation) and another part (restoring forces) resulting from a physical experiment on a real specimen may be further developed with the introduction of the sub-structuring concept. This concept corresponds to assume that the behaviour of a part of the structure may be fully modeled analytically i. e. including the simulation of the restoring forces and thus actual physical testing is only required for the part of the structure where the behaviour is more uncertain or difficult to predict, namely where nonlinear response is expected to occur. This sub-structuring concept is particularly suitable for bridge testing because in most cases the nonlinear response and damage concentrates in the piers whereas the deck remains elastic. Under these circumstances the deck may be simulated by any simple linear finite element software and only the piers have to be actually subjected to physical testing. Furthermore, and again especially relevant for bridge testing, pseudodynamic testing may be extended to reproduce situations of asynchronous seismic excitation.

The part of the structure not actually present (substructure) must be represented analytically by a suitable model and the time integration scheme needed to control the test of the specimen effectively present in the laboratory must be extended to the equations of motion established for such substructure. The part of the structure modeled analytically may have a number of degrees of freedom some orders of magnitude larger than the points actually controlled in the physical specimen. Thus, very large and complex structures may be "tested" even though the specimens actually subjected to actual testing may be relatively simple. Additionally, since a large part of the structure is not present in the laboratory, size constraints are less acute and consequently larger scales may be used in the constructions of the specimens, further approximating the tested response to the one expected in the prototype.

Testing control of "sub-structured" structures relies in having two processes running in parallel: one responsible for the pseudodynamic algorithm and running in the test controller and another one running the finite element software applied to the substructure in an independent workstation. Although independent the two processes must communicate to exchange information. The control of the tested structure needs the additional force terms coming from the substructure whereas the substructure requires the accelerations at the interface nodes to solve the equation of motion in its domain.

Directly linked to the time elongation in pseudodynamic testing is one of its potential drawbacks which is the fact that it is not able to capture the strain rate effects which may have some influence on the earthquake response of, for instance, reinforced concrete structures, Aktan and Bertero (1984). In fact, the velocities at which the structure is deformed during the test are two or three orders of magnitude smaller than those occurring during a real situation and so if this factor is relevant for the material response, the test results are affected.

Most materials have strain-rate dependencies in its mechanical response but these depend largely from material to material and also on the range of values of strain rates at hand. Generally, for civil engineering structural materials the strain rate effect is reflected by an increase of strength and stiffness for faster applications of load (i.e. for higher strain rates). Thus pseudodynamic testing could potentially provide results showing an artificially weaker structure.

However, for the frequencies associated with the seismic response of structures, this effect appears not to be particularly relevant. Some comparative studies of the pseudodynamic testing of structures with the testing in real dynamic conditions (both in free vibration situations as well as in shaking table), Kitagawa et al (1984) and Negro and Magonette (1998a) tend to support that in fact the differences are small and not larger than differences associated with the variability of material properties between specimens. Out of those comparisons, for reinforced concrete structures, bond behaviour seems to be more sensitive to the strain rate effect and this may affect the spread of damage in the critical zones of the specimens. Caution should however be exercised in this appreciation because the comparative studies were conducted on specimens of different scales and this could also be responsible for the modification of the bond response.

4 SHAKING TABLE TESTING

Shaking tables permit the performance of real dynamic tests on structures and are available with different number of components of the motion that may be applied (ranging from one to six DOF). Naturally this sort of test is, in principle, the most suitable to reproduce accurately the seismic response of structures. However, the necessarily limited power available to the actuators, together with force and geometrical constraints in the table itself, require in most cases the use of reduced scale specimens with the inherent complexities in terms of fulfillment of similitude laws and the construction of small specimens.

A shaking table is composed by a rigid platform, where the specimens can be fixed, which is moved by a set of actuators, in most cases servo-controlled hydraulic actuators. The more complex shaking tables have six degrees of freedom, i.e. three translational and three rotational which require a very sophisticated control system because it has to be able to counteract the coupling between these degrees of freedom. Taking into account the complexity of such control and also the fact that for most situations the rotational excitation is not very much important for the seismic response of structures (and besides the rotational excitation is not very well known) the largest shaking tables available have only translational degrees of freedom. In Europe the largest shaking table is the Azalée table at the CEA Laboratory in Saclay, France. It has two degrees of freedom, one vertical and one horizontal, with a platform of 6m by 6m and with the capability of testing specimens weighing (payload) up to 100 tons.

For the large dimension shaking tables the actuators have typically a force capacity in each direction of the order of two or three times the maximum payload of the table. The stroke of the actuators limits the maximum displacement applicable by the table and normally is of the order of 10 to 20 cm. The more constraining variable, in terms of the motion that can be applied by the table is the velocity because it is directly linked to maximum oil flow that the pumping station can provide and this, in turn, depends directly on the power of the pumping station. For usual accelerograms only a few large velocity peaks occur during the whole duration of the motion and so some tables are provided with accumulator banks with oil under pressure that can be used to service the peak flow demands associated with those velocity peaks (instead of increasing the power of the pumping station). Typical values of peak velocity in existing large shaking tables range from 40 to 100 cm/s.

As seen, the "motion" capabilities (acceleration, velocity and displacement) of each shaking table condition essentially the type of accelerograms that can be applied and thus the kind of tests that can be performed. Besides this aspect, and possibly even more constraining for the feasibility of shaking table tests, are the maximum dimensions and payload capacity of the table which in most cases call for the use of reduced scale specimens (typically in the range of 1:3 to 1:8 geometric scale for large shaking tables if a multi-storey structural system is to be tested).

The need for reduced scales in shaking tables tests of complete structural systems is one of the major drawbacks of this type of testing, due to the similitude problems that it raises, Okada (1978) and Bertero et al (1978). In fact, in order to be able to test an undistorted reinforced concrete model to its ultimate capacity, it is necessary to obtain a true simulation of

- The geometry;
- The stress-strain relationship of the materials;
- The mass and the gravity forces;
- The initial and boundary conditions.

Geometric similitude is normally achieved by a direct application of a geometric scale and does not present any difficulty, besides that associated with the construction of small specimens.

The accurate simulation of the stress-strain relationships, Bedell and Abrams (1983), is much more difficult, even when using essentially the same material in the prototype and in the model. Small geometric scales require the use of micro-concrete and specially manufactured reinforcing bars. Thus, it is extremely difficult to satisfy, throughout the complete spectrum of stresses, strains, amplitude of load reversals, strain rates and strain gradients, such simulation. For concrete, a correct simulation of the compressive and tensile strengths, together with the ultimate strain is of critical importance. For steel, the yielding stress, the strain hardening and the uniform plastic elongation are the critical characteristics to be simulated, and the sensitivity of the variables to the bar diameter is well-known. Furthermore, the bond characteristics, which simultaneously depend on the concrete and steel properties, is an important aspect conditioning the correct simulation of the structural behaviour. In this respect a large scatter of results is to be expected, cases being known of superior bond behaviour in small scale, Aktan and Bertero (1984), as well as cases with poorer bond behaviour in small scale, Abrams and Tangkijngamvong (1984).

The mass and gravity forces simulation is closely linked to the similitude law that has to be respected. In dynamic problems to be solved by experimental methods, the usual similitude laws are the Cauchy similitude and the Froude similitude, which ask, respectively, that the

$$\text{Cauchy value} = \frac{\rho\, v^2}{E} \tag{1}$$

or the

$$\text{Froude value} = \frac{v^2}{L\, g} \tag{2}$$

in which

ρ = specific mass
v = velocity
E = modulus of elasticity
L = length
g = acceleration of gravity

are the same in the prototype and in the model.

The Cauchy similitude is adequate for phenomena in which the restoring forces are derived from the stress-strain constitutive relationships. It is interesting to notice that the Cauchy value is exactly the ratio between the inertia forces and the elastic restoring forces

$$\left(\rho\, L^3\, v^2 \,/\, L\right) /\; E\, L^2 = \rho\, v^2 \,/\, E \tag{3}$$

whereas the Froude value is the ratio between inertia forces and gravity forces

$$\left(\rho\, L^3 v^2 \,/\, L\right) /\; \rho\, L^3 g = v^2 \,/\, L\, g \tag{4}$$

Alternatively, the Froude similitude is adequate for phenomena in which the gravity forces are important as, for instance, the pendular motion, or the flexural resistance of RC columns which depends on applied axial force.

It is evident that for the realistic modeling of strongly non-linear dynamic response of structures both similitude laws must be respected. However, assuming that the mechanical properties are the same both in the prototype and in the model, i.e. the same material is used, such a requirement asks for the specific mass scale to be inversely proportional to the geometric scale. For instance, for a model built at 1:5 scale, its specific mass should be five times the specific mass of the prototype.

Obviously, the fulfillment of such requirement for geometric scales not very close to unity would call for materials that do not exist, going well beyond what may be achieved with heavy-weight aggregate concrete, Silveira et al (1980). The difficulty is usually overcome by adding distributed masses to the model, so that the desired specific mass is obtained in an equivalent way. These masses are normally made of lead or steel ingots or concrete blocks and have to be rigidly linked to the model but should not affect its structural stiffness. The amount of additional masses rises very sharply as the size of the model is reduced. For instance, for a model built at 1:5 scale, the mass to be added represents 80% of the total mass of the model.

The simultaneously satisfaction of the Cauchy and Froude similitude values leads to the scale factors presented in Table 1 (assuming that $E_P / E_M = 1$, i.e. that the material is the same in the prototype and in the model). In the Table, the scale factors for the simpler situation of satisfying only the Cauchy similitude is also shown highlighting the variables for which different scale factors occur.

Table 1 Scale factors for the satisfaction of the Cauchy and Froude similitude laws.

Parameter	Symbol	Cauchy Scale factor	Cauchy + Froude Scale factor
Length	L	$L_P / L_M = \lambda$	$L_P / L_M = \lambda$
Modulus of elasticity	E	$E_P / E_M = e = 1$	$E_P / E_M = e = 1$
Specific mass	ρ	$\rho_P / \rho_M = \rho = 1$	$\rho_P / \rho_M = \rho = \lambda^{-1}$
Area	A	λ^2	λ^2
Volume	V	λ^3	λ^3
Mass	m	λ^3	λ^2
Displacement	d	λ	λ
Velocity	v	1	$\lambda^{1/2}$
Acceleration	a	λ^{-1}	1
Weight	w	λ^3	λ^2
Force	F	λ^2	λ^2
Moment	M	λ^3	λ^3
Stress	σ	1	1
Strain	ε	1	1
Time	t	λ	$\lambda^{1/2}$
Frequency	f	λ^{-1}	$\lambda^{-1/2}$

It is worth noting that if the Cauchy and Froude similitude laws are respected, the acceleration scale is unity, whereas the time scale is the square root of the geometrical scale λ. This means that in the model the time is compressed by a factor $\sqrt{(1/\lambda)}$. Thus, in comparison to the prototype, the accelerograms to be applied to the model have shorter duration, higher frequency content and the same accelerations. The effects of the two alternatives similitude conditions on the scales of the motion variables (displacement, velocity and acceleration) and on the scales of the excitation (represented by a response spectrum) are schematically depicted in figures 1 and 2.

In figure 1, d, v and a represent respectively the displacement, velocity and acceleration and α is the exponent to be applied to the geometrical scale λ to obtain the corresponding scales.

As indicated above, the inclusion of the Froude similitude is normally associated with a significant increase of the mass in the model. In practical terms, since this is not always feasible, it should be pointed out that intermediate solutions (in between the simple Cauchy similitude and the Cauchy and Froude similitudes) are also possible and reasonable in many cases. In such case, the scale factors shall be in between those presented in Table 1 and are very easily derived if the scale for the specific mass ρ is considered individually (and not expressed as a dependent of the geometry scale). The main consequence of such option shall be the distortion between the force and weight scales which shall not any longer be the same.

Finally, in what concerns the initial and boundary conditions, there are normally no excessively large difficulties, provided that there is no attempt to model any soil-structure interaction. It should anyway be referred that presently attempts are already being made to incorporate soil structure interaction in shaking table tests together with the concept of sub-structuring described above for the pseudo dynamic tests. Naturally this requires very complex experimental systems, both in terms of hardware (the "missing part" of the specimen is replaced by control actuators that have to be supported by a reaction frame that is installed on and "travels" with the shaking table itself) and in terms of software (the control system that controls those actuators has to be operated in real time to the contrary of pseudodynamic testing for which a "time elongation" is available as described above).

Even in the case of "conventional" shaking table tests, sophisticated control algorithms are needed for the correct simulation of the target motion, represented either by a specific accelerogram or a response spectrum. The control systems are based normally on acceleration and displacement control loops and rely on the experimental evaluation of the system (platform and model) transfer function(s) which is usually carried out before the test.

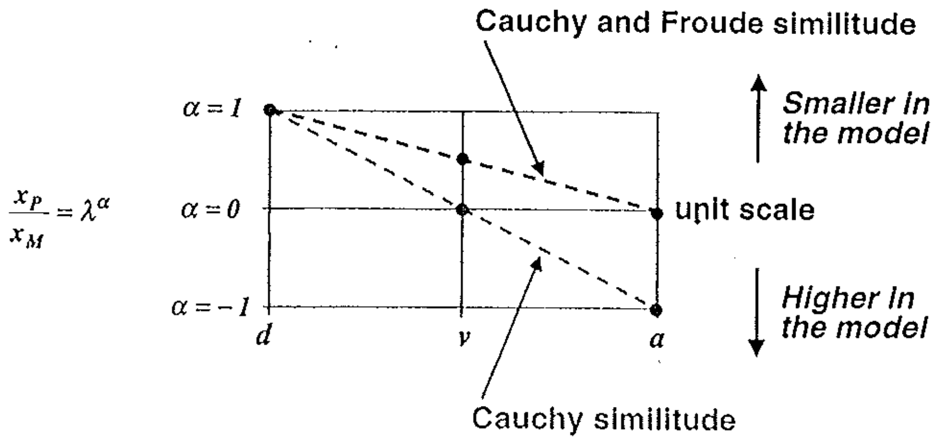

Figure 1. Scaling of the motion variables (displacement, velocity and acceleration).

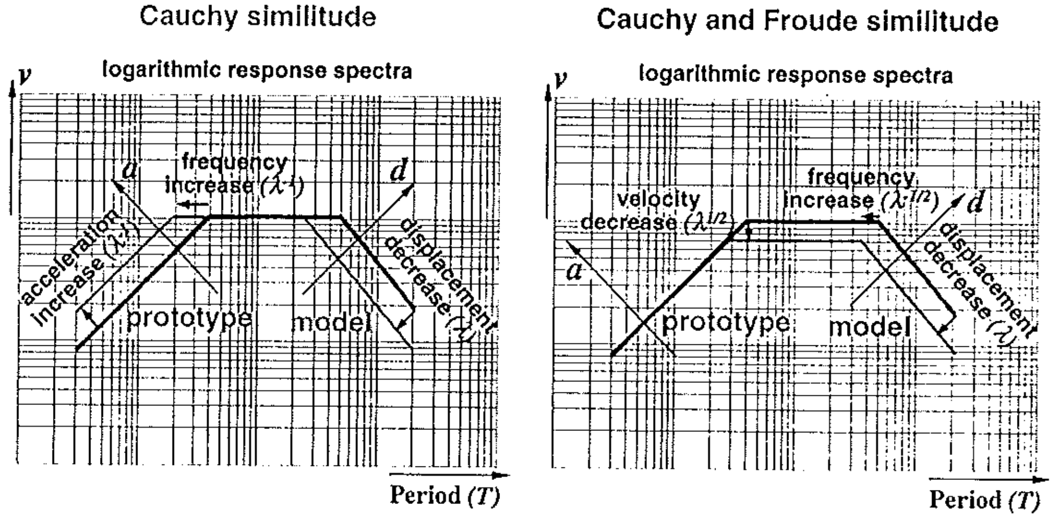

Figure 2. Scaling of the excitation.

As referred at the beginning, shaking table tests are usually used for complete structures and can be, by the means involved (equipment and specimens), quite expensive. Hence, all efforts should be applied to extract the maximum information of the testing of each specimen, both by an appropriate selection of the testing sequence as well as by a thorough interpretation of the test results. In many cases, the purpose of the test is the experimental evaluation of the vulnerability function of the structure which reflects the progression of the structural damage as the severity of the seismic action increases. Normally this requires the testing of the structure with progressively more severe earthquake actions, with such severity represented by a certain motion parameter (usually the peak acceleration). In principle, this would entail the necessity to test many individual virgin specimens since the sequential testing of one same specimen does progressively damage it and so the response is thereby affected.

To overcome this difficulty, methodologies are being developed, Coelho et al (1998) for the interpretation of a sequence of tests on one same specimen enabling to incorporate the effects of such accumulation of damage and still providing the obtaining of the vulnerability function. In such methodology, the evolution of the structural damage is approached in terms of the evolution of the stiffness associated with the variation of the dynamic properties of the specimens along the tests.

The identification of the characteristics of stiffness and damping of the structures is based on optimization procedures that force the analytical transfer functions to cope with the corresponding experimental functions. The conversion of the sequence of tests for increasing seismic intensities is based on the definition of an equivalent energy-based peak ground accêleration for each test. This parameter is evaluated from the maximum input acceleration of the nominal earthquake excitation considered in the experiment and the total "absolute" energy input accumulated until each test.

5 FINAL REMARKS

In spite of the enormous development of numerical methods and computer capabilities for the simulation of the behaviour of structures, physical testing is still an extremely valuable tool in the field of earthquake engineering. Both tools are complementary and, it is believed, are coming now closer by sharing, in an optimized way, the best features of each. Just to recall a few examples described above, this is clearly illustrated by the pseudodynamic testing technique in which part of the of the test is "physical" and part is "analytical" or by the ever more complex shaking table tests for the interpretation of which a good analytical "identification" of the specimen is essential.

Three types of seismic tests on structures were described (static, pseudodynamic and shaking table) but it is not possible to state whether one is better than the other. In general terms they may be considered complementary and the main criteria for choosing is the purpose of the tests. Although this aspect is sometimes somehow overlooked, for the success of any experimental program, it is essential that its purpose is clearly established "a priori". Out of such purpose the best experimental technique shall derive (as well as other aspects conditioning the definition of the program as for instance, the number and type of specimens, the instrumentation plan and the testing sequence).

Static tests are simple, both in terms of equipment and control requirements, and are useful mostly for the evaluation of responses of local nature with the testing of single elements or relatively simple subassemblages. By its nature, static tests are unable to catch the dynamic response of structures and so its use in complete, highly redundant structures (both at small or large scales), seems to be unbalanced since the large effort and cost to build the specimen(s) is not paid off by the very crude approximation with which the seismic response is obtained.

Pseudodynamic tests are relatively simple in terms of equipment (although space, stiffness and force requirements may be quite demanding) but are complex in terms of control. In spite of the non-reproduction of strain rate effects, this type of tests has the capability to simulate accurately the dynamic response of complete structures. It can be applied to reduced or full scale specimens. The use of full scale specimens, notwithstanding its large costs, is obviously more realistic since exactly the same materials and technologies may be used, thereby eliminating some difficulties and distortions that are unavoidable in small scale specimens. The possibility of using very realistic specimens in pseudodynamic tests is furthermore enhanced by the application of the sub-structuring concept, thereby limiting the portion of the structure to be reproduced in physical terms. On the other hand the major drawback of pseudodynamic tests is that they are ill suited to simulate the response of "distributed" structures for which the discretization of masses is not evident.

Finally, shaking table tests are demanding in terms of the testing equipment and in terms of the control systems and the size of the specimens is normally limited. Shaking table tests do not lend easily to sub-structuring, as pseudodynamic tests do, but, on the other hand, do not impose specific restrictions on the way mass is distributed in the structure, being, by this, particularly suitable for continuous systems or for the evaluation of multi-component responses. Construction of specimens is normally complex and costly due to the need to recourse to reduced scales which, additionally, may impose some extra difficulties in the strict compliance with the similitude laws. In spite of these limitations and difficulties shaking table tests allow a very close simulation of the behaviour of structures under strong earthquake excitation. In fact, this sort of loading induces strongly non-linear responses close to the ultimate capacity of structures and makes apparent its hidden under or over-strengths, which in complete, complex structures are still difficult to capture fully in purely analytical terms.

REFERENCES

Abrams, D.P. and Tangkijngamvong, S. 1984. Dynamic response of reduced scale models and reinforced concrete structures. Proc. 8th World Conf. On Earthq. Engng, Vol VI, 371-378, San Francisco.

Aktan, A.E. and Bertero,V.V. 1984. Earthquake response of a 1/5th scale model of a 7-story reinforced concrete frame-wall structural system. Proc. 8th World Conf. On Earthq. Engng, Vol VI, 651-658, San Francisco.

Bedell, R.J. and Abrams, D.P. 1983. Scale relationships of concrete columns. Res. Series 83-02, Dept. of Civil, Environmental and Architectural Engineering, University of Colorado, Boulder, Colorado.

Carvalho, E.C. 1979. Beam-column subassemblages tested under alternated loads for seismic design. In Proc. AICAP-CEB Symposium, Structural concrete under seismic actions, Rome, May 1979, Comite Euro-international du Béton, Bulletin d'Information nº 132, Paris.

Coelho, E., Campos-Costa, A., Rodrigues, J. & Bousias, S. 1998. Shaking table tests on reinforced concrete infilled frames. Proc. 11th European Conf. on Earthq. Engng, Paris, Sept 98, Balkema, Rotterdam.

Combescure, D. & Pegon, P. 1994. Alpha-operator splitting time integration technique for pseudodynamic testing: error propagation analysis. Special Publication No I.96.65, European Commission, Joint Research Center, Ispra.

Donea, J., Magonette, G., Negro, P., Pegon, P., Pinto, A. & Verzeletti, G. 1996. Pseudodynamic capabilities of the ELSA Laboratory for Earthquake Testing of Large Structures. Earthquake Spectra. Vol. 12, No. 1.

Kitagawa, Y. & al. 1984. Correlation study on shaking table tests and pseudodynamic tests by RC models. Proc. 8th World Conf. On Earthq. Engng, Vol VI, 667-674, San Francisco.

Nakashima, M., Akazawa, T. & Sakaguchi, O. 1993. Integration method capable of controlling experimental error growth in substructure pseudo-dynamic test. Journal of Struct. Constr. Engineering, AIJ, Tokio, No 454, 61-71.

Negro, P. & Magonette, G.E. 1998a. Experimental methods in Structural Dynamics. European Earthquake Engineering, Volume XII, 1.98, 29-39.

Negro, P. & Magonette, G.E. 1998b. Verification of the pseudodynamic test method. European Earthquake Engineering, Volume XII, 1.98, 40-50.

Okada, T. 1978. The experimental investigation of ERRCBC with emphasis in the use of earthquake response simulators in Japan. In Bertero V.V. (organ.), Proc. of a workshop on earthquake–resistant reinforced concrete building construction, vol. III, 1630-1651, Univ. of California, Berkeley, California.

Qi, X. & Pantazopolou, S.J. 1991. Response of RC frame under lateral loads. Am. Soc. Civ. Engrs, J. Struct. Engng, 117, 4, 1167-1188.

Shing, P.-S.B. & Mahin, S. 1984. Pseudodynamic Test Method for Seismic Performance Evaluation: Theory and Implementation. UCB/EERC-84/01, Earthquake Engineering Research Center, University of California, Berkeley, California.

Silveira, A. F., Azevedo, M. C., Ferreira, M. J. E. & Costa, C. A. P. 1980. High density and low strength materials for geomechanical models. Memória nº530, Laboratório Nacional de Engenharia Civil, Lisbon.

Thewalt, C.R. & Mahin, S. 1987. Hybrid Solution Techniques for Generalized Pseudodynamic Testing. UCB/EERC-87/09, Earthquake Engineering Research Center, University of California, Berkeley, California.