STRUCTURAL MODELS IN EARTHQUAKE ENGINEERING

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SYNOPSIS

This paper presents a brief state-of-the-art report on the use of structural models in earthquake engineering studies. Similitude requirements and dynamic properties of some commonly used materials are summarized. Some of the recent developments in testing techniques and laboratory facilities in North America, Europe and Japan are briefly reviewed.

Four examples of use of structural models for earthquake-resistant design are presented.

RESUME

Cette communication présente un bref état des connaissances sur l'usage des modèles structuraux dans les études des tremblements de terre. Les critères de similitude et les propriétés dynamiques de quelques matériaux couramment utilisés sont résumés. Quelques uns des développements récents sur les techniques d'essai et les installations de laboratoire en Amérique du Nord, en Europe et au Japon sont examinés brièvement.

Quatre exemples de modèles structuraux utilisés dans le calcul de résistance au séisme sont présentés.

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1. INTRODUCTION

Recent advances in high speed digital computers and the availability of powerful computer programs along with the progress made in experimental techniques such as modern data acquisition and processing systems and the construction of sophisticated shake tables and physical models have assisted greatly with improved understanding of behaviour of structures under various kinds of dynamic loads. Both experimental and computer dynamic analyses of structural systems are significantly more difficult to achieve than static analysis because of the complexities of dynamic load idealization and the variation of structural behaviour and material properties with time. Moreover, the basic complexity of the dynamic problems along with the lack of suitable experience makes it difficult to produce proper analytical models. Since the structural behaviour of a given system is often influenced by the neighbouring systems due to propogation of waves, it can only be adequately studied by extending the analysis to the entire system. This problem is not significant for buildings due to the small base dimensions, however for large structures such as suspension bridges and dams, the difference of vibration in nearby points due to the propogation of the waves deserves consideration (1).

Despite all the progress, the presently available mathematical methods can be used satisfactorily for linear elastic analysis and for nonlinear analysis of some problems, however they cannot be used for analysis of nonlinear behaviour of most highly indeterminate structures subjected to seismic loads, especially when conditions at higher load levels and failure need to be investigated. Over the past two decades, small scale models have been increasingly used to predict the structural behaviour of some important modern structures including their response to both static and dynamic loads in the non-linear range and at failure. Some examples of such structures are important building and tower structures, prestressed concrete pressure vessels, dams and bridges. Use of physical models appears to be an appropriate solution to study the behaviour of these complex structures subjected to dynamic loads. However, dynamic model testing can be quite complicated due to the difficulties arising from the adequate representation of model materials and earthquake loads, testing, acquisition and reduction of data, and in the analysis and interpretation of the results. Exact modeling is almost impossible in most cases and the effects of simplifications and approximations used must therefore be well understood.

This paper presents a state-of-the-art report in the field of structural models in dynamic loading applications especially for earthquake loadings.

2. SIMILITUDE REQUIREMENTS

The dynamics of structures is governed by an equilibrium of the time dependent forces on the structure, resisted by the inertia forces which are products of the local mass and acceleration, the resisting forces which are a function of the stiffness of the structure in the particular direction in which motion is occurring and the energy dissipative or damping forces which may be material or construction related. Besides these forces which cause dynamic stresses and deformations in the structure, there are certain types of massive structures in which gravity induced stresses play an important role and which must be accounted for in the modeling process. Determination of the necessary similitude requirements is the first step in the successful implementation of the dynamic modeling process. Some of these similitude requirements are shown in Table 1 which has been adopted from Reference 2 with slight modifications. It can be noted that these similitude requirements are dependent both on the geometry of the structure, material properties and on the type of loading. Details of derivation of similitude relationships between the model and the prototype can be found in any standard textbook on models (3).

These similitude relationships can be generalized to nonlinear behaviour of model and prototype structures if the stress-strain relationships of the model and the prototype materials are the same. The model can be studied until failure and at such state of cracking or damage where it is necessary to study the influence of this distress on the frequency ranges from the model to the prototype. Three sets of similitude requirements are presented in Table 1. Similitude requirements for true replica models (Col. 4) for which inertial, gravitational and restoring forces are correctly duplicated, are difficult to satisfy because of the severe restrictions on the model material properties, especially the mass density. Similitude relationships due to Froude (Col. 5) are based on the relationship (inertial forces/gravitational force) being a constant while those due to Cauchy (Col. 6) are based on the relationship (inertial force/elastic restoring force) being a constant. Considerable success has been achieved in the testing of reduced scale structures and structural components on shake tables where additional material of a non-structural nature has been added to simulate the required scale density of the model.

3. DYNAMIC PROPERTIES OF MODEL MATERIALS

Experimental evidence indicates that the strength and behaviour of most materials depends among other factors on the rate of strain during testing, the number of cycles of load reversal and the stress level at loading. The frequency of vibrations during most earthquakes is in the range 0-5 Hz which does not significantly influence the prototype material properties. However as the model scale is made smaller, it is clear from the similitude relationships in Table 1 that the frequency of vibrations in the model is much higher. Thus if the length scale factor is say 1/50, the model frequency range of interest becomes 0-250 Hz which may have a significant effect on the model material properties.

Pereira and Priestley (4) conducted a series of tests to obtain systematic information about the dynamic behaviour of different model materials. They studied the influence of frequency on the modulus of elasticity and the damping characteristics of steel, aluminum, brass, wood, perspex (plexiglas - an acrylic resin) and model concrete. The behaviour of rectangular bars of varying lengths, both with and without attached masses, was studied using a shake table to obtain the natural frequency and the associated damping. They noted that the effect of vibration frequency on the modulus of elasticity is significant and it must be considered in interpreting the model test results. They noted that because of the large variation in the values of the modulus of elasticity, materials such as wood and plexiglas cannot be readily used to model the response of a prototype structure to a random vibration sequence. More details can be found in Reference 4.

3.1 Modeling of Steel

The effect of the strain rate change on the stress-strain curve of ASTM A-7 structural steel is shown in Fig. 1. The effects of increasing rate of strain can be summarized as follows (5):

1. The yield stress increases to some dynamic value, σ_{dy};
2. The yield point strain, ε_y, increases.
3. The modulus of elasticity, E, remains constant in the elastic range.
4. The strain at the commencement of strain hardening, ε_{st}, also increases.
5. The ultimate strength increases slightly.

The most important effect which influences the design of steel structures to resist dynamic loads, is the increase in the yield stress. The percentage increase in yield stress is given as a function of the strain rate for two steels of different static yield stress (Fig. 2). It is evident that the increase in dynamic yield point is greater for steels with lower static yield point as is shown by the higher slopes of curve A (6). Variation of the dynamic yield stress as a function of the time, t_{yp}, required to reach the dynamic yield stress for ASTM A7 steel is shown in Fig. 3. Values of design yield stress can be obtained from Fig. 3 knowing the time to reach yield stress in a particular structure. The influence of frequency of vibration on the modulus of elasticity and damping characteristics of structural steel is shown in Figures 4 and 5 (4). The dynamic modulus of elasticity for a frequency of 500 Hz is 8 per cent higher than the static modulus.

Useful data on the effect of strain rate on smooth and knurled steel wire suitable for model reinforcement has been obtained by Staffier and Sozen (7), and is shown in Fig. 6. Krawinkler et al (2) have reported similar increases in yield strength with an increase in strain rate.

Besides studying the properties of structural steel as a model material, Krawinkler et al (2) investigated the mechanical properties of some copper and lead alloys to study their suitability to model structural steel. These tests included determination of stress-strain curves for low and high strain rates and under cyclic loadings, creep, size effects and ductility. Feasibility of fabricating structural elements and of providing welded and bolted connections was examined in each case. The damping of the model materials was observed to be small compared with other sources of damping in the structure. Harris et al (8) studied the properties of annealed phosphor bronze for dynamic modeling of structural steel.

3.2 Model Concrete

For modeling of concrete structures subjected to inelastic deformations it is not possible to use anything but cementitious materials such as model concretes of cement or gypsum mortar. The static properties of model concrete and model reinforcement have been studied by several investigators (9-11), however very little work has been done on the dynamic properties of model concretes. Strain rate effects on model concretes have been reported in connection with blast and impact loading effects (12, 13). Reversed cyclic loading tests have been conducted on small scale reinforced concrete models to simulate and study their behaviour under earthquake loadings (14, 15).

A series of 2in. x 4in.(50 x 100 mm) cylinders of model concrete with a water: cement: aggregate ratio of 0.9: 1: 4.5 was tested by Harris et al (12) at increasing strain rates in unconfined compression. A total of 44 specimens were tested at four rates of strain ranging from 10^{-5} in./in./sec. to 10^{-2} in./in./sec. The results are shown in Table 2 as the ratio of dynamic unconfined compressive strength, f'_{cD}, to the static strength, f'_c. The average values of f'_{cD}/f'_c are plotted in Fig. 7 versus the average rate of strain. A comparison with similar results of ordinary concrete shows a higher rate of increase of f'_{cD}/f'_c for the model-concrete over the range tested. This is partly due to the size effect when testing smaller specimens of model concrete.

Influence of the frequency of vibration on the modulus of elasticity and the damping characteristics of model concrete is shown in Figures 8 and 9 (4). It can be noted that for uncracked model concrete, the dynamic modulus of elasticity at a frequency of 150 Hz is about 15 per cent higher than the static modulus. Internal damping of model concrete increases with frequency (Fig. 9) and a local peak was observed at a frequency of 19 Hz, however Pereira and Priestley(4) attributed this to the properties of the base of the test set-up rather than those of the model concrete, thereby obscuring the experimental results.

4. TESTING TECHNIQUES AND LABORATORY FACILITIES

Elastic vibration studies can be performed on full scale structures using a variety of techniques (16) to excite the natural modes and frequencies of the structure. These studies are normally limited to low amplitude testing, and can be quite expensive; laboratory experimentation using scale models therefore remains the main source of useful experimental results for development of earthquake-resistant design.

Although free vibration measurements are sometimes easily accomplished by pulling on the structure, followed by a quick release, and measuring the free motions of the structure, most vibration tests in the laboratory are performed by forcing the structure to vibrate in one of its natural modes. This is accomplished by the use of mechanical or electromagnetic oscillators or by placing the model on a shake table or an earthquake simulator. A set-up for studying the natural modes and frequencies of a cantilever model plate is illustrated in Fig. 10. Using a small capacity electromagnetic shaker, the plate, which is connected through a small coupling rod and a load cell, can be made to oscillate sinusoidally or in other types of programmed periodic motions. By changing the frequency of the forcing signal, the plate is forced to vibrate at different frequencies; the frequencies which correspond to the natural frequencies of the plate will result in zero applied force through the load cell. This condition can be observed on the oscilloscope and facilitates the determination of the natural mode shape and frequency.

Clough and Bertero (17) summarized the work at the University of California at Berkeley on laboratory testing for earthquake model loading. They discussed several experimental laboratory facilities and techniques for pseudo-static and dynamic testing of building structures.

Sozen (18) suggested that the use of an earthquake simulator in research expands the possibilities of what can be studied without the necessity of a large number of buildings going through a large number of earthquakes however he cautioned that this can be as deceptive as it can be revealing. While earthquake simulation in the laboratories is in a primitive stage, it fulfills an important support role in structural research. Sozen discussed the choice of test specimens in terms of their initial stiffness and weight and described some of the University of Illinois tests on frames and coupled shear walls.

Bertero, Clough and Oliva (19) described laboratory procedures for studying the response of buildings under seismic excitations, with emphasis on the response of structures and structural components, using earthquake simulators and large-scale loading facilities. They discussed the advantages and disadvantages of each system and made some suggestions for future development and experimental research.

4.1 Seismic Modeling Techniques used at the Laboratorio Nacionel de Engnehria Civil (LNEC) and the Istituto Sperimentale Modellie Strutture (ISMES)

Response of various kinds of structures to earthquake loadings have been studied at the Laboratorio Nacionel de Engnehria Civil (LNEC), Lisbon, Portugal and the Istituto Sperimentale Modelli e Strutture (ISMES), Bergamo, Italy for more than two decades using small-scale models (1,20). A brief description of the basic techniques used by the two organizations follows:

Both the ISMES and the LNEC use two basic types of tests:

1. To study the generally dynamic behaviour of the structure, the model is subjected to a series of sinusoidal tests to obtain its natural frequencies, mode shapes, dynamic characteristics and stress distribution. This test is therefore equivalent to solving the eigenvalue problem and the model can be of the "elastic" type. The model is excited either by sinusoidal motion at its base, normally to determine the first natural frequency or by means of concentrated sinusoidal forces acting at several points on the structure. The frequency, amplitude and phase lag of each force can be selected so as to excite a single mode at a time, and to eliminate the contribution of the undesired modes. Under such conditions, the modal shapes can be obtained directly from the measurement of the response accelerations while damping coefficient can be obtained from the measurement of the input power. This technique was successfully used at the ISMES to study the response of the Ridracoli dam which was modelled to have 120 degrees of freedom. The first six vibration modes and the associated stress distributions were successfully determined (1).

2. To verify the response of a structure beyond the elastic range for a given earthquake, the model is constructed using the same materials as the prototype structure to reproduce the prototype behaviour both within and beyond the elastic range. It is ensured that the excitation has all the characteristics of the vibration source.

For models to be tested to failure, the acceleration intensity is slowly increased in a step-wise fashion and the natural frequency is determined using the shock test until the frequency starts decreasing. This is due to cracking and other types of distress in the model. The factor of safety is defined as the ratio of the intensity at rupture of the model to the intensity of the standard earthquake.

4.2 Simulation of Earthquakes

The experimental techniques to simulate earthquake loadings, presently used by leading laboratories around the world can be grouped as follows:

1. Shaking tests using earthquake simulators or shake tables.
2. Shaking tests by vibration generators.
3. Simulation using controlled loading test facilities.

A detailed discussion of these techniques can be found in References 1 and 17 through 22.

4.3 Earthquake Simulators or Shake Tables

Small to medium scale earthquake simulators have been used in many laboratories around the world as a reliable technique to investigate the response of structures to a simulated earthquake ground motion. Due to expense and time requirements, fewer shaking tests have been carried out on building models than pseudo-static or pseudo-dynamic loading tests although more than twenty earthquake simulators have been installed around the world over the last decade (21). Because of size constraints of shake tables, shaking tests are limited to very simple prototypes, small sub-structures or models of prototype structures. Besides, the model is usually very small due to limitations of the driving power and in some cases it is difficult to satisfy the similitude relationships between the prototype and the model.

According to Okada (21), a shaking test has lower benefit-to-cost ratio than a pseudo-dynamic loading test and/or a computer analysis. A large number of cyclic or alternate loading tests on members and frames including the existing buildings have been performed in Japan. Computer analyses using the analytical models derived from the pseudo-dynamic loading tests have been performed and a combination of the pseudo-dynamic loading test and computer analysis is considered to be an optimum choice to simulate the earthquake response of building structures. This technique has two limitations: (a) the difficulty to derive a general hysteretic rule to represent the real nonlinear characteristics from the test data obtained under a specific loading path and (b) the inability to consider the effect of strain rate. It is well known that nonlinear characteristics of a reinforced concrete structure are very sensitive to the loading path. Efficient use of the shake table was recommended to investigate structural behaviour during earthquakes and to develop more realistic analytical models.

Krawinkler (22) suggested that to obtain complete seismic response of specific structures, the complete prototype soil-structure system must be reproduced in the laboratory, which is almost impossible to achieve even on large shake tables. Therefore, laboratory earthquake simulation becomes a compromise which limits the usefulness of shake table experimentation. "The purpose of experimentation on earthquake simulators is to investigate within the limitations of rational table sizes seismic response phenomena which cannot be studied in a more reliable and cost efficient manner by other experimental means. Such phenomena could include rate of loading effects, dynamic response characteristics under realistic seismic excitation ranging from low amplitude vibrations to excitations producing inelastic response and failure, failure mechanisms, effects of mass and stiffness irregularities, torsional effects, overturning effects, dynamic instability, idealized soil-structure interaction effects, and interaction between structural and nonstructural elements." More emphasis needs to be placed on the role of shake table experimentation as a means of verifying mathematical modeling techniques. Also, there is an urgent need to study the scale effects between the model and prototype materials, elements and structures resulting from strain rate effects, strain gradient effects, dimensional tolerances, detailing requirements, etc.

Shake tables can be constructed for a wide range of sizes and capabilities. Some are hydraulically actuated and others such as the 6.5' x 10' table at LNEC, are actuated by electromagnetic shakers as shown in Fig. 11. A classification of some shake tables (Table 3) indicates that the smaller facilities are more suited to testing scale models. The model to be tested under simulated earthquake motions is usually bolted to the moving table as shown in Fig. 11 which illustrates the testing of a model dam and its surroundings. Description of the shake tables used at the LNEC, the ISMES and the University of California, Berkeley can be found in References, 1, 4, 18 and 19. Krawinkler et al (2) examined the design and performance requirements for shake tables and summarized the basic information for various shake tables being used around the world.

4.4 Forced Vibration Tests - Vibration Generators

Forced vibration tests are an acceptable technique to determine the dynamic characteristics of a structure such as its natural frequencies, mode shapes, damping coefficient, etc. The model is excited by sinusoidal forces concentrated at one or more points on the structure. It is well known that the response of a structure subjected to a system of external forces can be expressed as a sum of the oscillations corresponding to the different vibration modes, therefore with a suitable choice of exciting forces, it is possible to eliminate the contribution of any given mode to the overall response of the structure. Thus the problem reduces to one of finding the particular set of forces for which the response of all other modes, except the one desired, is zero. This can be achieved by a prior knowledge of the mode shape and by an iterative experimental technique to minimize the influence of the undesired modes. After each iteration, the location of the exciters and the force generated can be suitably adjusted.

In recent years, Japanese investigators have frequently used this forced vibration technique to study the dynamic behaviour of existing reinforced concrete buildings and large size models along with the pseudo-dynamic loading tests up to the collapse stage. This technique was also successfully used in the ISMES laboratorie on an elaborate model of Zarate-Brazo Largo Bridges shown in Fig. 12 (23); the bridge deck was excited by four exciters (Fig. 13) acting normally to the bridge deck (Fig. 14). All exciters worked at the same frequency; the phase difference between the individual exciters and the intensity of the forces were regulated for an easy determination of the symmetrical and antisymmetrical modes. The deck displacements were measured in all three directions to determine the bridge response. The transducer signals were amplified and recorded simultaneously on multi-channel tape recorders.

The disadvantage of this technique is the difficulty of controlling the amplitude of the model response. This causes a vibration mode different from the earthquake response and therefore the data has to be modified to simulate the earthquake response. This can be achieved with relative ease if the structure remains within the elastic range however it becomes difficult if the response exceeds the elastic limit.

4.5 Controlled Load and Deformation Testing Facilities

Many investigators have used quasi-static or pseudo-static cyclic load tests (slowly applied reversing loads) to study the load-deformation behaviour of structural elements and subsystems. Through the use of hydraulic actuators and synchronized electronic control it is possible to directly apply a predetermined history of external forces, and/or deformations at several points on the test structure to simulate the effects of an earthquake tremor. Based on the time rate at which these forces or deformations are applied, the test can be classified either as dynamic or pseudo-static. For the pseudo-static tests, the effects of strain rate and velocity-dependent damping characteristics are not significant. It is possible to stop the test at any time to inspect the test specimen and/or to check the instrumentation. Moreover if necessary, the loading program can be altered dependent upon the test results. This technique facilitates a better knowledge of the stiffness and strength mechanisms and their deterioration and of the failure modes of the structure. However, the strain rate effects must be taken into consideration before final acceptance of the test results; this can be achieved through a judicious combination of dynamic and pseudo-static loading tests.

Normally, it is quite tedious to select the test loading or deformation history to induce stresses and deformations at homologous points in the test structure similar to those induced by the earthquake shaking in the real structure. According to Bertero et al (19), "Usually the only recourse is to simulate the critical combination of forces that could develop at a certain time and then to vary the intensity of these forces according to simple time functions. Rational selection of this critical combination requires integrated experimental and analytical studies because this combination will vary depending upon the specific subject of the study".

After selecting a rational combination of forces, the investigator faces the problem of how to vary these forces with time keeping in mind that the behaviour of some structures especially the reinforced concrete structures is quite sensitive to the loading path (19). As a starting point in an iterative procedure, one selects a load pattern and assumes a simplified mathematical model and the response of the test structure is evaluated. The mathematical model is suitably modified based on the above data and the analysis is repeated until satisfactory convergence is obtained.

Excellent load and deformation control can be achieved using this technique. Moreover, there is no limitation on instrumentation as was the case with earthquake simulators. The basic disadvantages of this technique (19, 22) are as follows:

(a) The time-dependent effects are not simulated. Although sufficient data is presently unavailable to draw definite and general conclusions on strain rate effects, recent studies have shown that these time-dependent effects may not be of a major importance.

(b) Cyclic loading tests involve a subjective decision on the predetermined loading history to be applied to the structure. Analysis prior to experimental work can be helpful but this prediction of the loading history based on unknown strength and stiffness characteristics seems to defeat the purpose of the experimental work.

This technique has been used for many research programs at various laboratories and a typical test set-up for a coupled wall system tested at the University of California at Berkeley is shown in Fig. 15.

4.6 Computer-Actuator On-Line System

Okada (21) reported a significant new development to simulate the seismic response of structures using a hybrid "computer actuator on-line system". This involves computer solution of the nonlinear differential equation expressing the seismic response of the structural system to the earthquake ground motion. The computer utilizes the electronically measured stiffness characteristics of the test specimen and computes the incremental response of the specimen to the earthquake ground motion which is fed back to the electronically controlled actuators. The system can simulate the response of multi-degree of freedom systems to bidirection components of the earthquake ground motion. An example of a simple test set-up is shown in Fig. 16 and for completeness, the flow chart for the on-line system has been reproduced in Fig. 17.

Okada (21) summarized the advantages of the on-line system as follows:

(1) Earthquake response can be simulated taking into account the real non-linear restoring force characteristics of structures or structural elements. There is no need to assume an idealized mathematical model.

(2) Realistic loading paths can be used in tests on large size structures using electro-hydraulic actuators having a comparatively small capacity.

(3) The test can be performed using an enlarged "non-real" time controlled by the computer which facilitates observation of failure mechanisms, and data acquisition.

(4) Unlike the simulation of dead loads by heavy weights attached to the models tested on a shake table, gravity load can be easily applied to the structure prior to the on-line test to consider the stresses resulting from the gravity loads.

Since the computer-actuator on-line system is a hybrid system, it has disadvantages of both the loading test and the computer analysis. As mentioned previously the difficulties in the experimental work are in the areas of the loading path and strain rate effects. While the former can be improved by the on-line system, more research is needed into the effects of the strain rate. The accuracy of simulation can be ensured by an adequate numerical integration and careful choice of an accurate measuring system. Some applications of this technique are presented in Reference 21.

4.7 Impact Loading Tests

Shaking tests on three-dimensional building models have been conducted in Japan using an impact loading table (21). The impact table is suspended from a steel frame as shown in Fig. 18(a), the weight of the pendulum constitutes the impact load. The maximum impact load used at the Obayashi-gumi laboratories was 30 tons and the intensity of impact loading can be controlled by the weight of the pendulum and the number of shock absorbers made of steel and synthetic rubber. The intensity of shock applied to a model can be increased in stages until the response displacement becomes approximately three times the yield displacement. The maximum response acceleration attained was 1.5 g. One building model was tested until the collapse stage; the response displacement at this stage was more than nine times the yield response. A typical frame tested on the impact loading table is shown in Fig. 18(b).

5. APPLICATIONS IN DESIGN AND RESEARCH

Several papers have been presented at the recent World Conference on Earthquake Engineering and other Earthquake Engineering Conferences describing the use of models in studying the response of a structure subjected to earthquake loadings. Some examples of static and dynamic modeling of reinforced and prestressed concrete building, bridge, prestressed concrete pressure vessel, and hanger structures are presented in Reference 24. These include the results of shaking tests on the model of a curved bridge which collapsed during the San Fernando earthquake of 1979 (25). Several design and research model tests are described in Reference 19. Because of space limitation only the following four model investigations are summarized in this section.

1. A 22-storey building in Caracas, Venezuela partially damaged during the 1967 earthquake.

2. A two-storey reinforced concrete frame.

3. The south pier of the Tagus River suspension bridge.

4. The Ruck-A-Chucky Bridge over the American River in California.

5.1 Model of 22-Storey Venezuelan Building Tested at the Laboratorio Nacional de Engenharia Civil, Portugal

The LNEC (4, 26) was involved in an interesting earthquake model study of a 22-storey building which had suffered partial damage during the 1967 earthquake in Caracus, Venezuela. This 22-storey reinforced concrete building (59.4 m (195 ft.) high and 12.1 m (40 ft.) by 11.6 m (38 ft.) in plan) contained 16 tapering columns and a sufficient number of partitions and exterior brick walls to ensure that the building behaved as a cantilever.

Subsequent studies of the building revealed that the flexibility of the foundation had contributed significantly to the lateral deflections. A 1/30-scale elastic model was constructed at the LNEC to investigate this phenomenon. The columns and walls of the basic structure were simulated by a plexiglas box, with steel floors flexibly connected to the plexiglas walls. These steel floors correctly simulated the mass at each floor level. Changes in the moments of inertia of the prototype building were simulated by reducing the total area of the walls by an appropriate number of holes. The assembled model ready for test and some of the instrumentation used are shown in Fig. 19.

Variable foundation flexibility was modeled by supporting the rigid steel base of the model on four plexiglas columns (Fig. 20). Overturning of the model was prevented by prestressing the columns by a line of bolts along an axis perpendicular to the direction of vibration. The range of variation of the prototype subgrade modulus between 3.5 kg/cm^3 and 25 kg/cm^3 was modelled by changing the lengths of the perspex columns between 17.5 mm and 140 mm. An effectively rigid foundation (subgrade modulus - 1000 kg/cm^3) was obtained by replacing the plexiglas columns with steel columns.

Natural frequencies, mode shapes and associated damping of the first four mode shapes were determined by subjecting the model to sinusoidal excitation on a shake table. The model was then subjected to random noise tests to establish expected elastic response to an earthquake of 30 sec. duration in the prototype. In each case the effect of the foundation flexibility was studied by supporting the model on deformable perspex or steel columns which simulated the various foundation conditions examined in the investigation.

Good agreement was observed between measured and computed mode shapes and the frequencies of vibration of the first four modes. This showed that the model behaved satisfactorily and that the assumptions made about the behaviour of the structure were valid. Random noise tests indicated that in the second vibration mode, significant stresses resulted at the fifth floor level; the effects of the second and higher modes are normally ignored in design.

5.2 Reinforced Concrete Frames Tested at the Earthquake Engineering Research Center of the University of California, Berkeley

Four single-bay, two-storey frames were tested for their seismic response at the Earthquake Engineering Research Center of the University of California, Berkeley as a part of an on-going research program on earthquake-restraint design of reinforced concrete structures (19). These relatively large (7/10 - scale) model frames were designed and constructed according to the standard seismic codes. This large scale also permitted the use of normal reinforcing bars and fabrication procedures. Additional concrete blocks were added for ballast to give the frame a typical period of vibration and to induce significant seismic forces during the tests (Figure 21). The frames were constrained against out-of-plane and torsional movements by means of suitable lateral bracing (Fig. 21).

Instrumentation used to record the response of the frame included accelerometers, displacement potentiometers and DCDT's (Direct Current Differential Transformers) at each floor level and on the shake table. Strain gauges were installed on column and beam reinforcing bars at locations where maximum strain readings were expected. Externally mounted gauges were used to measure curvatures at these stations. Moments and shearing and axial forces at midheight of the columns were measured using suitable calibrated force transducers. Approximately 100 channels were used for data acquisition and interpretation in each test.

The design of each test frame was varied by altering the member details to study the effect of the different member mechanisms on the overall structural response. Additional special instrumentation was used to monitor the anticipated response of this particular member.

The earthquake input signal used to test all frames was derived from the 1952 Taft Earthquake (N69°W Component) with no vertical components (Figure 22). The excitation was applied with increasing intensities starting with a peak acceleration of 0.07g in the first run and increasing to a peak acceleration of 0.44g in the final run. The objective was to introduce initial cracking, followed by a low excitation test on the cracked structure in the elastic range, and finally subjecting the frame to strong motions forcing the structure into the inelastic range. After each test, the flexibility, the free vibration frequency and damping of the test structure were determined by suddenly releasing a 1,000 lb horizontal force applied either at the first or the second storey level. The successive changes of these characteristics (Fig. 23) demonstrate the extent of damage imposed on the structure in each test run. A comparison of the top storey displacement measured during the most intense test run with a nonlinear response analysis is shown in Fig. 24.

The experimental data was used to assess the accuracy of available nonlinear analysis procedure and to improve the mathematical models used in these analyses.

The first frame failed due to flexural plastic hinging at the column and beam ends. Poor detailing of slab reinforcement anchorage into the transverse girders caused its early slipping and considerable flexural cracking at the column interface with bottom storey girders. Adequate correlation was achieved between analytical and experimental results. The second frame was almost identical to the first, except that the slab reinforcement was properly anchored. The damage caused to this frame during the test was similar to that observed for the first frame.

The third frame was designed to have an increased column moment capacity (heavier longitudinal steel with a higher yield strength) and therefore a relatively reduced shear resistance. Hinges were again formed at the column ends preceded by yielding of the main steel and crushing and spalling of the concrete. Significant bond deterioration and slippage resulted at the beam-column and column-footing joints on account of high cyclic straining. The experimental response is being analysed at present.

For the fourth frame, column length and sizes were decreased and special detailing was used for column and beam longitudinal bars. In additional the stirrup spacing was increased and extra concrete mass blocks were added to the frame. These modifications were intended to vary the mechanical characteristics for the frame and to accentuate the shear effects on its response. The frame failed due to diagonal shear-induced cracks in the columns with anchorage failure of the stirrups. Hinges were formed at the column bases where high steel strains showed loss of bond. Special instrumentation consisting of multiple diagonal displacement measurement devices and new force transducers was used to monitor the anticipated shear effects in this frame.

The increased flexural strength of beams and columns made the frame too strong to fail under the maximum intensity Taft earthquake that could be applied using the shake table (19). A special signal consisting of multiple cyclic square acceleration pulses with a frequency near the natural frequency of the structure was used to initially build up a resonant response with low amplitude input (Fig. 25). A significant inelastic displacement excursion was then induced using a large enough pulse in one direction which brought the table to its peak velocity of 25 in/sec (63.5 cm/sec). The observed table accelerations differed from the desired square wave due to depletion of oil in accumulators and limited pump capacity, however these were sufficient to cause shear failure of the columns. The velocity and displacement limitations of the shake table have caused difficulties in bringing the test structure into a collapse state or to inducing severe damage on the structure.

5.3 South Pier of Tagus River Bridge Tested at the LNEC, Portugal

Ferry Borges and Pereira (20) studied the dynamic behaviour of the south pier of the Tagus River suspension bridge at the LNEC Laboratories. This 86 m high reinforced concrete pier weighed 82,500 metric tons and had a base of 22.4 x 38.4 m. The pier supported a 180 m high steel tower weighing 5,700 metric tons. Details of the pier are shown in Fig. 26.

The pier was modeled at 1/150-scale by a timber prism. The foundation environment was simulated in a cylindrical steel tank as shown in Fig. 27. The steel tower was simulated by a steel bar and the effect of the dead load was modeled by a spring system as shown. The cylindrical tank and the foundation system model were mounted on a shake table. The reaction transmitted to the foundation for horizontal motion of the pier was measured by dynamometers at the base of the model.

To simulate the varying environmental conditions existing around the pier foundation, the following four conditions were studied:

1. Model without any surrounding medium.

2. Model surrounded by water.

3. Model surrounded by water, silt and sand.

4. Same as in 3 above but with air under pressure in an attempt ro reproduce the total pressure in the silt.

To determine the values of the seismic coefficient for the standard earthquake used at the LNEC (duration T = 30 seconds, a power spectral density S_o = 675 $cm^2\ sec^{-4}/Hz$ and frequency range 0.2 to 5 Hz), four values of spectral density namely, 250, 500, 750 and 1000 $cm^2\ sec^{-4}/Hz$, were used and the value of the seismic coefficient was calculated by interpretation of the results. Approximate magnitude of damping for each of the four test conditions was also evaluated. The experimental results were summarized as follows:

Condition of the Test	Seismic Coefficient T = 30 sec $S_o = 675\ cm^2\ sec^{-4}/Hz$	Damping, Percent (Percussion Test)
1	0.18	3
2	0.16	8
3	0.12	20
4	0.08	-

Once the construction of the prototype pier was completed, it was subjected to vibration tests by applying an impulsive load of 15 tons which was applied by breaking a calibrated steel bar stressed by a cable which was pulled by a tugboat. The natural frequency and damping measured in these tests were 1.4 Hz and 5 per cent respectively showing satisfactory agreement with the model test results.

5.4 Ruck-A-Chucky Bridge Tested at the Earthquake Engineering Research Center of the University of California, Berkeley

The proposed Ruck-A-Chucky bridge over the American River in California (28, 29) consists of a curved box girder bridge with a width of 48.9 ft. (14.9 m), and a depth of 8 ft. (2.4 m) and a radius of curvature of 1500 ft. (475 m), supported on 48 cables individually anchored to the walls of the river canyon (Fig. 28). The dynamic response of this unprecedented bridge to an earthquake especially in the vertical direction is dependent on its length, curvature and its flexibility and is therefore quite complex to analyse. Recognizing this, Godden and Aslam (27, 28) undertook a model study to verify the accuracy of the analytical model used to determine the seismic response of the bridge and to identify such behaviour characteristics that may not have been considered in the analysis and which may influence the design of the structure. These tests were performed on the shake table at the University of California at Berkeley (19).

The girder has a constant cross-section along the entire span and it lies in a horizontal plane. The 48 cables are anchored at the neutral axis at transverse girder sections 50 ft. (15.2 m) apart; these cables vary in length from 225 ft. (68.6 m) to 1250 ft. (381 m) and in cross-sectional area from 5.65 in^2 (3650 mm^2) to 7.80 in^2 (5030 mm^2). The total bridge span between cable anchorages is about 2806 ft. (85.5 m).

No provision was made for an expansion joint in the steel design and the girder is fixed at each abutment (27, 28). The continuity eliminates the large dynamic forces that can develop at points of discontinuity and therefore it improves the bridge seismic response significantly. However the alternate concrete design may have required an expansion joint at midspan and therefore it was decided to study the bridge behaviour for both cases - with and without the expansion joint. This relative data can be helpful if consideration is to be given to the concrete design.

The model was very flexible and therefore the support system and the instrumentation required had to be carefully designed to eliminate any significant interaction with the model. Therefore, the natural frequency of the supporting frame was made much higher than the model frequencies studied, and it was ensured that the table motions were accurately applied to the model.

Fine-stranded stainless steel wires weighted with lead shots to compensate for the prototype dead weight were used for cables (Fig. 29). The model girder was constructed from a 0.16 in. (4.1 mm) thick sheet of aluminum alloy. Transverse steel plates were used as lumped masses to simulate the prototype dead weight. After the tests on the bridge model were completed, the midspan joint was introduced. This consisted of a typical reinforced concrete bridge joint design reproduced to scale by machined aluminum components.

As mentioned before, the instrumentation was designed not to add any significant mass, stiffness or damping to the system. Eight cable forces were measured by small cantilever spring devices acting as measurement anchorages. The axial flexibility of the cable consisted of the wire, tension spring, and measurement anchorage acting in series. Girder forces were calculated from measurements at mid-span, quarter-span and abutment sections. Vertical girder accelerations at mid-span and quarter-span were measured using miniature accelerometers.

The only prototype characteristic which was distorted in the model was the axial rigidity (AE). It could not have been practically reproduced and the influence of the distortion of axial stiffness on the dynamic response of the bridge was studied at an early stage. Godden and Aslam (27, 28) noted that the lowestmode significantly influenced by the axial stiffness was f_6 which was primarily a symmetric transverse mode and had a relatively minor influence on the vertical modes or the transverse antisymmetric modes. They concluded that a disproportionately large AE value would have only a secondary effect on the total seismic response of the bridge. However, at a later stage the value of AE in the computer model was altered to that of the model to obtain a valid comparison between the experimental and computed data.

Preliminary static and dynamic tests were conducted on the model to establish the accuracy of the model and the experimental procedure. Two sets of dynamic tests were performed on the shake table at the Earthquake Engineering Research Center of the University of California, Berkeley - one to shake the model in the x-and z-directions and the second to shake the model in the y-and z-directions. An artifical earthquake of 15 second duration and with horizontal and vertical peak accelerations of 0.12 g and 0.08 g respectively was generated for the bridge site. These characteristics were time-scaled and used for shake table command motions.

Godden and Aslam (27, 28) conducted an intensive study on the continuous bridge model and a comparison of measured and computed response data led to the conclusion that the bridge was effective in resisting all horizontal components of ground motion due to its continuity and the horizontal curvature of the girder. The primary response of the bridge was due to the vertical components of the ground motion, and for the motions studied it was largely first mode response. It was noted that although the linear analysis neglected the cable vibrations, it was reasonably accurate for design purposes. Cable vibrations had little effect on the total behaviour of the bridge. It was further concluded that if the bridge were to sustain a local damage due to very severe ground motions, or due to differential ground motions (which were not studied in this investigation), this damage would be in the girder and would not impair the overall integrity of the bridge since the cable forces would remain relatively small.

After all tests on the continuous model had been completed, the bridge model was modified by introducing the midspan expansion joint. Godden and Aslam (27, 28) observed that although the continuous and the modified bridge had similar overall vertical responses, transverse bending and axial forces may be larger in the modified model and may include impacting.

The interested reader can find more details in Reference 27.

6. CONCLUSIONS

Similitude requirements governing the design and behaviour of structural models subjected to dynamic loads, especially due to similuated earthquake forces and ground motions are presented. Dynamic properties of some commonly used model materials are summarized and the effect of vibration frequency on their elastic moduli and damping characteristics is illustrated.

Some of the recent developments in testing techniques and laboratory facilities in North America, Europe and Japan are briefly described. These include pseudo-static and pseudo-dynamic tests using controlled load and deformation testing facilities and the newly developed computer-actuator on-line system. The use and limitations of earthquake simulators or shake tables for investigation of response of structures to simulated earthquake ground motion is discussed. Also, some of the instrumentation used is briefly described.

Four examples of structural models used for earthquake-resistant design are presented.

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TABLE 1

SUMMARY OF SCALE FACTORS FOR EARTHQUAKE RESPONSE OF STRUCTURES

		DIMENSION	SCALE FACTORS		
			TRUE REPLICA MODEL	ARTIFICAL MASS SIMULATION	GRAVITY FORCES NEGLECTED PROTOTYPE MATERIAL
1	2	3	4	5	6
Loading	Force, P	F	$S_E S_\ell^2$	$S_E S_\ell^2$	S_ℓ^2
	Pressure, q	FL^{-2}	S_E	S_E	1
	Acceleration, a	LT^{-2}	1	1	S_L^{-1}
	Gravitational Acceleration, g	LT^{-2}	1	1	neglected
	Velocity, v	LT^{-1}	$S_\ell^{1/2}$	$S_\ell^{1/2}$	1
	Time, t	T	$S_\ell^{1/2}$	$S_\ell^{1/2}$	S_ℓ
Geometry	Linear Dimension, ℓ	L	S_ℓ	S_ℓ	S_ℓ
	Displacement, δ	L	S_ℓ	S_ℓ	S_ℓ
	Frequency, ω	T^{-1}	$S_\ell^{-1/2}$	$S_\ell^{-1/2}$	S_ℓ^{-1}
Material Properties	Modulus, E	FL^{-2}	S_E	S_E	1
	Stress, σ	FL^{-2}	S_E	S_E	1
	Strain, ε		1	1	1
	Poisson's Ratio, ν		1	1	1
	Mass Density, ρ	$FL^{-4}T^2$	S_E/S_ℓ	*	1
	Energy, EN	FL	$S_E S_\ell^3$	$S_E S_\ell^3$	S_ℓ^3

$$^*\left(\frac{g\rho\ell}{E}\right)_m = \left(\frac{g\rho\ell}{E}\right)_p$$

TABLE 2

RESULTS OF STRAIN RATE EFFECTS ON UNCONFINED COMPRESSIVE STRENGTH

Average Strain Rate (in./in./sec.)	Head Speed of Instron Machine (in./min.)	$f'_c = 1730$ psi		$f'_c = 1860$ psi		$f'_c = 2270$ psi		Average
		No. of Specimens	$\frac{f'_{cD}}{f'_c}$	No. of Specimens	$\frac{f'_{cD}}{f'_c}$	No. of Specimens	$\frac{f'_{cD}}{f'_c}$	$\frac{f'_{cD}}{f'_c}$
1×10^{-5}	0.02	5	1.040	7	1.023	2	.968	1.021
1×10^{-4}	0.20	4	1.109	7	1.116	3	1.153	1.122
1×10^{-3}	2.0	4	1.206	3	1.199	3	1.160	1.187
1×10^{-2}	20.0	3	1.367	0	-	3	1.303	1.335

TABLE 3

CLASSIFICATION OF VARIOUS SHAKE TABLES

LOCATION	DIMENSIONS ft	PAYLOAD LIMIT lb	a_{max},g		d_{max} ±in.		f_{max} Hz	TYPE OF SUPPORT
			HOR	VERT	HOR	VERT		
1	2	3	4	5	6	7	8	9
SMALL (< 10 ft.)								
Stanford University	5 x 5	5000	5	-	2.5	-	50	Roller Bearing
University of Calgary	4.5 x 4.5	2000	20	-	3	-	*	*
ISMES, Italy	10 x 6.5	300	100	-	*	-	800	Oil Film
Drexel University	4 x 6	205	10	-	.25	-	2000	Roller Bearing
MEDIUM (10 - 30 ft.)								
University of California, Berkeley	20 x 20	100,000	1.5	1.0	5	2	15	Air Pressure
University of Illinois	12 x 12	10,000	7	-	4	-	100	Flexible Support
Corps of Engineers	12 x 12	12,000	34	60	2.2	1.8	200	*
Wylie Laboratory	17 x 11	9,500	8	8	3	3	500	*
LARGE (> 30 ft.)								
National Research Center, Japan	50 x 50	1,000,000	0.6	1.0	1.2	*	16	*
Berkeley-Proposed	100 x 100	4,000,000	0.6	0.2	6	3	-	*

* Information not available

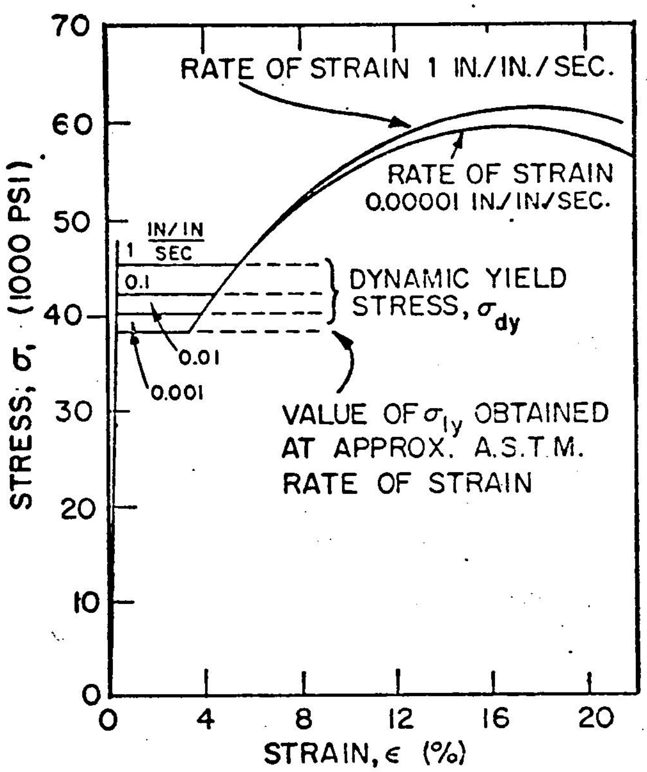


Fig. 1 Effect of Rate of Strain on Stress-Strain Curve for Structural Steel (Reference 5)

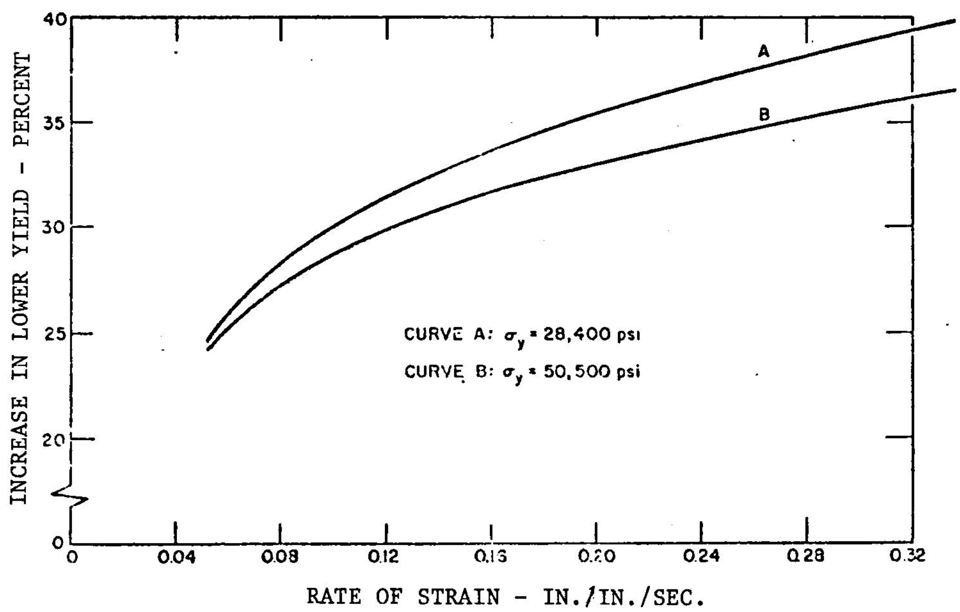


Fig. 2 Increase of Lower Yield Point of Steel with Strain Rate (Reference 6)

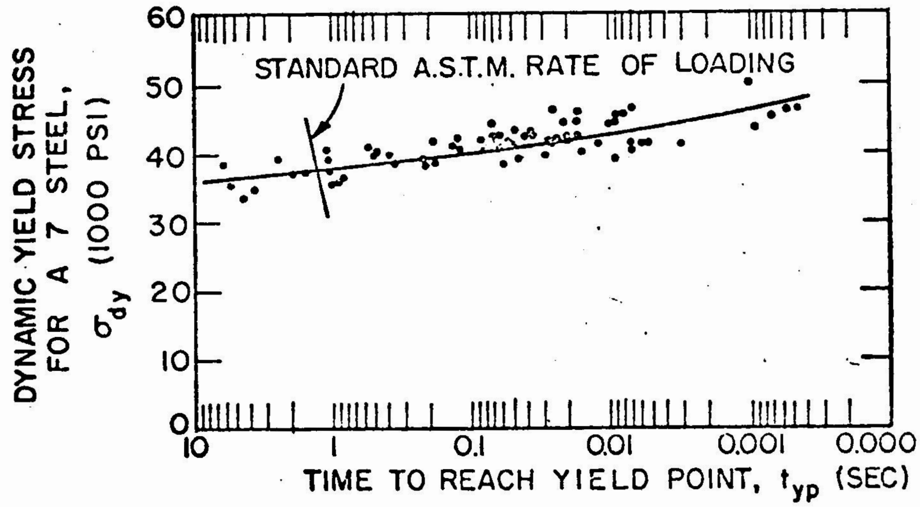


Fig. 3 Effect of Rate of Strain on Yield Stress (Reference 5)

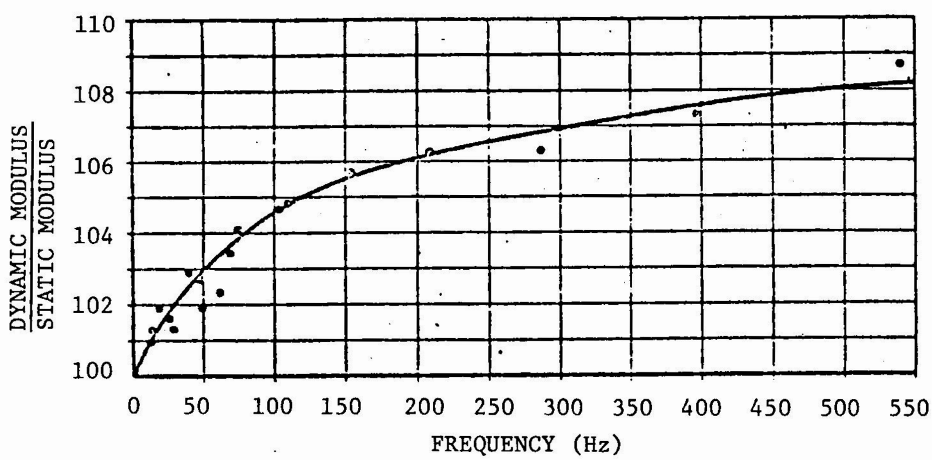


Fig. 4 Modulus of Elasticity vs Frequency for Structural Steel (Reference 4)

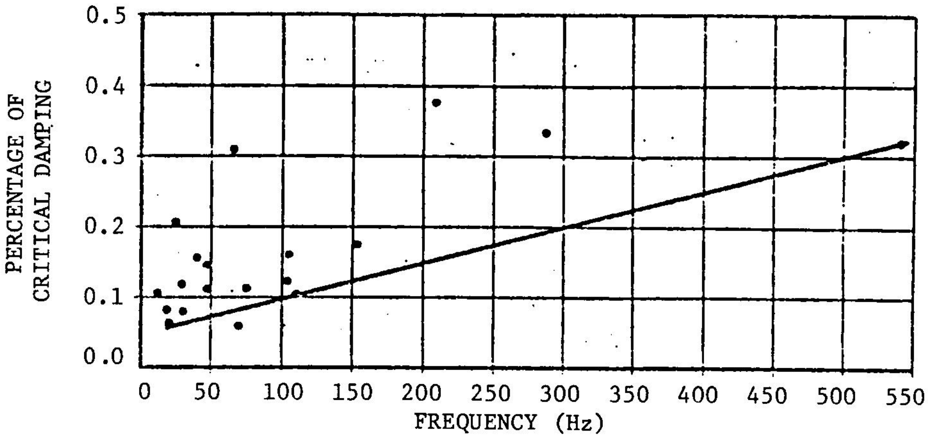


Fig. 5 Damping vs Frequency for Structural Steel (Reference 4)

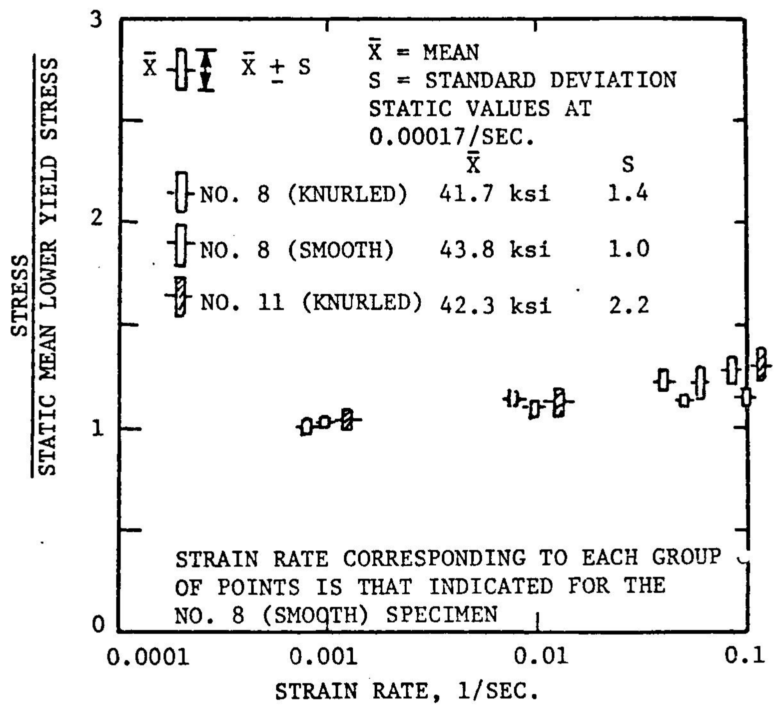


Fig. 6 Ratios of Standard Deviation and Mean Lower Yield Stress to Static Mean Lower Yield Stress at Different Strain Rates For No. 8 and No. 11 Gage Black Annealed Wire (Reference 7)

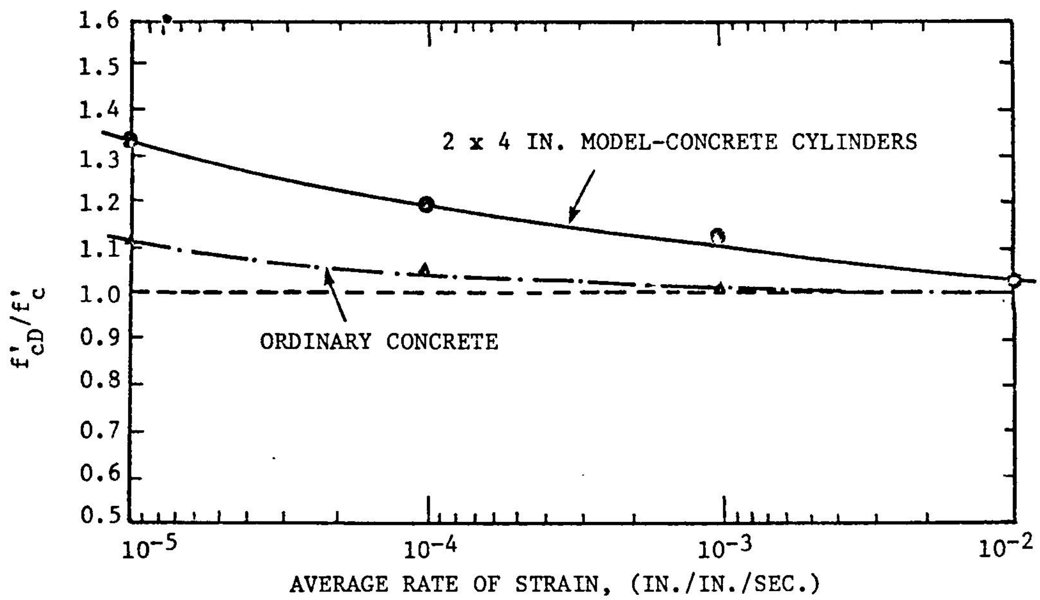


Fig. 7 Effect of Increased Strain Rate on the Unconfined Compressive Strength (Reference 12)

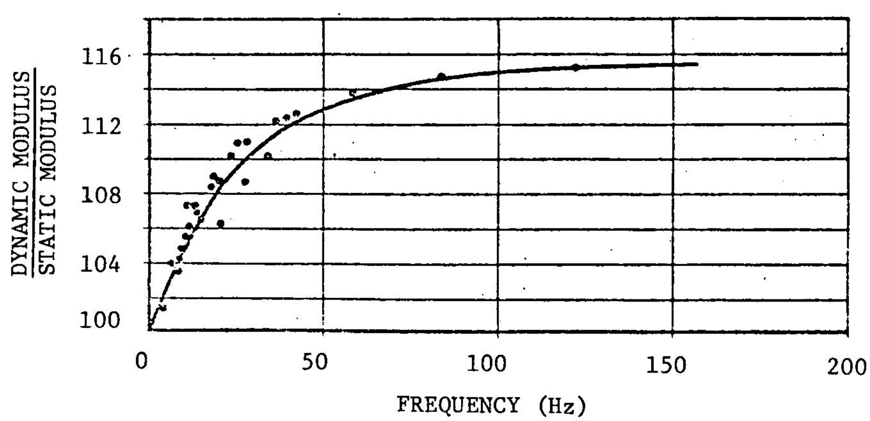


Fig. 8 Modulus of Elasticity vs Frequency for Model Concrete (Reference 4)

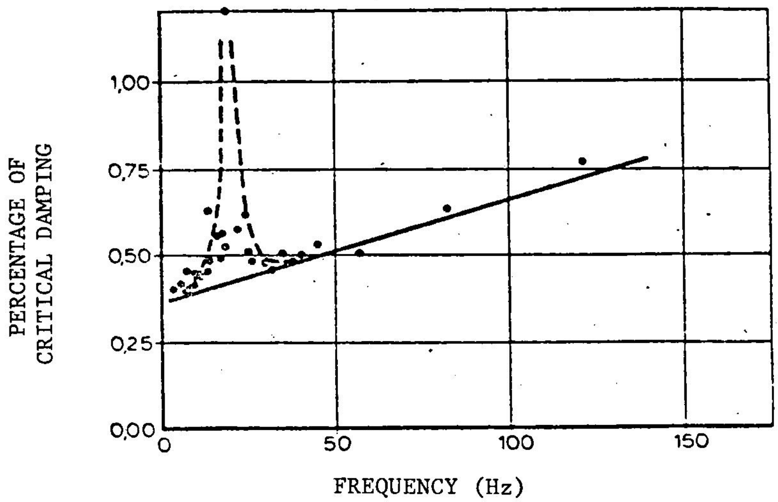


Fig. 9 Damping vs Frequency for Model Concrete
(Reference 4)

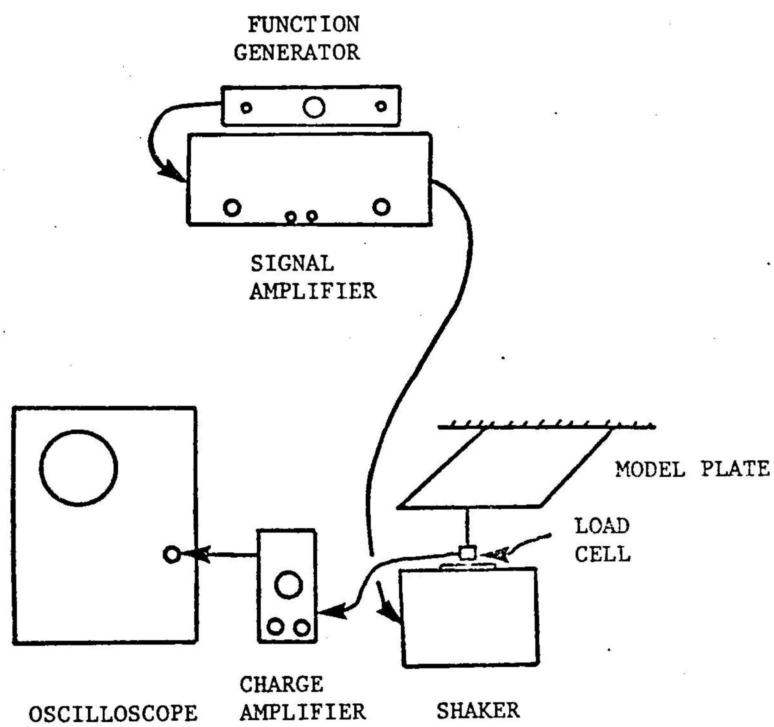


Fig. 10 Vibration Test Set-Up

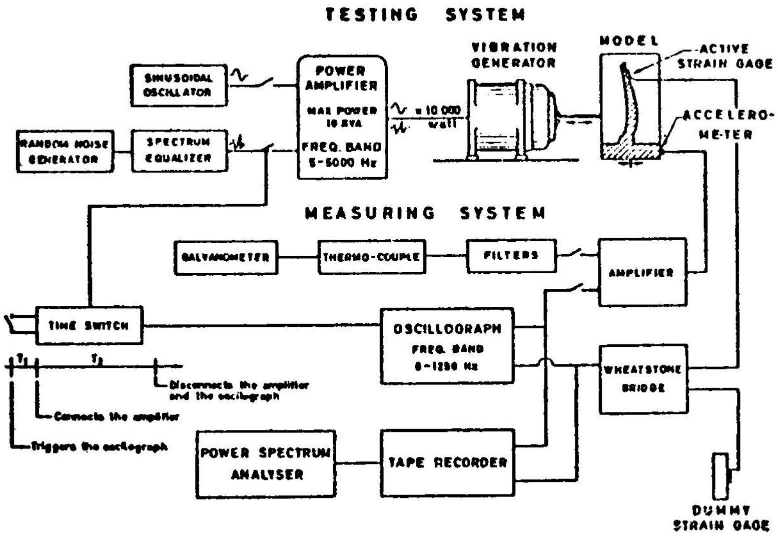


Fig. 11 Testing and Measuring Systems at the Laboratorio Nacional de Engenharia Civil, Lisbon (Reference 4)

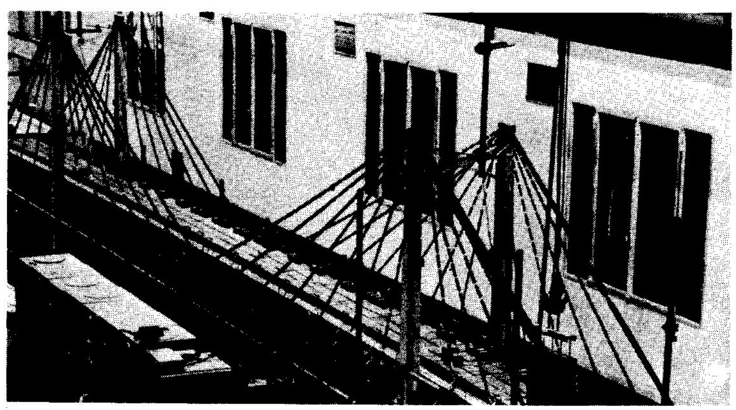


Fig. 12 Model of Zarate-Brazo Largo Bridges Tested at the Istituto Sperimentale Modelli e Strutture (Reference 23)

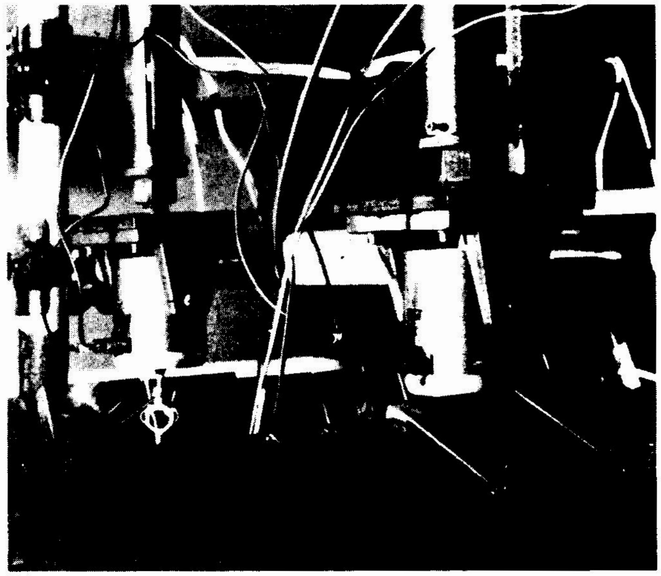


Fig. 13 Location of Vibration Generators for Determination of Dynamic Characteristics of the Bridge Model (Reference 23)

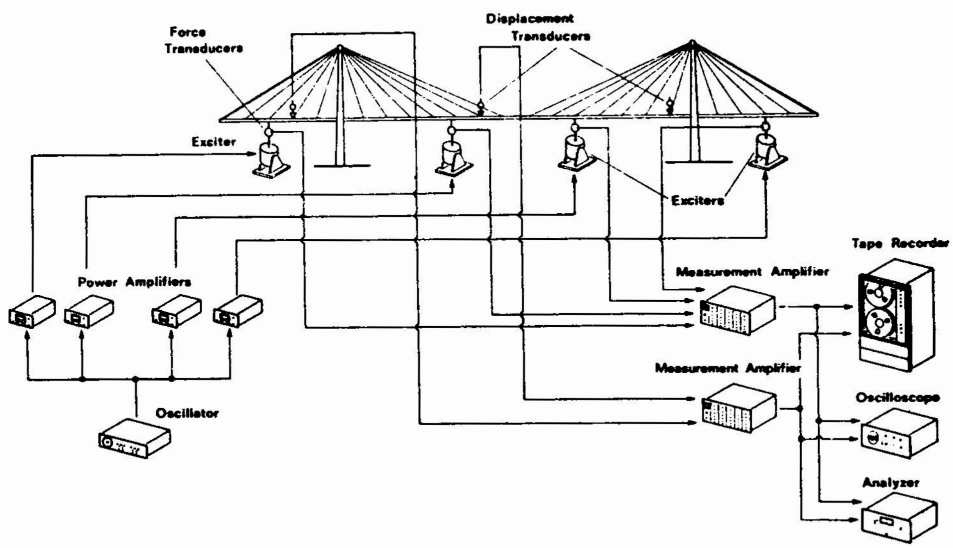


Fig. 14 Block Diagram of Excitation and Measurement System to Determine Vibration Modes by Means of Concentrated Forces (Reference 23)

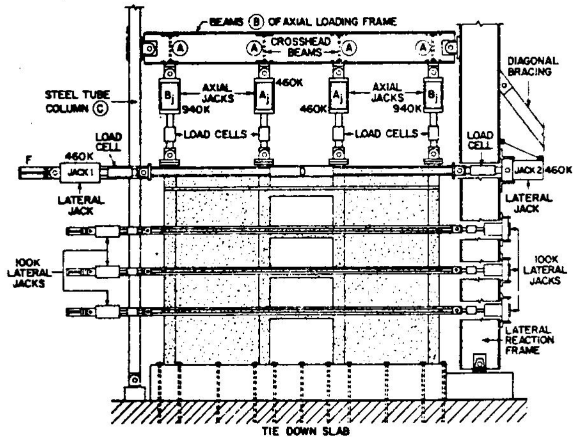


Fig. 15 Test Setup for Coupled Walls (Reference 19)

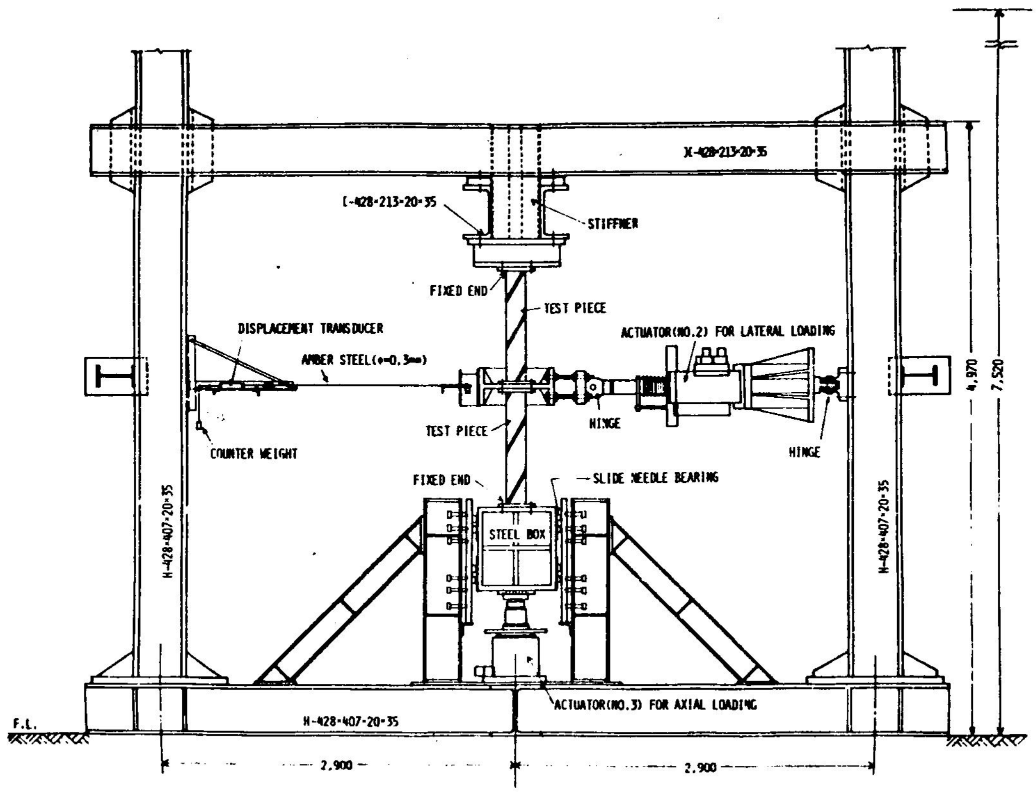


Fig. 16 Test Arrangement - Computer-Actuator On-Line System (Reference 21)

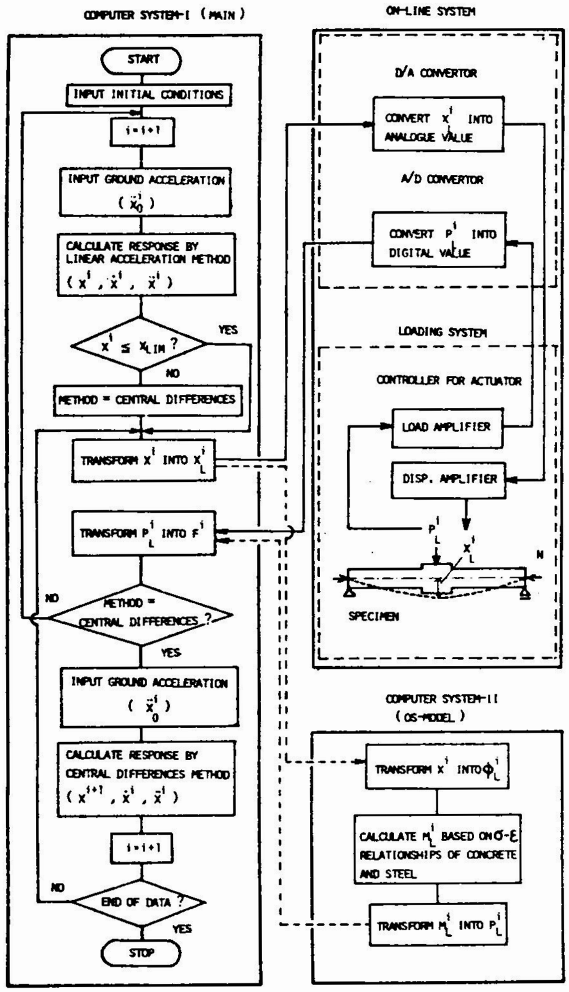


Fig. 17 Flow Diagram of On-Line System
(Reference 21)

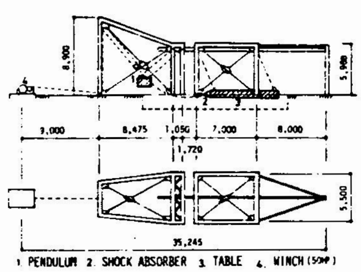


Fig. 18(a) Impact Loading System

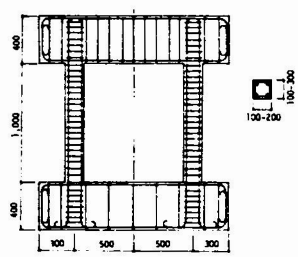


Fig. 18(b) A Typical Test Frame

(Reference 21)

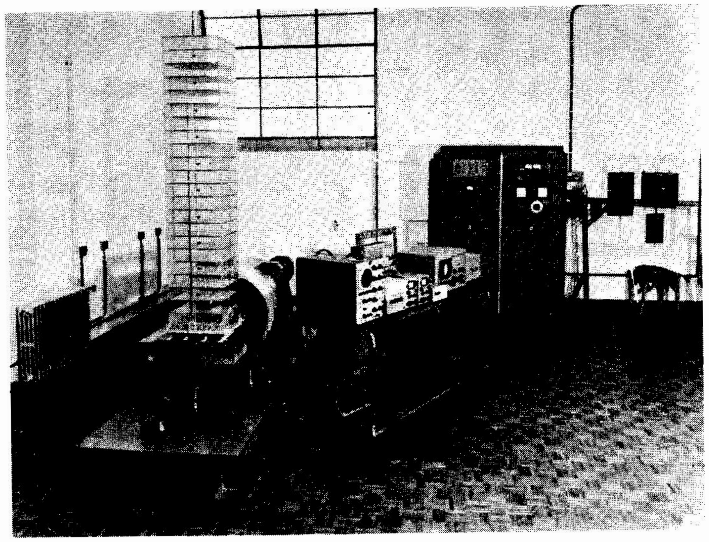


Fig. 19 22-Storey Building Model (Reference 4)

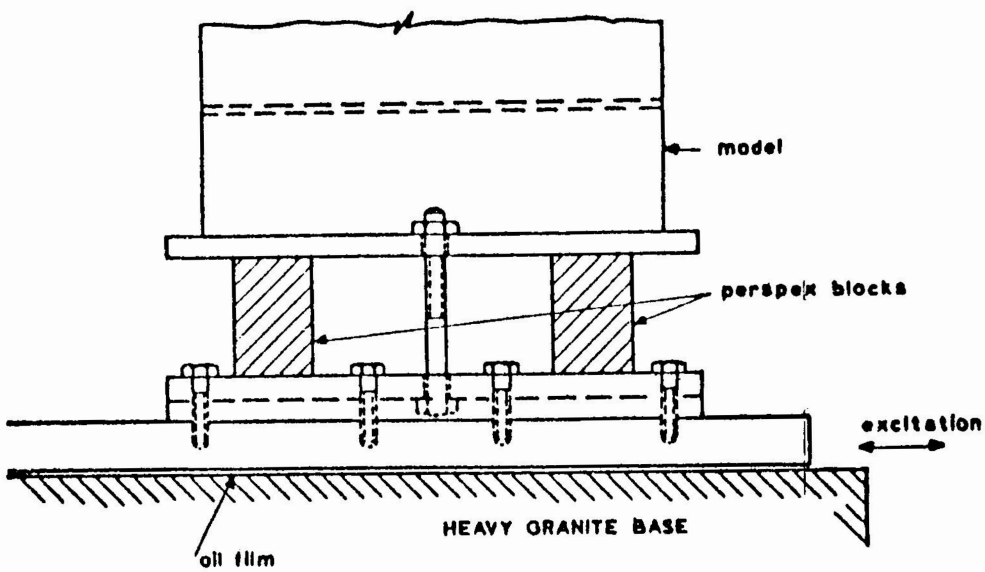


Fig. 20 Simulation of Foundation Conditions Using Perspex or Plexiglas Columns (Reference 4)

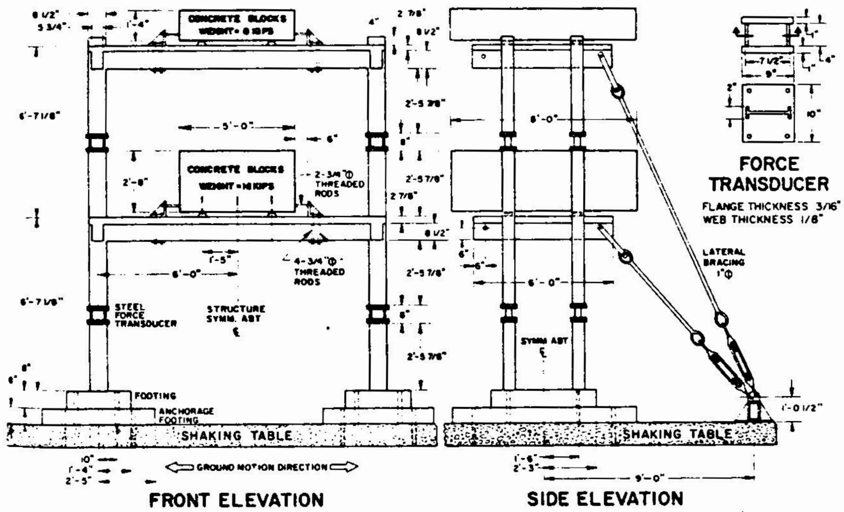


Fig. 21 Test Structure and Test Arrangement on Shake Table (Reference 19)

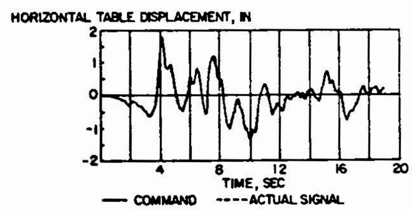
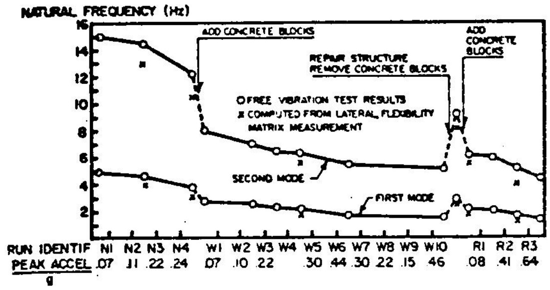
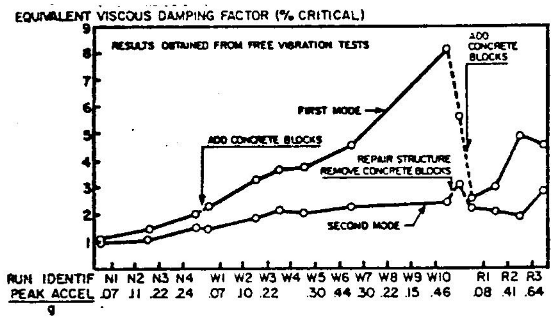


Fig. 22 1952 Taft No-69^{o}-W Component: Comparison of Command and Actual Table Signal (Reference 19)



(a) Frequency Variation During Test Sequence



(b) Damping Variation During Test Sequence

Fig. 23 Changes in Dynamic Characteristics of Frame 1 During Testing (Reference 19)

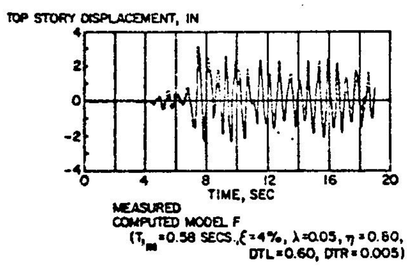


Fig. 24 Correlation of Computed and Measured Top Storey Displacement for Frame 1 (Reference 19)

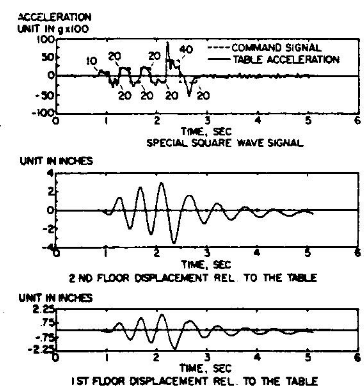


Fig. 25 Square Wave Input Acceleration Motion and Floor Displacement Responses for Frame 4 (Reference 19)

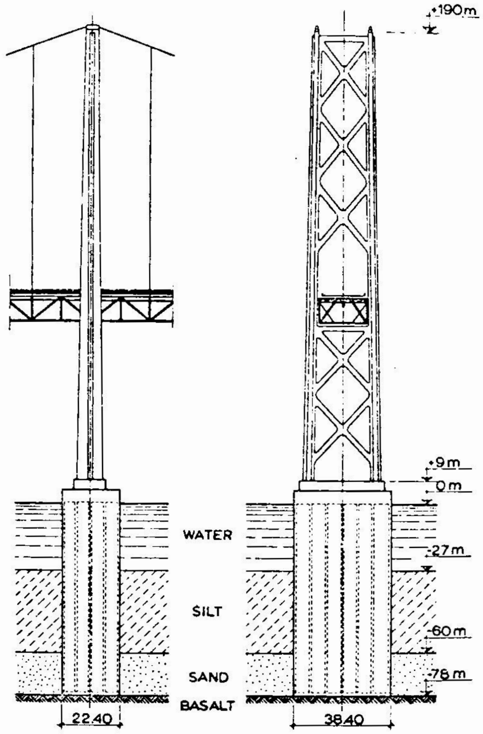


Fig. 26 South Pier of Tagus River Bridge (Reference 20)

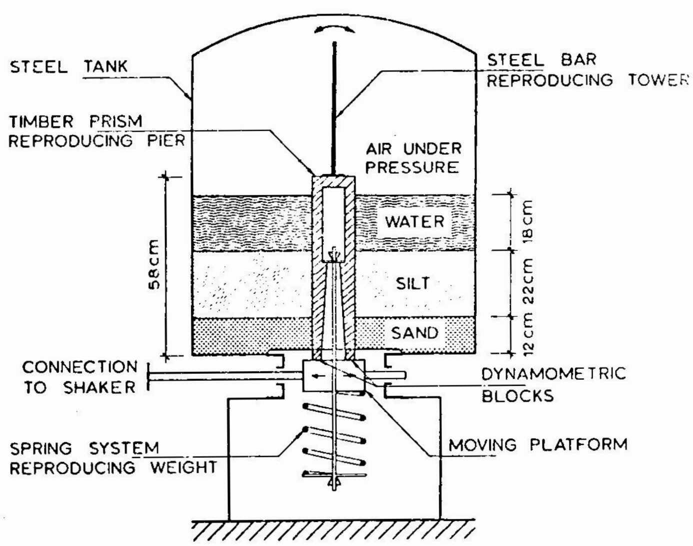


Fig. 27 Test Setup - Model of the Pier (Reference 20)

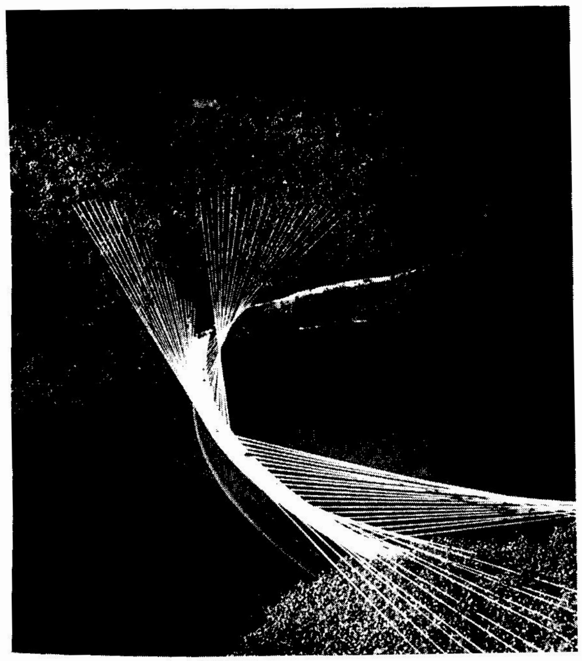


Fig. 28 Artist's Model of Ruck-a-Chucky Bridge
(Courtesy: Progressive Architecture)

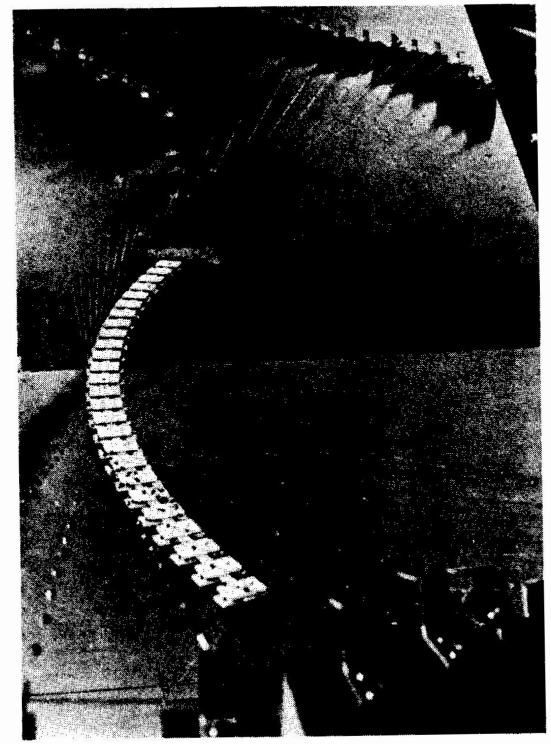


Fig. 28 Ruck-a-Chucky Bridge Model Under Test
(Reference 27)