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VIBRATION ISOLATION CONCEPT IN ASEISMIC DESIGN OF STRUCTURES

by

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IN

ASEISMIC DESIGN OF STRUCTURES

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İstanbul, June 1985 Ahmet ÇİVİ

ABSTRACT

The philosophy of conventional design principles is reviewed for earthquake resistant structures. It is emphasized that in order to achieve a higher degree of reliability and also to prevent the occurence of any cracking at critical points, especially for important structures, like nuclear power plants, a new technique of aseismic design should be developed.

The concept of vibration isolation, although used extensively in connection with machine foundations and bridge supports during the last one hundred-years, is introduced for important structures as an indispensable measure of security against earthquakes. A comprehensive review is presented about the types of vibration isolation available and their specific applications on real structures.

The mathematical formulation is introduced for the time history dynamic response analysis of structures with base isolation. In order to assess the degree of accuracy as well as to determine the validity of various assumptions of analytical studies, a steel model frame is experimentally investigated at the shaking table facilities of the *Institute of Earthquake Engineering and Engineering Seismology*, Skopje, Yugoslavia.

For each floor level, peak response values of accelerations and displacements have been calculated analytically and also have been measured instrumentally at the shaking table. A wide spectrum of peak input accelerations, ranging from 0.05 g to 0.70 g have been

considered in association with the 1940 El Centro, U.S.A. and the 1979 Montenegro, Yugoslavia earthquakes. The frequency contents of these earthquakes have been also varied by using a reduced time scale. The ratio of reduction has been taken as $\frac{1}{2}$ corresponding to the square root of the ratio of the geometric scale of the model.

Generally, the analytical investigations produced in very close agreement results to those obtained by laboratory measurements. Some discrepancies occured however, in the response values of the fixed base case, under real time earthquakes, on account of nonlinear behaviour of the model at the shaking table tests.

It has been established both by analysis and experiments that the peak response values of the structure are significantly reduced. Moreover the structure practically moves only in the rigid body modes, remaining always in the elastic range, when vibration isolation is used. It has been also concluded that the rubber elements are unable to provide any vibration isolation in the vertical direction, thereby being very susceptible to high degree of acceleration amplifications.

Finally, the influence of damping on the displacement response has been also investigated. The use of viscodampers in connection with helical springs as recommended by the writer, proved to be very successful in supplying adequate energy absoprtion capacity as well as in providing necessary isolation in all possible modes of vibrations.

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CHAPTER I

INTRODUCTION

1.1 CONVENTIONAL DESIGN PRINCIPLES

There seems to be sufficient knowhow and scientific knowledge available today (1985) for predicting the period of occurrence and intensities of strong earthquakes at a given region. Since, it is considered impossible to decrease the seismic energy in any way, the structures are expected to be designed safely and reliably such that they withstand the effects of the severest earthquakes possibly to occur within their life times.

The ultimate objective of an aseismic design is to accomplish safety and reliability at the least cost possible. The contemporary principle of aseismic design requires that in addition to possessing adequate strength, a structure should be capable of undergoing large deformations without collapse and should be able to absorb energy by experiencing excessive elasto-plastic deformations.

The emphasis in this principle is to store into the structure as much energy absorption capability as possible, such that in the event of a large earthquake, the critical load carrying elements of the structure undergo large plastic deformations without collapse. It is allowed however, that all nonstructural elements, partitions, plasters, glassware, ornaments, pipes etc., may be damaged. As a consequence, this principle does not only involve additional cost due to special construction and detailing requirements, but the secondary elements in the structure may be damaged so substantially that prohibitively expensive repairs may be necessary after a strong earthquake. Additional disadvantages of this principle are;

a) the requirements of ductile design while increasing the cost are difficult to implement in the construction, especially for structures like shear towers, shells, braced frames, deep beams, masonry walls, etc.,

b) the necessity of achieving a high degree of strength and deformability in structural as well as nonstructrual members may impose severe restrictions on the architecture and functionality of these elements. For instance, the service lines in a nuclear power plant, when failed prematurely during an earthquake, will not allow a safe and orderly shut-down of the reactor,

c) the resulting design is questionable and may not be considered completely safe and reliable, because of the assumptions made on the material nonlinearity and plastic action, and also because of the uncertainties involved in assessing the dynamic characteristics of both the structure and the ground motion,

d) the safety of structure relies on and is ensured by the large plastic deformations which virtually mean damage to the structure itself. This is a very paradoxical and self denying concept since it requires to conceive that the safety, which literally allows for no damage, is accomplished by allowing for damage.

1.2 NEED FOR VIBRATION ISOLATION

Classical design philosophy for earthquake resistant structures is that the load carrying elements are expected to withstand the largest earthquake loads possible without collapse. The secondary structural elements, as well as the machinery, equipment and piping system may undergo however, severe damage to the extent beyond repair. Main structural members are allowable to undergo extensive plastic deformations developing plastic hinges and cracking at critical points.

In fact, the energy induced by the earthquake is absorbed through these plastic hinges. Therefore, the structural members are expected to deform inelastically under the action of loads caused by rarely occurring, extremely strong earthquakes, while preserving overall safety. This phenomenon is reflected in the design codes by dictating certain ductility requirements to ensure adequate deformation capability in the plastic range.

However, the absolute safety against earthquakes is not adequately ensured when conventional design principles are used. Further, no cracking or plastic hinging is allowed in some important structures like nuclear power plants, etc. Moreover, relatively higher level of ground motion intensities and much wider ranges of design spectrum characteristics are used in the aseismic design of these important structures than those required for ordinary structures. Consequently, under these stringent requirements of safety against earthquakes, the conventional design principles become cumbersome, unreliable, technically impractical and in most cases economically infeasible.

Hence, in order to achieve a higher level of safety against earthquakes and also to meet the new requirements of structural behaviour, earthquake engineers were obliged to modify the conventional design principles and in fact develop entirely different earthquake resistant design methodologies. The concept of vibration isolation is an outcome of such a pressing need.

1.3 OBJECTIVES OF VIBRATION ISOLATION

The aim of vibration isolation of structures is to control the seismic energy by dissipating it at ground level and also reducing the energy transmitted to the superstructure through period lengthening. This is achieved by decoupling the structure from the earthquake shaken surrounding. Although complete decoupling is not possible, a properly designed vibration isolation system substantially reduces the structural response and ensures absolute safety against earthquakes.

Special isolation devices like rubber pads or helical springs placed under the structure increase the flexibility of the structural system causing the fundamental frequency of vibration of the structure to be shifted far away from the energetic region of the earthquake spectra. Fig. 1.1 shows a typical acceleration response spectrum of an earthquake together with the possible ranges of the natural periods of vibration of medium rise buildings with and without isolation elements installed at the base. It is apparent that even

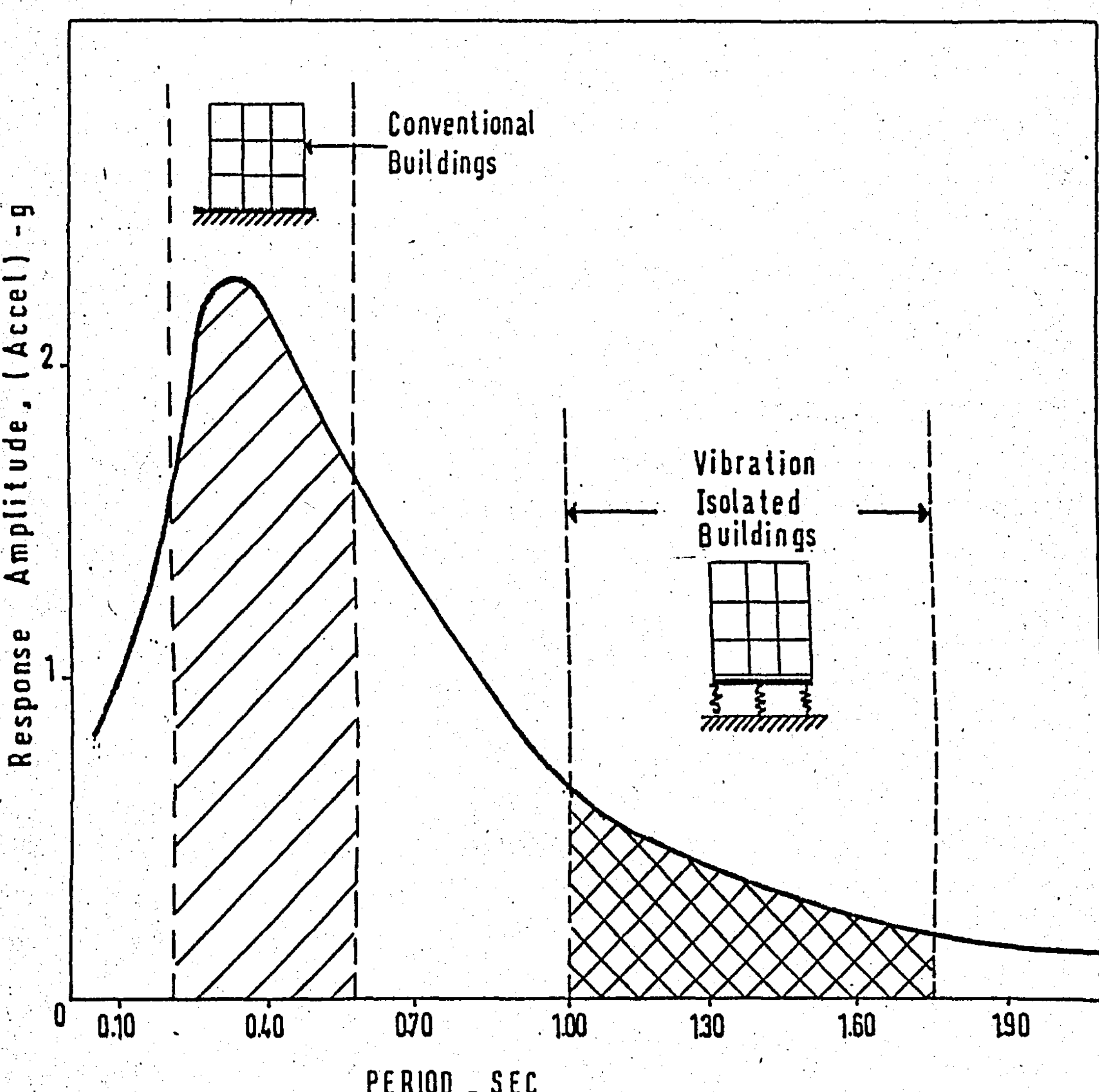


FIG. 1.1 - INFLUENCE OF VIBRATION ISOLATION

if resonance condition is not possible, a quasi-resonance condition is unavoidable in the non-isolated structures. However, the response is significantly reduced with isolation and the success of isolation is increased with increased flexibility.

The concept of vibration isolation is an entirely different philosophy of aseismic design, which allows the structure to behave only in the elastic range and does not require any plastic deformations. The energy of the earthquake motion is absorbed primarily by means of the vibration isolation devices installed at the base of the structure.

The purpose is to isolate the ground motion from the structure and thus to reduce the inertia loads and internal stresses. Contrary to the contemporary earthquake resistant design procedure, vibration isolators enable the structure to behave like a rigid body and restrict all deformations and stresses to remain within the elastic range, thus prevent damages to any structural or nonstructural elements.

To produce an earthquake proof design by means of vibration isolators is not only easier and less costly, but it also makes the safety of structures against earthquakes more reliable.

1.4 ADVANTAGES OF VIBRATION ISOLATION

Some of the advantages of vibration isolation may be summarized as follows:

a) The cost of achieving an earthquake-proof structure is less than that designed by contemporary aseismic design principles. The cost of a nuclear power plant for instance, in a region of very high seismic activity may be as much as 40% higher than the cost of the same nuclear power plant in a nonseismic region. When vibration isolators are used however, the increase in cost due to providing complete protection against earthquakes, becomes only a few percent of the overall cost of the nuclear power plant.

b) A greater degree of safety is ensured, since the uncertainties of the complex nonlinear behaviour are avoided and stresses are not allowed to exceed the yield levels. Analytical accuracy is less sensitive to approximations and assumptions in determining dynamic behaviour.

c) Designing and detailing of structural members become much simpler, and consequently a saving is achieved in construction time and cost.

d) Structural as well as nonstructural elements are subject to less number of load reversals, since the natural period of vibration is made longer.

e) Safe shut-downs in nuclear power plants would be much less frequent, since the earthquake response is much reduced. To shut-down a power plant facility could cost somewhere about US $ 350 000 to $ 400 000 a day.

f) A prompt restarting of the nuclear power plant and also the immediate use of the structural facilities become possible after a major earthquake, since neither the structural nor the nonstructural elements are expected to be damaged.

g) The nuclear power plant will not face the risk of being shut-down forever because of unexpected earthquake hazards not considered during the design phase. (As specific examples, the nuclear power plant in Diablo Canyon, California, designed to withstand an earthquake with a certain magnitude using elastic criteria, is not permitted by authorities to operate after new geological faults endangering the plant have been discovered in the vicinity. Similarly, Humbolt Bay nuclear power plant also in California was shut-down for maintenance and, after discovering certain cracks, is not allowed to reopen because of expected earthquake hazard.)

h) Design of structures and nuclear power plants already completed for nonseismic regions is largely valid for the same structures to be built in seismic regions. Thus, the standardized nuclear plant design for non-seismic regions may be duplicated for use in highly seismic regions.

i) Simplified and sophisticated mathematical models give almost identical results, while in the conventional fixed base analysis relatively more refined and sophisticated mathematical models are necessary.

k) Soil characteristics do not influence the response of structures in the same degree as in a conventional design.

l) Degree of seismic risk is very much reduced for important structures with vibration isolation.

1.5 THREE DIMENSIONAL NATURE OF VIBRATON ISOLATION

The concept of vibration isolation has been utilized in connection with rotating machinery and delicate instruments since over a century. However, only in the last decade, it is introduced as a practical technique for earthquake resistant design. Rubber elements have been already used as vibration isolators in the construction of some conventional structures and nuclear power plants. Many other base isolation systems such as ball bearings and floating platforms are yet in the stage of theoretical development.

The vibration isolation provided by the rubber pads is limited to the horizontal direction only. Rubber elements, although very flexible in the horizontal direction, are very stiff in the vertical direction. Therefore, it is not possible to eliminate the amplification of the vertical accelerations in the superstructure, due to the very high rigidity of these elements in the vertical direction.

The secondary structural elements, piping systems, machinery and other equipment would be subject to the excessive amount of vertical component of the response. Thus, although the horizontal response is significantly reduced the overall safety of the structure would be hindered on account of the existence of such large vertical amplifications. In addition, due to lack of sufficient damping in the rubber elements, horizontal displacements also become unacceptably large.

Earthquakes excite the structures not only horizontally, but also vertically, i.e. in a three dimensional way. This fact further increases the importance of the flexibility of the isolation elements in the vertical direction.

In order to achieve vibration isolation in all three directions, a number of new vibration isolation systems with appropriate elastic properties in all directions have been introduced. Helical springs and velocity proportional dampers recommended by the writer, proved to be very efficient in providing vibration isolation practically in all possible directions of motion.

The helical springs are very suitable for isolation of earthquake excitations since the ratio between their vertical and horizontal stiffnesses may be easily varied to meet the requirements of three dimensional isolation. The necessary damping is supplied by means of viscodampers so that both the acceleration and displacement response of the structure are reduced to any desired level.

Relative merits of helical springs and viscodampers over other base isolation systems will be studied extensively in the subsequent chapters.

1.6 OUTLINE OF THE THESIS

The state-of-the-art survey of vibration isolation systems available in literature is presented in Chapter 2. The theory, method of analysis and the computational aspects have been explained in Chapter 3. Shaking table test results of a five-storey model frame with and without isolation elements subjected to simulated earthquake excitations are given in Chapter 4.

Parallel to the tests, the model structure is analytically investigated for the same earthquake loads in order to be able to compare the results of tests with those of analytical studies. Results of analytical studies are discussed in Chapter 5.

Comparative evaluation of the analytical studies with those of the experimental works are presented in Chapter 6. Chapter 7 deals with other aspects of vibration isolation such as; application to nuclear power plants, influence of local soil conditions, safety precautions, etc. In Chapter 8, conclusions of the study as well as recommendations for further research are presented.

CHAPTER 2

LITERATURE SURVEY

2.1. GENERAL

The probable coincidence of the natural frequency of vibrations of medium rise buildings with the frequency of the earthquake waves is the major reason for the undesirable acceleration amplifications resulting in substantial damages in structures. In order to provide safety for the public a well as to reduce earthquake induced damages to structures, basically a conventional aseismic design approach is followed.

In the conventional aseismic design approach, the structure is expected to survive the largest earthquake ever to occur during the lifespan of the building, without collapse. But, it is indirectly envisaged and assumed that the nonstructural elements, such as partition walls, equipment, piping system, etc., may be damaged to the extent beyond repair. The nonlinear behaviour of the structural elements, as well as the plastic deformations to be developed at critical locations provide the main source of energy absorption. Even though the additional cost involved in achieving a proper aseismic design is considerable, the degree of assurance in safety is still questionable. There are also uncertainities involved in methods of analysis, in estimating the characteristics of the strong ground motion, in determining structural and soil material properties, etc. Moreover, difficult and complex construction details are necessary, which always add to the cost.

In buildings with sufficiently strong partition walls, as especially in relatively low rise rigid buildings, like power plants, hospitals, schools, etc., the possibility of total collapse is remote due to the inherent rigidity. The structure, however, will be shaken more violently than that of a flexible structure and a greater amount of earthquake energy will be transferred. Consequently, the machinery and piping system may be severely damaged rendering the building to be completely out of function. This result is an unavoidable but obviously an undesirable situation.

As an alternate solution, the structure is isolated at its base, from the incoming ground motions, and the aseismic design principles are completely altered. In this new concept of vibration isolation, the natural frequency of the building is reduced to the low energy portion of the earthquake spectrum, and the response of the structure is controlled at the foundation level, and no earthquake energy is transferred into the superstructure. The frequency shift is achieved by means of installing special flexible elements under the structure at the foundation level or by means of implementing special structural details to produce a relatively flexible structure.

When vibration isolation is introduced into a structure, its response to severe earthquake motions is reduced to a minimum. In fact, the whole structure behaves like a rigid body not allowing for any significant interstorey displacements. Thus the safety against earthquakes becomes more reliable and accurate.

2.2. HISTORICAL PERSPECTIVE

The idea of vibration isolation of structures against manmade vibrations has been envisaged in principle as early as the beginning of this century. Although a very early proposal for a base isolation system was described by J.A. Calantarients (1909) in England, the subject has drawn increased attention only in the last fifteen years. In fact, the Table 2.1 prepared by Eidingen (1983) and expanded by the writer shows the number of published references on vibration isolation since 1972. It is indicative of the ever increasing extensive research which has taken place on the topic in recent years.

The historical perspective on seismic isolation and its modern applications starts with the original idea first proposed by Calantarients in 1909. As explained by Kelly (1979) Calantarients, a medical doctor by profession, proposed a system in which the decoupling of the structure from the foundation is achieved by means of a layer of talc. Further, a set of ingenious connections are designed to allow the gas lines and sewage pipes to accomodate for large displacements. Hence, Calantarients was aware that the isolation system reduced the accelerations at the expense of large relative displacements to take place between the building and the foundation.

TABLE 2.1: PUBLISHED REFERENCES on VIBRATION ISOLATION SYSTEMS

YEAR	TOTAL REFERENCES	MAIN TOPICS			
		SEISMIC ISOLATION	ENERGY ABSORBERS	TEST RESULTS	ANALYTICAL STUDIES
1972	2	2	-	1	1
1973	4	1	3	-	1
1974	4	4	-	-	4
1975	7	5	-	1	6
1976	5	2	3	1	1
1977	13	8	5	7	6
1978	11	5	5	3	5
1979	8	7	5	4	4
1980	12	3	9	5	4
1981	8	7	1	3	4
1982	7	5	2	2	4
1983	9	5	2	2	3
1984	14	11	6	5	4
1985	11	6	3	4	5
TOTAL	115	71	44	38	52

2.3. FLEXIBLE FIRST STOREY CONCEPT

It is known from Engineering Mechanics that any irregularity of shape causes stress concentrations at these locations. Likewise in building structures, existence of any stiftness or mass irregularity at any storey results in concentration fo responses at that level. Flexible first storey concept, which has been proposed as a means for reducing structural response by controlling the earthquake effects at a prescribed location, is based on the above fact and described by several researchers.

In this method, rather than using special isolation devices, the first storey of the structure is designed purposely flexible. Earthquake energy and most of the deformations would be taken up in this flexible first storey allowing very little to be transmitted to the upper stories. Although the flexible first storey concept proposes an easily deformable, beam-column arrangement , it is expected that all the columns would remain fully in the elastic range and survive the earthquake with little or no cracks. This design concept is first proposed as early as in the 1920's by Nishkran (1927), Synden (1927), Marel (1929) and Green (1935).

2.4. SOFT STOREY CONCEPT

As notification to the flexible first storey concept Fintel and Khan (1969) introduced a new idea based on inelastically deformable soft storey. This idea was based essentially on controlling the amount of lateral forces to be transmitted to the upper storeys by building the first storey columns of the structure to be relatively weaker. They will act as a soft link between the ground and the upper structure and will yield at a specified horizontal shear force thus preventing the transfer of forces and deformations to the upper storeys. This concept is illustrated in Fig. 2.1.

Rintel and Khan, expressed after observing the Shopje, Yugoslavia earthquake in 1963 and the Carakas, Venezuela earthquake in 1967, that buildings having first storeys without shear walls suffered less dmage than those with shear walls.. The higher flexibility gained by the open first storey caused the system to behave like an invested pendulum and inelastic deformations are confined in the soft region. This so called soft storey portion of the building undergoes bilinear forms of elasto-plastic hysteresis providing an energy absorbing mechanism.

At higher levels, however, the structure remains within the elastic range due to the reduced response even during very high intensity earthquakes. With analysis performed on a single degree-of-freedom systrem, reduction in the acceleration responses to levels of 10 to 30 percent of the imput ground acceleration, is claimed to be achieved.

However, later in a study by Chopra et al (1973) on an eight storey building with a soft first storey, it was determined that the reduction of acceleration response is actually more modest than those values mentioned above, due to the complexity of multi-storey dynamic behaviour, which is ignored in the single mass example of Fintel and Khan.

The soft storey concept, has been also found to be impractical, since the first storey must exhibit essentially an elastic-perfectly plastic behaviour for the system to be considered effective. Any significant post-yield stiffness of hte columns would increase the shear force transmitted to upper storeys over the design values. Further, in order the full earthquake energy to be absorbed, very large lateral displacements, beyond desirable level, are required.

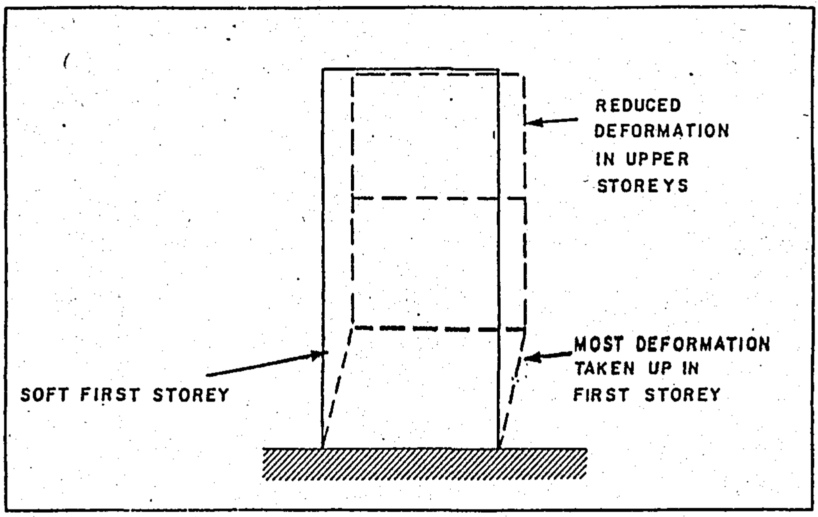


FIG. 2.1 - ILLUSTRATIVE EXAMPLE OF SOFT STOREY DESIGN CONCEPT

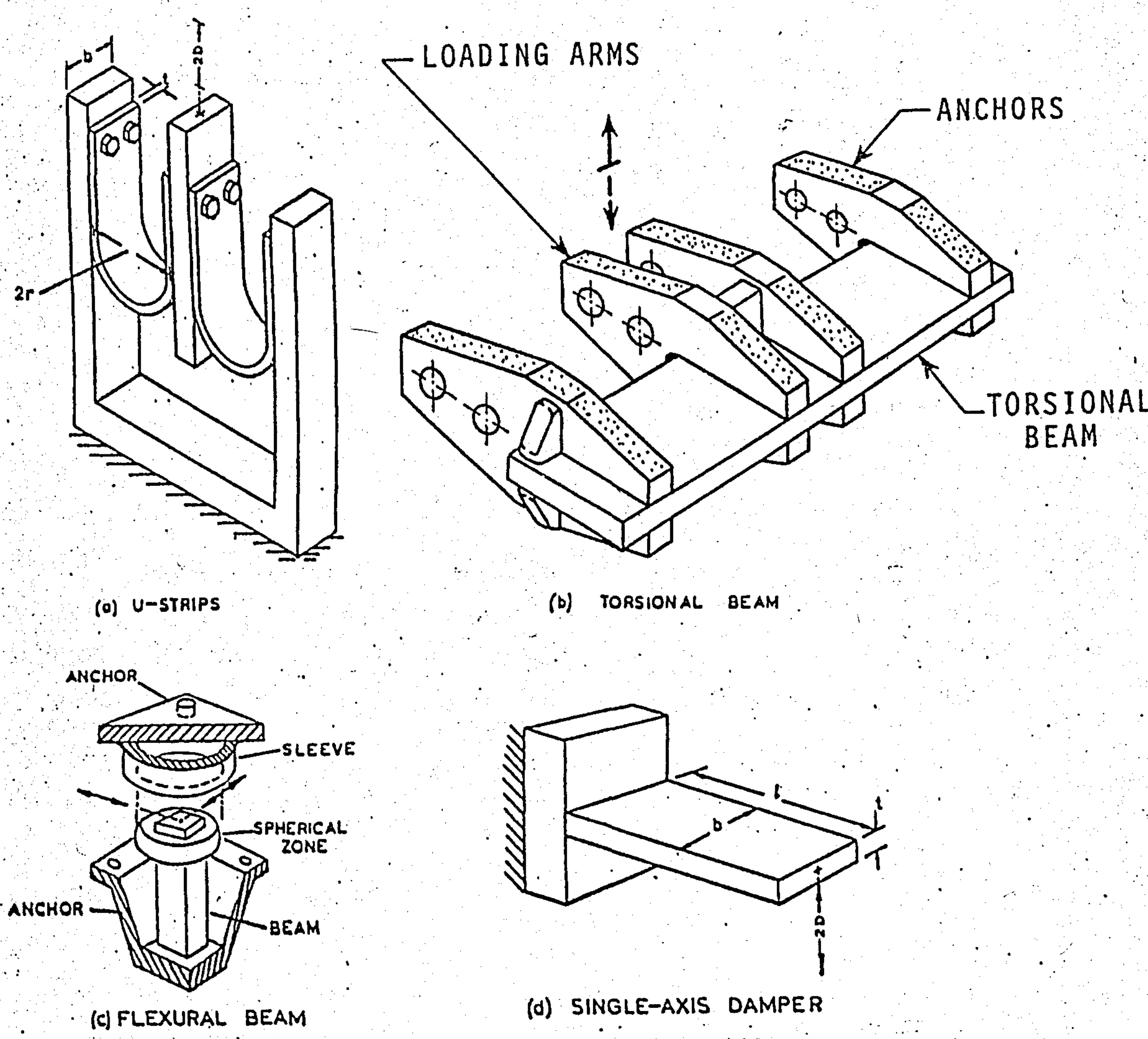


FIG. 2.2 - MECHANICAL HYSTERETIC DAMPERS

As alternatives to the soft storey concept, other special structural detailings have been introduced as tools of vibration isolation. Matsushita and Izumi (1965) have proposed a "double basement" concept whereby the rigid part of the basement is seperated structurally from the main building. Matsushita and Izumi also reported other structural mechanisms such as rod mechanism, flexible column system and double column system, etc. developed by the Japanese researchers for the aseismic design of buildings.

2.5. ENERGY ABSORBING DEVICES

A) HYSTERETIC DAMPERS out of MILD STEEL

Energy absorbing devices may be used to increase the earthquake resistance of structures. These special devices may provide damping by hydraulic pumping, viscous materials, surface friction, or hysteresis deformation of solids. A comprehensive reivew of energy absorbing devices have been given by Kelly and Skinner (1979).

Hysteretic dampers are the most common types of energy absorbing devices and extensive testing has been conducted to determine their load-displacement relationships, and fatique resistances, etc. Kelly et al (1972) and Skinner et al (1923, 1975) have visualized a variety of mechanical devices in the form of bending, flexural and torsional dampers. Energy is dissipated when the metal or along in the device is strained beyong its yielding point. This energy is absorbed by a change in the microstructure of the metal and by an increase in temperature. The basic damper types, which absorb energy by plastic deformation of steel members, as outlined by Skinner et al (1975), are shown in Fig. 2.2.

The first type of steel energy absorbers utilizes the rolling or bending energy of U-shaped strips of mild steel, while the second type is based on the deformations of square or rectangular bars in torsion and flexture with torsion dominating. The third type steel damper is a single cauti-lever of square or circular cross section, and is designed to operate for loads along any direction perpendicular to the beam axis. The fourth type steel damper is designed to operate when short rectangular steel beams are plastically deformed under a flexural load.

It is reported by Kelly et al (1972) that the torsional device has a life span of several hundred cycles which is more than adequate for earthquakes if suitable connections are made between the loading arms and the inelastically deforöed beam. The flexural damper and the U-shaped strip dampers also have long lifetimes within the range of 20 to 200 cycles.

The hysteretic dampers are designed basically to absorb seismic energy and they are not at all required to withstand any portion of the main structural load. On the other hand, the main structural components will no longer require large energy absorbing capacities and thus may be optimized for their required stiffness to resist gravity and wind loads only. This seperation of component functions lead to increased economy, reliability and safety of design.

Skinner et al (1977) reports that energy absorbing devices based on plastic torsion of mild steel bars are designed for use in the piers of the reinforced concrete Rangitikei railway bridge in New Zealand. The energy absorbers are so arranged that they become operative when the leg of the piers lift off from the base in a stepping action under large earthquake loading. A reinforced concrete chimney of 35 m high with cruciform section constructed at Christchurch Airport in New Zealand has been also designed to rock on its foundation over the energy absorbing devices. These mild steel torsion bars are activated and start absorbing energy as soon as vertical forces due to rocking exceed a prescribed value (12 tons).

B) LEAD EXTRUSION DEVICES

A very efficient method of absorbing energy is achieved by the cyclic extrusion of lead inside a dashpot-like absorber as introduced by Robinson and Greenbank (1976). A block of lead is forced by compression through an orifice changing its microstructre as it deforms. (Fig. 2.3). This restructuring of material produces heat, i.e. the mechanical energy is converted to heat. Lead is chosen as the metal to be extruded since it recrystallizes at room temperatures thereby recovering most of its mechanical properties almost immediately. Therefore, lead extrusion devices are very suitable for cyclic applications like seismic forces.

The extrusion energy absorber is a plastic, rate independent, creep resistant device. The amount of energy absorbed is not limited by work-hardening or fatique of the lead but by the heat capacity of the device, the melting point of lead being the upper limit. Because of interrelated processes of recovery, recrystalization and grain growth, which occur during and after the extrusion of the lead, the energy absorber is not affected by work hardening or fatique, but instead the lead is forever returning to its original undeformed state. Therefore the device when correctly designed has a very long life and does not have to be replaced after an earthquake. The tests have shown that extrusion devices sustained cyclic forces of more than 1000 cycles for exceeding the requirements for seismic applications.

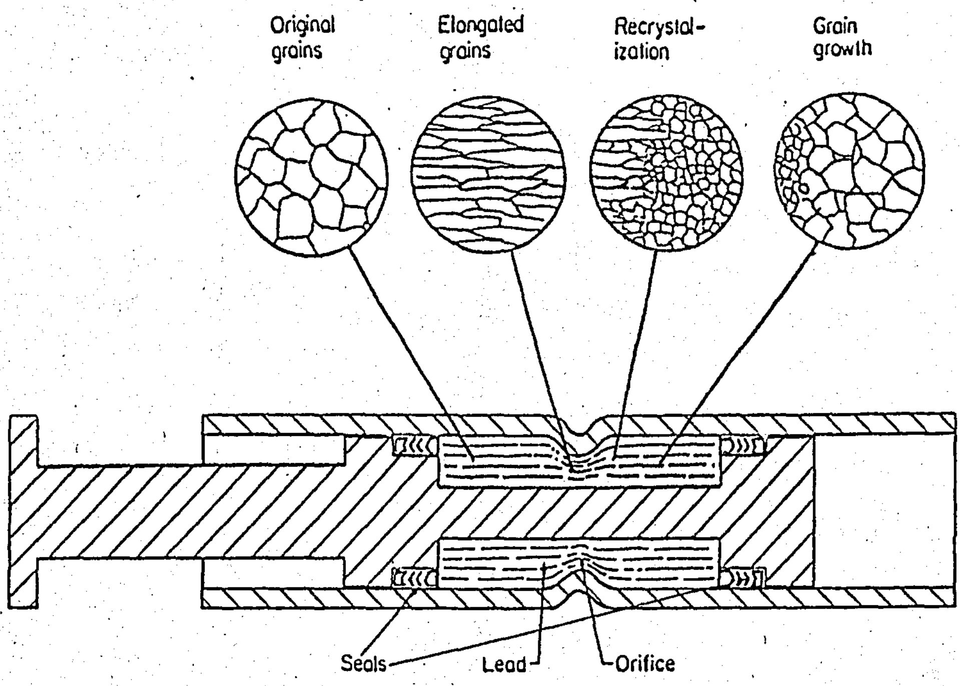


FIG. 2.3 - EXTRUSION OF A METAL SHOWING THE CHANGES IN MICROSTRUCTURE

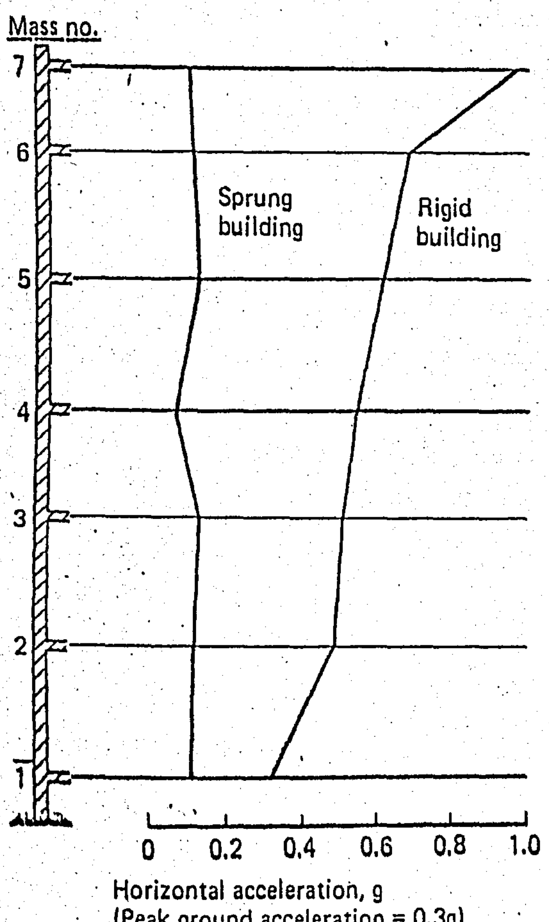


FIG. 2.4 - COMPUTED ACCELERATIONS AT FLOOR LEVELS OF A 5-STOREY BUILDING

Robinson and Greenbank (1976) also report that the lead extrusion energy absorber has been adopted by the New Zealand Ministry of Works and Development for two bridges for the Wellington motorway. The problem was to protect a sloping bridge from seismic ground motion while at the same time to resist the forces arising during emergency braking of downward-moving vehicles. Six extrusion energy absorbers are provided to resist the motion in the longitudinal direction and vertically mounted beams are designed to absorb the energy of transverse motion.

C) FRICTION DAMPERS

Energy is dissipated by converting the kinetic energy into heat when two contact surfaces are rubbed by friction. This principle is utilized in developing energy absorbing devices and a variety on friction dampers are proposed by Ezra and Fay (1971). The resisting force in a friction device is proportional to the coefficient of friction between contact surfaces which depends on various factors such as, temperature, humidity, surface conditions, relative velocities , etc.

Due to difficulties in determining these physical properties, friction dampers have not been widely used as mechanisms of energy absoption in aseismic design. Recently a friction damper system comprising an annular member of an inverted U-shaped section and several friction plates set on this member is proposed by Fujita et al (1983). It is claimed by the authors that the dampers when used in conjunction with laminated rubber bearings pressed the necessary qualities required for the earthquake isolation of heavy equipment.

D) TUNED MASS DAMPERS

A tuned mass damper composed of a mass, spring and a dashpot appended to a structure is an energy absorbing device used to minimize the earthquake response of structures. Gupta and Chandrasekanan (1969) studied this kind of a vibration absorbers for a single-degree-of-freedom (sdf) system subject to earthquake excitations. They argued that the tuned mass dampers are effective at only one frequency, thus they are not as effective for earthquake type excitations as compared to sinusoidal excitations.

Wirsching and Yao (1973) reported that by means of tuned mass dampers a reduction of fourty percent in the lateral displacements was possible at the roof of a structure. The heavy equipment at the roof can also be utilized as a tuned mass damper by mounting it to a spring and damper.

An alternate tuned mass absorbee systems may be developed by designing the top storey columns relatively more flexible and also utilizing the weight of the roof floor on the necessary mass. If necessary, addtional dampers may be added.

In a later paper, Wirsching and Campbell (1974) concluded from their studies on a 10 degree-of-freedom system that increase in the absorber mass has no significance on the energy absorbing effectiveness.

It happens that the energy absorbing devices of this type is best suited to regular flexible structures like high rise buildings. In fact, these absorbers have been already successfully used for wind protection in the Centerpoint Tower in Sydney, Australia; in the Citicorp Building in New York, in the John Hancock Building in Boston, etc.

2.6. PERIOD LENGTHENING DEVICES

A) HISTORICAL DEVELOPMENTS

Medium rise structures usually have natural frequencies of vibration falling inside the energy concentrated portion of typical earthquake spectrum and consequently they are exposed to larfe earthquake forces. One method of achieving reduced structural response is to arrange a base isolation system installed between the foundation and the superstructure so that the fundamental frequency of the structure is moved well far away from the governing frequencies of strong ground motion. Devices that isolate the structures from ground motions, in this way are usually spring-like elements such as helical springs, rubber pads, air pads, etc.

The principle of vibration isolation is a very old concept and has been widely utilized in the design of antivibration mountings for machinery, instruments, etc. and many books have been written on the subject as by Den Hartog (1956) and Harris and Crede (1961).

Vibrations produced by rotating machines are usually minimized by keeping the natural frequency of vibration of the structure far beyond the range of the dominant frequencies of the disturbing machinery. In cases, where this is not possible, the bqse of the rotating machinery is isolated from the main structure by means of soft springs in the form of either helical springs or rubber cushions, such that the fundamental natural frequencies of the structure and the machinery are seperated. In general, absolute protection against the undesirable vibration of the rotating machinery is achieved and safety is assured at the highest degree possible when the vibration isolation scheme is used.

BOĞAZİÇİ ÜNİVERSİTESİ KÜTÜPHANESİ

Also, vibration isolation systems are used to protect building structures from ground vibrations generated by nearby machinery, traffic, etc. Albany Court, for instance, a block of seven flats in London, (Waller, 1966) is constructed on rubber springs to isolate the structure from underground railway vibrations. Similarly, a cinema at Marble Arch, London (Mann, 1967) and a hotel in Swiss Cottage, London (Woottan, 1975) have been spring isolated to reduce vibrations from nearby railways.

Vibration isolation of structures against earthquake motions is similar in principle to isolation of machinery induced vibrations. However, there exist some important differences which have hindered the utilization of the concept in the seismic design of structures until recently. Firstly, it should be remembered that the predominant frequencies of rotating machinery are in the 10 Hz to 50 Hz range. The predominant excitation frequencies of earthquakes, however are generally between 1 Hz to 10 Hz. The predominant frequencies of rotating machinery are very much greater than the natural frequencies of ordinary structures, thus the possibility of quasi-resonance is very remote when compared with that of the earthquake. Secondly, the peak forces induced due to earthquake ground motions are much greater than those of machinery induced vibrations. Finally, due to hgher risks of loss of life and property as consequences of unpredictable behaviour or inadequate isolation, the safety standards are kept much more stringent in the aseismic design of structures. Therefore, further research is needed to fully develop the technology for seismic isolation.

B) APPLICATIONS of RUBBER PADS in BUILDINGS

1. SCHOOL BUILDING in SKOPJE

In spite of the above difficulties, especially in 1970's, vibration isolation concept has found practical implementation in the field of earthquake engineering. The first notable application of vibration isolation technique to resist the earthquake induced vibrations is reported by Roth et al (1970). Heinrich Pestalozzi School in Skopje, Yugoslavia was completed in 1969 using pure rubber elements for seismic isolation. Mechanical fuses in the form of ceramic rods were used in order to trigger the participation of the rubber elements only after wind or earthquake loads exceed certain limits.

2. MALAYSIAN RUBBER SYSTEM (MRPRA)[1]

Malaysian Rubber Producers' Research Accosiation (MRPRA)[(1)] proposed the use of natural rubber as the isolation element. Relying upon their successful experience on the use of natural rubber highway bridge bearings and building mounts for the isolation of noise and ground borne vibrations from various sources, they sugegsted that their product might in some form provide the needed isolation potential to achieve protection against earthquakes.

A joint study has been initiated between MARPRA and Atkins Research and Development[(2)] to study the effect of earthquakes on buildings with and without rubber bearings.

[(1)] Malaysian Rubber Producers Research Association, Brickendonbury, Hertford, SG13 8NP, England.

[(2)] Atkins Research and Development, Ashley Road, Epsom, Surrey, England.

Fig. 2.4 taken from Derham et al (1977) shows the predicted response of a five storey shear wall structure to an earthquake having a peak ground acceleration of 0.3 g. It is concluded that if the bearings could be constructed so as to make the horizontal natural frequency of the mounted structure equal to 0.5 Hz, the peak accelerations of the rubber mounted structure was calculated to be one-tenth the maximum accelerations experienced by the fixed-foundation structure.

3. SHAKING TABLE TESTS of RUBBER ELEMENTS

Experimental studies of natural rubber isolated structural models have been performed at the Earthquake Engineering Research Center at the University of California, Berkeley, U.S.A. A three-storey steel frame test structure of 40 ton in weight shown in Fig. 2.5 is excited on the 6 m by 6 m square shaking table. Comparisons were made between the rubber isolated and fixed base conditions. The first made horizontal natural frequency of the model structure was 2 Hz at fixed base condition and it is moved to 0.6 Hz by means of rubber bearings. The dimensions of the bearings installed are shown in Fig. 2.6.

It was not possible for these experimental bearings to be made by the usual commercial process of direct chemical rubber-to-steel banding during vulcanization. Instead, they were hand fabricated from sheets of rubber that were banded by vulcanization to aluminium foil. The aluminium foil was in turn bonded to the mild steel interleaves using double sided adhesive tape on the edges of the plate, and an epoxy resin adhesive in the center to increase their shear strength. The bearings made in this way, although neither as strong nor as durable as commercially produced bearings, were adequately strong for the attempted tests.

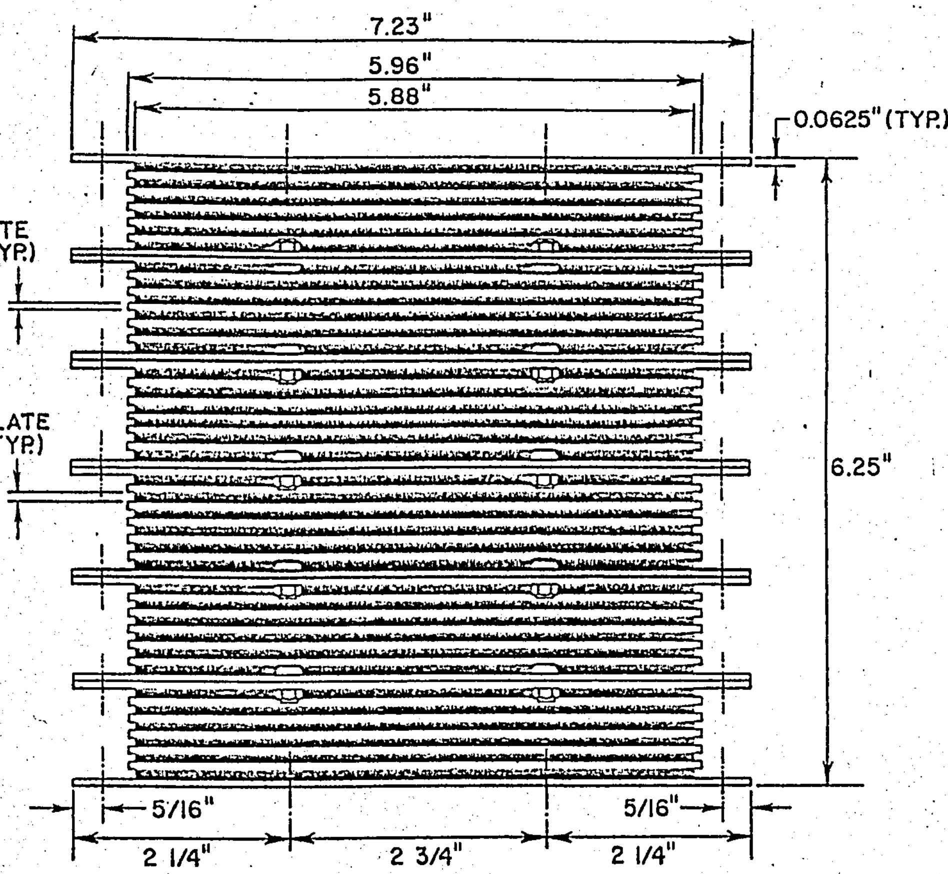


FIG. 2.6 - DIMENSIONS OF THE RUBBER BEARING PADS

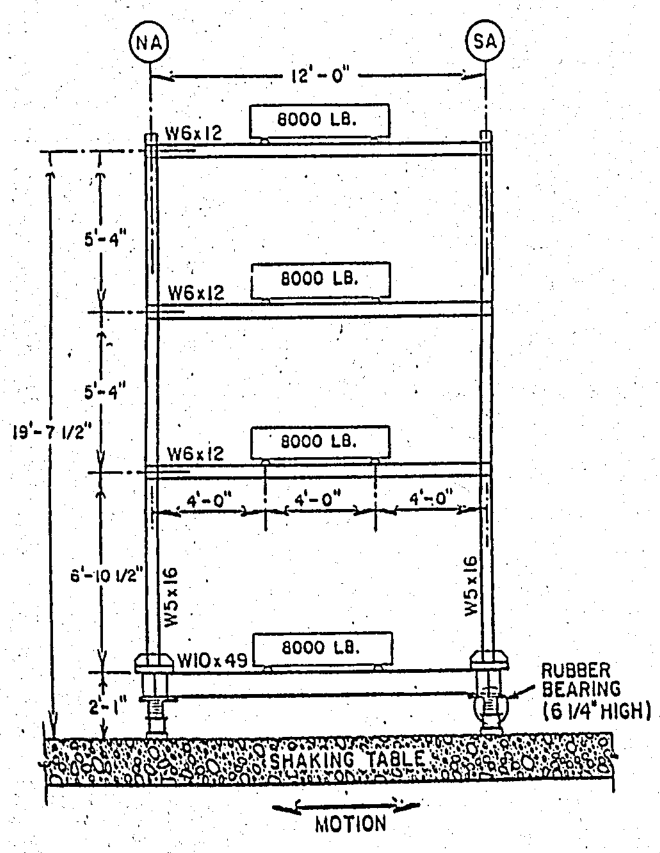


FIG. 2.5 - THREE STOREY TEST MODEL ON ISOLATED FOUNDATION

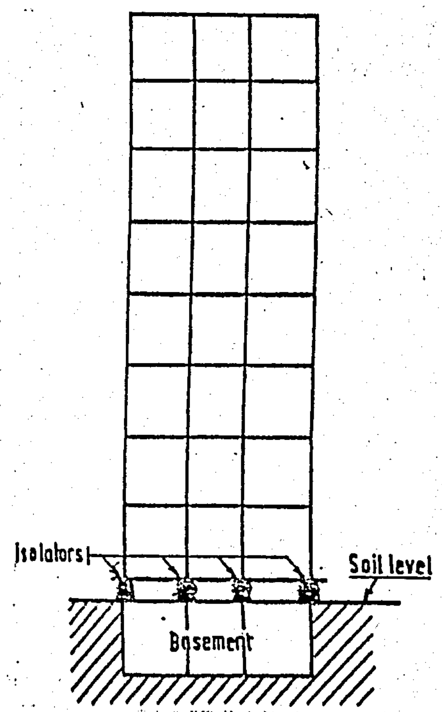


FIG. 2.7 - THE GAPEC SYSTEM

C) GAPEC SYSTEM of LAMINATED STEEL-RUBBER ELEMENTS

Another base isolation element is laminated steel-rubber bearing pad named as "GAPEC" system, as presented by Delfosse (1977). The bearing pads are placed between the first floor and basement as shown in Fig. 2.7. The system is developed at the Centre National de la Recherche Scientifique in Marseille, France. It is introduced as belonging to the same class of soft-storey vibration isolation systems but representing a further step in the sense that the soft storey is perfectly elastic acquired by the isolation bearings. The rubber bearings used are very stiff in the vertical direction. Horizontal stiffness is hundred times less than the vertical one and therefore the bearings are soft in shear and torsion.

Shaking table tests with a scaled model of a twenty storey building have been performed at the Mechanics and Acoustics Laboratory at the Centre National. The model frame weighing 940 kg is tested with and without vibration isolators. The first natural period of 0.10 sec of the fixed based case is increased to 0.18 sec with the isolators. It is reported that accelerations, shear forces and overturning moments were all reduced by a factor of approximately 8, when the isolation system was used.

A school building complex composed of three buildings seperated from each other by 10 cm, was built in the town of Lambesc near Marseilles in 1976. The school is approximately 77 m by 30 m in plan and three storeys high and placed on 157 steel reinforced natural rubber sheets of GAPEC type each of which is 30 cm in diameter and 5 cm tick. The first natural periods of vibration increased from 0.18 sec in the longitudinal direction and from 0.25 sec in the transverse direction to 1.70 secs in both directions when vibration isolators installed.

The building was constructed with prefabricated concrete panels and was designed to resist a MM intensity VIII earthquake. The accelerations, shear forces and everturning moments are drastically reduced with isolators that seismic requirements would not be otherwise satisfied unless large increase in the thickness of concrete and the cross-sectional area of reinforcement as pointed out in a review paper by Kelly (1979).

In order to prevent vibrations of the building during wind excitations or during mild earthquakes, proposed the use of simple mechanical devices called wind-stabilizers installed at the same level as the isolators. Since the building is fixed against ordinary wind loads and mild earthquakes, the occupants will not be disturbed by excessive oscillations. These wind-stabilizers will be automatically disconnected when the base shear exceeds a predetermined design vale, which is accepted to be exceeded also during a strong earthquake, leaving the structure free on isolators.

D) "SEISMAFLOAT" RUBBER ELEMENTS

A relatively new vibration isolation system consisting of solid natural rubber cylinders bonded to top and steel base plates has been proposed by Staudacher (1984). This system, which has been designed by the two Swiss Companies, Seisma AG, Zürich and Huber and Suhner AG, Pfaffikon is known as "Seismafloat" system.

These isolation pads are highly flexible in all directions and the elements with a height of h= 90 cm and diameter of Ø= 105 cm can accommodate cyclic deformations of ± 50 cm horizontally and ± 20 cm vertically without damage. The system is different from other rubber base isolation systems in the sense that isolation in the vertical direction as well as in the horizontal directions is provided. This is also confirmed by the shaking table tests of a 5-storey and 3-bay steel model frame conducted at the Earthquake Engineering Research Center of the University of California at Berkeley, U.S.A. The hysteretic damping capacity of these elements were determined to be in the range of 3% to 6% of critical.

Foam glass elements are added to the system as mechanical fuses to stabilize the structure under the action of wind and mild earthquake forces.

2.7. REFINED RUBBER ELEMENTS WITH DAMPERS

A) GENERAL

It is already mentioned above that the use of base isolation systems without any damping element may lead to significant amplification of displacements. Rubbre elements may inherently have a certain amount of damping, at best equivalent to approximately 5% to 10% of critical. In more earthquake input motion cases, higher damping capacity is required to reduce the displacements. Additional damping may be supplied by the inclusion of energy-absorbing devices into the isolation system. In this way, the structure is still isolated from its foundation on rubber bearings but displacements are limited by means of the energy absorbing devices.

Skinner et al (1976) proposed the use of laminated rubber bearings combined with energy absorbing hysteretic devices as a practical system for isolating nuclear power plants. The isolated system was about twenty times as flexible as the non-isolated structure which increased the displacements by 70 times. With the addition of hysteretic absorbers, however the displacements decreased drastically.

B) RUBBER ELEMENTS
with TORSION DAMPERS

A suitable combination of energy abcosrbing devices and rubber bearings is proposed by Kelly et al (1977). The authors pointed out that the energy abosrbing device used was based on the cyclic plastic torsion of rectangular bars of low carbon hot-rolled mild steel. The details of the design of the device are given in Fig. 2.8. The key energy absorbing element is the rectangular torsion bar to which torque is applied through the moment arms. The devices have been designed in such a way that welding is either well away from highly stressed regions or not present.

The authors also point out that the energy absorbig devices play two distinct roles in the response of the rubber-mounted system to earthquake loading. Since they are elastic for small displacements and their elastic sitffness is high relative to that of the rubber bearings they act as mechanical fuses and cause the system to behave as a high frequency system for small excitations. Thus, under small excitations the system amplifies the ground acceleration. As the excitation increases in intensity, the device yields and the frequency of the system drops. The system acts as an isolatior with a very high damping and no longer amplifies the input accelerations.

In addition to the effects due to the change in frequency, the devices also, through hysteresis, dissipate a very large amount of energy and thus act as dampers. This process also serves to reduce the displacements experienced by the rubber bearings under extreme excitation.

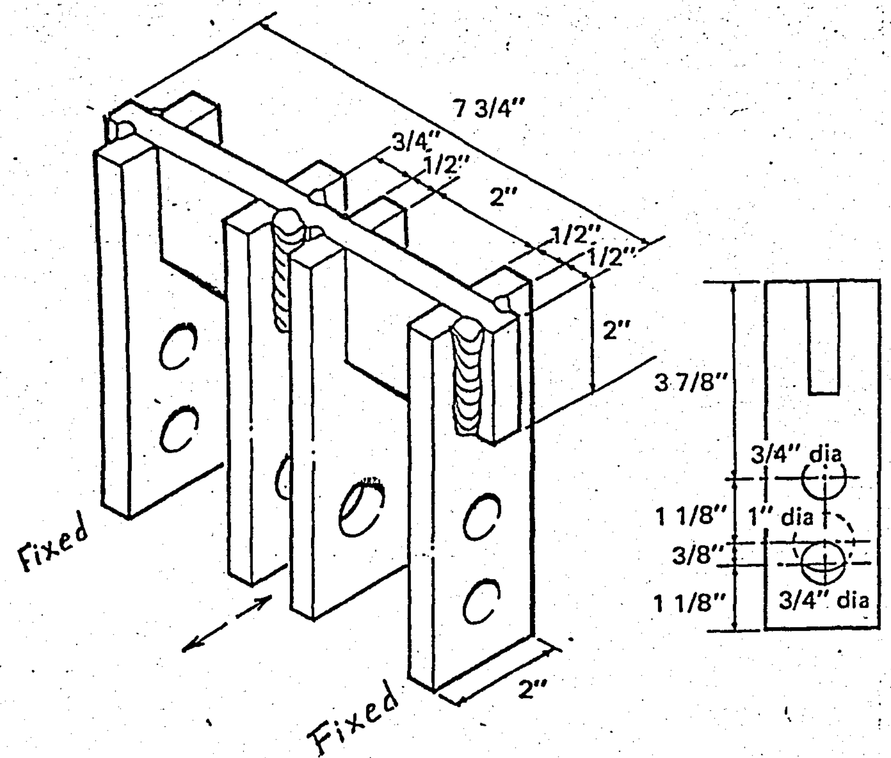


FIG. 2.8 - TORSIONAL ENERGY ABSORBING DEVICE AND ITS DIMENSIONS

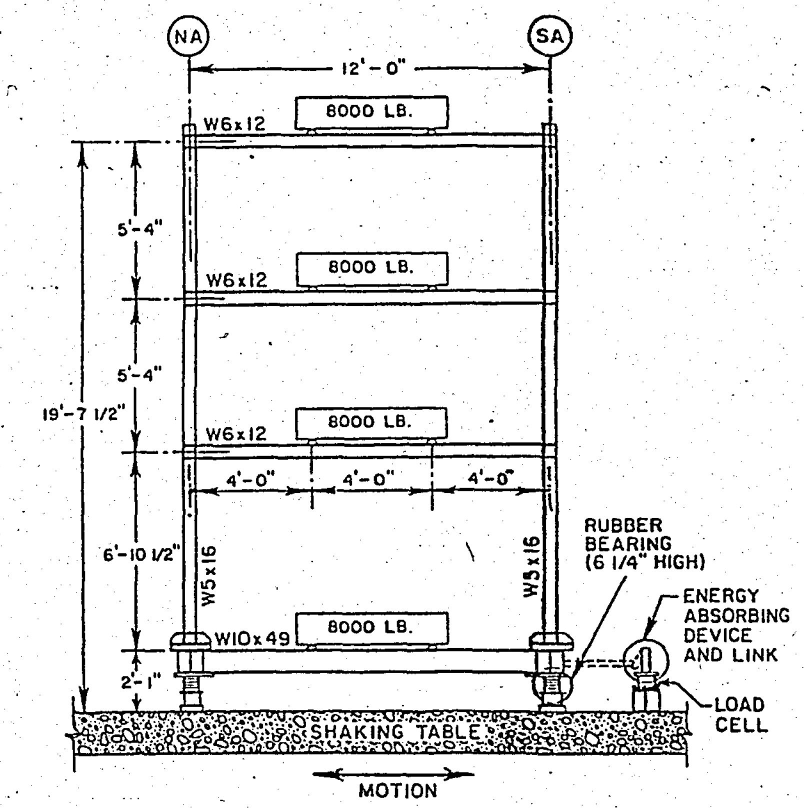


FIG. 2.9 - THREE STOREY ISOLATED TEST MODEL WITH TORSIONAL ENERGY ABSORBER

Experimental results of a typical structural model shown in Fig. 2.9 with rubber elements and torsional energy absorbing devices are reported by Erdinger and Kelly (1978). The tests carried out showed that accelerations were reduced by a factor of ten when a low damping rubber isolation system was used. Increased damping did not affect the peak structural accelerations; however, the displacements were decreased by 20% to 30% when the critical damping ratio was increased from 3% to 10%.

C) RUBBER ELEMENTS
with LEAD PLUG DAMPER

A four storey reinforced concrete government building located in Wellington, New Zealand is designed with laminated rubber bearings to provide adequate resistance to seismic forces as reported by Megget (1978). The laminated rubber pads utilized in the isolation system shown in Fig. 2.10 are of the type commonly used under bridge superstructures. A lead plug is insertd in a critical hole which has been drilled in the middle of the rubber pads. It is planned to deform in shear with the bearings to provide damping into the system. The maximum bearing displacement is estimated to be 150 mm and special service connections between the exterior and the interior have been designed to accommodate the horizontal motion.

D) RUBBER and
LAMINATED STEEL PLATES

1. GENERAL DESCRIPTION

A vibration isolation system consisting of laminated steel plates and rubber is developed by Spie Batignolles in collaboration with *Electricite de France* as reported by Jolivet and Richli (1977). The vibration isolation is achieved by means of a large number of elastomer rubber pads

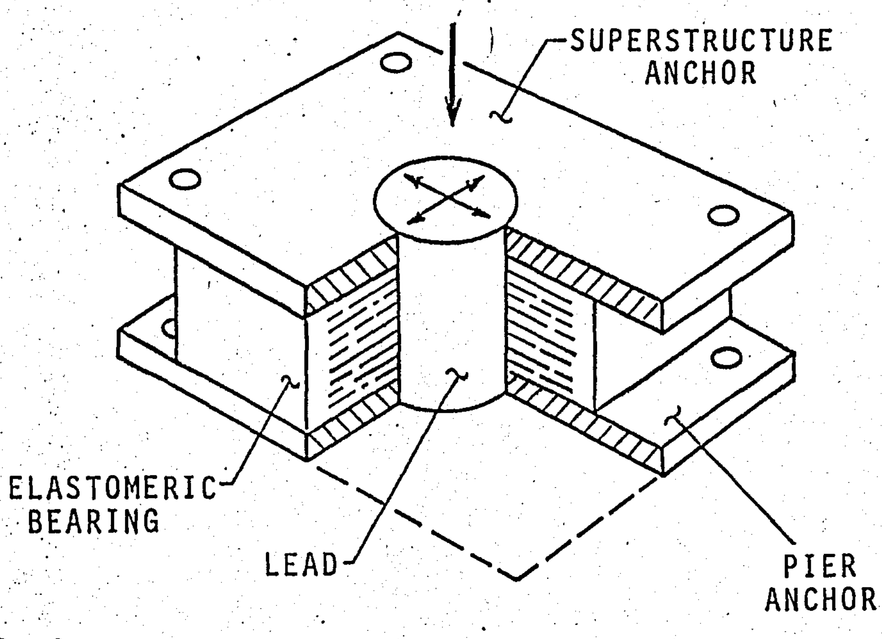


FIG. 2.10 - LEAD-RUBBER BEARING OF THE NEW ZEALAND SYSTEM

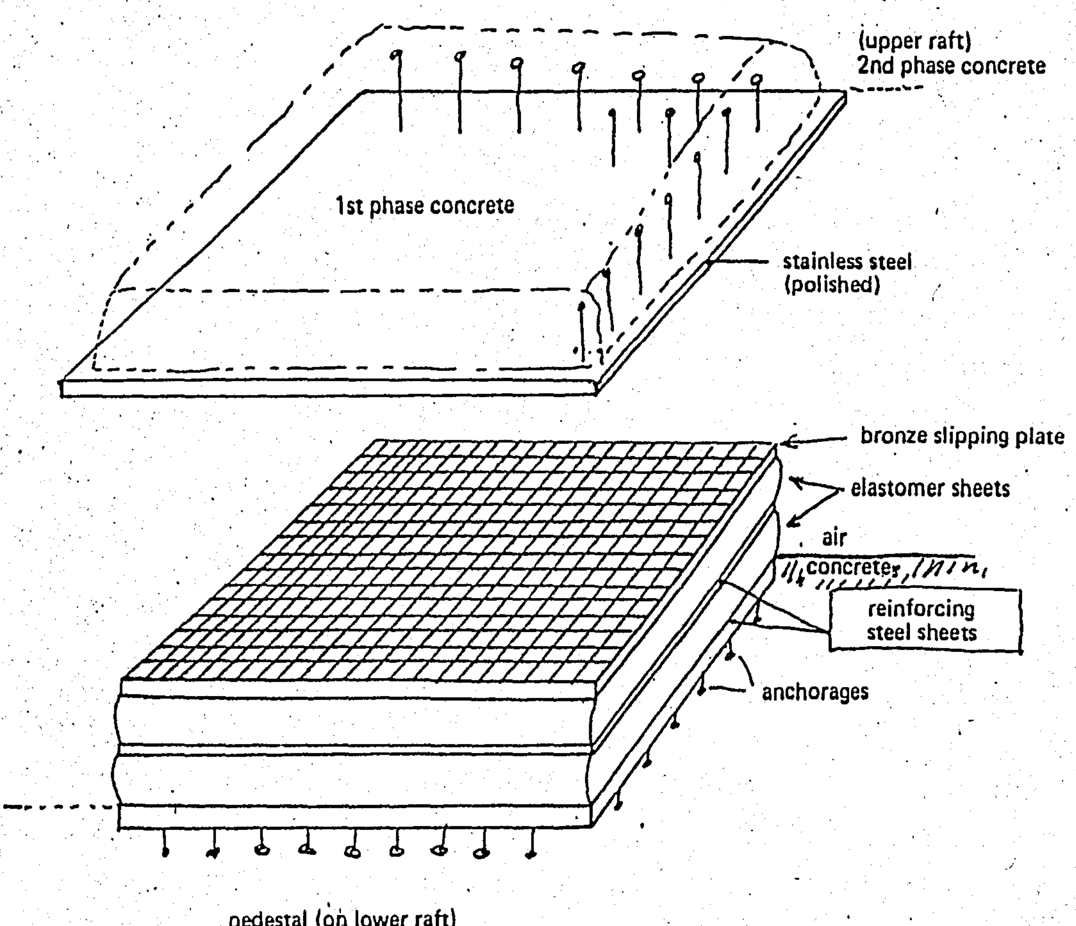


FIG 2.11 - ELASTOMER RUBBER PADS WITH FRICTION PLATES

with laminated steel and a pair of frivtion plates, one of stainless steel and the other of a leaded bronze alloy as shown in Fig. 2.11.

The elastomer pads is made up of artificial rubber (neoprene), chemical composition of which is a polychloroprene. The laminated steel plates provide confinement of the elastomer and thereby minimize the shearing stresses in the elastomer.

The purpose of the friction plate is to limit the horizontal forces that can be transmitted to the super structure. The stainless steel and leaded bronze combination is designed to provide a constant friction factor of approximately 0.20. The top friction plate is vulcanized to the upper part of the rubber pads and the lower plate is anchored in an intermediate concrete pedestal.

The bearings are characterized by having very low stiffness in the horizonzal direction so that interstorey deflections are greatly reduced and the structure vibrates predominantly in a rigid body made. In the vertical direction, however, the bearings are very stiff that they do not provide any isolation in thsi direction.

2. CONCEPT of SLIDING

For small earthquakes with peak horizontal acceleration not exceeding 0.20 g, the structure vibrates on the elastomer blocks and finally returns to its initial position. The elastomer bearings are deformed in shear and the system behaves just like a rubber isolated structure. As soon as the ground accelerations exceed the predetermined friction value of 0.20 sliding occurd between the two plates and the shear forces transmitted to the structure is limited. This phenomenum is illustrated in Fig. 2.12 as reproduced from Jolivet

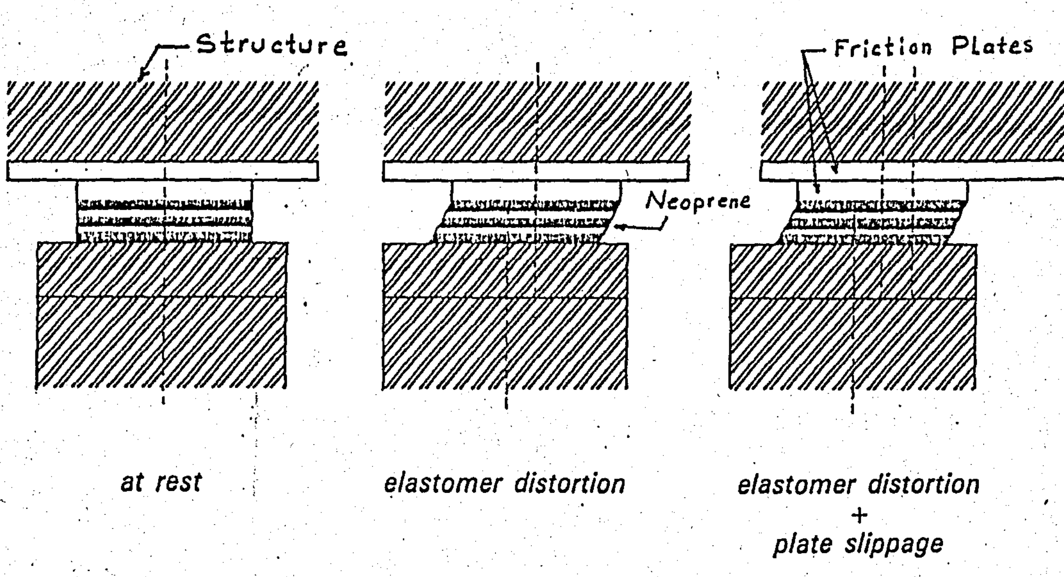


FIG. 2.12 - DYNAMIC BEHAVIOUR OF THE ELASTOMER PADS WITH FRICTION PLATES

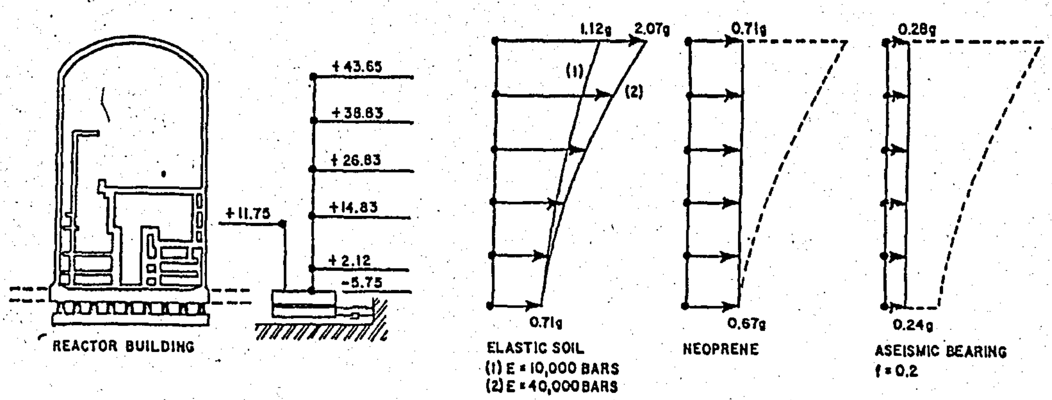


FIG. 2.13 - ACCELERATION RESPONSES FOR A REACTOR BUILDING

and Richli (1977). It is very likely that permanent off-set displacements occur in the event of sliding. Although no technical document has been published yet, it is claimed by Plichon and Jolivet (1978) that an elaborate operation scheme is developed to recenter the building back to its original centerline by moving each of the bearing pads one after the other.

Jolivet and Richli have also shown that the effects of sliding on the response of a atructure can be quite significant. For example, the acceleration response at the top of a fixed base model reactor building is reduced from 2.07 g to 0.71 g when isolation pads without friction plates are installed. Further, the response is reduced to 0.28 g when friction plates are added as shown in Fig. 2.13. The input peak acceleration is taken to be 0.60 g and the friction coefficient of the pads is 0.20.

3. LABORATORY TESTS

A series of shaking table tests have also been carried out by the *Commissariat a L'Energie Atomique*, France on the Vesuvius vibrating table which is capable of reproducing recorded earthquake accelerations. The test model was a concrete structure with 14.5 ton weight representing $1/10^{th}$ scaled reactor building supported by aseismic bearings. Tests repeated for more than 70 simulated earthquakes indicated that the bearings behaved satisfactorily and the isolation system is reliable as reported by Jolivet and Richli (1977).

4. APPLICATION in NUCLEAR POWER PLANTS

This isolation system with laminated steel plates and is specifically designed for important structures like nuclear power plant structures to allow the use of standard power plant design under any site conditions including areas susceptible to very strong earthquakes. In fact, the system has been utilized for the nuclear power plant in Koeber, South Africa, with a safe shutdown earthquake (SSE) of 0.3 g. More than 1800 aseismic bearings are installed under the foundation of the 2 x 900 MW power plant. A similar system is also utilized in 2 x 900 MW Karun nuclear power plant in Iran, the construction of which is currently, (1985) suspended.

2.8. ISOLATION WITH BALL BEARINGS

Rocking ball bearings placed under the structure also provides seismic isolation by decoupling the superstructure and the foundation black. Bednanski (1935) in his discussion to *Green's* paper (1935) recommended the use of viscous dashpots in conjunction with rollers to isolate buildings during an earthquake (Fig. 2.14). This was one of the earliest examples of vibration isolation systems which do not necessitate any special detailing in the structure. Also, in contrast to weak columns in the soft storey concept, the special isolation devices can be replaced easily after a damaging earthquake. Other poller systems in which the structure is supported by rocking ball bearings have been introduced by Katsuta and Mashiju (1965) and Matsushita and Izumi (1969).

In this isolation scheme, the base system comprises a bed of ball bearings to decouple the upper structure from ground and dampers to limit the lateral displacements. Fiction provides resistance to small excitations like small earthquake and wind loads.

Caspe (1970) utilized ball bearings to isolate the

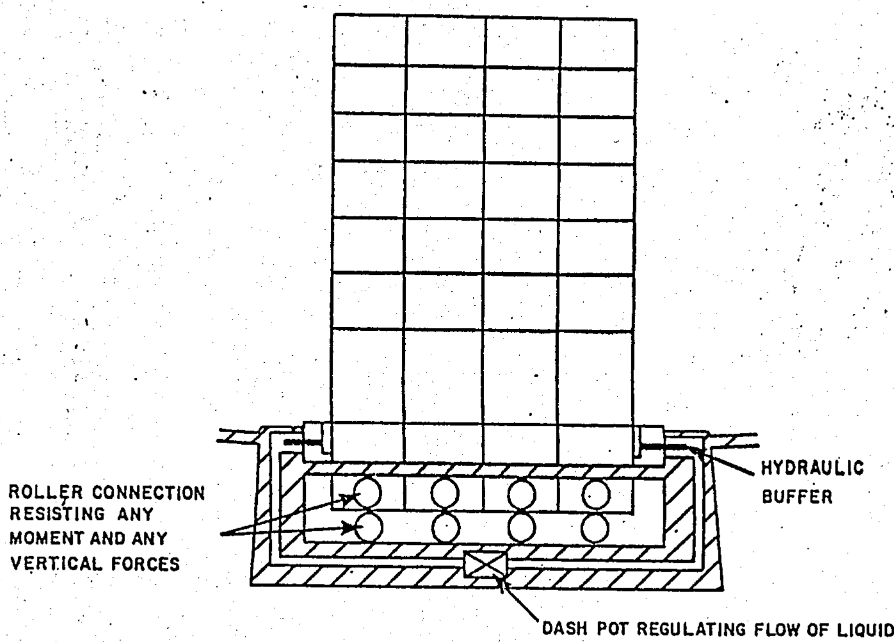


FIG. 2.14 - BALL BEARINGS AND VISCOUS DASHPOTS TO ISOLATE STRUCTURES

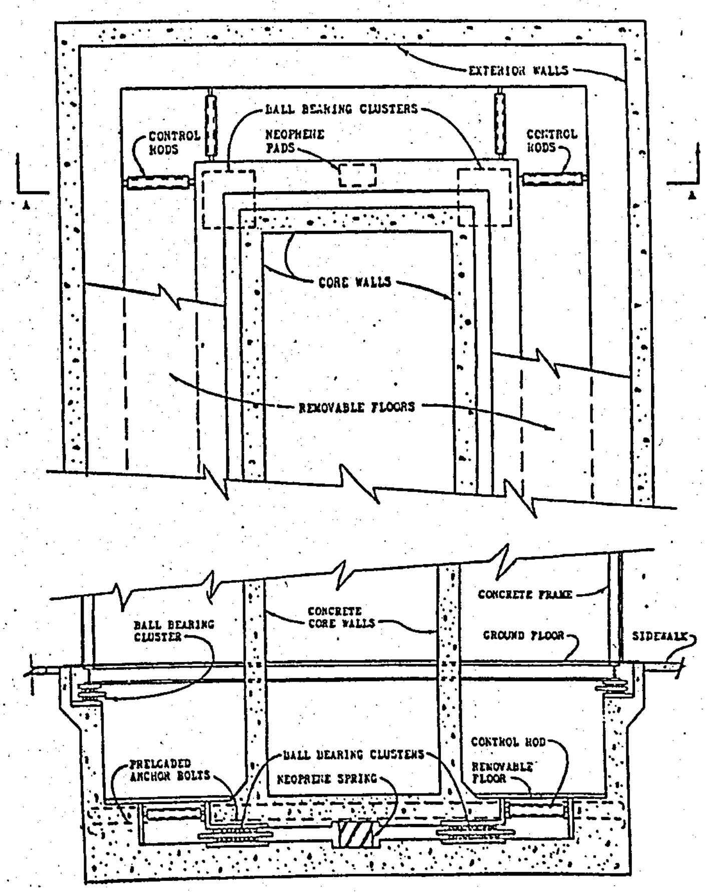


FIG. 2.15 - THE SYSTEM PROPOSED BY CASPE

structure and a neoprene element to produce a restoring force and yielding control rods to provide wind restraint and high hysteretic damping (Fig. 2.15). The control nods and the neoprene elements function as effective vibration barriers limiting the accelerations transmitted to the upper storeys at a certain level and they complement each other in smoothing out the sharp vibratory acceleration peaks.

2.9. ISOLATION WITH FLOATING PLATFORMS

Similar to the ball bearing concept, placing the structure on a floating platform is also a very efficient decoupling isolation scheme in which no destructive shear wave components of the ground motion is transmitted to the structure as illustrated in Fig. 2.16. Busey (1969) propsoed several alternative systems for floating nuclear power plants.

One concept involves floating only the nuclear steam generators and associated fuel handling equipment and safety systems. All other structures are built on the surrounding breakwater platforms. This would certainly reduce the size of the platform and the breakwater. However, due to the required flexibility of high pressure steam piping of the water cannot be maintained at a constant level, Busey considers this system impractical.

Floating the entire nuclear complex on a common platform as illustrated in Fig. 2.17 is recommended to be practical by Busey. The platform should be moored to a breakwater which encloses the floating nuclear island as shown in Fig. 2.18. For a 1000 MW nuclear power station 840 meters offshore from Huntington Beach, Califronia, a concrete platform with plan

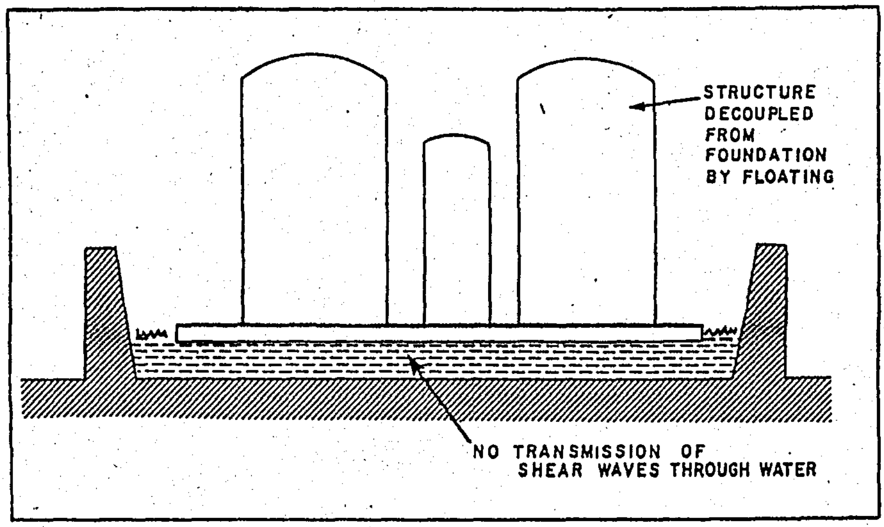


FIG. 2.16 - ILLUSTRATIVE EXAMPLE OF DECOUPLING ISOLATION CONCEPT

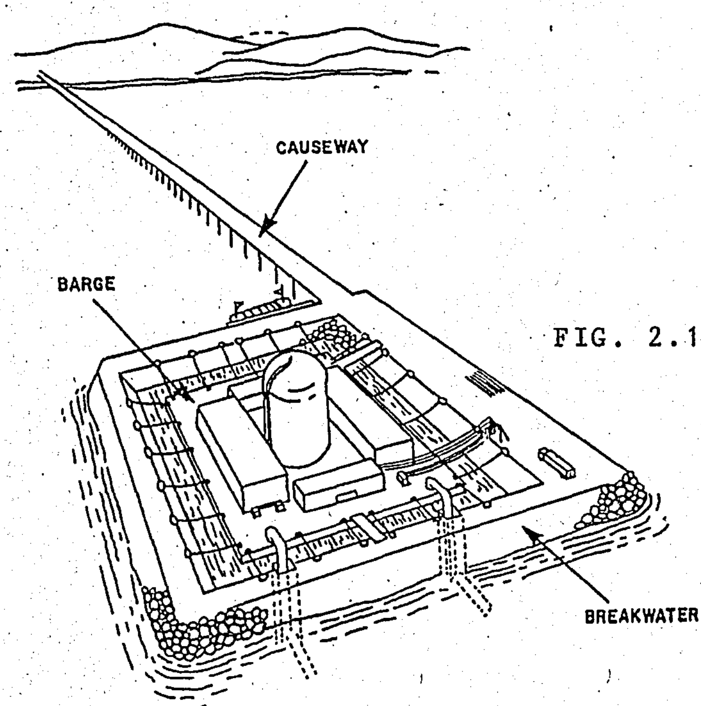


FIG. 2.17 - FLOATING NUCLEAR POWER PLANT (After Busey, 1969)

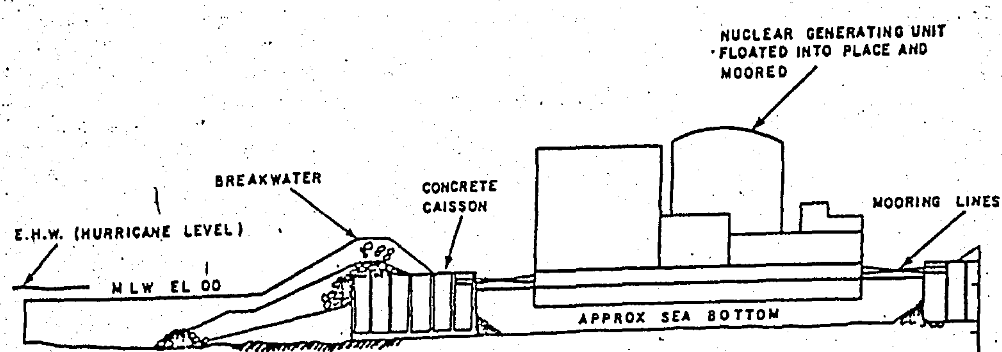


FIG. 2.18 - BASIC COMPONENTS OF A FLOATING OFFSHORE PLANT

For a floating platform, complete seismic isolation is achieved in the horizontal direction. However, for shallow waters vertical accelerations can induce substantial rocking. The insertion of air springs is recommended by Busey (1969) to provide additional isolation in the vertical direction. The breakwater must be constructed to withstand the effects of environmental hazards like storms, tsunamis, earthquakes etc. Transmission of forces to the nuclear island through the mooring lines must be accounted for.

Kehnemuyi (1974) reported an alternative floating nuclear plant of 1150 MW capacity sitting on a floating platform with dimensions of 120 m x 110 m x 13 m is to be marketed by Offshore Power Systems. The plant is shown in Fig. 2.19.

2.10. HELICAL SPRINGS and VISCODAMPERS

A new vibration isolation scheme utilizing helical steel springs and viscous damper combination is proposed by Tezcan and Çivi (1979 and 1981), Huffmann (1980) and Tezcan et al (1980). In this system, vibration isolation is provided for all possible horizontal, vertical, rocking and torsional modes.

The philosophy of this new concept as well as a complete discussion of analytical investigations and laboratory testings are presented in this study.

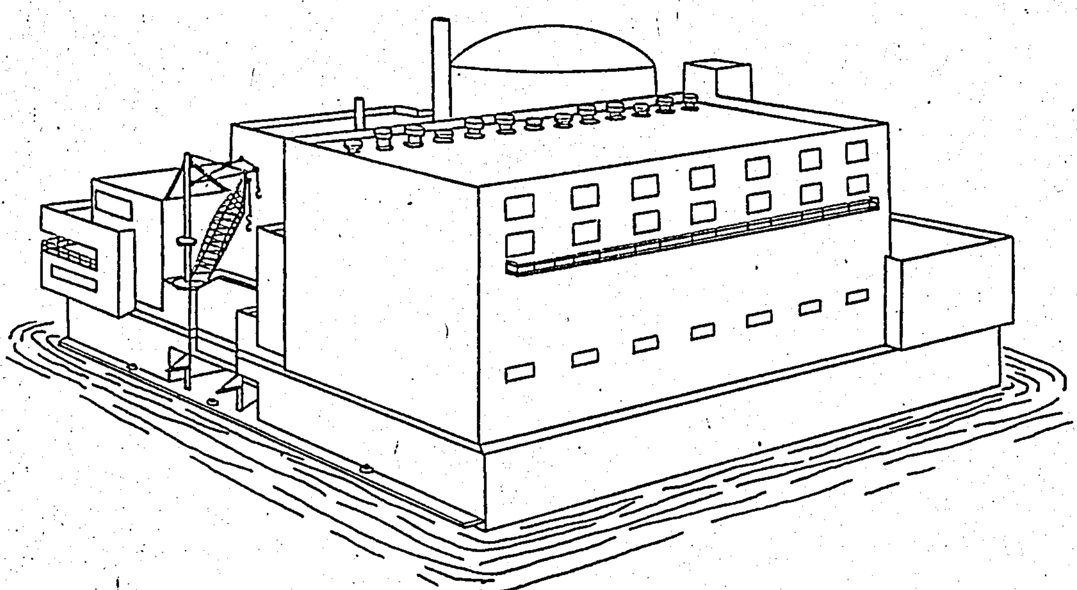


FIG. 2.19 - FLOATING NUCLEAR POWER PLANT
(After Kehnemuyi, 1975)

CHAPTER 3

METHOD OF ANALYSIS

3.1. MATHEMATICAL MODELLING

For conventional structures without base isolation, realistic dynamic response analysis depends mainly on the mathematical modelling of the system. As the model becomes more sophisticated the results become more accurate, but longer and more expensive calculations are necessary.

Mathematical modelling is not important however, when vibration isolation is used. Because, the basic idea behind vibration isolation is to reduce the governing motion of the system into that of rigid body. Therefore, once a mathematical model capable of representing the rigid body motions in the horizontal and vertical directions is designed, the details in the superstructure is no longer important for dynamic analysis purposes. Moreover, since the amplitudes of the response to earthquake loads are much reduced very little happens in the superstructure. This brings simplicity to the seismic analysis of highly complex structural systems like nucelar power plants.

A typical nuclear reactor building and a possible

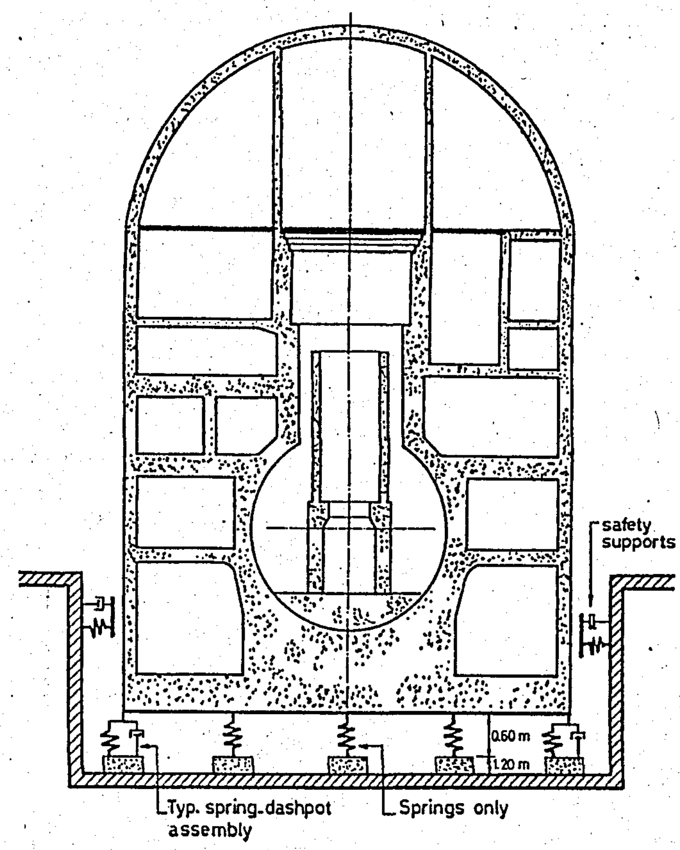


FIG. 3.1 - A TYPICAL REACTOR BUILDING

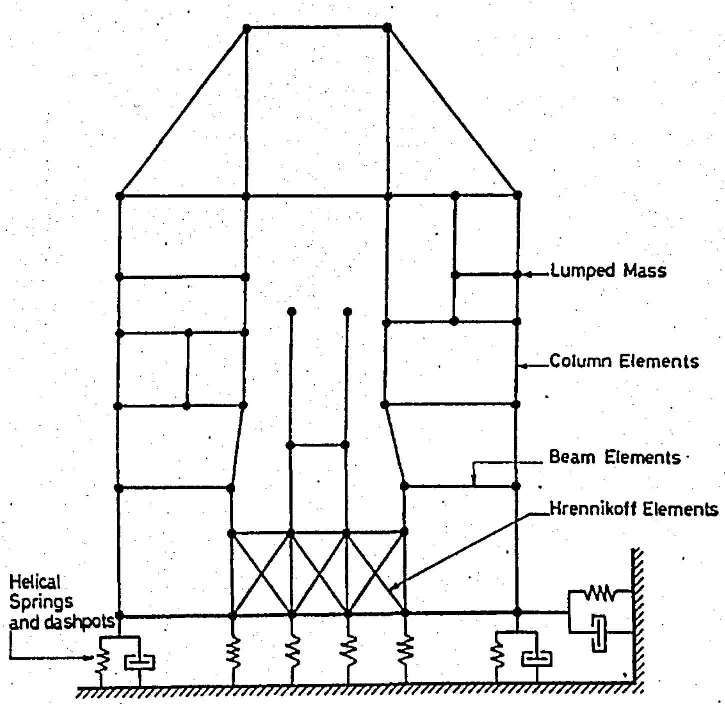


FIG. 3.2 - MATHEMATICAL MODELLING

mathematical modelling are shown in Figs. 3.1 and 3.2, respectively. In most cases however, much simpler model with one or a few lumped masses is sufficient to produce satisfactory results. It is therefore, obvious that the cost of dynamic analysis is minimum when the vibration isolation is used. For the purpose of determining the time history response of the structure a step-by-step integration procedure is used. Simple mathematical models are more convenient, since much smaller number of time steps would be sufficient for the same degree of accuracy.

Hence, in order to illustrate the principle of base isolation, an extremely simple system, like the tyo mass and two spring model are shown in Fig. 3.3, is selected. The upper mass represents the whole superstructure and the upper spring represents the base vibration isolation system. If, the foundation soil is soft, an assembly of finite elements can be used for a sophisticated mathematical modelling. However, soft soil condition may also be satisfactorily idealized by means of an appropriate spring-mass assembly. The spring represents the stiffness of the soil while the mass represent the lower foundation raft as well as some portion of the soil moving "in-phase" with the foundation. In the case of hard soil however, the lower part of the simplified model is not necessary. Typical mathematical models for hard soil and soft soil conditions are shown in Fig. 3.4.

Analyses performed on various structures using simple as well as sophisticated mathematical models, have shown that natural frequencies of vibration and the mode shapes were in close agreement in lower frequency modes. Only, the higher natural frequencies and mode shapes could not be obtained when siöplified models were used.

In addition to the mass amd stiffness parameters, the

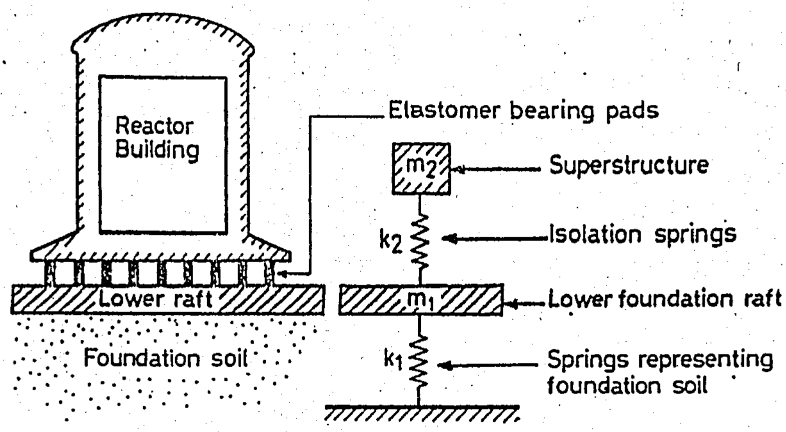


FIG. 3.3 - TWO-MASS ASSEMBLY OF A REACTOR BUILDING

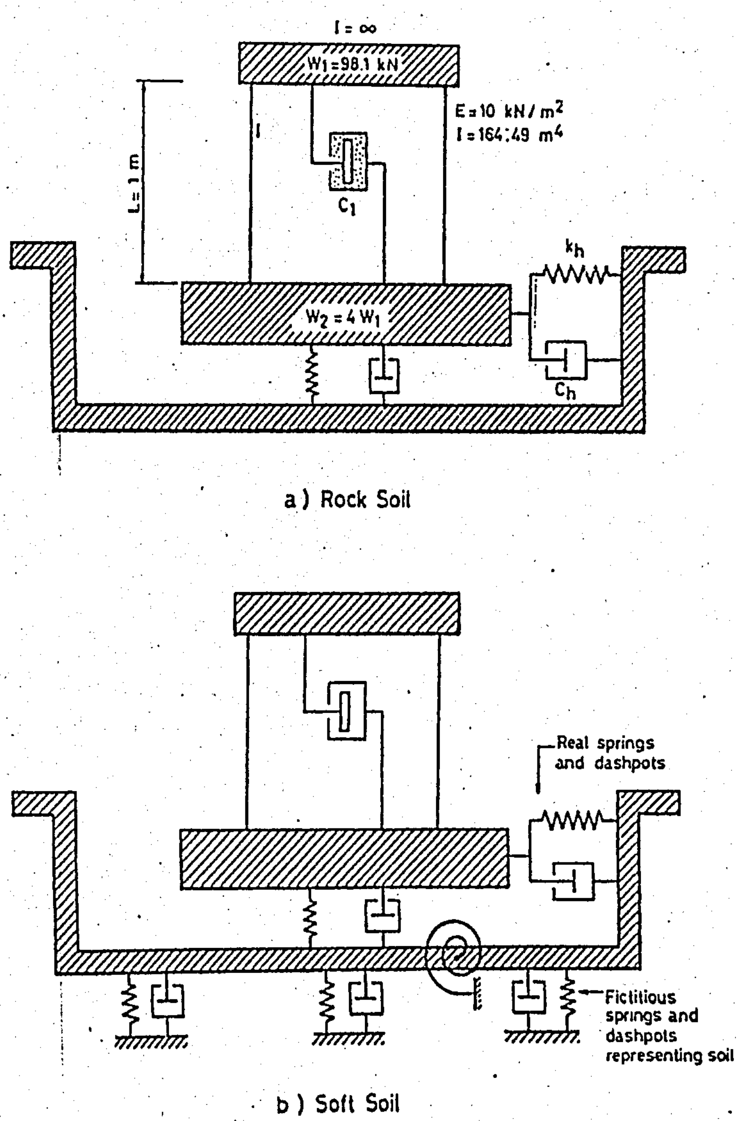


FIG. 3.4 - VIBRATION ISOLATION MODELS

amount of damping is also an important parameter in mathematical modelling. The degree of damping present in both the isolation elements and the superstructure, is usually determined by means of laboratory or full scale testing of the relevant structural systems or their components. Generally, viscous damping is used in mathematical modelling since it suits well with the available computation techniques.

Finally, the yielding behaviour of the vibration isolation elements should also be considered in mathematical modelling if a non-linear analysis is to be performed.

3.2. MATHEMATICAL FORMULATION

The stiffness equation of static equilibrium of a structural system is characterized by

$$[K]\{x\} = \{P\} \qquad \ldots \quad (3.1)$$

in which,

$[K]$ = the system stiffness matrix of the system of size n by n,

$\{X\}$ = the column vector of unknown displacements and rotations of size n, and

$\{P\}$ = the column vector of external loads acting along the same directions of displacements and rotations, also of size n.

n = the number of unknowns in the structure.

The differential equation of motion of a multidegree of freedom system is

$$[M]\{\ddot{X}\} + [C]\{\dot{X}\} + [K^*]\{X\} = \{F(t)\} \qquad \ldots \quad (3.2)$$

in which,

$[M]$ = the diagonal lumped mass matrix of the structure after the rotational degrees of the freedom are eliminated,

$[C]$ = the viscosity matrix of the system corresponding to the same degrees of freedom of the mass matrix,

$[K^*]$ = the reduced stiffness matrix corresponding to the primary degrees of freedom.

The reduced stiffness matrix is obtained from the original system stiffness matrix $[K]$ by means of Gaussian elimination procedure,

$\{F(t)\}$ = the forcing function ordinates, variable in time and acting along each of the primary degrees of freedom;

$\{X\}$ = the time history ordinates of displacements,

$\{\dot{X}\}$ = velocities, and

$\{\ddot{X}\}$ = accelerations in primary vibrating directions.

A detailed discussion of the reduced stiffness matrix is given in the next section.

If the structure is subject to the ground accelerations, $a_h(t)$ and $a_v(t)$, in the horizontal and vertical directions, respectively, the forcing functions should be replaced by the following expression:

$$\{F(t)\} = -\begin{Bmatrix} m_1 \\ o \\ m_2 \\ o \\ \cdot \\ \cdot \\ \cdot \\ \cdot \end{Bmatrix} a_h(t) - \begin{Bmatrix} o \\ m_1 \\ o \\ m_2 \\ \cdot \\ \cdot \\ \cdot \\ \cdot \end{Bmatrix} a_v(t) \qquad \ldots \quad (3.3)$$

in which,

m_i = the i^{th} lumped mass,

$a_n(t)$ = the time history of ground shaking in the horizontal direction,

$a_v(t)$ = the time history of ground shaking in the vertical direction

3.3. STEPS OF CALCULATION

The analysis is essentially executed at three major steps as follows:

Step 1: Generation of the main stiffness matrix, [K]

Step 2: Determination of the reduced stiffness matrix, [K*]

Step 3: Step-by-Step direct integration procedure

A) STEP 1: GENERATION of the MAIN STIFFNESS MATRIX, [K]

For dynamic analysis, the structure is first idealized into an assembly of base elements assuming the distributed masses being lumped at the joints. Then, degrees of freedom at every joint are claffied as vibrating and non-vibrating directions. It is possible that the mass at a particular joint vibrates in only one or two translational degrees of freedom, while there may be several other degrees of freedom in which no vibration takes place. Rotational degrees of freedom, for instance, are regarded as non-vibrating directions, since the rotary moments of inertia of the masses are generally neglected. Vibrating and non-vibrating directions may be also called as "primary" and "secondary" directions, respectively.

At first, the stiffness matrices of the individual members comprising the structure are evaluated and stored in the memory. Then, the primary and secondary vibrating directions are numbered at each joint. In order to apply the Guassian elimination procedure for the reduction process conveniently, the secondary degrees of freedom are numbered frist at all joints. Then, the primary directions are numbered sequentially succeeding the last number of the secondary degrees of freedom. This numbering process is automatically achieved by means of a special subroutine which reads the information about the supported directions as input data.

Generation of the complete stiffness matrix of the system is performed by means of the code number techniques as described by Tezcan (1963). Due to symmetry, it is sufficient to generate only the upper triangle of the main stiffness matrix. Half-band width of the stiffness matrix, corresponding to the secondary degrees of freedom, is also taken into account in order to economize the available memory space. The zero triangle on the right-hand side of the band is neither stored nor treated in the operations. Detailed description of the code number technique as well as the generation of the main stiffness matrix is given in Appendix A.

B) STEP 2: DETERMINATION of the REDUCED STIFFNESS MATRIX, [K*]

In the beginning, the main stiffness matrix contains both the primary and the secondary vibrating directions. Taking advantage of the zero force values corresponding to to the secondary directions, the Gaussian elimination procedure is applied to the condensation of the main stiffness matrix into a smaller size containing the primary vibrating directions, only. This condensed matrix is generally called the "reduced stiffness matrix".

For the purpose of separating the primary and secondary degrees of freedom, the stiffness equation containing a symmetrical square matrix [K] may be subdivided as

$$\begin{array}{c} n_1 \\ \\ n_2 \end{array}\left[\begin{array}{c|c} [A] & [B] \\ \hline [B]^T & [C] \end{array}\right]_{(nxn)} \left\{\begin{array}{c} \{D_s\} \\ \cdots\cdots \\ \{D_p\} \end{array}\right\}_{(nx1)} = \left\{\begin{array}{c} \{P_s\} \\ \cdots\cdots \\ \{P_p\} \end{array}\right\}_{(nx1)} \begin{array}{c} n_1 \\ \\ n_2 \end{array} \qquad \ldots \quad (3.4)$$

in which,

n = total number of degrees of freedom,
n_1 = number of secondary degrees of freedom,
n_2 = number of primary degrees of freedom,
$\{D_s\}$ = displacement vector for the second directions,
$\{D_p\}$ = displacement vector for the primary directions,
$\{P_s\}$ = external forces in the secondary directions,
$\{P_p\}$ = external forces in the primary directions.

Since, there are no masses in the secondary directions the inertia forces do not exist in those directions. Hence, all the elements of the $\{P_s\}$ are zero.

Substituting $\{P_s\} = \{0\}$ in the above matrix equation, the secondary degrees of freedom $\{D_s\}$ may be eliminated and a relation between the primary directions $\{D_p\}$ and the external forces $\{P_p\}$ is obtained as follows:

$$[K^*]\{D_p\} = \{P_p\} \qquad \text{size } (n_2 \times n_2) \qquad \ldots \quad (3.5)$$

in which,

$[K^*]$ = the reduced stiffness matrix,

given by,

$$[K^*] = [C] - [B]^T [A]^{-1}[B] \qquad \ldots \quad (3.6)$$

In vibration problems, where there are secondary degrees of freedom, in which no mass is vibrating, the reduction of the main stiffness matrix is most essential. The size of the reduced stiffness matrix is equal to the number of vibrating masses. When there are no secondary degrees of freedom the reduction is obviously not necessary.

For large size matrices, the reduced stiffness matrix is not recommended to be obtained using Eq. (3.16), since the inversion and the triple matrix product operations require excessive computing time. On the other hand, however, the Gaussian elimination procedure is simple and convenient.

The algorithm for the Gaussian elimination of a symmetrical matrix for the first n_1, rows is given as follows:

$$a_{kj} = a_{kj}/a_{kk}\,, \qquad j = k,\, k+1, \ldots, n \qquad \ldots \quad (3.7)$$

$$a_{ij} = a_{ij} - a_{ik}a_{kj}\,, \quad \left[\begin{array}{l} k = 1,2,\ldots,\, n_1 \\ i = k+1,\, k+2, \ldots, n \\ j = i,\, i+1,\, n \end{array}\right. \qquad \ldots \quad (3.8)$$

The elimination is terminated at the last row of the secondary degrees of freedom. The remaining matrix of size $n_2 \times n_2$ at the lower right side is the desired reduced stiffness matrix.

C) STEP 3: STEP-by-STEP
DIRECT INTEGRATION PROCEDURE

Once, the numerical contents of the [M], [C], and [K] matrices are available, the step-by-step numerical integration procedure is followed for the purpose of determining the response of the structure. Using a linear acceleration method of numerical integration as applied to the direct integration of the differential equations of motion in matrix notation, the time history response of the selected lumped masses of the system are determined. As outlined in Wilson and Clough (1962), the following steps are followed:

$$[E] = [M] + \tfrac{1}{2}\Delta t[C] + \tfrac{1}{6}\Delta t^2[K] \qquad \ldots \quad (3.9)$$

$$\{\ddot{X}\}_1 = [M]^{-1}\left[\{F\}_1 - [C]\{\dot{X}\}_1 - [K]\{X\}_1\right] \qquad \ldots \quad (3.10)$$

in which,

E = an axuiliary matrix calculated only once, and

$\{\ddot{X}\}_1$ = the acceleration vector at the first time step.

The response vectors at the remaining time steps are calculated using the following equations:

$$\{a\}_{i-1} = \{\dot{X}\}_{i-1} + \tfrac{1}{2}\Delta t\,\{\ddot{X}\}_{i-1} \qquad \ldots \quad (3.11)$$

$$\{b\}_{i-1} = \{X\}_{i-1} + \Delta t\{\dot{X}\}_{i-1} + \tfrac{1}{3}\,\Delta t^2\{X\}_{i-1} \qquad \ldots \quad (3.12)$$

$$\{\ddot{X}\}_i = (E)^{-1}\left[\{F\}_i - (C)\ \{a\}_{i-1} - (K)\ \{b\}_{i-1}\right] \qquad \ldots \quad (3.13)$$

$$\{\dot{X}\}_i = \{a\}_{i-1} + \tfrac{1}{2}\Delta t\{X\}_i \qquad \ldots \quad (3.14)$$

$$\{X\}_i = \{b\}_{i-1} + \tfrac{1}{6}\Delta t^2\{\ddot{X}\}_i \qquad \ldots \quad (3.15)$$

in which, Eqs. 3.11 to 3.15 are repeated sequentially in cyclic order at every time station. In order to prevent divergence in numerical integration, the time interval Δt between any two consecutive calculation points should be selected to be less than $\frac{1}{5}$ to $\frac{1}{8}$ of the smallest natural period of vibration of the structure.

3.4. COMPUTER PROGRAMMING FEATURES

The computer program DIR201 developed for this purpose is capable of considering lineal and/or rotational elastic springs directly attached to the vibrating lumped masses. There may be real physical dampers in the form of lineal dashpots attached also to the vibrating masses. Equal deformations of joints can be also taken into account. Member bending, axial and shear deformability, semi-rigid connections and finite size point dimensions may be accounted for. Large differences in member stiffness, straight members with constant or variable sections are acceptable.

Automatic node labelling for regular and rectangular frames is possible. The exciting force may be either one or more externally acting time dependent forcing functions or a ground motion occurring in horizontal and/or vertical direction.

The program DIR 201 expects the joint coordinates. member properties, boundary conditions, elastic supports, dampers and the input motion ordinates as the input data. All input data is echoed as output for verification purposes. At the end of analysis, the program produces the absolute acceleration, the relative velocity and displacement of all joints for which the response values are required. Finally, the maximum response values for each joint are also printed out.

3.5. NUMERICAL EXAMPLES

For the purpose of illustrating the use of vibration isolators, several example structures have been analyzed and the results of the investigations are herewith summarized. All mathematical models are planer structures and subjected to the N-S component of the 1940 El Centro earthquake motion as taken from Anonymous (1972). The peak ground acceleration is 0.35 g (3.46 m/sec²) in the horizontal and 0.23 g (2.31 m/sec²) in the vertical directions. The acceleration record and its corresponding response spectra are given in Figs. 3.5 and 3.6 respectively.

Two degrees of freedom for each mass, one in the horizontal and the other in the vertical direction are considered. Rotational degrees for each mass are defined for stiffness calculation purposes and later eliminated at the stage of the reduced stiffness matrix calculations.

A) MODEL 1

An extremely simple mathematical model of a structure with vibration isolators is shown in Fig. 3.7. It is a model of a building on soft soil where the soil-structure interaction is considered by means of a set of lineal and rotational springs and dashpots. These springs and dashpots are fictitions and their characteristics are selected to represent the prevailing soil conditions. It is also important to note that the model is actually a one-mass model if the structure is assumed to be fixed at the foundation level.

In order to illustrate the effect of vibration isolators on the acceleration response, the structure is subjected to the 1940 El Centro ground motion and the history response of the roof is calculated for the fixed base case as well as for varying values of horizontal springs and dam-

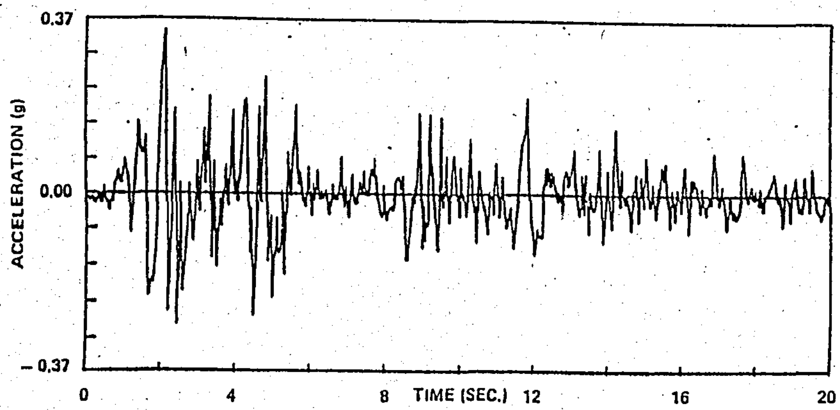


FIG. 3.5 - ACCELERATION RECORD OF THE 1940 EL-CENTRO EL-CENTRO EARTHQUAKE, N-S COMPONENT

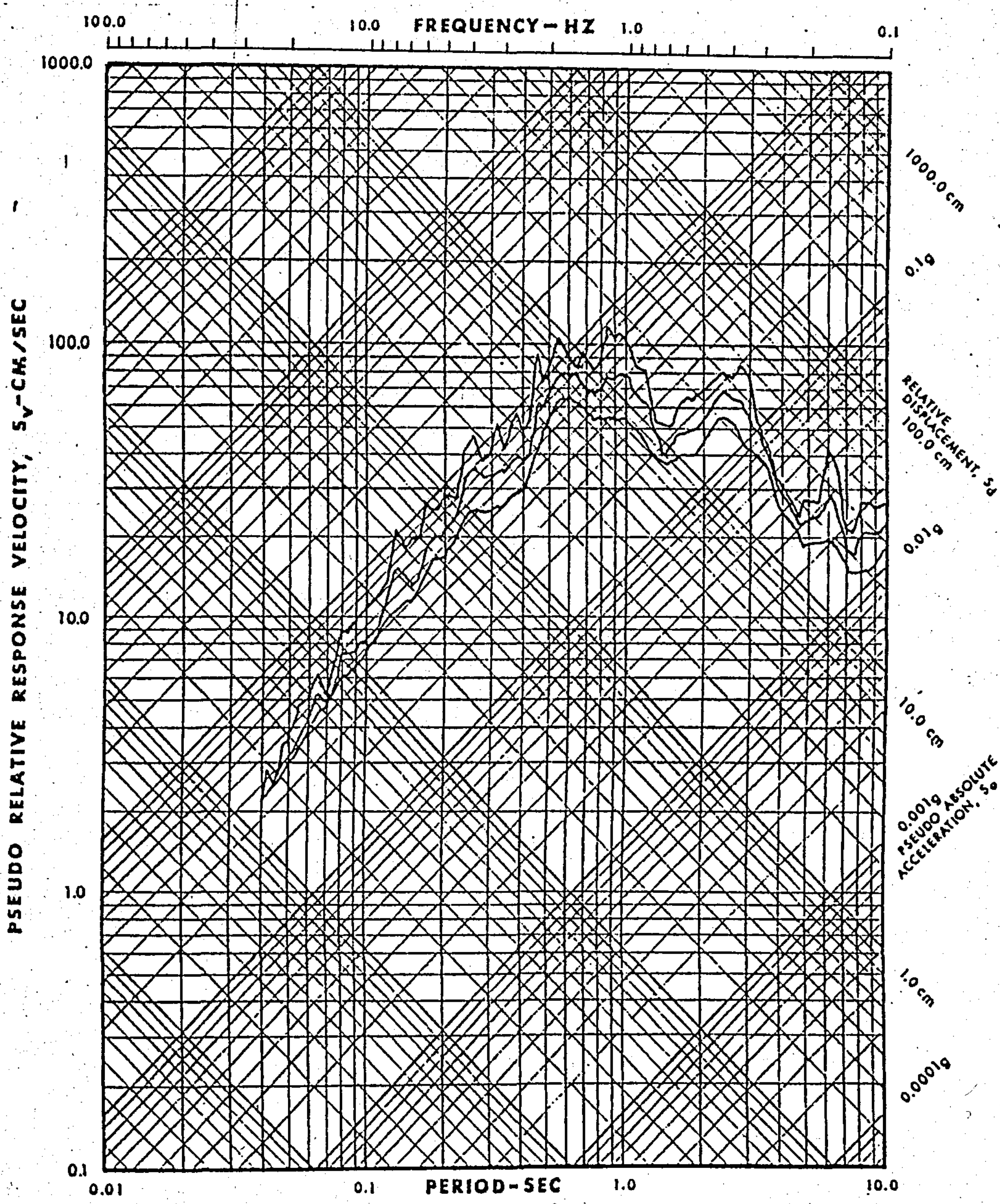


FIG. 3.6 - RESPONSE SPECTRA OF THE 1940 EL-CENTRO EQ., N-S COMPONENT (β = 0.02, 0.05, and 0.10)

pers. The magnification factor MF, defined as the ratio of the maximum acceleration of the roof to the peak ground acceleration, is plotted in Fig. 3.8 against the natural periods of vibration. It is seen that the response is reduced significantly and the success of isolation is increased with decrease in spring constants and increase in damping.

B) MODEL 2

A mathematical model for a typical BWR Reactor Building is shown in Fig. 3.9. Similar numerical data values as those given in p. 44 of John A. Blume (1967) Report TID-25021 are used except the number of masses is reduced from to three. The acceleration and displacement response values of the top mass are illustrated in Figs. 3.10 to 3.13. The fundamental natural period of vibration is T= 0.20 sec for the fixed base case and it is increased to T= 1.07 sec after the vibration isolators are attached. It is seen that, for the damping case of $C_h = 10^8$N sec./m, the horizontal accelerations are reduced from 24.00 m/sec² for the fixed base case, to 2.55 m/sec² for the isolated case. The reduction is more than nine times. The structure behaves like a rigid body and the relative displacements between storeys are almost nil.

C) MODEL 3

The model shown in Fig. 3.14 represents a typical BWR Reactor-Building complex and the data is the same as in Liu et al (1973). The horizontal accelerations and displacements of Mass No. 1 are shown in Figs. 3.15 & 3.16. The fundamental period of vibration at fixed base case is T= 0.18 sec and it is increased to T= 1.00 sec by means of horizontal springs. The acceleration response is reduced by about ten times, but the displacements are increased from 1.35 cm to 4.32 cm for the damping case of $c_h = 25 \times 10^7$N sec./m.

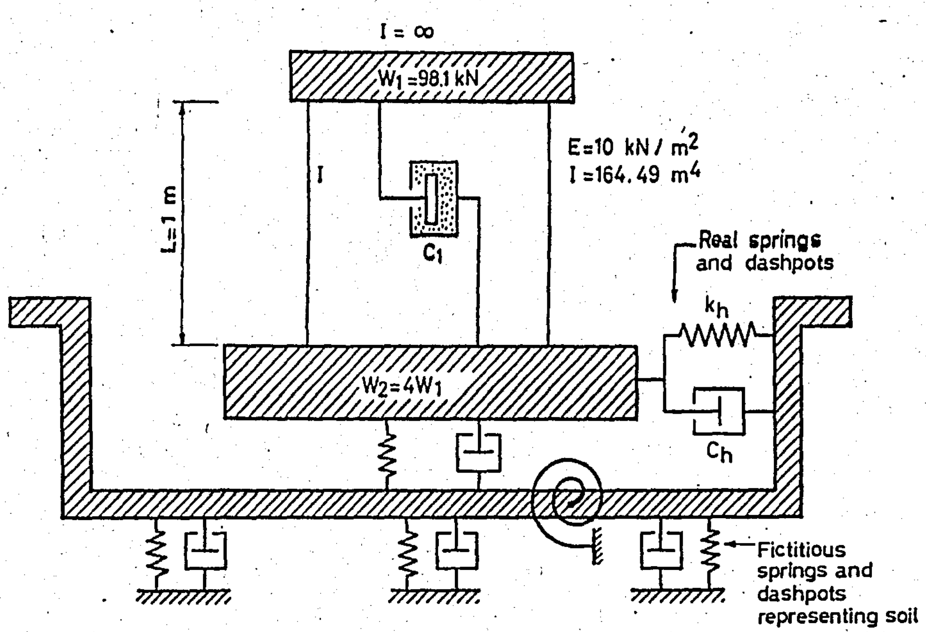


FIG. 3.7 - VIBRATION ISOLATION , MODEL 1

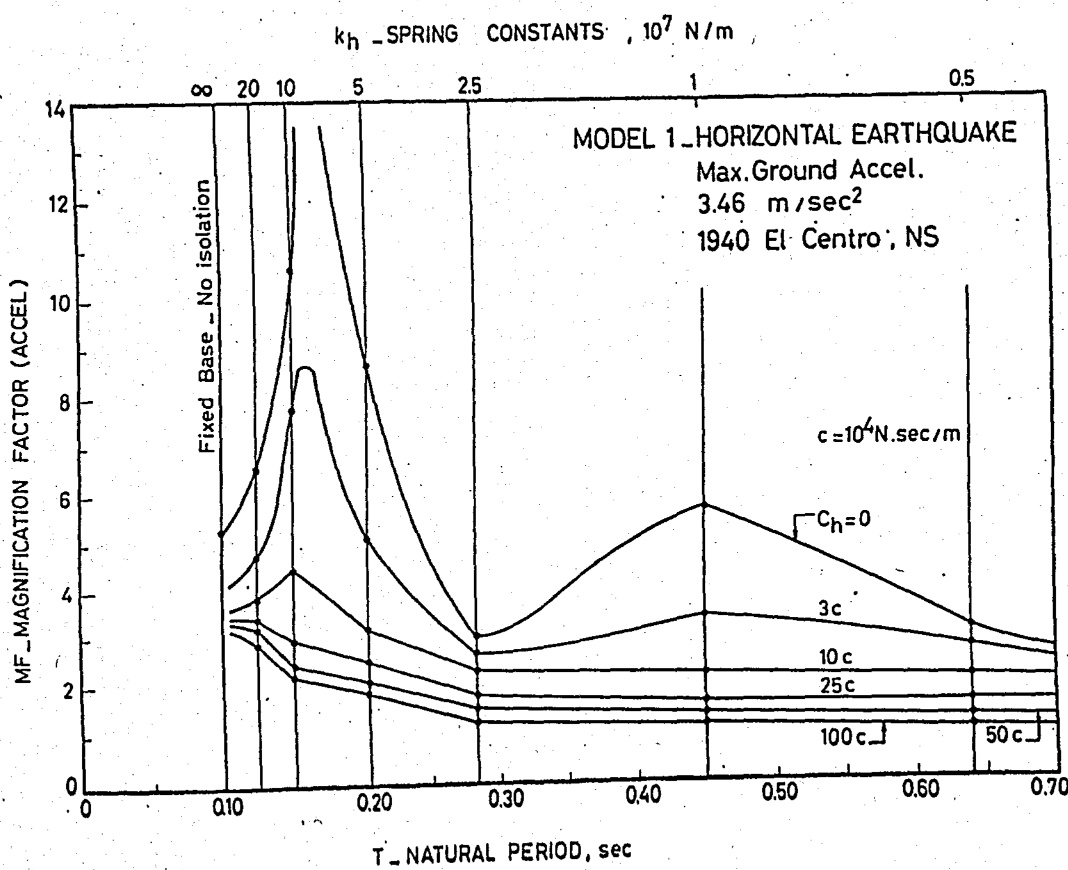
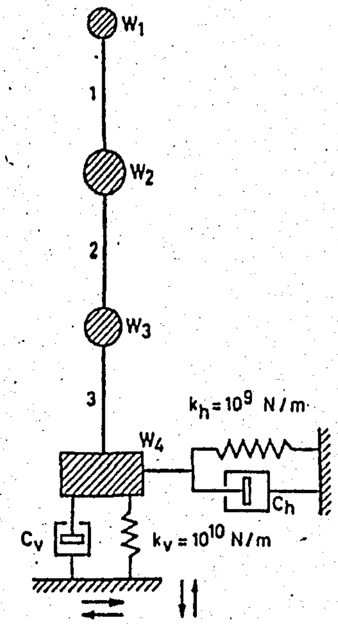


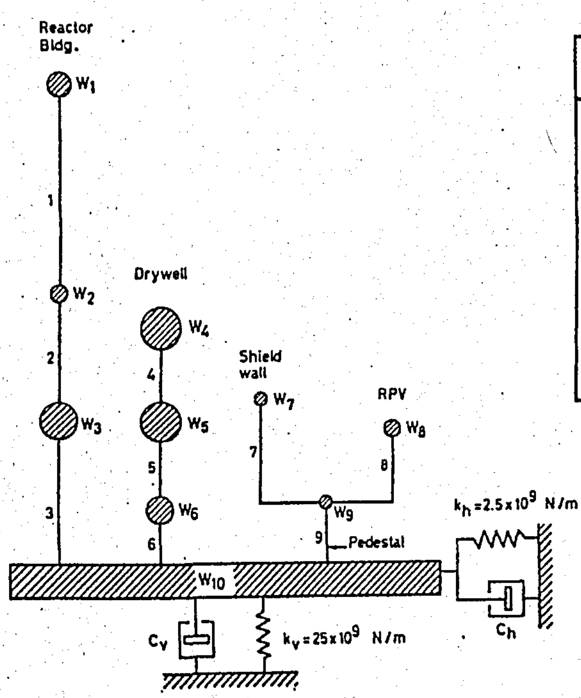
FIG. 3.8 - EARTHOUAKE RESPONSE , MODEL 1



No	L	A	I	W
	m	m^2	10^3 m^4	10^6 N
1	20	70	10	40
2	20	90	16	60
3	20	130	24	60
4	-	-	-	120

$E = 2 \times 10^{10}$ N/m^2

FIG. 3.9 - VIBRATION ISOLATION, MODEL 2



NO	ELEV.	A	I	W
	m	m^2	m^4	10^6 N
1	57.30	56.4	22 460	35.85
2	29.70	56.4	22 460	5.07
3	11.85	58.5	22 680	54.87
4	25.05	117.0	14 580	64.42
5	11.85	63.0	8 100	71.06
6	-1.35	51.0	6 885	49.15
7	15.12	0.4	7.36	2.34
8	10.50	900.0	67.2	14.65
9	-0.39	31.5	12 312	10.49
10	-9.48	-	-	266.53

$E = 2.29 \times 10^{10}$ N/m^2

FIG. 3.14 - VIBRATION ISOLATION , MODEL 3

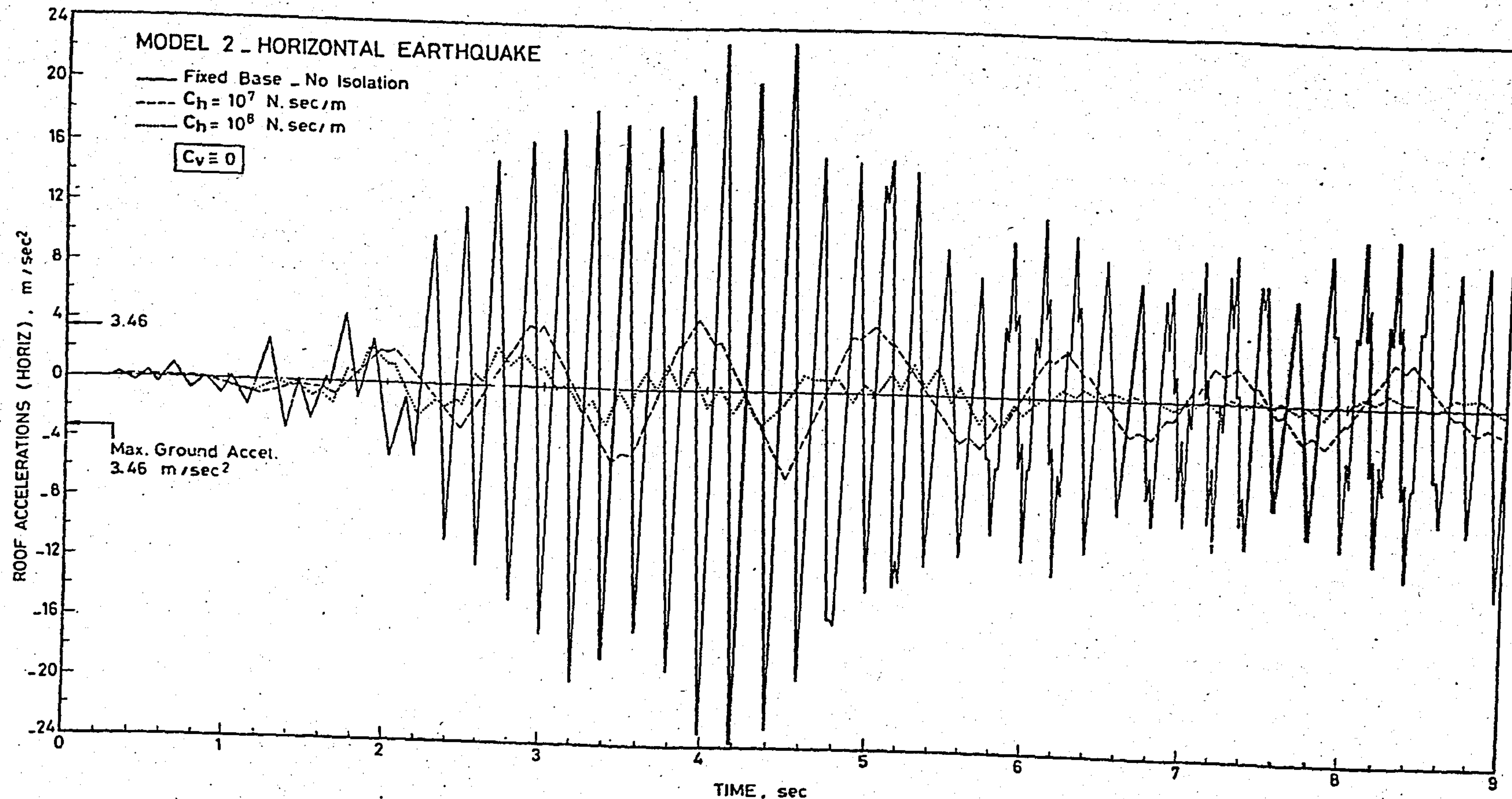


FIG. 3.10 - ACCELERATION RESPONSE OF MASS NO. 1 DUE TO HORIZONTAL EARTHQUAKE

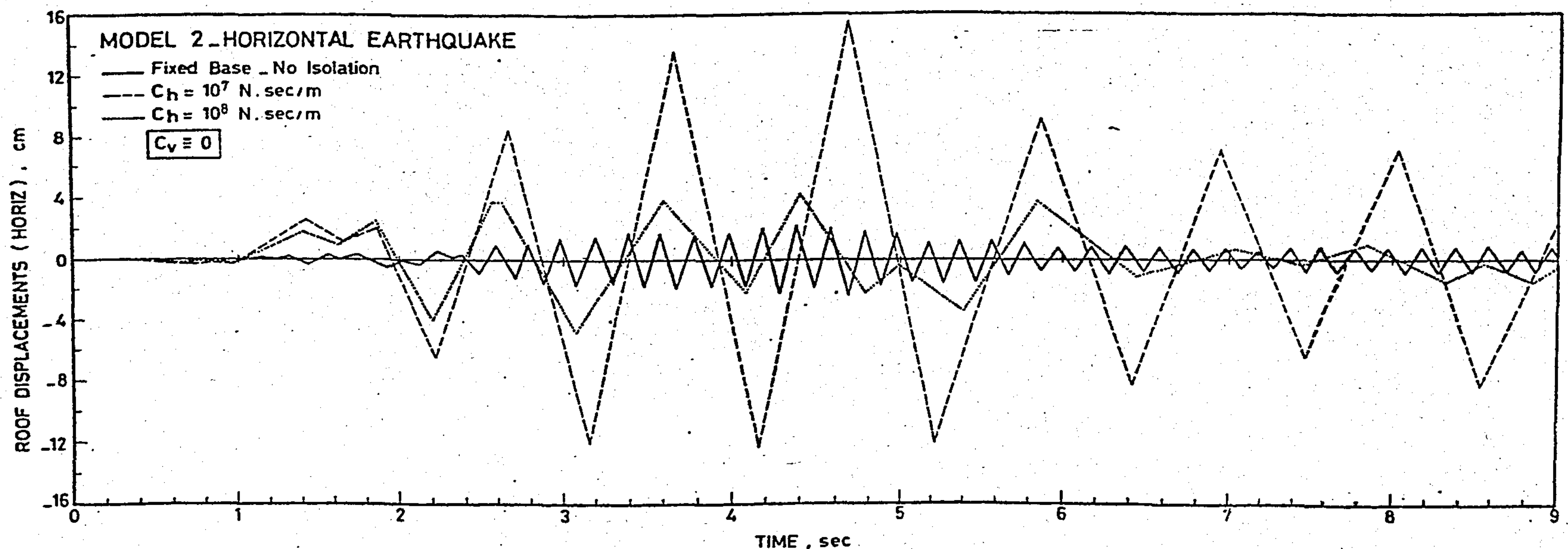


FIG. 3.11 - DISPLACEMENT RESPONSE OF MASS NO. 1 DUE TO HORIZONTAL EARTHQUAKE

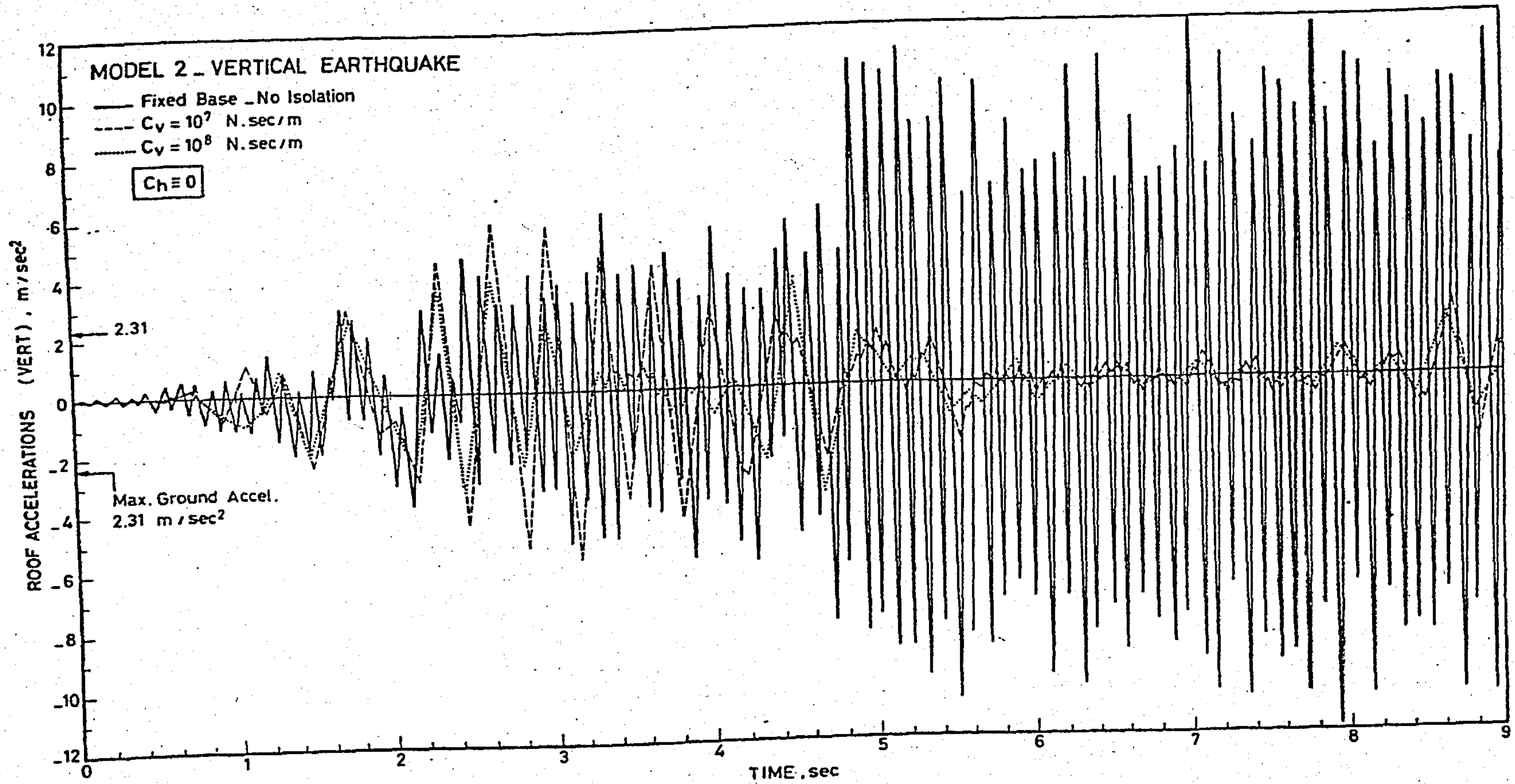


FIG. 3.12 - ACCELERATION RESPONSE OF MASS NO. 1 DUE TO VERTICAL EARTHQUAKE

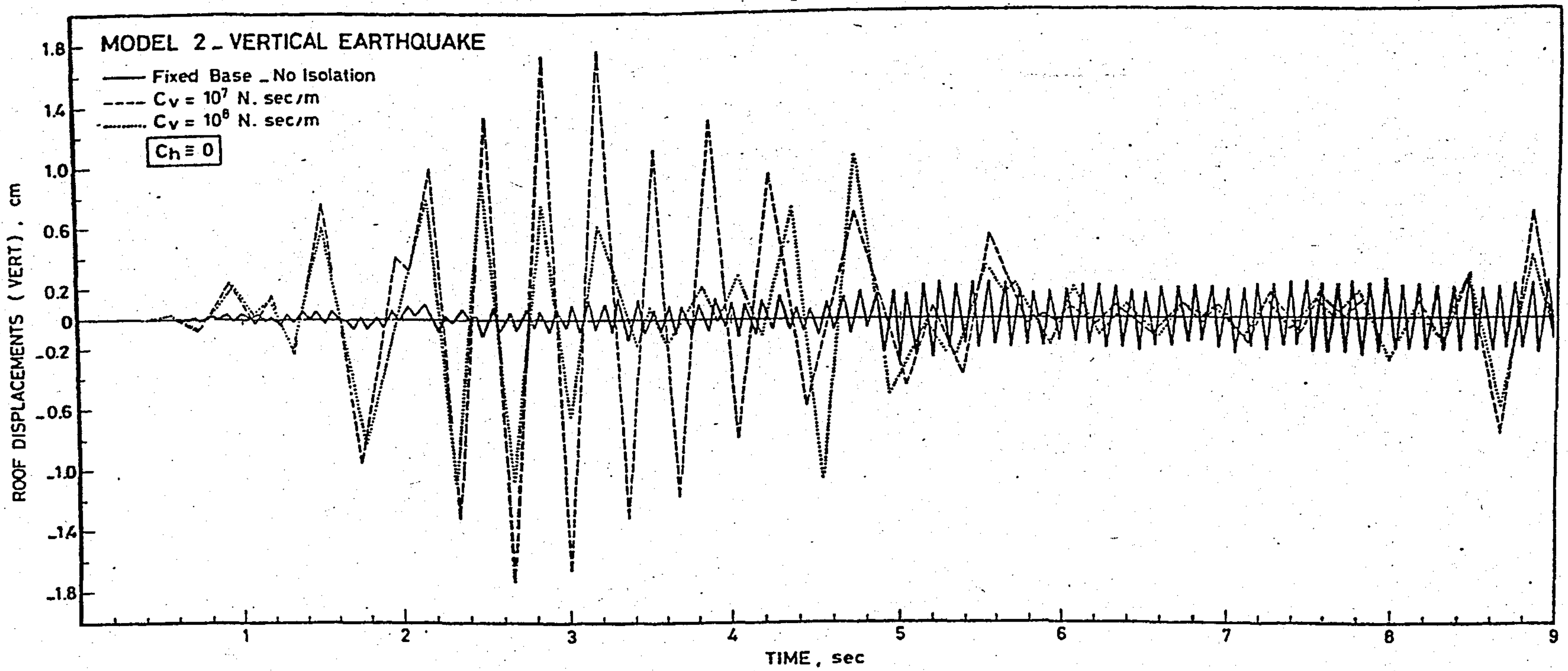


FIG. 3.13 - DISPLACEMENT RESPONSE OF MASS NO. 1 DUE TO VERTICAL EARTHQUAKE

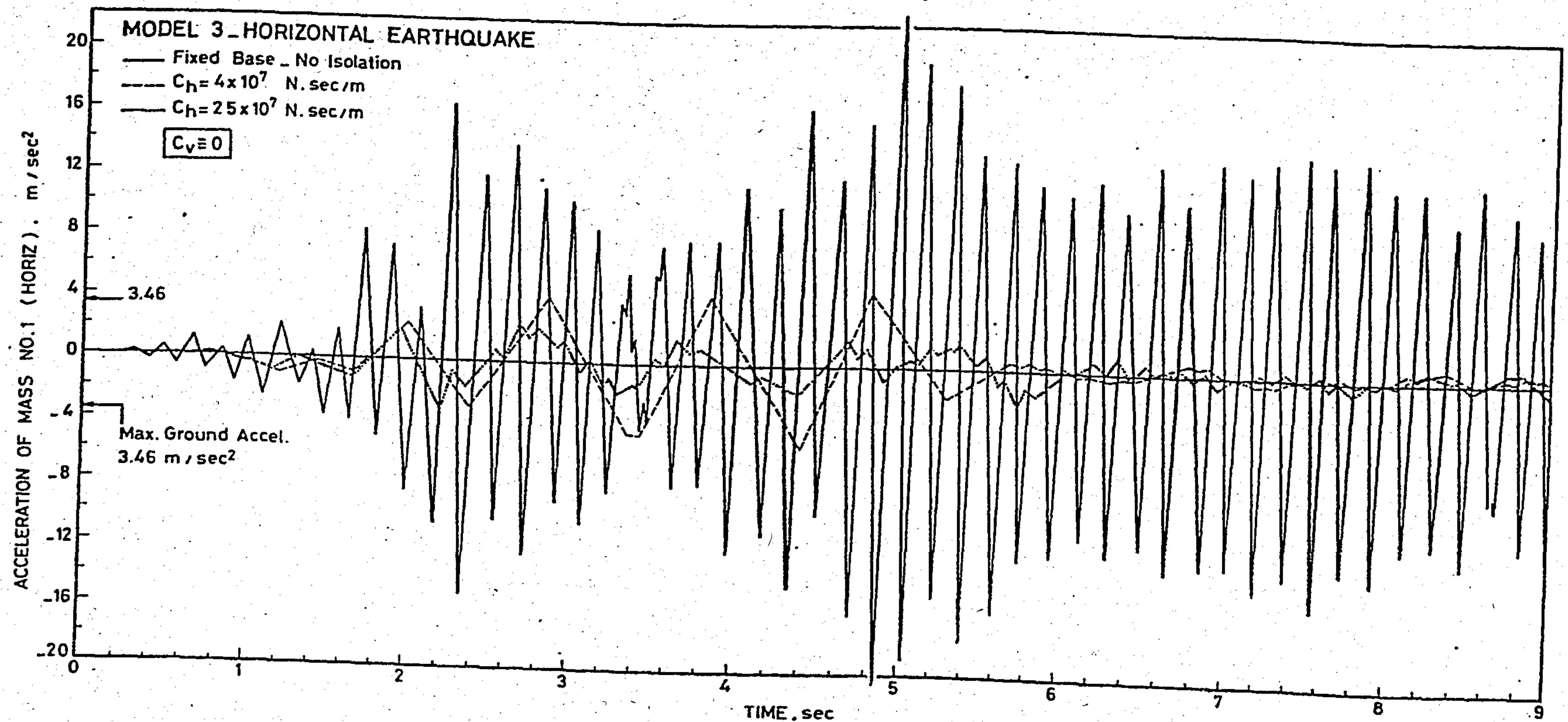


FIG. 3.15 - ACCELERATION RESPONSE OF MASS NO. 1, MODEL 3

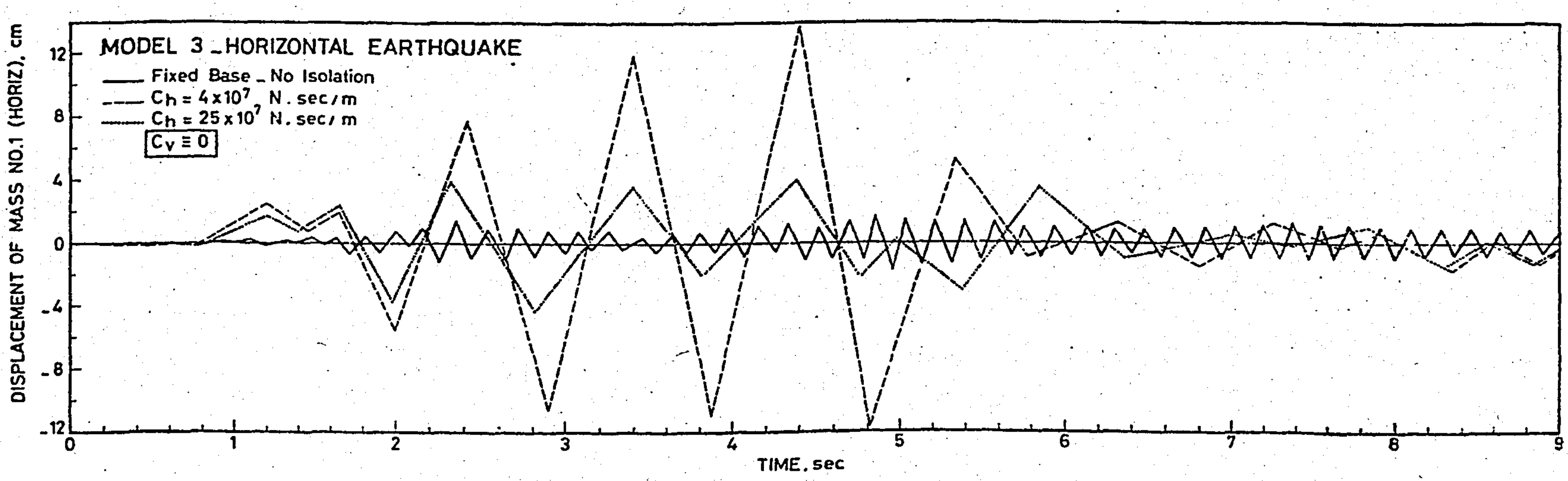


FIG. 3.16 - DISPLACEMENT RESPONSE OF MASS NO.1, MODEL 3

Although, the overall displacement for example, between Mass No. 1 and No. 10, which is significant for design, is only on the order of 0.2 cm.

D) MODEL 4

A typical frame of a seven storey shear wall structure is shown in Fig. 3.17 and it is subjected to the 1940 El Centro earthquake ground motion. The calculated acceleration and displacement responses of each storey are illustrated in Figs. 3.18 & 3.19. With the use of vibration isolators the natural period of vibration is increased from T= 0.44 sec to T= 1.02 sec in the horizontal direction. The accelerations throughout the building are very much less than those for the fixed base case, particularly in the upper storeys. The rigid body displacements are on the order of 7 cm and they are almost identical at all levels. The maximum relative displacement of the roof with respect to the foundation however is only 1.8 cm.

E) GENERAL COMMENTS on MODAL STUDIES

1. Vibration isolators reduce the earthquake loads on a structure by more than five to ten timse with respect to those of a fixed base structure.

2. Vertical springs influence primarily the vertical period of vibration, while the horizontal period of vibration is controlled only by the horizontal springs. Rocking motion is dependent primarily on the vertical springs.

3. The spring constants and amount of damping should be determined by a trial process such

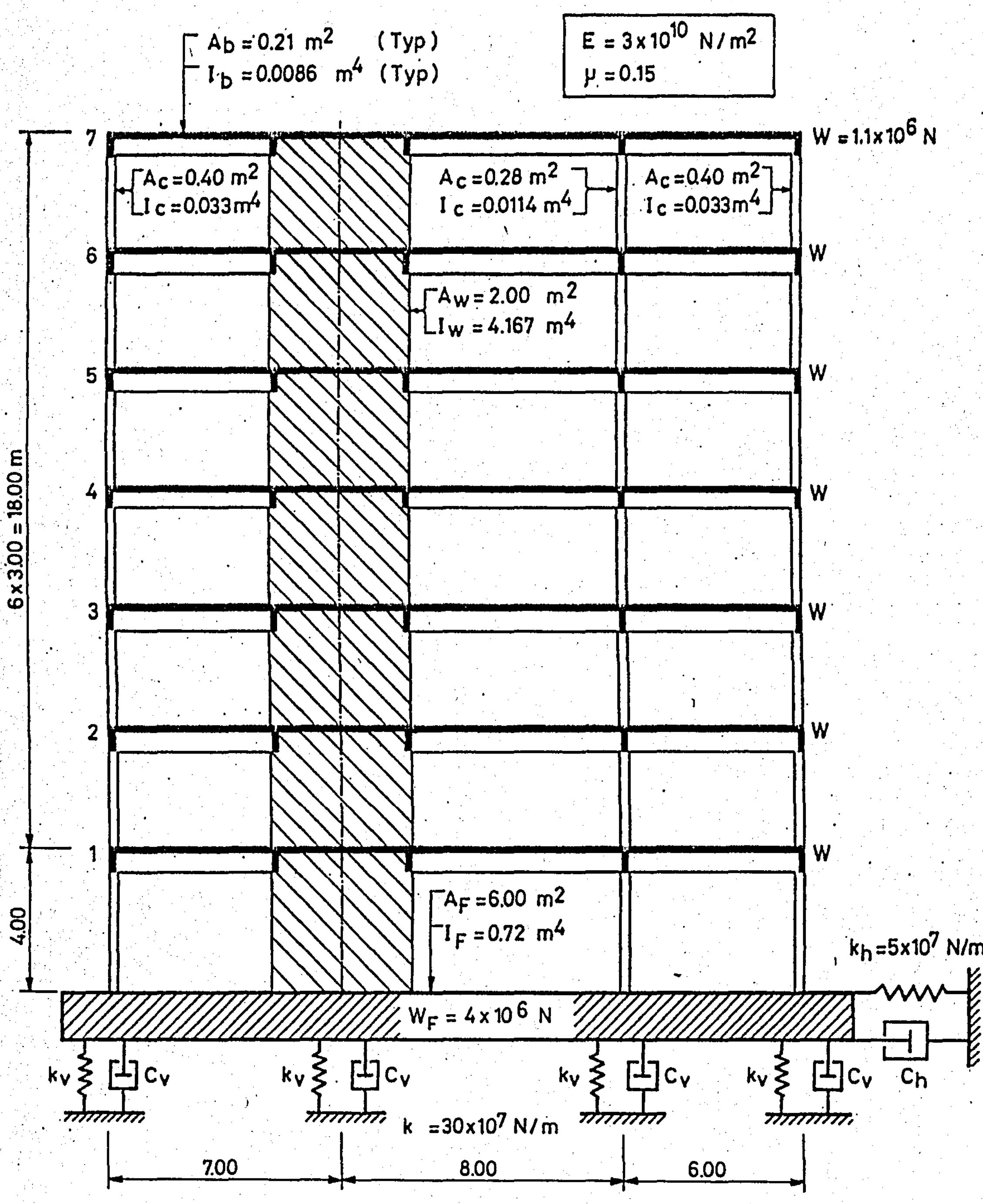


FIG. 3.17 - SEVEN STOREY FRAME , MODEL 4

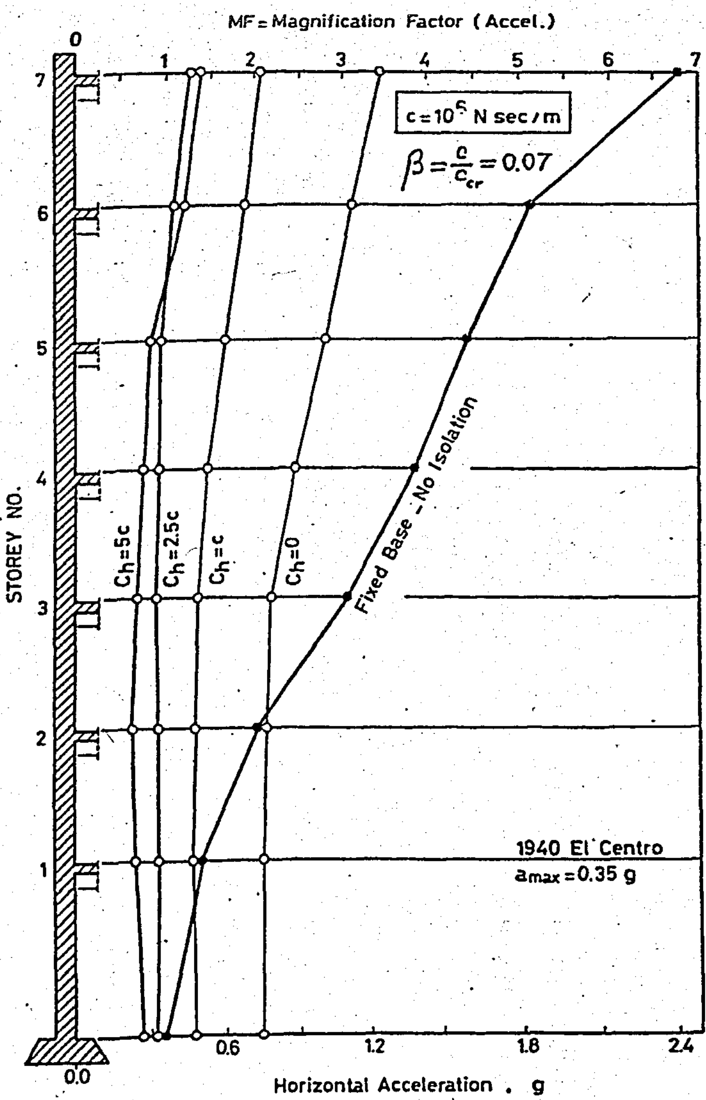


FIG. 3.18 - ACCELERATION RESPONSE, MODEL 4

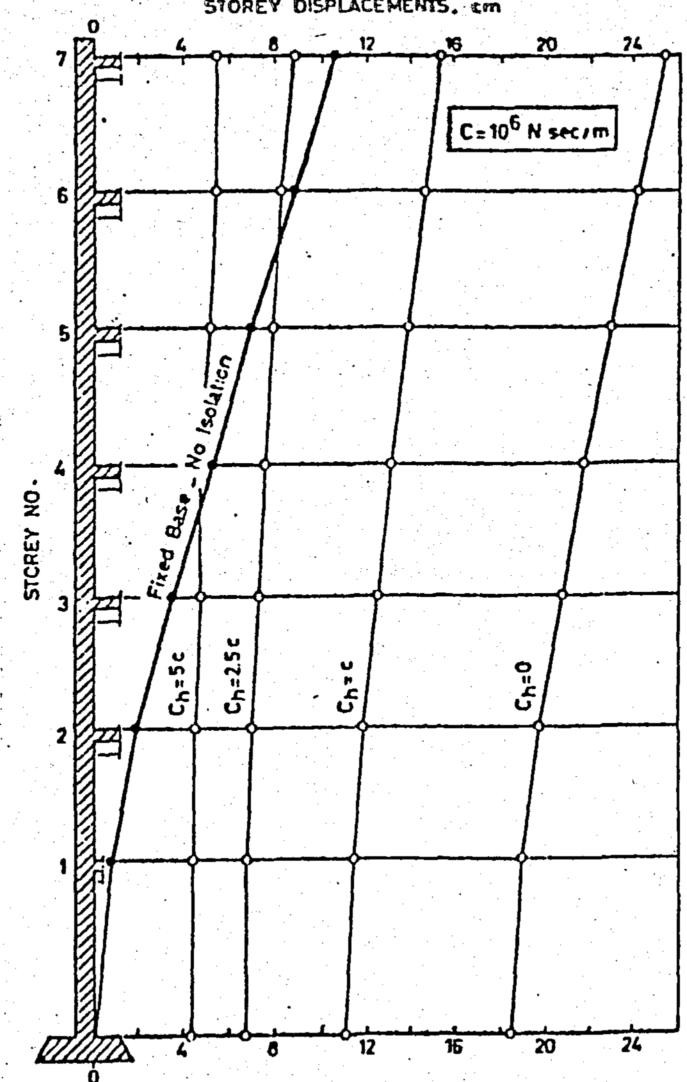


FIG. 3.19 - DISPLACEMENT RESPONSE, MODEL 4

that the vibration isolators reduce the earthquake response with a high degree of efficiency and for a wide range of ground motion characteristics.

4. Vibration isolators are very efficient for structures subjected to ground motions with a predominant period less than about 0.4 to 0.5 secs, which is usually the case for stiff soil and rock concidtions. For longer period ground motions or for structures with very long natural periods of vibration (T>2 sec), vibration isolation may be economically unfeasible.

3.6. NONLINEAR BEHAVIOUR of the ISOLATION ELEMENTS

In the vibration isolation technique, it is desirable that the superstructure remains purely elastic during a strong earthquake. In fact, during small earthquakes, the whole system, including the isolation elements, remains elastic. In the case of unexpectedly very strong earthquakes however, it is possible that the isolation elements may behave nonlinearly, while the superstructure still remains elastic.

On the other hand, using purely linear vibration isolation elements, results in designs, which are overly uneconomical. The intention that the structural system will behave elastically does not necessitate the elastic behaviour of the isolation elements. Vibration isolation systems can also be improved in terms of cost without sacrifice of safety if inelastic deformation is allowed. Furthermore, as analytical procedures become more sophisticated and extensive computer utilization is facilitated, there appears a possibility towards increased safety by determining the behaviour of the systems more accurately.

Therefore, the computational scheme already developed has to be improved for the analysis of structures with localized non-linear elements subjected to earthquakes. A time-history analysis is necessary if elastic-inelastic criteria is used and it required more effort. However, there exists limited number of isolation elements, which may behave non-linearly. Hence, non-linearity is confined to the vibration isolation elements and relatively simple non-linear analysis techniques may be employed.

The main matrix equation of motion developed in the earlier sections of this chapter may be modified to take the probable non-linearity into account. Afterwards, numerical solution algorithm is modified as well. The equation of motion takes the form,

$$[M]\{\ddot{X}\} + [C]\{\dot{X}\} + [K^*]\{X\} + \{R(t)\} = \{F(t)\} \quad \ldots \quad (3.16)$$

where, $\{R(t)\}$ = the force function of time varying displacements, velocities or accelerations. $R(t)$ represents the residual force in the vibration isolators and only those elements on the diagonal, which corresponds to the degrees of freedom at which vibration isolation elements are attached, are non-zero. After seperating the linear and the non-linear parts of the main equation, a sub-structuring algorithm leading to very efficient results may be developed by employing a reduction scheme.

The non-linear behaviour of the isolation elements may be taken into account by a number of models specifying the force-deformation relatoinship developed for the inelastic structural elements under cyclic loading. The most widely used models available in the literature are bilinear, Ramberg-Osgood models, etc. It is reported by Matzer and McNiven

(1976) that both bilinear and Ramberg-Osgood models are not suitable for random earthquake type excitations. However, recently a series of newly proposed models for cyclic behaviour of structural elements has been described by Özdemir (1976). These models are given in the form of differential equations and are sufficiently general to include the mechanical properties of materials such as strain-hardening, stiffness degradation, etc. A single equation governs initial loading, unloading and reloading and it behaves well in the case of arbitrary excitations and suits well to the iterative computation schemes like the ones used in this study.

CHAPTER 4

SHAKING TABLE TESTS
OF A
5- STOREY STEEL FRAME

4.1. OBJECTIVES and SCOPE of the LABORATORY INVESTIGATIONS

The experimental studies conducted at *The Institute of Earthquake Engineering and Engineering Seismology,* Skopje, Yugoslavia (I2II3), consist of testing two distinct types of energy-absorbing systems, most favorable for seismic isolation, (a) helical springs and visco dampers, and (b) rubber pads.

In order to investigate the behaviour of these isolation systems and also to study their respective advantages over the other systems, a series of experiemntal tests were conducted on a five storey, three bay steel frame, which is almost a copy of the one which was already tested earlier in the University of California at Berkeley with rubber base isolation systems (Kelly et al, 1980).

The purpose of selecting a frame model similar to the frame previously tested at Berkeley, was to compare the relative performances of the rubber and spring-dashpot isolation systems.

One spring and one dashpot unit were placed at each extreme corner of the base beam (4D-case). In order to determine the influence of damping however, same tests have been repeated using one additional dashpot unit at each corner (8D-case). As an alternate set of investigations, springs and dashpots were substituted by one rubber pad at each corner. Both types of base isolation, as well as the fixed base configuration, were subjected to simulated earthquake motions at the shaking table under pure horizontal or vertical motions and simulatneous action of biaxial excitations.

After all test results were available, it was possible to make a comparative study about the relative advantages of various support conditions and base isolation systems. In general, it was determined that the spring-dashpot system considerably decreased the acceleration amplitudes of the model and provided adequate energy absorption in reducing displacement response values. In the case of rubber isolation however, the storey displacements were found to be excessively large even at relatively low magnitude input motions.

Summary of test results and their comparison with those of anaytical investigations have been given at later chapters.

4.2. SHAKING TABLE CHARACTERISTICS

The experiments were carried out at the *Earthquake Simulator of the Institute of Earthquake Engineering and Engineering Seismology,* University, *Kiril and Metodij* , Skopje, Yugoslavia (I2IIS). The shaking table is a prestressed reinforced concrete rigid slab five to five meters in plan. The table is supported and excited by four vertical electro-hydraulic actuators having a total capacity of 880 kN. In horizontal direction the table is excited by two actuators with a capacity of 850 kN.

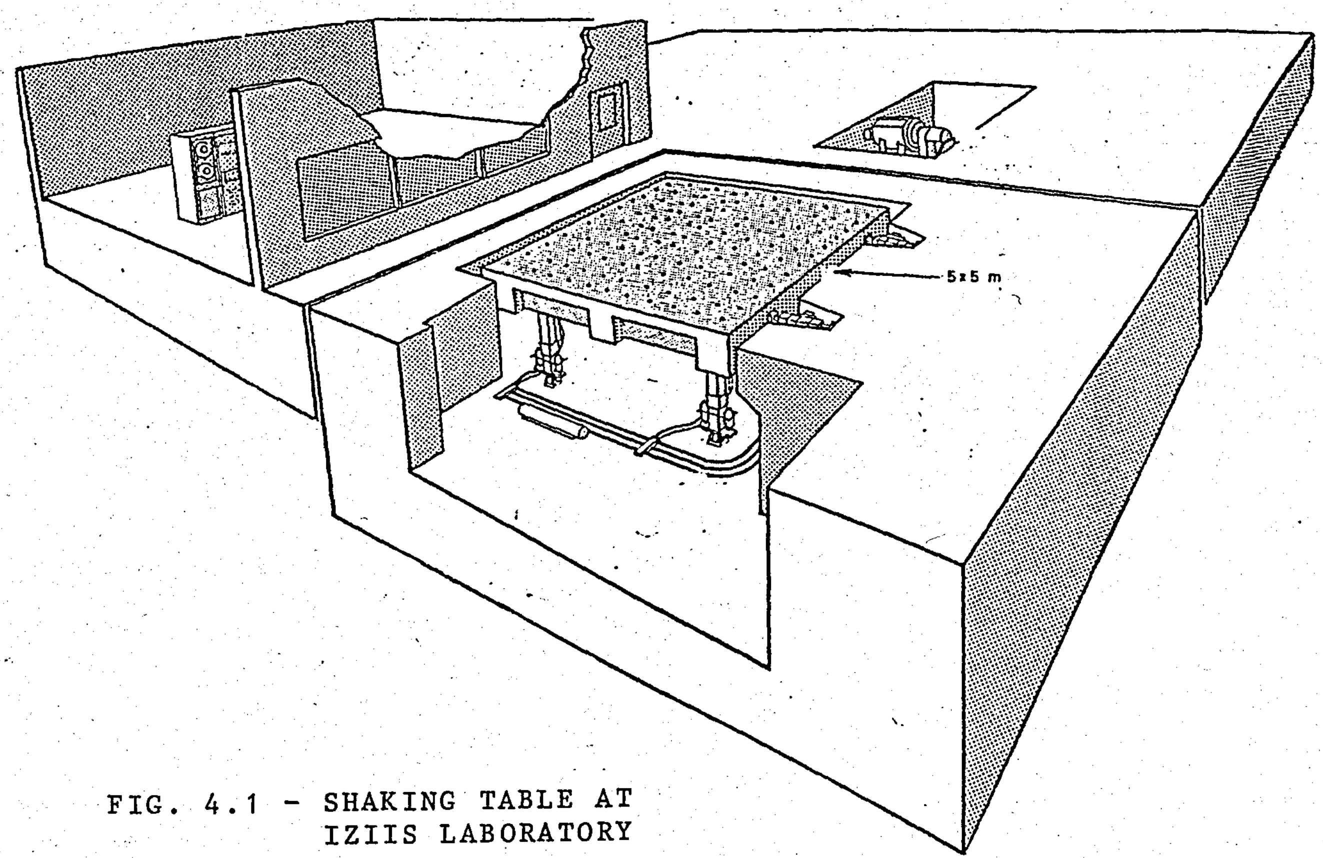


FIG. 4.1 - SHAKING TABLE AT IZIIS LABORATORY

A general outline of the shaking table is given in Fig. 4.1.

The gravity loads of the table and the model are supported by means of a special system of static supports using nitrogen, gas during the operations, thus relieving the vertical actuators of any static-load. The total static load carrying capacity of the supporting system is 720 kN (32 tons of shaking table plus 40 tons of the model). The height of the model is limited to 6 m.

Excitation is possible in the frequency range from 0 Hz to 30 Hz. Under certain circumstances however, frequencies up to 80 Hz may be also reached. Figs. 4.2 and Fig. 4.3 show the excitation characteristics in horizontal and vertical direction for total weight of 35 tons, 50 tons and 70 tons. Maximum displacements are 12.5 cm in horizontal and 5 cm in vertical direction. Corresponding maximum velocities are 63 cm/sec and 38 cm/sec, respectively. Maximum accelerations depend on the mass of the model and range from 1.20 g to 2.4 g in the horizontal and from 1.05 g to 2.14 g is the vertical direction.

The analogue control system has a capacity of controlling five degrees of freedom; one horizontal and one vertical translations plus three rotations. Translation of the platform along the second horizontal direction is prevented by means of special hydraulic supports, one controlling the force and the other displacement. A reverse control is provided by three variable servo control system which is capable of simultaneous control of displacements, velocities and accelerations. This three variable control method is a new technological solution offering many advantages. One advantage is that the system at low frequencies provides control of displacement while at higher frequencies it provides control of acceleration.

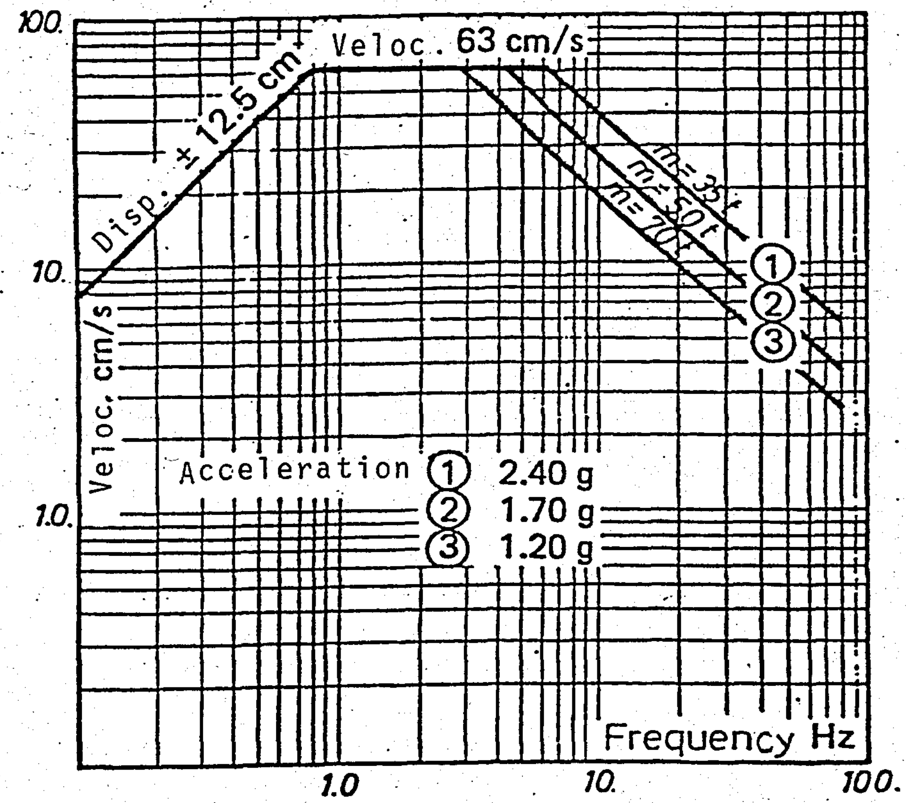


FIG. 4.2 - SHAKING TABLE CHARACTERISTICS - HORIZONTAL DIRECTION

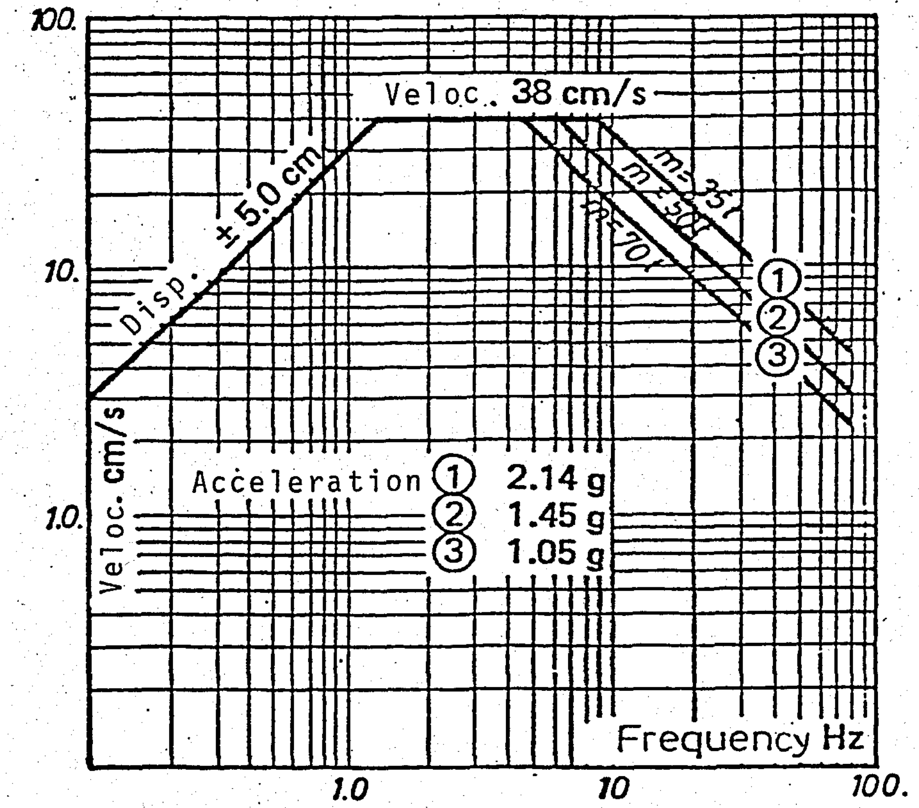


FIG. 4.3 - SHAKING TABLE CHARACTERISTICS - VERTICAL DIRECTION

In order to generate earthquake motions the time history ordinates in the form of displacements or accelerations are stored in a PDP 11/45 computer as digital data. A data acquisition is accomplished by a fast electronic system connected to the PDP 11/45 computer. A package of computer programs has also been developed to provide an on-line control of operations.

4.3. TEST FRAME and BASE ISOLATION ELEMENTS.

A) PROPERTIES of TEST FRAME

The experimental model used is shown in Fig. 4A. It is a five storey three bay steel frame mountde on two heavy base girders, which are supported on the shaking table for simulation of the fixed base model. In order to simulate the base isolated cases however, sets of spring-dashpot units as well as rubber elements are used at the corners of base girders.

Although the structural members have different cross sectional properties, the I2IIS test frame has similar dynamic characteristics as the test frame used in Berkeley, California for testing of rubber base isolation (Kelly et al, 1980). The dead load is provided by emans of steel blocks tied down to the frame at each floor level. The dead weight at upper floors is 4700 kg. The total dead load at the upper floors is approximately 23.5 ton, while the weight of the two frames including the base girders and the bracing, is 2.5 ton. An additional dead weight of 6.4 ton is attached onto the base girders, thus exerting a total of 32.4 ton weight on four springs.

The dead load provided by the steel blocks produces stress levels comparable to those in a full-scale structural

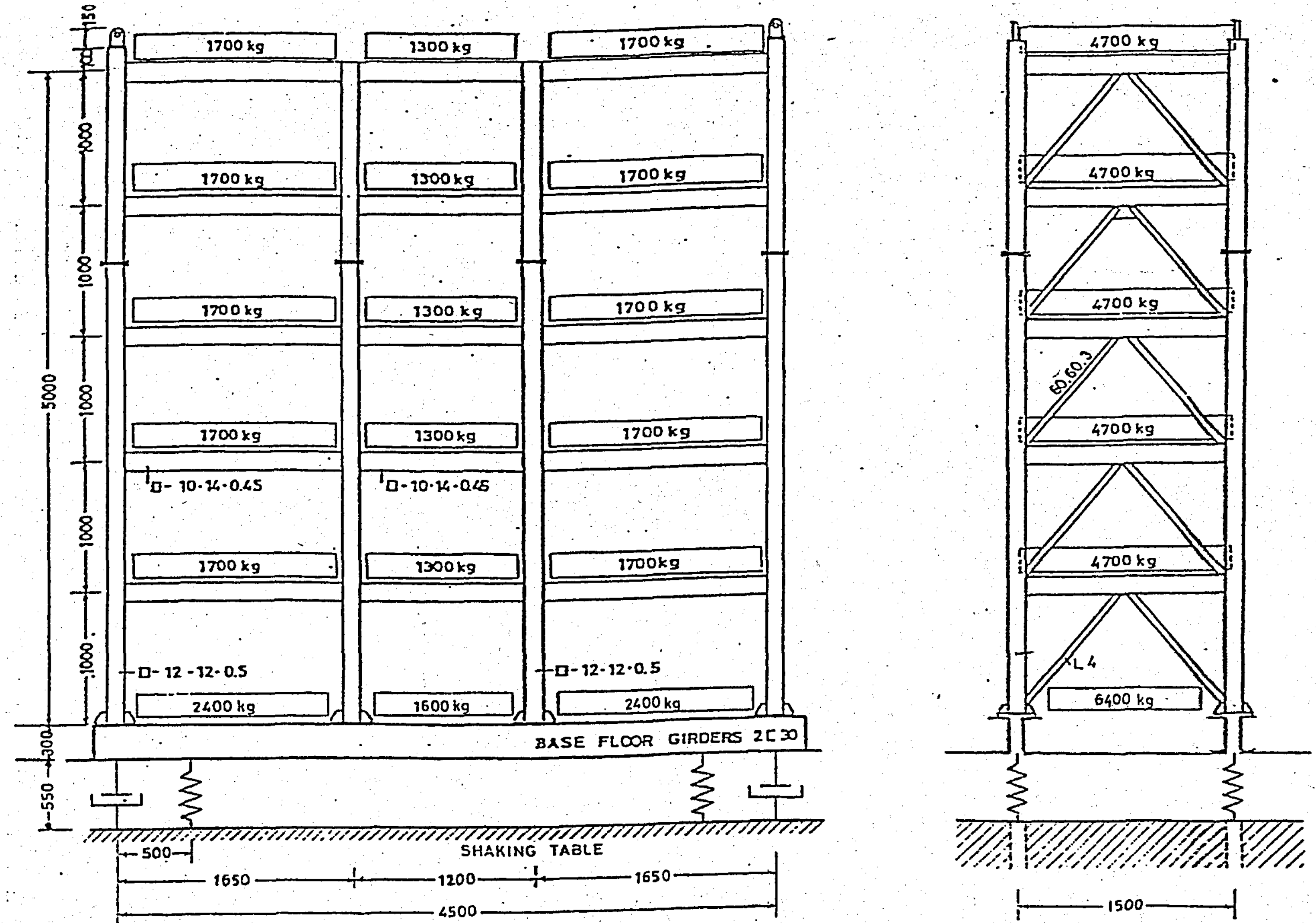


FIG. 4.4 - THE TEST MODEL ON THE SHAKING TABLE

frame. The geometrical scale factor of the model is roughly 1 to 4. The corresponding time scale factor will then be $\sqrt{4} = 2$.

The experimental program includes four cases of structural support conditions. The first series of experimental tests refer to the fixed base model in which the floor girders are fixed to the shaking table.

The other three cases correspond to base isolated models, two of which are with spring-dashpot units and one is with rubber elements.

B) SPRING-DASHPOT UNITS

These spring-dashpot systems consists of four springs and four dashpot units (4D-case) placed under the base floor girders as shown in Fig. 4.5. The other systems consists of four springs and eight dashpot units (8D-case). The dashpots are placed at each end of the model along the end column lines. The total weight of the vibration isolation system in the 4D-case is 3 ton, thus the total weight of the model and the isolation elements on the shaking table becomes 35.4 ton.

The spring constants of a single helical spring are:

in vertical direction, $k_v = 0.748$ kN/mm
in horizontal direction, $k_h = 0.395$ kN/mm

The damping resistance of the viscodampers is very much dependent on temperature and frequency. In the range of the natural frequencies of the test model, working as a rigid body on top of the springs, and at an ambient temperature of 20°C the damping coefficient for each viscodamper unit is:

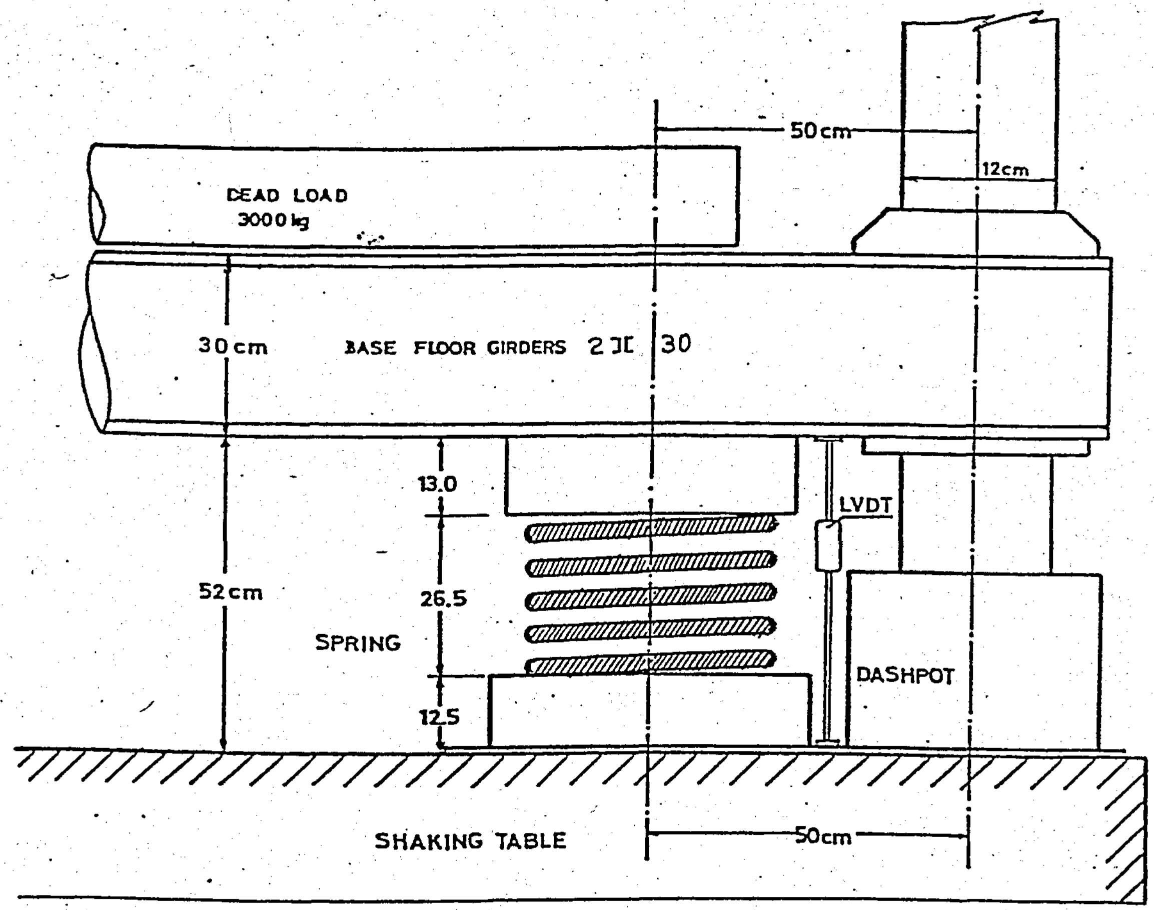


FIG. 4.5 - POSITION OF THE ISOLATION ELEMENT UNDER THE STRUCTURAL MODEL

in vertical direction, C_v = 30 kN-sec/m

in horizontal direction, C_n = 25 kN-sec/m

It is intended that the structural model has similar dynamic properties as the model tested at Berkeley. Free vibration analyses of these two separate frames have been conducted, and the natural periods of vibration for the first four modes for both the I211S and the Berkeley models are summarized in Table 4.1. It is expected that with the coincidence of the natural periods of vibration and the mode shapes, the two test models will show similar dynamic behaviour.

TABLE 4.1: NATURAL PERIODS of VIBRATION, sec

Mode	I211S Model	Berkeley Model
1	0.253	0.227
2	0.081	0.073
3	0.046	0.043
4	0.032	0.031

4.4. SEISMIC PROPERTIES of RUBBER PADS

A) GENERAL FORMULATION

In order to be able to achieve any desired level of vibration isolation, it is essential to establish the dynamic characteristics of the rubber elements. Normally, the rubber elements are either cylindrical or prismatic in shape. Some rubber elements are laminated and reinforced at horizontal layers with steel plates. The shearing strain capacity of a rubber element is determined by means of laboratory tests under statis and dynamic loadings. The shear strain-γ, is related to the shear modulus-G, by means of Hooke's law as,

$$\gamma = \frac{\tau}{G}, \quad \text{or} \quad \gamma = \frac{V}{AG} \qquad \ldots \qquad (4.1)$$

in which,

A = cross sectional area of the rubber pad

For small deformations γ also represent the pad distortion and is defined by

$$\gamma = \frac{d}{h}, \quad \text{or} \quad d = \gamma h = \frac{Vh}{AG} \qquad \ldots \qquad (4.2)$$

in which,

d = lateral displacement, and

h = height of the pad

Considering shear stiffness-k defined as

$$k = \frac{V}{d} \qquad \ldots \qquad (4.3)$$

the horizontal stiffness of the rubber pad, after substituting Eq. 4.2 into Eq. 4.3, becomes

$$k = \frac{AG}{h} \qquad \ldots \qquad (4.4)$$

For a single degree of freedom system with a mass-m, and stiffness-k, the natural period of vibration, T is:

$$T = 2\pi\sqrt{\frac{m}{k}} \quad \ldots \quad (4.5)$$

or, for a rubber element

$$T = 2\pi\sqrt{\frac{Wh}{gAG}} = 2\pi\sqrt{\frac{\sigma h}{gG}} \quad \ldots \quad (4.6)$$

in which,

W = vertical load supported by the rubber pad,

σ = vertical stress on the rubber pad due to vertical loads, and

g = gravitational acceleration.

It is possible to relate the horizontal acceleration-a, to the pad distortion-γ, by considering maximum value of acceleration in a harmonic motion as,

$$a = w^2 \, d \quad \ldots \quad (4.7)$$

Therefore, from eq. 2

$$\gamma = \frac{a}{w^2 h} \quad \ldots \quad (4.8)$$

Substituting, from eq. 6 and using $w = 2\pi/T$

$$w^2 = \frac{gAG}{Wh} = \frac{gG}{\sigma h} \quad \ldots \quad (4.9)$$

and,

$$\gamma = \frac{aW}{gAG} = \frac{a\sigma}{gG} \quad \ldots \quad (4.10)$$

$$a_{max} = \frac{gG}{\sigma} \gamma_{max} \quad \ldots \quad (4.11)$$

If, maximum horizontal acceleration $-a_{max}$ is given, for a specific rubber pad, maximum pad distortion $-\gamma_{max}$ may be obtained from Eq. 4.10 given above. It is seen from this expression that, in order to increase the capacity against maximum accelerations, the vertical load on the rubber pad should be proportionally reduced.

B) EXPERIMENTAL DATA

Extensive cyclic tests conducted at the laboratories of the *Electricite de France-Septen, Paris,* indicate that the laminated rubber pads may safely carry vertical design loads up to

$\sigma_{max} = 50$ MPa or

$\sigma_{max} = 500$ kg/cm² (1 MPa $\neq$ 10 kg/cm²)

The elastomer and the reinforcing steel plates start to seperate (unbond) from each other only after several cycles of alternating distortions with $\gamma > 2$ (Guerand, 1981).

For instance, considering an elastomer pad with

$G = 1.10$ MPa

$\sigma = 7$ MPa

the maximum acceleration that can be applied at the top of the rubber pad, for a maximum distortion of $\gamma=2$,

$$a_{max} = \frac{gG}{\sigma}\gamma_{max} = \frac{1.10g}{7} \cdot 2 = 0.31 \text{ g}$$

The laminated rubber pads used to support the five storey steel frame at the I2IIS laboratories had the following properties:

$$h = 0.20 \text{ m}$$

$$A = 0.20 \times 0.20 = 0.04 \text{ m}^2$$

$$G = 1.15 \text{ MPa} \quad = 11 \text{ kg/cm}^2$$

Therefore,

$$\sigma = W/A = 32.4 \text{ ton}/(0.04 \times 4) = 202.5 \text{ ton/m}^2$$

$$\sigma = 2.025 \text{ MPa}$$

$$T = 2\pi\sqrt{\frac{\sigma h}{Gg}} = 2\pi\sqrt{\frac{2.025 \cdot 0.20}{1.15 \cdot 9.81}} = 1.19 \text{ sec}$$

$$k_h = \frac{AG}{h} = \frac{0.04 \cdot 1.15}{0.20} = 0.23 \text{ MN/m} = 0.23 \cdot 10^6 \text{ N/M}$$

$$k_v = 115.65\ k_h = 26.6 \cdot 10^6 \text{ N/m} \quad \text{(tested in the laboratory)}$$

For an assumed maximum distortion of $\gamma=0.35$, the maximum acceleration that can be applied at the top of these rubber pads, is

$$a_{max} = \frac{gG}{\sigma}\gamma_{max} = \frac{1.15g}{2.025}\ 0.35 = 0.20 \text{ g}$$

In fact, during the shaking table tests at the I2IIS laboratories, in Skopje, Yugoslavia, the peak accelerations of the shaking table were limited to be less than 0.20 g. Consequently, the maximum pad distortions were not allowed to exceed $\gamma = 0.35$ which corresponds to a maximum lateral displacement, at the top of the pad, as defined by

$$d_{max} = \gamma_{max}\ h = 0.35 \cdot 20 = 7\ \text{cm}$$

4.5. EXPERIMENTAL PROGRAM

The experimental program has been prepared to provide as many as possible experimental data necessary for accurate interpretation of the dynamic behaviour of the structure with and without base isolation. The input motions utilized in the experimental program is grouped in two categories:

A) SINUSOIDAL and IMPULSE TESTS

In order to evaluate the natural frequencies of the test model with and without isolation, the test series began with sinusoidal or impulse vibration excitations at different frequencies. Data were gathered only on two channels (displacement and acceleration) and were recorded on an oscilloscope which yields immediate results.

Tests were run for frequency band in the range of the expected natural frequencies of teh test model. For fixed base model,more accurate results were obtained from impulse tests. For sinusoidal tests, however, significant rolling effect was observed at the shaking table which results in a decrease of the actual value of natural frequencies, especially for the frist mode of vibration.

In the base isolated cases more accurate values of natural frequencies were obtained by simulating sinusoidal motion on the shaking table both for horizontal and vertical directions.

B) SIMULATED EARTHQUAKE MOTIONS

The earthquake testing program was selected to be the most representative for the most common destructive earthquakes. The following earthquakes were selected:

1. El Centro 1940, component N-S, real time (Δt=0.02 sec)
2. El Centro 1940, component N-S, time scaled (Δt=0.01 sec)
3. El Centro 1940, vertical component, real time
4. El Centro 1940, component N-S and vertical component simultaneous motion, real time
5. Montenegro earthquake, record at Petrovac, component N-S, real time
6. Montenegro earthquake, recorded at Petrovac, component N-S, time scaled.
7. Montenegro earthquake, recorded at Petrovac, vertical component, real time

For each of the selected types of earthquakes and their combinations, the simulation of each earthquake was achieved by 3 to 4 different levels of accelerations given to the shaking table by scaling the displacement amplitudes.

Altogether 73 tests of the structural model with and without vibration isolation elements were carried out using the above earthquakes as simulated input motion.

In order to provide an easy test identification, each test has its own identification number consisting of 8 digits. The first two digits identify the base condition as follows:

FB = Fixed base model
D4 = Model with four dashpot elements
D8 = Model with eight dashpot elements

The next two digits identify the type of earthquake:

EN = El Centro earthquake real time (Δt= 0.02 sec)
EB = El Centro earthquake time scaled (Δt= 0.01 sec)
PN = Petrovac earthquake real time (Δt= 0.02 sec)
PB = Petrovac earthquake time scaled (Δt= 0.01 sec)

The earthquake component has been denoted by the fifth and the sixth digits of the identification number.

NS = Component North-South
VK = Vertical component

The last two digits denote the value of displacement amplitude scaling factor SPAN(*) comprising only two out of the three used numbers. Thus, if the identification code ends up with 30 it means that the SPAN value is 300, etc.

This scheme of notation applies only to single component motion. In case of biaxial motion a change has been introduced in the last four digits. Thus, the direction should not be dnoted in this case and therefore four numbers denoting the SPAN of each direction have been selected. The first two numbers denote the SPAN of horizontal direction while the last two in vertical direction. As an example, the identification code FBEN4020 denotes:

FB = Fixed base

EN = El Centro real time

40 = SPAN for horizontal component, which means 400

20 = SPAN for vertical component meaning 200

The list of "Tests" performed using the El-Centro and the Petrovac earthquakes is given in Tables 4.2 & 4.3, respectively. The peak input acceleration for each test is also supplied in these tables.

(*) For definition of SPAN, please refer to Section 4.6

TABLE 4.2: TESTS WITH THE 1940 EL CENTRO EARTHQUAKE

(a_{max} = 342 cm/sec^2)

(a_{max} = 206 cm/sec^2)

CODE	FIXED BASE	D4-DAMPERS	D8-DAMPERS
EN NS 20	1 FB 137 [2]	18 D4 137	44 D8 128
EN NS 40	2 FB 257	19 D4 256	45 D8 252
EN NS 60	3 FB 365	20 D4 352	46 D8 377
EN NS 70	-	21 D4 388	47 D8 411
EN NS 80	-	-	48 D8 440
EB NS 10	4 FB 105	22 D4 111	49 D8 131
EB NS 20	5 FB 204	23 D4 203	50 D8 221
EB NS 30	6 FB 299	-	-
EB NS 40	-	24 D4 397	51 D8 384
EB NS 50	-	25 D4 483	52 D8 493
EN VK 10	-	-	53 D8 17
EN VK 20	7 FB 45	26 D4 50	54 D8 34
EN VK 40	8 FB 87	27 D4 86	55 D8 66
EN VK 60	-	28 D4 122	56 D8 207
EN 20 10	-	29 D4 221/44 [3]	57 D8 247/36
EN 30 15	-	30 D4 302/53	-
EN 40 20	-	31 D4 416/81	58 D8 370/62
EN 60 30	9 FB 485/153	-	59 D8 493/94
EB 20 10	-	32 D4 306/87	60 D8 312/80
EB 30 15	-	33 D4 411/147	61 D8 430/114
EB 40 20	-	-	62 D8 556/149

(1) D8-case analytical study is repeated using accelerations recorded at the Shaking Table (ST).

(2) 1 FB 137 has the following meaning:
1 = Run number used in the laboratory
FB = Fixed Base condition
137 = Peak acceleration of the input (cm/sec^2)

(3) 221/44 has the following meaning:
221 = The peak acceleration in horizontal direction
44 = The peak acceleration in vertical direction

TABLE 4.3: TESTS WITH THE APRIL 15, 1979 PETROVAC EARTHQUAKE

($a_{max,h}$ = 427 cm/sec^2)
($a_{max,v}$ = 198 cm/sec^2)

CODE	FIXED BASE	D4-DAMPERS	D8-DAMPERS
PN NS 10	10 FB 178	34 D4 71	63 D8 70
PN NS 20	11 FB 141	35 D4 142	64 D8 138
PN NS 30	12 FB 211	36 D4 220	-
PN NS 40	13 FB 270	-	65 D8 282
PN NS 50	-	37 D4 359	-
PN NS 60	-	-	66 D8 417
PB NS 10	-	38 D4 280	67 D8 175
PB NS 20	14 FB 345	39 D4 354	68 D8 344
PB NS 30	15 FB 480	40 D4 464	-
PB NS 40	-	-	69 D8 634
PN VK 10	-	-	70 D8 40
PN VK 20	16 FB 71	41 D4 80	71 D8 60
PN VK 40	17 FB 138	42 D4 146	72 D8 108
PN VK 60	-	43 D4 220	73 D8 136

(1) D8-case analytical study is repeated using accelerations recorded at the Shaking Table (ST).

(4) Table peak acceleration is unnecessarily larger than that for SPAN = 200.

4.6. INSTRUMENTATION and RECORDING of DATA

The instrumentation of the table is permanently incorporated and average vertical and horizontal table displacements and accelerations as well as pitch, roll and twist motions are recorded.

The frame was instrumented to measure accelerations, displacements and stresses. Identification number and position of these pick-ups are shown in Fig. 4.6. Horizontal accelerations on the frame were recorded by accelerometers on the base and each floor level. The accelerometers were mounted onto the exterioe transversal beams of the structural model at each floor level close to the center line between the two frames.

Displacements were recorded by linear potentiometers with respect to e reference column located outside the shaking table at the foundation block. The horizontal displacements were measured at the base girder and also at each floor level. Vertical displacements were measured only at the point of two front springs. The axial and bending stresses were measured by strain gages at the external columns of the lowest storey.

Data samples were taken at a rate of 200 samples per second for each channel and then stored on a magnetic tape. The records were taken from the following 30 channels:

Channel 1 : Horizontal table displacement
Channel 2 : Horizontal table acceleration
Channel 3 : Vertical table displacement
Channel 4 : Vertical table acceleration
Channel 5-10: Horizontal displacements at each different floor level

.../..

Channels 11-16:	Horizontal accelerations at each different floor level
Channels 17-18:	Vertical displacement at the front springs. By this records the rocking could be identified, too.
Channels 19-20:	Vertical accelerations at the ends of of the base girders.
Channels 21-22:	Accelerometer measuring the motion in perpendicular direction.
Channel 23 :	Horizontal acceleration of a cantilever having natural frequency of the first natural frequency of the structural model.
Channel 24 :	Horizontal acceleration of a cantilever having natural frequency of the second natural frequency of the structural model.
Channel 25 :	Vertical acceleration of a vertical beam having natural frequency of the structural model in horizontal direction.
Channel 26 :	Vertical acceleration of a vertical beam having natural frequency of the second natural frequency of the structural model in horizontal direction.
Channels 27-28:	Strain gages measuring axial stresses at the external columns of the first floor of the structural model
Channels 29-30:	Strain gages measuring bending stresses of the external columns of the first floor of the structural model.

4.7. INPUT MOTIONS at the SHAKING TABLE

For the generation of earthquake motion, the digital computer PDP 11/45 is used. The shaking table is displacement controlled. Therefore, it was necessary to transform the original acceleration ordinates into a displacement time history through a double integration process.

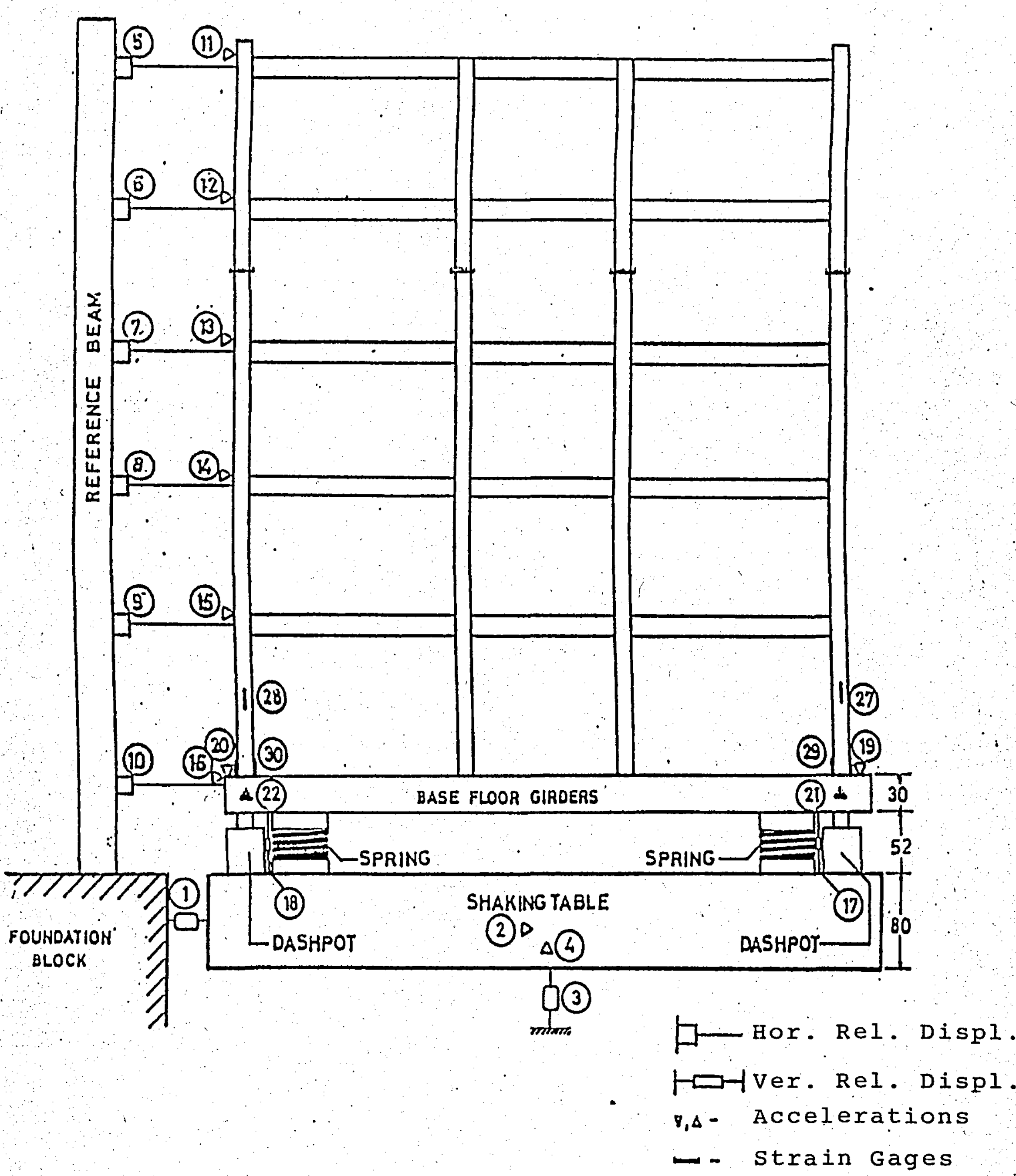


FIG. 4.6 - INSTRUMENTATION SCHEME

The tests are performed with the El Centro Earthquake of 1940 and the Petrovac, Montenegro Earthquake of 1979. In both cases horizontal N-S and vertical components are used.

The time interval for the input motion data points is normally specified to be $\Delta t = 0.02$ sec (*Real Time Interval*). For aprt of the tests however, the time interval is scaled down by a factor of 2, and is taken as $\Delta t = 0.01$ sec (*Scaled Time Interval*). In this way, the governing peaks of the input motion are shifted to higher frequencies, and more realistic results are obtained for the model, because the geometrical scale between the model and the prototype is 1 to 4.

The maximum input acceleration is also modified by means of a variable factor which determines the intensity of a particular earthquake during the tests. The amplitudes are varied by means of the SPAN feature- which is directly correlated to the maximum table displacement. A peak table displacement of ±12.5 cm - the limit of the table- corresponds to a span number of 1000. Lower span numbers correspond to proportionately lower displacements. It must also be noted that the peak table acceleration for the same SPAN number for different earthquakes may vary considerably. This is because the table motion is displacement controlled and, in addition, there is probably small amount of structure - table interaction which varies with different base conditions.

The time history accelerations recorded at the shaking table are given in Tables 4.4 & 4.5 for various input motions.

TABLE 4.5 - D8 PN NS 40 - SHAKING TABLE RECORDS

IDENTIFICATION CODE: D8PNNS40.INV

CHANNEL: 2 TABLE ACCELERATION TIME HISTORY (MM/SEC**2)
DT = 0.02 SEC

PETROVAC, Real Time D8 PN NS 40 $F = \frac{4273}{2717} = 1.572$

1	-14.38	43.13	345.05	100.64	71.89	115.02	-14.38	0.00	-86.26	158.15
11	-100.64	186.90	43.13	100.64	43.13	43.13	-71.89	158.15	-14.38	57.51
21	0.00	71.89	0.00	86.26	-57.51	115.02	14.38	57.51	43.13	43.13
31	57.51	57.51	-86.26	172.53	-14.38	-71.89	28.75	14.38	14.38	57.51
41	-71.89	28.75	-43.13	14.38	-28.75	43.13	-28.75	14.38	-28.75	28.75
51	14.38	14.38	43.13	14.38	43.13	-14.38	-43.13	14.38	0.00	-14.38
61	-57.51	-14.38	-14.38	-71.89	-57.51	-28.75	-14.38	-71.89	-28.75	-28.75
71	0.00	-57.51	57.51	-57.51	43.13	-28.75	-14.38	-28.75	-43.13	-43.13
81	57.51	-86.26	43.13	0.00	-57.51	-14.38	-71.89	-28.75	86.26	-86.26
91	43.13	0.00	-43.13	14.38	-28.75	-14.38	86.26	-100.64	43.13	-57.51
101	0.00	-28.75	-115.02	-57.51	57.51	-201.28	-143.77	-115.02	143.77	-273.17
111	0.00	-43.13	143.77	-115.02	71.89	14.38	0.00	57.51	14.38	-100.64
121	0.00	-244.41	-244.41	-86.26	-172.53	-129.39	57.51	-143.77	186.90	-115.02
131	43.13	-71.89	14.38	57.51	-14.38	14.38	71.89	-129.39	-28.75	-71.89
141	-28.75	-14.38	14.38	-14.38	115.02	-100.64	-43.13	-14.38	-14.38	-28.75
151	14.38	-14.38	100.64	-129.39	0.00	-43.13	-43.13	57.51	100.64	-71.89
161	115.02	0.00	14.38	-71.89	-43.13	-71.89	-86.26	-115.02	43.13	-115.02
171	14.38	-28.75	43.13	-28.75	100.64	57.51	71.89	86.26	57.51	100.64
181	57.51	57.51	14.38	-14.38	-71.89	43.13	43.13	43.13	43.13	71.89
191	28.75	28.75	-100.64	158.15	57.51	71.89	201.28	43.13	201.28	86.26
201	-115.02	258.79	-43.13	71.89	14.38	71.89	14.38	-71.89	71.89	100.64
211	316.30	172.53	158.15	115.02	28.75	115.02	-71.89	301.92	86.26	215.66
221	14.38	-57.51	43.13	100.64	-86.26	258.79	129.39	172.53	115.02	115.02
231	129.39	86.26	-345.05	431.31	14.38	215.66	891.38	-445.69	215.66	-57.51
241	-172.53	186.90	-57.51	43.13	186.90	57.51	43.13	-100.64	-948.89	675.73
251	877.01	-301.92	143.77	186.90	115.02	57.51	-460.07	531.95	-43.13	287.54
261	129.39	186.90	230.03	86.26	-273.17	560.71	43.13	186.90	-57.51	1178.93
271	-474.45	115.02	-1049.53	819.50	-28.75	43.13	43.13	-1164.55	244.41	-560.71
281	-1164.55	-172.53	-1365.83	186.90	-273.17	-71.89	-86.26	-28.75	215.66	690.10
291	-345.05	1509.60	0.00	1667.75	1078.29	86.26	948.89	201.28	158.15	-963.27
301	-675.73	115.02	-1754.01	244.41	-345.05	-201.28	-1423.34	201.28	-1437.71	100.64
311	-1581.49	-1035.15	-460.07	-747.61	-1236.43	920.14	-517.58	-704.48	301.92	-172.53
321	373.81	-690.10	805.12	-28.75	2156.57	848.25	1408.96	1940.91	1150.17	1768.39
331	129.39	488.82	-1107.04	-920.14	-201.28	-805.12	-1150.17	-1308.32	-934.51	-1121.42
341	-1322.70	-1121.42	-158.15	-445.69	575.09	445.69	862.63	1092.66	1337.07	2501.62
351	57.51	1984.04	1121.42	1279.57	-100.64	416.94	-833.87	488.82	-71.89	57.51
361	-359.43	517.58	0.00	158.15	-43.13	-28.75	-1049.53	460.07	-531.95	158.15
371	-143.77	-1020.78	373.81	-690.10	-1394.58	-1365.83	-1452.09	-1279.57	-1265.19	-445.69
381	-1207.68	-258.79	-330.67	733.23	1495.22	603.84	1351.45	2386.60	1207.68	1696.50
391	1509.60	1552.73	761.99	1667.75	1135.79	1035.15	416.94	1236.43	14.38	-675.73
401	-1351.45	-287.54	-1178.93	-2573.51	-2271.59	-287.54	-2070.31	-1107.04	1308.32	-790.74
411	1207.68	57.51	1595.86	244.41	1754.01	891.38	-431.31	1969.67	-158.15	503.20
421	-43.13	-1207.68	-776.37	-1092.66	-1466.47	-445.69	$\boxed{-2717.28}$	115.02	-1682.13	172.53
431	-186.90	1035.15	1035.15	-445.69	1754.01	-100.64	618.22	1567.11	920.14	28.75
441	819.50	100.64	-718.86	546.33	-1207.68	-43.13	-1495.22	-115.02	-201.28	-488.82
451	86.26	143.77	445.69	258.79	1523.98	-244.41	-57.51	373.81	86.26	215.66
461	172.53	158.15	71.89	43.13	-1293.94	460.07	-517.58	-230.03	-431.31	-1250.81
471	373.81	-287.54	14.38	-1610.24	-718.86	-172.53	-1595.86	71.89	-287.54	1049.53
481	1150.17	460.07	115.02	1567.11	14.38	503.20	1610.24	-86.26	920.14	273.17
491	1351.45	-86.26	1380.21	-14.38	1207.68	-230.03	-1380.21	1006.40	-201.28	-948.89

4.8. EXPERIMENTAL DETERMINATION of NATURAL FREQUENCIES

On the basis of the prepared experimental program, a series of tests to evaluate the natural periods of vibration has been conducted using sinusoidal and impulse tests.

For definition of the fundamental mode of vibration of a fixed base model a displacement impulse is generated at the fifth floor level of the structural model and the fourier transform of the recorded time history response is used to identify the first translational frequency. The fourier transform is accomplished by applying the Fast Fourier Analyser amnufactured by the Hewlett Packard Company. The impulse test has yielded a first natural frequency of 3.38 Hz corresponding to a natural period of 0.30 sec (Fig. 4.7).

Applying sinusoidal input motion at different frequencies to the shaking table however, values which are ten percent higher are obtained for the natural period of vibration due to the rolling effect.

In the cases when vibration isolation systems are installed, for definition of the fundamental mode of vibration, a series of harmonic sinusoidal motion is simulated on the shaking table and the natural periods of vibration are identified for each case of base isolation with four and eight dashpots, respectively. The results of simulation of horizontal sinusoidal motion are shown in Fig. 4.8.

The defined frequency response curves show resonant peaks around f_1 = 0.87 Hz (T=1.15 sec) and f_2= 1.70 Hz (T=0.59 sec) for the case with four dashpots (D4-Case). Based on the calculations for a rigid body model, neglecting the elastic properties of the structural model the first frequency corres-

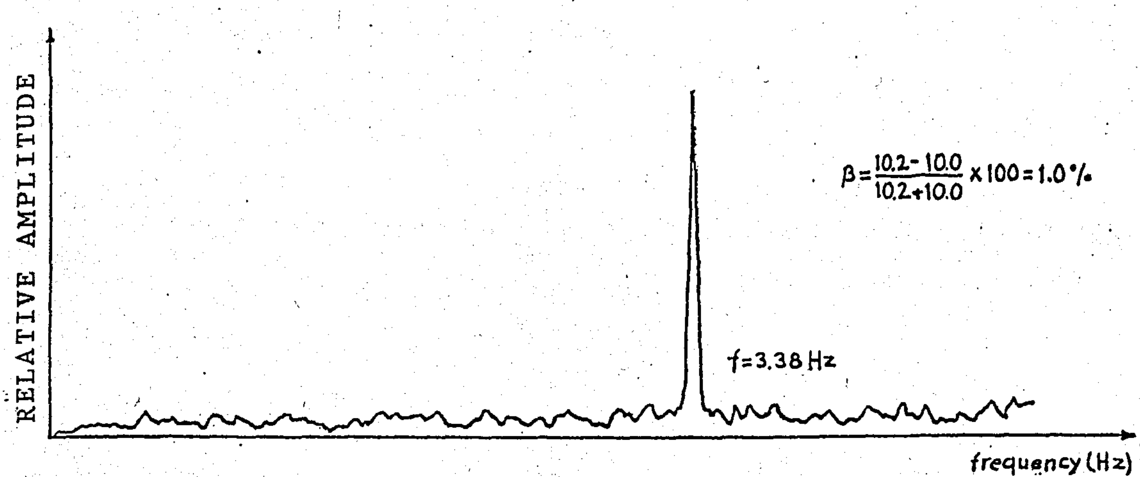


FIG. 4.7 - FOURIER AMPLITUDE SPECTRUM FOR FIXED BASE MODEL

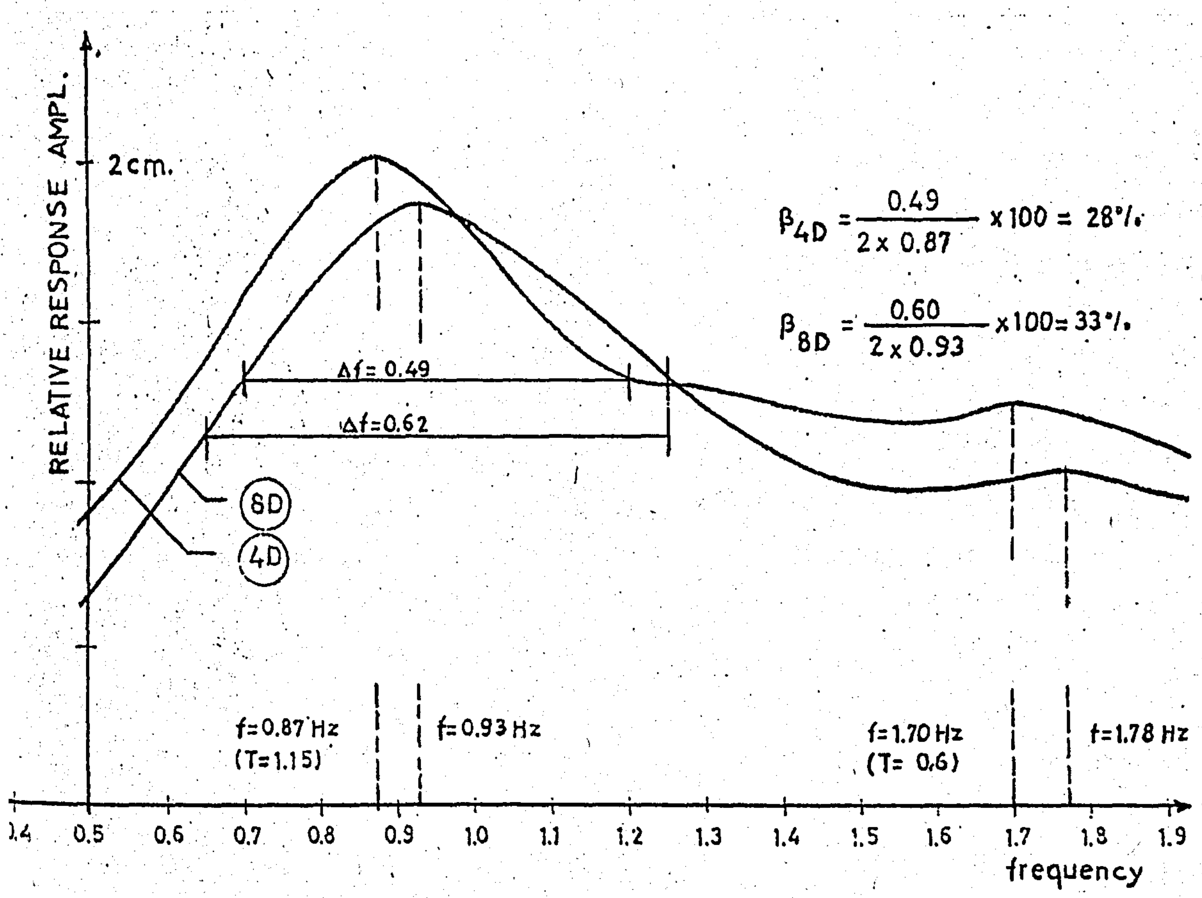


FIG. 4.8 - FREQUENCY RESPONSE CURVES FOR ISOLATED MODELS - HORIZONTAL DIRECTION

ponds to the lower rocking mode, while the second resonant peak will probably correspond to the higher rocking mode.

For the case when eight dashpots are included (D8-case) slightly higher frequencies (0.93 Hz and 1.78 Hz) have been obtained due to the increased energy absorbtion capacity of the system and the increased stiffness.

It is apparent that curves for both D4-case and D8-case are rather wide. This is due to the coupling of the two frequencies which are rather close and their response curves are not well separated. The response of the system between the frequencies of 0.90 and 1.70 Hz is a sum of responses of the two resonant curves corresponding to the uncoupled systems.

For the vertical direction, the frequency response curves look even more complicated (Fig. 4.9). Besides the reason mentioned above, in this case an effect of the base girder stiffness is also involved which causes the coupling of the vertical mode of vibration with the rocking mode. The stiffness of these girders is not of that kind so that the base of the frame model should be considered as a rigid one, and for all frequencies the same amplitude of vibration appears in all rows of columns. Also, due tp small eccentricity of the gravity load (because of the small differences in the masses of some blocks and their distribution, the gravity load could not be ideally distributed with respect to the girder center) multi-coupling in response of teh system takes place. However, in spite of all the above mentioned reasons, the resonant peaks could be distinguished in the range between f_1 = 1.55 Hz (T= 0.65 sec) and f_2= 2.90 Hz (T=0.34 sec) in the vertical direction. The first resonant peak corresponds to the first vertical mode of vibration of the system. In summary, the natural periods of vibration, as determined by means of sinusoidal

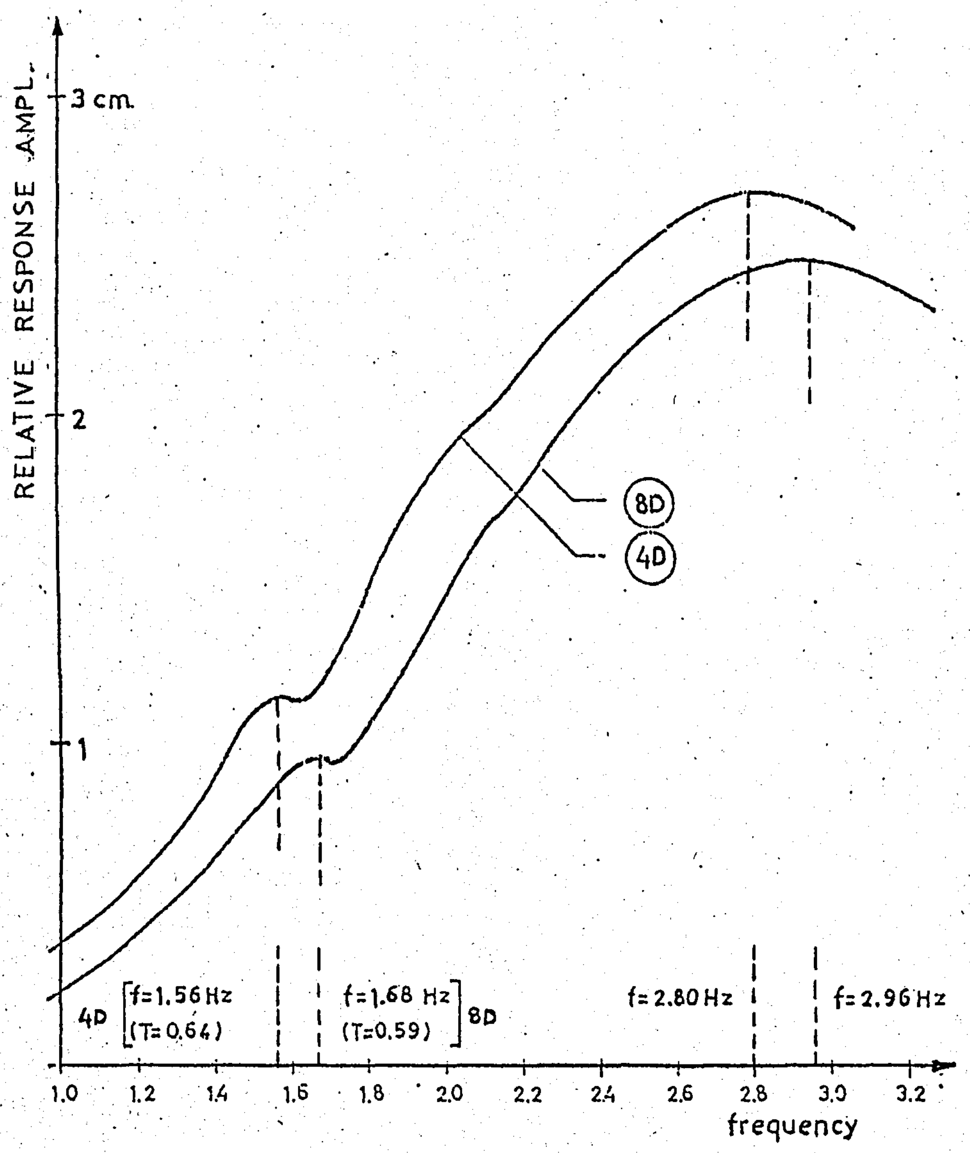


FIG. 4.9 - FREQUENCY RESPONSE CURVES FOR ISOLATED MODELS - VERTICAL DIRECTION

waves and impulse motions, as well as by analyses are shown in Table 4.6.

TABLE 4.6: NATURAL PERIODS of VIBRATION, sec

Base Condition	T/A	MODES 1	2	3	4
Fixed Base Case	T A	0.30 0.29	– 0.09	– 0.05	– 0.03
Springs, 4D-Case	T A	1.15 R 1.47 R	0.64 V 0.65 V	0.60 H 0.54 R	0.36 V 0.10 R
Springs, 8D-Case	T A	1.08 R 1.47 R	0.59 V 0.65 V	0.56 R 0.54 R	0.34 V 0.10 R
Rubber Base Case	T A	1.19 1.21 H	– 0.19 R	– 0.10 V	– 0.08 R

T= *Tests*, A= *Analysis*, H= *Horizontal*, V= *Vertical*, R= *Rocking*.

4.9. PEAK RESPONSES at FLOOR LEVELS

The experimental data of all thirty channels for 73 different runs are stored on magnetic tapes. In this study, only a minimum number of the most essential data important for understanding of the structural model behaviour is presented.

For each of the runs, extreme values of the records taken by 30 channels have been summarized in tabular form. Some of these tables have been presented in Tables 4.7 to 4.12, for illustrative purposes. Based on the analyses on these data it was found that the response data in all the tests recorded on channels 10 and 26 are not correct and they should not be used in evaluations.

From the collected experimental data, the mechanism of dynamic behaviour of base isolated models may be defined fairly precisely, and at the same time the reduction of the level of forces, strains and stresses in the representative points and sections of the model as compared to the behaviour of fixed base model may be evaluated. These comparative values are very important and provide an adequate explanation if a comparison is made between the results from single component simulations for selected types of earthquakes. Figs.4.10 to 4.17 show the maximum values of the measured real accelerations at each floor level for different base conditions and different types of earthquakes. Most of these tests for the same earthquake have been carried out with the same SPAN for the three base conditions. However, some modifications have been introduced in order to test the linear behaviour of the model or the need to simulate higher accelerations on the shaking table.

It is evident from these results that the base isolation systems in the form of springs and dashpots act as eenrgy absorbers and significantly decrease the acceleration values along the height of the model. The reduction level of maximum accelerations under base isolation conditions is different and for some levels it is as large as 20 times. It is also interesting to note that in the case of base isolation with four dashpots, the acceleration reduction level is considerably higher compared to the base isolation with eight dashpots. This is a consequence of the increased stiffness of the system due to the additional viscodampers.

Comparing the measured peak displacements at each floor level for different base conditions as shown in Figs. 4.10 to 4.17, some conclusions could be reached. In most of the figures the displacement amplitudes for the fixed base model are smaller than those of the base isolation model, excluding a few cases corresponding to the Petrovac earthquake time scaled. As different from the accelerations, the displacement amplitudes of the base isolated model with eight dashpots are smaller than those recorded under the same conditions with four dashpots.

Peak floor response values of acceleration and displacements, corresponding to El Centro real time (EN NS 20) and Petrovac real time (PN NS 20) earthquakes for different base conditions are illustrated in Figs. 4.18 and 4.19. It is seen that although the accelerations of the rubber base isolation are within the acceptable range, the horizontal displacements become excessively large even at relatively moderate earthquake input intensities.

4.10. TIME HISTORY RESPONSES

In order to obtain a better insight about the reduction of acceleration responses, the complete time histories of accelerations from base to the fifth floor level have been plotted for different base conditions for the Petrovac earthquake of time scaled with SPAN 200.

Figure 4.20 shows the acceleration response of the fixed base model where it is shown that there is a significant amplification. The next two figures (Figs. 4.21 and 4.22) show the behaviour of the base isolated model with four dashpots and eight dashpots, respectively. From inspection of these acceleration time histories, considerable reduction in acceleration response level is evident.

Similar to the acceleration time histories for the three base conditions of the model, a representative series of displacement responses have been shown for the four floors of the model, for the fixed base model and the two cases of base isolation (Figs. 4.23 to 4.25). As expected, the displacements are very small for the fixed-base condition. In the case of spring-isolated systems however, the displacements are relatively larger, but the increased rigid body displacements may be reduced to tolerable limits by means of an ample supply of viscous dampers.

It is very important to note that in the case of spring supported system the horizontal displacements grow from base to top of the model similar to the fixed base model. At the spring level the horizontal deflections are negligible. However, horizontal displacements become significantly larger along the character of the height of the structure due to coupling of rocking and horizontal motions. Contrary to the considerable amount of relative inter story displacements in the fixed base case, the structural model with base isolation behaves almost like a rigid body causing almost no interstorey displacements.

Figure 4.26 shows the time histories of the spring displacements in the vertical direction for test 24 (El Centro NS component, time scaled, 4D-case, SPAN 400 - D4 EB NS 40) and tests (El Centro NS - component, time scaled, 8D-Case, SPAN 400- D8 EB NS 40). In these figures the responses for the vertical displacements of the springs from channels 17 and 18 show an evident assymmetry, identifying a rocking motion. This points out the importance of the base rotation. Namely, if the distance of gages of channels 17 and 18 is considered to be roughly 4.00 m, then the vertical base motion of 2 cm recorded at the spring will produce a horizontal displacement amplitude at the fifth floor of approximately 5.00 cm as already illustrated in Fig. 4.26. In this test, a total displacement amplitude of 6.5 cm has been recorded at the fifth floor (Fig. 4.15), which means that the horizontal motion alone excluding the rocking effect produces a deformation of 1.5 cm. This shows the importance of the rocking effect in base isolated systems. The same conclusion applies to the test No. 51, except a reduction in the displacement amplitudes is apparent because of additional visco-dampers.

The stress measurements of the structural model at the recording points also vary depending upon the base support conditions. Figs. 4.27 and 4.28 show the time histories of axial stresses and stresses due to moments of an end column at the first floor of the model respectively. The axial forces and stresses in the end columns (Ch. No. 27) show a significant reduction (about eight times) compared to fixed base model, while the stresses due to moment recorded at the bottom of the end columns show a reduction even higher than ten times. The stress state of a structure should be considered as an indicator of its stability when it is subjected to strong earthquakes. Thus, in test No. 14, the stresses due to moments approach the yield point of the steel, while in test No. 39 for a model having four dashpots, the stresses in the same section for the same time history have been reduced to more than 14 times (155.84 N/mm²: 10.81 N/mm²).

4.11. DISCUSSION of TEST RESULTS

It has been shown that the acceleration of the structural model, which has been excited by strong ground motion, are effectively reduced if the model is isolated by helical springs and viscodampers. Accelerations are smaller than the input acceleration on the shaking table and no amplification of acceleration takes place at various levels of the model.

It is evident that in the case of base isolation which consists of four dashpots, the acceleration reduction level is considerably higher (almost two times) compared to base isolation with eight dashpots. This is due to the increase in the rigidity at the base level of the system caused by increasing the energy absorption capacity (by adding more dashpots). It is also evident that by increasing the excitation level the reduction level would decrease slightly, which means that the reduction is more pronounced at lower acceleration values of the shaking table. This nonlinearity is not a characteristics of the higher excitation levels in which the reduction level is more or less independent of the excitation level.

Analysing the envelope shape of the peak accelerations as shown in Figs. 4.10 to 4.17, several regularities are ob - served. First, the acceleration level continually decreases going from the base to the third floor and then starts to increase again. This shows that the deformational mechanism of the test model in the case of base isolation is complicated. It could be assumed that the type of deformation is controlled by the rocking, however, for higher levels there is an influence of the horizontal inertia forces.

Second, in all cases when the same type of earthquake motion is repeated with the same SPAN values and for diffe-rent base conditions, the pick value of acceleration of the shaking table is almost the same. This shows that the obtained results with respect to the simulation conditions are rather consistent.

The tests have, also, shown that the reduction of the model accelerations, in the cases of horizontal motion simulation, is considerably larger, compared to the reduction at vertical excitation of the model. However, in the case of frame systems, as well as small-span systems, the vertical earthquake component does not cause any significant stresses in the structure.

All the above advantages in the form of the reduction of accelerations are off-set by the problem of larger horizontal and vertical displacements which result from the rocking effect at the base of the model. Although the scale of these displacements is in the order of several centimeters, they can be controlled by the special system which has to reduce the level of the rocking effect at the base level. From the tests conducted, it is observed that with the increase in energy absorption capacity the displacement amplitude level decreases considerably. The additional viscodampers reduce the effects of rocking, resulting in the decrease of horizontal displacements.

Due to the position of the rocking center near to the spring level, and the governing motion being in the rocking mode, the horizontal displacement of the springs are negligible. In none of the tests conducted the vertical displacements were significant. Even the maximum vertical displacements ranging from 1 cm - 3 cm, were less than one quarter of the overall static deflections of teh springs.

For simultaneous horizontal and vertical excitations, the results were practically similar to those corresponding to the excitations. Based on the series of bi-axial tests, it may be concluded that there is no considerable interaction effect of the model responses to excitations in the horizontal and vertical directions. The results of independent simulations in each direction are easily available and sufficiently accurate to represent the results of biaxial excitation.

TABLE 4.7 — SHAKING TABLE TEST RESULTS — ENNS 20

TEST NUMBER : 1

IDENTIFICATION CODE : FBENNS20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-19.79	22.41
2	MM/S**2	-934.51	1365.83
3	MM	-0.03	1.17
4	MM/S**2	-71.89	105.43
5	MM	-5.54	5.98
6	MM	-4.28	5.27
7	MM	-3.46	4.26
8	MM	-2.96	3.46
9	MM	-2.37	3.06
10	MM	0.00	0.00
11	MM/S**2	-1810.45	1783.83
12	MM/S**2	-1293.94	1198.09
13	MM/S**2	-1413.75	1150.17
14	MM/S**2	-1222.06	1030.36
15	MM/S**2	-910.55	934.51
16	MM/S**2	0.00	0.00
17	MM	0.00	0.00
18	MM	0.00	0.00
19	MM/S**2	0.00	0.00
20	MM/S**2	0.00	0.00
21	MM/S**2	-127.00	100.64
22	MM/S**2	-91.06	136.58
23	MM/S**2	-11861.14	11381.90
24	MM/S**2	-11142.28	11980.95
25	MM/S**2	-694.89	670.93
26	MM/S**2	0.00	0.00
27	N/MM**2	-3.76	3.78
28	N/MM**2	-3.06	3.76
29	N/MM**2	-15.28	12.93
30	N/MM**2	-16.92	13.87

TEST NUMBER : 18

IDENTIFICATION CODE : D4ENNS20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-19.85	22.35
2	MM/S**2	-877.01	1365.83
3	MM	-0.06	0.12
4	MM/S**2	-57.51	76.08
5	MM	-15.39	17.25
6	MM	-12.80	13.65
7	MM	-9.98	10.72
8	MM	-8.23	8.39
9	MM	-6.17	6.45
10	MM	0.00	0.00
11	MM/S**2	-143.77	143.77
12	MM/S**2	-143.77	143.77
13	MM/S**2	-143.77	167.73
14	MM/S**2	-119.81	143.77
15	MM/S**2	-143.77	143.77
16	MM/S**2	-119.81	143.77
17	MM	-0.22	0.22
18	MM	-0.22	0.26
19	MM/S**2	-167.73	143.77
20	MM/S**2	-167.73	143.77
21	MM/S**2	-0.58	0.58
22	MM/S**2	-0.10	0.10
23	MM/S**2	-1437.71	2156.57
24	MM/S**2	-5391.43	5870.66
25	MM/S**2	0.00	0.00
26	MM/S**2	0.00	0.00
27	N/MM**2	-0.39	0.78
28	N/MM**2	-0.39	0.70
29	N/MM**2	-1.57	1.96
30	N/MM**2	-1.96	2.35

TEST NUMBER : 44

IDENTIFICATION CODE : D8ENNS20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-19.97	22.53
2	MM/S**2	-905.76	1279.57
3	MM	-0.06	1.14
4	MM/S**2	-76.68	86.26
5	MM	-12.89	11.21
6	MM	-10.32	9.42
7	MM	-7.91	7.44
8	MM	-7.08	5.90
9	MM	-5.53	4.86
10	MM	0.00	0.01
11	MM/S**2	-532.49	851.96
12	MM/S**2	-431.31	479.24
13	MM/S**2	-383.39	479.24
14	MM/S**2	-383.39	455.28
15	MM/S**2	-359.43	575.09
16	MM/S**2	-455.28	718.86
17	MM	-2.92	2.88
18	MM	-2.72	2.84
19	MM/S**2	-355.47	215.06
20	MM/S**2	-287.54	383.39
21	MM/S**2	-93.45	50.32
22	MM/S**2	-14.38	67.09
23	MM/S**2	-3714.09	4073.52
24	MM/S**2	-3714.09	4193.33
25	MM/S**2	-335.47	431.31
26	MM/S**2	0.00	0.00
27	N/MM**2	-0.94	2.12
28	N/MM**2	-1.41	1.41
29	N/MM**2	-4.94	5.64
30	N/MM**2	-5.17	6.58

TABLE 4.8 — SHAKING TABLE TEST RESULTS — ENNS 40

TEST NUMBER : 2

IDENTIFICATION CODE : FBENNS40.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-39.20	45.00
2	MM/S**2	-1955.29	2573.51
3	MM	-0.03	4.63
4	MM/S**2	-167.73	177.32
5	MM	-10.73	13.07
6	MM	-9.50	10.38
7	MM	-7.62	8.14
8	MM	-6.44	6.96
9	MM	-5.32	6.20
10	MM	0.00	0.00
11	MM/S**2	-3354.67	3381.29
12	MM/S**2	-2516.00	2563.92
13	MM/S**2	-2444.11	2132.61
14	MM/S**2	-2084.69	1845.07
15	MM/S**2	-1725.26	1797.14
16	MM/S**2	0.00	0.00
17	MM	0.00	0.00
18	MM	0.00	0.00
19	MM/S**2	0.00	0.00
20	MM/S**2	0.00	0.00
21	MM/S**2	-234.83	218.05
22	MM/S**2	-184.51	242.02
23	MM/S**2	-20487.42	19409.13
24	MM/S**2	-18570.47	18091.23
25	MM/S**2	-1030.36	910.55
26	MM/S**2	0.00	0.00
27	N/MM**2	-7.52	7.52
28	N/MM**2	-6.82	6.82
29	N/MM**2	-30.09	29.85
30	N/MM**2	-33.38	31.26

TEST NUMBER : 19

IDENTIFICATION CODE : D4ENNS40.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-39.20	44.82
2	MM/S**2	-1754.01	2559.13
3	MM	-0.03	4.59
4	MM/S**2	-215.66	189.90
5	MM	-30.55	35.93
6	M	-24.37	29.08
7	MM	-19.72	22.73
8	MM	-15.82	18.14
9	MM	-12.14	13.28
10	MM	0.00	0.00
11	MM/S**2	-71.89	95.85
12	MM/S**2	-646.97	599.05
13	MM/S**2	-670.93	623.01
14	MM/S**2	-670.93	718.86
15	MM/S**2	-886.59	1006.40
16	MM/S**2	-1677.33	1365.83
17	MM	-8.69	10.05
18	MM	-9.48	8.36
19	MM/S**2	-862.63	790.74
20	MM/S**2	-742.82	1006.40
21	MM/S**2	-0.10	0.19
22	MM/S**2	0.00	0.00
23	MM/S**2	-3354.67	4432.95
24	MM/S**2	-8506.47	8626.28
25	MM/S**2	0.00	0.00
26	MM/S**2	0.00	0.00
27	N/MM**2	-1.18	1.57
28	N/MM**2	-1.18	1.18
29	N/MM**2	-3.53	3.53
30	N/MM**2	-3.92	3.92

TEST NUMBER : 45

IDENTIFICATION CODE : D8ENNS40.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-39.63	44.82
2	MM/S**2	-1682.13	2516.00
3	MM	-0.06	0.09
4	MM/S**2	-129.39	129.39
5	MM	-22.78	21.46
6	MM	-18.52	17.60
7	MM	-15.28	13.45
8	MM	-12.99	11.43
9	MM	-10.52	8.51
10	MM	0.00	0.00
11	MM/S**2	-1144.85	1757.21
12	MM/S**2	-866.59	1174.13
13	MM/S**2	-766.78	1030.36
14	MM/S**2	-886.59	1030.36
15	MM/S**2	-742.82	1269.98
16	MM/S**2	-886.59	1461.68
17	MM	-5.14	5.38
18	MM	-4.92	5.06
19	MM/S**2	-575.09	431.31
20	MM/S**2	-599.05	718.86
21	MM/S**2	-45.53	141.38
22	MM/S**2	-93.45	64.70
23	MM/S**2	-8506.47	6506.47
24	MM/S**2	-6230.09	5870.66
25	MM/S**2	-599.05	838.67
26	MM/S**2	0.00	0.00
27	N/MM**2	-2.35	4.00
28	N/MM**2	-3.29	2.59
29	N/MM**2	-10.11	12.69
30	N/MM**2	-11.05	14.34

TABLE 4.9 — SHAKING TABLE TEST RESULTS — ENNS 60

TEST NUMBER : 3

IDENTIFICATION CODE : FBENNS60.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-58.62	69.06
2	MM/S**2	-2544.75	3651.79
3	MM	-0.09	10.90
4	MM/S**2	-359.43	268.37
5	MM	-17.68	20.38
6	MM	-15.23	16.17
7	MM	-11.93	12.75
8	MM	-10.40	11.08
9	MM	-8.48	9.57
10	MM	0.00	0.00
11	MM/S**2	-5005.37	5191.74
12	MM/S**2	-3929.75	4001.64
13	MM/S**2	-3809.94	3474.47
14	MM/S**2	-3258.82	2659.77
15	MM/S**2	-2587.88	2683.73
16	MM/S**2	0.00	0.00
17	MM	0.00	0.00
18	MM	0.00	0.00
19	MM/S**2	0.00	0.00
20	MM/S**2	0.00	0.00
21	MM/S**2	-246.81	369.01
22	MM/S**2	-349.84	254.00
23	MM/S**2	-26957.13	23243.04
24	MM/S**2	-24800.56	24081.70
25	MM/S**2	-1509.60	1293.94
26	MM/S**2	0.00	0.00
27	N/MM**2	-11.52	11.52
28	N/MM**2	-11.28	10.85
29	N/MM**2	-46.54	46.78
30	N/MM**2	-52.42	49.83

TEST NUMBER : 20

IDENTIFICATION CODE : D4ENNS60.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-59.42	67.11
2	MM/S**2	-2559.13	3522.40
3	MM	-0.09	0.09
4	MM/S**2	-148.56	177.32
5	MM	-47.05	53.94
6	MM	-37.81	43.90
7	MM	-30.23	34.92
8	MM	-24.18	28.29
9	MM	-18.34	20.94
10	MM	0.00	0.00
11	MM/S**2	-95.85	119.81
12	MM/S**2	-958.48	934.51
13	MM/S**2	-982.44	910.55
14	MM/S**2	-1006.40	1150.17
15	MM/S**2	-1389.79	1413.75
16	MM/S**2	-2252.42	2468.07
17	MM	-13.40	15.16
18	MM	-15.03	12.88
19	MM/S**2	-1246.02	1078.29
20	MM/S**2	-1293.94	1485.64
21	MM/S**2	0.00	0.19
22	MM/S**2	0.00	0.00
23	MM/S**2	-6469.71	7068.76
24	MM/S**2	-11861.14	12699.80
25	MM/S**2	0.00	0.00
26	MM/S**2	0.00	0.00
27	N/MM**2	-1.57	1.96
28	N/MM**2	-1.57	1.57
29	N/MM**2	-5.09	5.48
30	N/MM**2	-5.88	5.48

TEST NUMBER : 46

IDENTIFICATION CODE : D8ENNS60.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-59.72	67.35
2	MM/S**2	-2487.24	3760.81
3	MM	-0.06	0.09
4	MM/S**2	-182.11	201.28
5	MM	-35.41	31.71
6	MM	-28.84	26.11
7	MM	-23.35	21.04
8	MM	-20.00	17.22
9	MM	-16.20	13.35
10	MM	0.00	0.00
11	MM/S**2	-1730.58	2662.43
12	MM/S**2	-1293.94	1773.16
13	MM/S**2	-1174.13	1605.45
14	MM/S**2	-1174.13	1509.60
15	MM/S**2	-1222.06	1773.16
16	MM/S**2	-1413.75	2132.61
17	MM	-7.99	7.99
18	MM	-7.52	7.68
19	MM/S**2	-862.63	694.89
20	MM/S**2	-1006.40	1150.17
21	MM/S**2	-67.09	148.56
22	MM/S**2	-107.83	103.04
23	MM/S**2	-14017.71	14137.52
24	MM/S**2	-8865.90	8626.28
25	MM/S**2	-958.48	1174.13
26	MM/S**2	-0.00	0.00
27	N/MM**2	-3.76	5.64
28	N/MM**2	-5.17	4.23
29	N/MM**2	-14.57	18.57
30	N/MM**2	-16.69	20.92

TABLE 4.10 — SHAKING TABLE TEST RESULTS — EBNS 20

TEST NUMBER : 5

IDENTIFICATION CODE : FBEBNS20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-19.79	22.41
2	MM/S**2	-1624.62	2041.55
3	MM	-0.15	1.14
4	MM/S**2	-206.07	201.28
5	MM	-23.43	20.63
6	MM	-19.58	17.45
7	MM	-15.41	13.58
8	MM	-11.07	9.50
9	MM	-6.59	4.95
10	MM	0.00	0.00
11	MM/S**2	-6283.34	7587.93
12	MM/S**2	-4936.15	5750.85
13	MM/S**2	-4265.22	4672.57
14	MM/S**2	-3043.16	3067.12
15	MM/S**2	-2156.57	2587.88
16	MM/S**2	0.00	0.00
17	MM	0.00	0.00
18	MM	0.00	0.00
19	MM/S**2	0.00	0.00
20	MM/S**2	0.00	0.00
21	MM/S**2	-345.05	428.92
22	MM/S**2	-335.47	337.86
23	MM/S**2	-31989.13	34744.75
24	MM/S**2	-27556.18	25519.42
25	MM/S**2	-1341.87	1317.90
26	MM/S**2	0.00	0.00
27	N/MM**2	-14.57	15.75
28	N/MM**2	-16.69	13.40
29	N/MM**2	-51.71	60.65
30	N/MM**2	-58.76	64.88

TEST NUMBER : 23

IDENTIFICATION CODE : D4EBNS20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-19.91	22.53
2	MM/S**2	-1840.27	2027.18
3	MM	-0.06	1.11
4	MM/S**2	-158.15	138.98
5	MM	-30.69	34.11
6	MM	-24.78	27.92
7	MM	-19.76	21.28
8	MM	-15.94	16.73
9	MM	-12.39	11.72
10	MM	-0.22	0.87
11	MM/S**2	-1198.09	1464.34
12	MM/S**2	-766.78	768.78
13	MM/S**2	-814.70	766.78
14	MM/S**2	-786.78	1174.13
15	MM/S**2	-1385.85	1605.45
16	MM/S**2	-2348.27	2492.04
17	MM	-9.58	10.09
18	MM	-10.00	9.04
19	MM/S**2	-1128.21	1006.40
20	MM/S**2	-1293.94	1222.06
21	MM/S**2	-64.70	67.09
22	MM/S**2	-81.47	79.07
23	MM/S**2	-4313.14	4792.38
24	MM/S**2	-10303.81	9824.38
25	MM/S**2	-886.89	1293.94
26	MM/S**2	0.00	0.00
27	N/MM**2	-2.35	2.82
28	N/MM**2	-2.82	2.59
29	N/MM**2	-8.46	8.46
30	N/MM**2	-9.40	9.64

TEST NUMBER : 50

IDENTIFICATION CODE : D8EBNS20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-19.97	22.53
2	MM/S**2	-1782.77	2214.08
3	MM	-0.06	0.12
4	MM/S**2	-143.77	148.56
5	MM	-19.24	19.06
6	MM	-15.76	15.82
7	MM	-12.48	12.48
8	MM	-10.27	10.13
9	MM	-8.54	7.61
10	MM	0.00	0.00
11	MM/S**2	-1304.59	1783.83
12	MM/S**2	-838.67	910.55
13	MM/S**2	-866.59	766.78
14	MM/S**2	-910.55	982.44
15	MM/S**2	-838.67	1293.94
16	MM/S**2	-1198.09	1916.95
17	MM	-4.64	4.79
18	MM	-4.76	4.48
19	MM/S**2	-846.97	814.70
20	MM/S**2	-1006.40	718.86
21	MM/S**2	-74.28	153.36
22	MM/S**2	-55.11	115.02
23	MM/S**2	-10423.42	9824.38
24	MM/S**2	-8386.66	8027.23
25	MM/S**2	-886.59	934.51
26	MM/S**2	-0.00	0.00
27	N/MM**2	-2.35	2.82
28	N/MM**2	-2.59	2.59
29	N/MM**2	-9.87	8.70
30	N/MM**2	-11.28	10.11

TABLE 4.11 — SHAKING TABLE TEST RESULTS — ENVK 20

TEST NUMBER : 7

IDENTIFICATION CODE : FBENVK20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-0.12	0.18
2	MM/S**2	-28.75	57.51
3	MM	-6.68	10.58
4	MM/S**2	-378.60	445.69
5	MM	-0.62	1.11
6	MM	-0.38	0.59
7	MM	-0.18	0.57
8	MM	-0.38	0.32
9	MM	-0.32	0.32
10	MM	0.00	0.00
11	MM/S**2	-79.87	106.50
12	MM/S**2	-71.89	95.85
13	MM/S**2	-143.77	191.70
14	MM/S**2	-191.70	167.73
15	MM/S**2	-23.96	47.92
16	MM/S**2	0.00	0.00
17	MM	0.00	0.00
18	MM	0.00	0.00
19	MM/S**2	0.00	0.00
20	MM/S**2	0.00	0.00
21	MM/S**2	-158.15	141.38
22	MM/S**2	-91.06	165.34
23	MM/S**2	-838.67	1078.29
24	MM/S**2	-2995.24	2875.43
25	MM/S**2	-1246.02	1246.02
26	MM/S**2	0.00	0.00
27	N/MM**2	-0.47	0.94
28	N/MM**2	-0.47	0.47
29	N/MM**2	-0.94	0.94
30	N/MM**2	-0.71	0.94

TEST NUMBER : 26

IDENTIFICATION CODE : D4ENVK20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-0.12	0.24
2	MM/S**2	-43.13	57.51
3	MM	-6.65	10.55
4	MM/S**2	-488.82	503.20
5	MM	-0.18	0.95
6	MM	-0.24	0.32
7	MM	-0.44	0.38
8	MM	-0.24	0.38
9	MM	-0.52	0.58
10	MM	-0.01	0.02
11	MM/S**2	0.00	48.41
12	MM/S**2	-71.89	23.96
13	MM/S**2	-71.89	71.89
14	MM/S**2	-119.81	95.85
15	MM/S**2	-23.96	71.89
16	MM/S**2	-47.92	47.92
17	MM	-0.90	0.70
18	MM	-0.84	0.56
19	MM/S**2	-119.81	215.66
20	MM/S**2	-143.77	215.66
21	MM/S**2	0.00	119.81
22	MM/S**2	-71.89	71.89
23	MM/S**2	-119.81	359.43
24	MM/S**2	-2516.00	2635.81
25	MM/S**2	-167.73	359.43
26	MM/S**2	0.00	0.00
27	N/MM**2	0.00	0.71
28	N/MM**2	-0.24	0.24
29	N/MM**2	-0.24	0.47
30	N/MM**2	-0.24	0.17

TEST NUMBER : 54

IDENTIFICATION CODE : D6ENVK20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-0.12	1.28
2	MM/S**2	-43.13	71.89
3	MM	-5.13	8.15
4	MM/S**2	-335.47	263.56
5	MM	-0.61	0.99
6	MM	-0.57	0.76
7	MM	-0.59	0.52
8	MM	-0.60	0.73
9	MM	-0.73	0.38
10	MM	0.00	0.01
11	MM/S**2	-26.62	79.87
12	MM/S**2	-47.92	71.89
13	MM/S**2	-71.89	95.85
14	MM/S**2	-95.85	167.73
15	MM/S**2	-23.96	71.89
16	MM/S**2	-23.96	71.89
17	MM	-0.51	0.35
18	MM	-0.40	0.40
19	MM/S**2	-143.77	167.73
20	MM/S**2	-95.85	215.66
21	MM/S**2	-45.53	91.06
22	MM/S**2	-43.13	52.72
23	MM/S**2	-359.43	599.05
24	MM/S**2	-2396.19	2755.62
25	MM/S**2	-287.54	359.43
26	MM/S**2	0.00	0.00
27	N/MM**2	-0.24	0.47
28	N/MM**2	0.00	0.47
29	N/MM**2	-0.71	0.94
30	N/MM**2	-0.47	0.71

TABLE 4.12 — SHAKING TABLE TEST RESULTS — EN 4020

TEST NUMBER : 31

IDENTIFICATION CODE : D4EN4020.INV

CHA-NNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-39.51	44.88
2	MM/S**2	-4154.99	2990.44
3	MM	-4.92	10.70
4	MM/S**2	-309.91	450.48
5	MM	-32.46	39.42
6	MM	-20.24	31.91
7	MM	-20.48	25.40
8	MM	-16.40	20.52
9	MM	-12.31	15.14
10	MM	-4.13	4.06
11	MM/S**2	-1011.72	1198.09
12	MM/S**2	-646.97	694.89
13	MM/S**2	-742.82	646.97
14	MM/S**2	-694.89	910.55
15	MM/S**2	-1006.40	1102.25
16	MM/S**2	-1821.10	1797.14
17	MM	-8.77	10.95
18	MM	-10.80	9.16
19	MM/S**2	-1030.36	766.78
20	MM/S**2	-766.78	1102.25
21	MM/S**2	-107.83	64.70
22	MM/S**2	-50.32	105.43
23	MM/S**2	-4552.76	4552.76
24	MM/S**2	-8626.28	8865.90
25	MM/S**2	-646.97	958.48
26	MM/S**2	-0.01	0.01
27	N/MM**2	-1.88	2.35
28	N/MM**2	-2.12	2.12
29	N/MM**2	-8.23	6.82
30	N/MM**2	-9.17	7.76

TEST NUMBER : 58

IDENTIFICATION CODE : D8EN4020.INV

CHA-NNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-39.57	44.88
2	MM/S**2	-3694.92	3033.58
3	MM	-4.72	10.73
4	MM/S**2	-623.01	335.47
5	MM	-21.67	31.73
6	MM	-18.71	24.98
7	MM	-17.09	18.09
8	MM	-10.60	18.87
9	MM	-7.84	14.76
10	MM	-3.76	0.74
11	MM/S**2	-1624.08	1198.09
12	MM/S**2	-1030.38	886.59
13	MM/S**2	-1437.71	263.58
14	MM/S**2	-1485.64	287.64
15	MM/S**2	-1150.17	1030.36
16	MM/S**2	-1797.14	886.59
17	MM	9.00	21.70
18	MM	-2.88	10.80
19	MM/S**2	1198.09	2659.77
20	MM/S**2	1389.79	2851.47
21	MM/S**2	-251.60	-56.11
22	MM/S**2	-79.07	86.26
23	MM/S**2	-7787.62	6948.95
24	MM/S**2	-6948.95	5870.66
25	MM/S**2	3091.03	4792.38
26	MM/S**2	0.33	0.37
27	N/MM**2	258.60	291.47
28	N/MM**2	122.00	127.40
29	N/MM**2	-69.34	-49.60
30	N/MM**2	40.20	62.29

TABLE 4.13 — SHAKING TABLE TEST RESULTS — EB 3015

TEST NUMBER : 33

IDENTIFICATION CODE : D4EB3015.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-29.62	33.52
2	MM/S**2	-2760.41	4111.86
3	MM	-3.55	7.62
4	MM/S**2	-953.68	1471.26
5	MM	-48.96	55.36
6	MM	-39.58	44.80
7	MM	-31.47	35.10
8	MM	-25.65	27.29
9	MM	-19.44	19.26
10	MM	-8.07	6.68
11	MM/S**2	-1863.70	2050.07
12	MM/S**2	-1174.13	1150.17
13	MM/S**2	-1222.06	1365.83
14	MM/S**2	-1461.68	1485.64
15	MM/S**2	-2060.72	2180.53
16	MM/S**2	-3474.47	3354.87
17	MM	-14.92	16.01
18	MM	-17.11	14.71
19	MM/S**2	-1701.29	1389.79
20	MM/S**2	-1916.95	1869.03
21	MM/S**2	-67.09	95.85
22	MM/S**2	-136.58	86.26
23	MM/S**2	-11022.47	11381.90
24	MM/S**2	-18690.28	16413.90
25	MM/S**2	-1461.68	1749.22
26	MM/S**2	-0.01	0.01
27	N/MM**2	-3.53	4.00
28	N/MM**2	-4.00	3.76
29	N/MM**2	-11.99	13.87
30	N/MM**2	-13.63	15.04

TEST NUMBER : 61

IDENTIFICATION CODE : D8EB3015.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-29.74	33.46
2	MM/S**2	-2961.69	4298.76
3	MM	-3.40	7.09
4	MM/S**2	-790.74	1140.59
5	MM	-27.92	29.82
6	MM	-23.12	24.69
7	MM	-18.89	19.28
8	MM	-15.48	15.54
9	MM	-12.44	11.54
10	MM	-3.13	2.96
11	MM/S**2	-1970.20	2555.94
12	MM/S**2	-1246.02	1198.09
13	MM/S**2	-1293.94	1222.06
14	MM/S**2	-1389.79	1533.56
15	MM/S**2	-1605.45	1821.10
16	MM/S**2	-2252.42	2444.11
17	MM	-6.93	7.32
18	MM	-7.84	6.64
19	MM/S**2	-1078.29	1054.32
20	MM/S**2	-1293.94	1078.29
21	MM/S**2	-88.66	153.36
22	MM/S**2	-122.21	134.19
23	MM/S**2	-21206.28	21326.09
24	MM/S**2	-13658.28	11381.90
25	MM/S**2	0.00	0.00
26	MM/S**2	-0.02	0.02
27	N/MM**2	-3.29	4.23
28	N/MM**2	-3.29	4.00
29	N/MM**2	-15.28	13.87
30	N/MM**2	-17.16	14.57

TABLE 4.14 — SHAKING TABLE TEST RESULTS — PNNS 20

TEST NUMBER : 11

IDENTIFICATION CODE : FBPNNS20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-21.13	15.39
2	MM/S**2	-1308.32	1408.96
3	MM	-0.06	0.94
4	MM/S**2	-124.60	105.43
5	MM	-12.59	9.94
6	MM	-10.45	8.44
7	MM	-8.35	6.65
8	MM	-6.31	4.78
9	MM	-4.36	3.26
10	MM	0.00	0.00
11	MM/S**2	-3248.17	3354.67
12	MM/S**2	-2468.08	2611.85
13	MM/S**2	-2180.53	2492.04
14	MM/S**2	-1845.07	2036.76
15	MM/S**2	-1413.75	1533.56
16	MM/S**2	0.00	0.00
17	MM	0.00	0.00
18	MM	0.00	0.00
19	MM/S**2	0.00	0.00
20	MM/S**2	0.00	0.00
21	MM/S**2	-179.71	196.49
22	MM/S**2	-170.13	210.86
23	MM/S**2	-10663.04	12460.18
24	MM/S**2	-13059.23	13059.23
25	MM/S**2	-862.63	742.82
26	MM/S**2	0.00	0.00
27	N/MM**2	-7.05	8.23
28	N/MM**2	-7.29	7.05
29	N/MM**2	-26.56	31.50
30	N/MM**2	-29.15	33.38

TEST NUMBER : 35

IDENTIFICATION CODE : D4PNNS20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-21.13	15.33
2	MM/S**2	-1408.96	1423.34
3	MM	-0.03	0.97
4	MM/S**2	-105.43	105.43
5	MM	-25.56	19.25
6	MM	-20.88	16.05
7	MM	-16.26	12.37
8	MM	-12.58	9.07
9	MM	-2.94	6.06
10	MM	-0.10	0.02
11	MM/S**2	-958.48	1064.97
12	MM/S**2	-670.93	670.93
13	MM/S**2	-575.09	646.97
14	MM/S**2	-623.01	646.97
15	MM/S**2	-718.86	838.67
16	MM/S**2	-1246.62	1198.09
17	MM	-8.10	7.13
18	MM	-6.44	7.36
19	MM/S**2	-742.82	646.97
20	MM/S**2	-766.78	766.78
21	MM/S**2	-47.92	119.81
22	MM/S**2	-47.92	143.77
23	MM/S**2	-2755.62	2516.00
24	MM/S**2	-6589.52	5511.24
25	MM/S**2	-670.93	623.01
26	MM/S**2	0.00	0.00
27	N/MM**2	-2.35	2.12
28	N/MM**2	-2.35	2.12
29	N/MM**2	-6.11	6.58
30	N/MM**2	-6.58	7.05

TEST NUMBER : 64

IDENTIFICATION CODE : D8PNNS20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-21.25	15.45
2	MM/S**2	-1351.45	1380.21
3	MM	-0.06	0.94
4	MM/S**2	-110.22	100.64
5	MM	-21.56	13.81
6	MM	-17.45	11.72
7	MM	-13.77	9.40
8	MM	-11.22	7.86
9	MM	-8.81	5.98
10	MM	0.00	0.01
11	MM/S**2	-985.10	1198.09
12	MM/S**2	-670.93	886.59
13	MM/S**2	-623.01	814.70
14	MM/S**2	-503.20	838.67
15	MM/S**2	-646.97	790.74
16	MM/S**2	-538.67	934.51
17	MM	-5.06	4.29
18	MM	-4.08	5.28
19	MM/S**2	-575.09	431.31
20	MM/S**2	-527.16	503.20
21	MM/S**2	-57.51	93.45
22	MM/S**2	-50.32	98.24
23	MM/S**2	-4073.52	3953.52
24	MM/S**2	-4672.57	4912.19
25	MM/S**2	-646.97	599.05
26	MM/S**2	0.00	0.00
27	N/MM**2	-1.88	3.06
28	N/MM**2	-2.59	2.35
29	N/MM**2	-7.52	9.64
30	N/MM**2	-8.46	11.05

TABLE 4.15 — SHAKING TABLE TEST RESULTS — PBNS 20

TEST NUMBER : 14

IDENTIFICATION CODE : FBPBNS20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-21.07	16.33
2	MM/S**2	-3450.51	2832.30
3	MM	-0.47	0.85
4	MM/S**2	-474.45	761.99
5	MM	-50.36	51.79
6	MM	-42.06	43.76
7	MM	-33.45	34.24
8	MM	-24.52	23.37
9	MM	-14.42	11.69
10	MM	0.00	0.00
11	MM/S**2	-17625.30	14137.52
12	MM/S**2	-12675.84	10950.59
13	MM/S**2	-10495.31	9968.15
14	MM/S**2	-6589.52	7907.43
15	MM/S**2	-4720.49	5583.12
16	MM/S**2	0.00	0.00
17	MM	0.00	0.00
18	MM	0.00	0.00
19	MM/S**2	0.00	0.00
20	MM/S**2	0.00	0.00
21	MM/S**2	-570.29	747.61
22	MM/S**2	-809.91	745.21
23	MM/S**2	-27316.56	25998.65
24	MM/S**2	-41813.50	44089.88
25	MM/S**2	-4073.52	4385.03
26	MM/S**2	0.00	0.00
27	N/MM**2	-34.79	32.91
28	N/MM**2	-32.20	35.02
29	N/MM**2	-135.39	133.04
30	N/MM**2	-165.84	140.80

TEST NUMBER : 39

IDENTIFICATION CODE : D4PBNS20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-21.01	15.45
2	MM/S**2	-3536.78	2904.18
3	MM	-0.06	0.91
4	MM/S**2	-258.79	201.28
5	MM	-26.99	26.66
6	MM	-20.63	21.77
7	MM	-16.11	16.90
8	MM	-12.80	13.07
9	MM	-10.79	9.63
10	MM	-1.15	10.76
11	MM/S**2	-2129.95	2263.07
12	MM/S**2	-766.78	982.44
13	MM/S**2	-1126.21	1078.29
14	MM/S**2	-2132.61	2158.57
15	MM/S**2	-3306.74	3210.89
16	MM/S**2	-5463.31	5217.85
17	MM	-8.92	7.95
18	MM	-7.76	7.56
19	MM/S**2	-2204.49	2060.72
20	MM/S**2	-2468.07	2683.73
21	MM/S**2	-177.32	122.21
22	MM/S**2	-119.81	165.34
23	MM/S**2	-7787.62	8147.04
24	MM/S**2	-23842.09	25759.04
25	MM/S**2	-2516.00	2396.19
26	MM/S**2	0.00	0.00
27	N/MM**2	-3.53	4.23
28	N/MM**2	-4.23	3.76
29	N/MM**2	-8.93	8.93
30	N/MM**2	-10.81	11.99

TEST NUMBER : 68

IDENTIFICATION CODE : D8PBNS20.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-21.07	15.45
2	MM/S**2	-3436.14	2889.80
3	MM	-0.06	0.97
4	MM/S**2	-225.24	182.11
5	MM	-19.12	20.70
6	MM	-16.16	17.14
7	MM	-13.05	13.46
8	MM	-10.37	11.12
9	MM	-8.46	8.90
10	MM	0.00	0.00
11	MM/S**2	-2236.44	2449.44
12	MM/S**2	-1317.90	1509.60
13	MM/S**2	-1126.21	1413.75
14	MM/S**2	-1174.13	1509.60
15	MM/S**2	-1725.26	1821.10
16	MM/S**2	-2755.62	2587.86
17	MM	-4.91	5.03
18	MM	-5.16	4.52
19	MM/S**2	-1485.64	1365.83
20	MM/S**2	-1557.52	1581.48
21	MM/S**2	-170.13	186.90
22	MM/S**2	-165.34	182.11
23	MM/S**2	-13778.09	14017.71
24	MM/S**2	-15695.04	17492.18
25	MM/S**2	-2252.42	2180.53
26	MM/S**2	-0.00	0.00
27	N/MM**2	-4.00	5.17
28	N/MM**2	-4.70	4.00
29	N/MM**2	-13.16	15.04
30	N/MM**2	-14.34	16.92

TABLE 4.16 — SHAKING TABLE TEST RESULTS — PNVK 40

TEST NUMBER : 17

IDENTIFICATION CODE : FBPNVK40.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-0.12	0.18
2	MM/S**2	-57.51	86.26
3	MM	-13.75	20.84
4	MM/S**2	-1375.41	1265.19
5	MM	-1.45	2.07
6	MM	-0.85	0.85
7	MM	-0.63	0.83
8	MM	-0.70	0.51
9	MM	-0.64	0.38
10	MM	0.00	0.00
11	MM/S**2	-399.36	346.12
12	MM/S**2	-287.54	287.54
13	MM/S**2	-335.47	335.47
14	MM/S**2	-287.54	239.62
15	MM/S**2	-71.89	119.81
16	MM/S**2	0.00	0.00
17	MM	0.00	0.00
18	MM	0.00	0.00
19	MM/S**2	0.00	0.00
20	MM/S**2	0.00	0.00
21	MM/S**2	-450.48	452.88
22	MM/S**2	-397.77	330.67
23	MM/S**2	-718.88	958.48
24	MM/S**2	-4432.95	3953.71
25	MM/S**2	-4504.84	4648.61
26	MM/S**2	0.00	0.00
27	N/MM**2	-1.65	2.35
28	N/MM**2	-2.12	1.88
29	N/MM**2	-3.29	3.06
30	N/MM**2	-3.06	3.53

TEST NUMBER : 42

IDENTIFICATION CODE : D4PNVK40.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-0.12	0.24
2	MM/S**2	-57.51	115.02
3	MM	-13.63	20.72
4	MM/S**2	-1461.68	1083.06
5	MM	-0.74	0.95
6	MM	-0.45	0.59
7	MM	-0.63	0.63
8	MM	-0.44	0.51
9	MM	-0.18	0.64
10	MM	0.00	0.00
11	MM/S**2	0.00	23.96
12	MM/S**2	-47.92	95.85
13	MM/S**2	-71.89	119.81
14	MM/S**2	-95.85	143.77
15	MM/S**2	-191.70	191.70
16	MM/S**2	-670.93	814.70
17	MM	-6.08	5.10
18	MM	-6.20	5.20
19	MM/S**2	-1126.21	1293.94
20	MM/S**2	-1198.09	1413.75
21	MM/S**2	-0.10	0.10
22	MM/S**2	0.00	0.00
23	MM/S**2	-239.62	239.62
24	MM/S**2	-3234.86	3234.86
25	MM/S**2	0.00	0.00
26	MM/S**2	0.00	0.00
27	N/MM**2	-0.39	1.18
28	N/MM**2	-1.18	1.18
29	N/MM**2	-0.78	1.18
30	N/MM**2	-0.39	1.18

TEST NUMBER : 72

IDENTIFICATION CODE : D6PNVK40.INV

CHANNEL	UNITS	MINIMUM VALUES	MAXIMUM VALUES
1	MM	-0.06	5.01
2	MM/S**2	-330.67	273.17
3	MM	-11.14	16.91
4	MM/S**2	-1078.29	977.65
5	MM	-1.84	4.19
6	MM	-2.01	3.49
7	MM	-2.34	2.31
8	MM	-1.78	2.19
9	MM	-1.48	2.12
10	MM	0.00	0.00
11	MM/S**2	-399.36	266.24
12	MM/S**2	-215.66	119.81
13	MM/S**2	-263.58	167.73
14	MM/S**2	-239.62	167.73
15	MM/S**2	-191.70	191.70
16	MM/S**2	-383.39	215.66
17	MM	-3.04	1.71
18	MM	-3.04	1.60
19	MM/S**2	-862.63	910.55
20	MM/S**2	-862.63	1126.21
21	MM/S**2	-91.06	55.11
22	MM/S**2	-21.57	69.49
23	MM/S**2	-2156.57	2396.19
24	MM/S**2	-3594.28	3354.67
25	MM/S**2	-1557.52	1557.52
26	MM/S**2	0.00	0.00
27	N/MM**2	-1.18	2.12
28	N/MM**2	-1.41	2.12
29	N/MM**2	-3.29	3.06
30	N/MM**2	-3.29	3.29

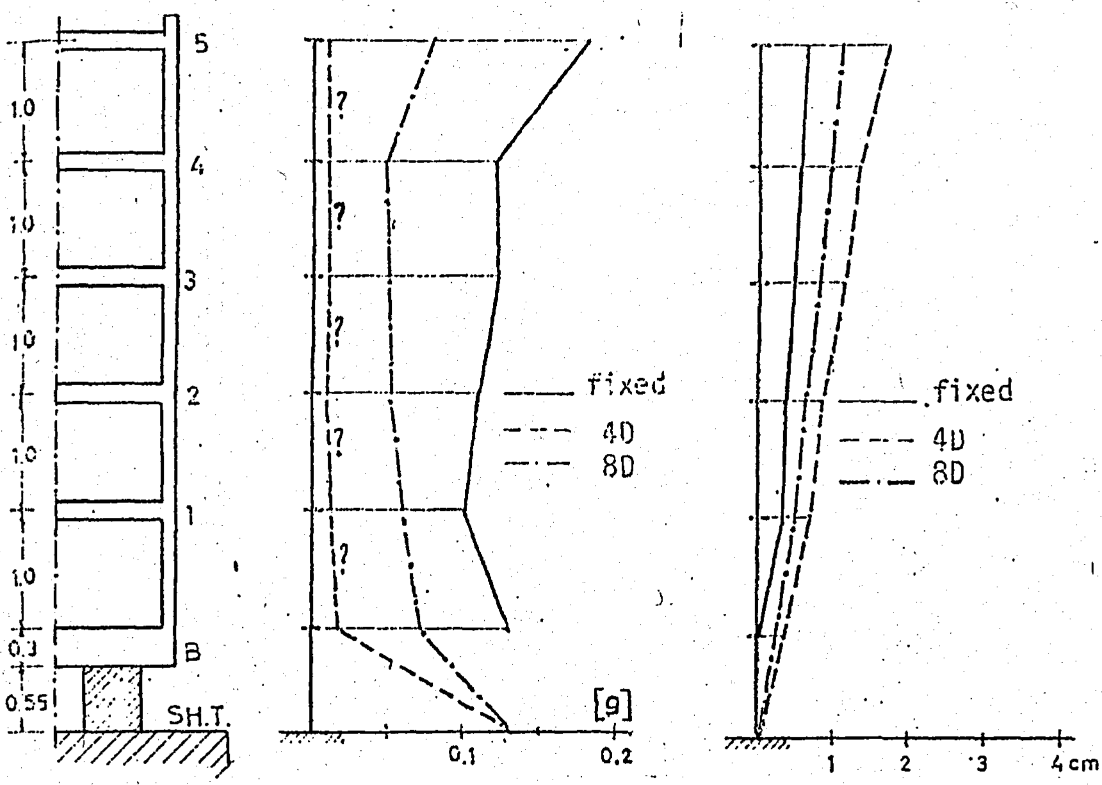


FIG. 4.10 - PEAK ACCELERATIONS AND DISPLACEMENTS - EN 200

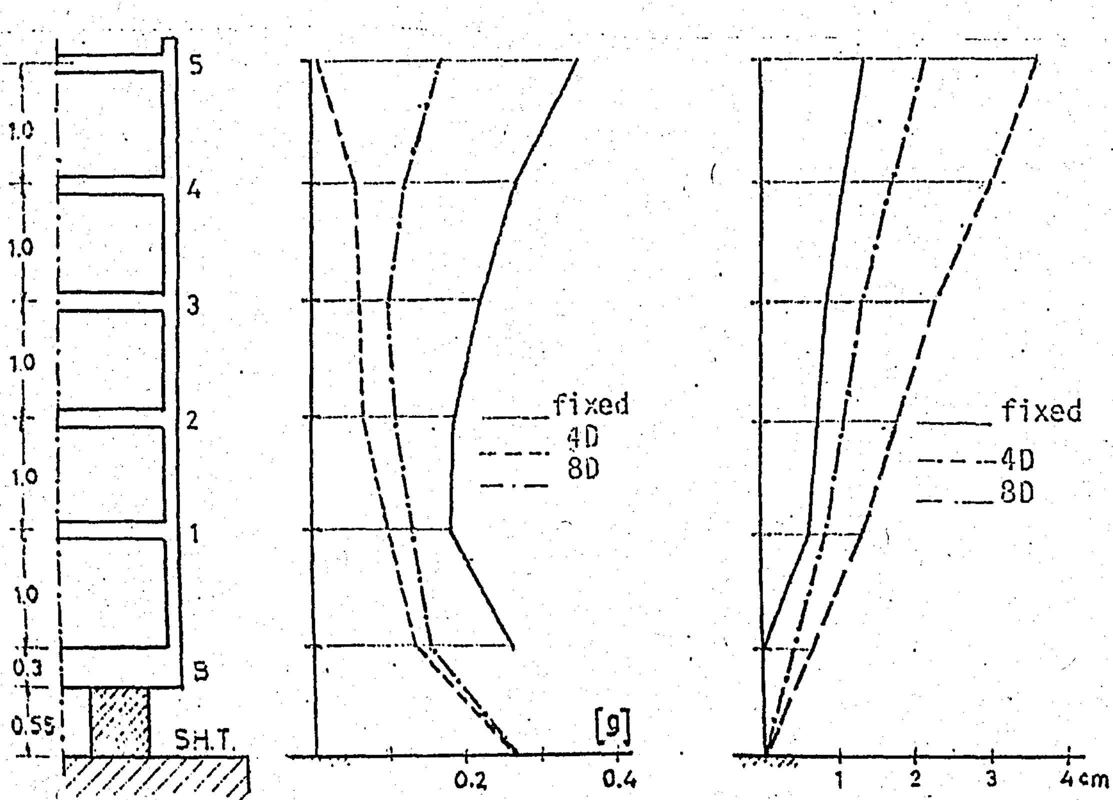


FIG. 4.11 - PEAK ACCELERATIONS AND DISPLACEMENTS - EN 400

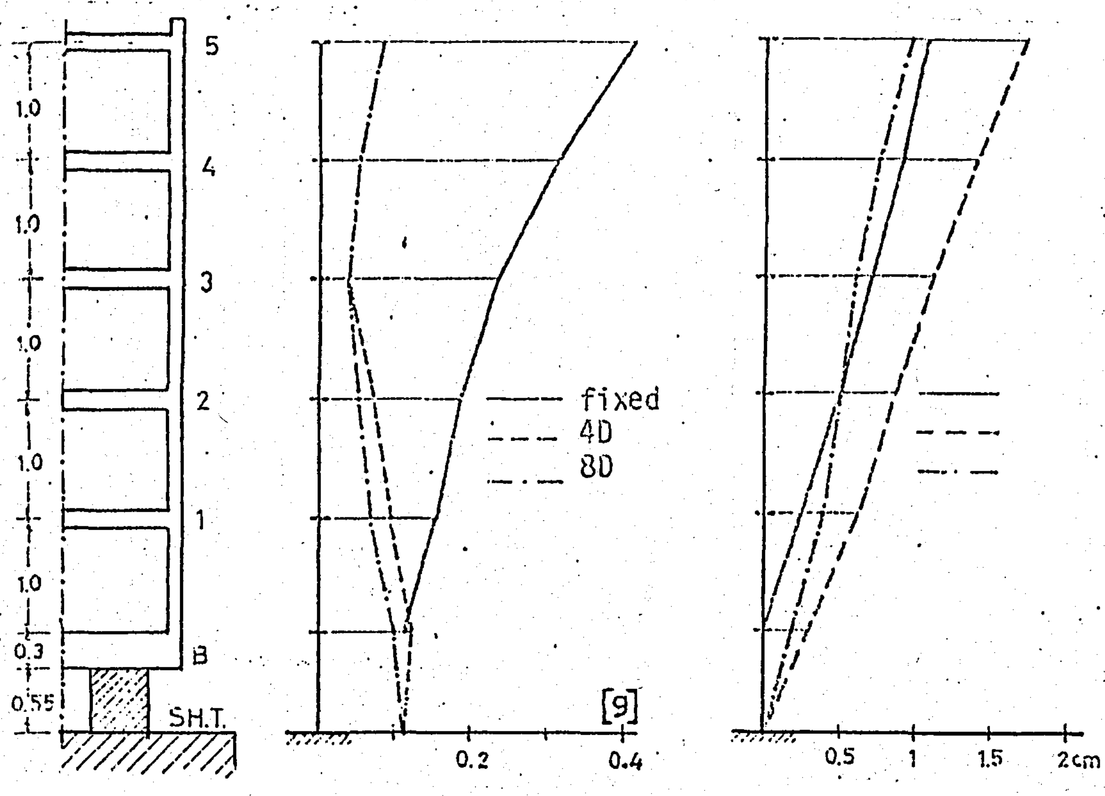


FIG. 4.12 - PEAK ACCELERATIONS AND DISPLACEMENTS - EB 100

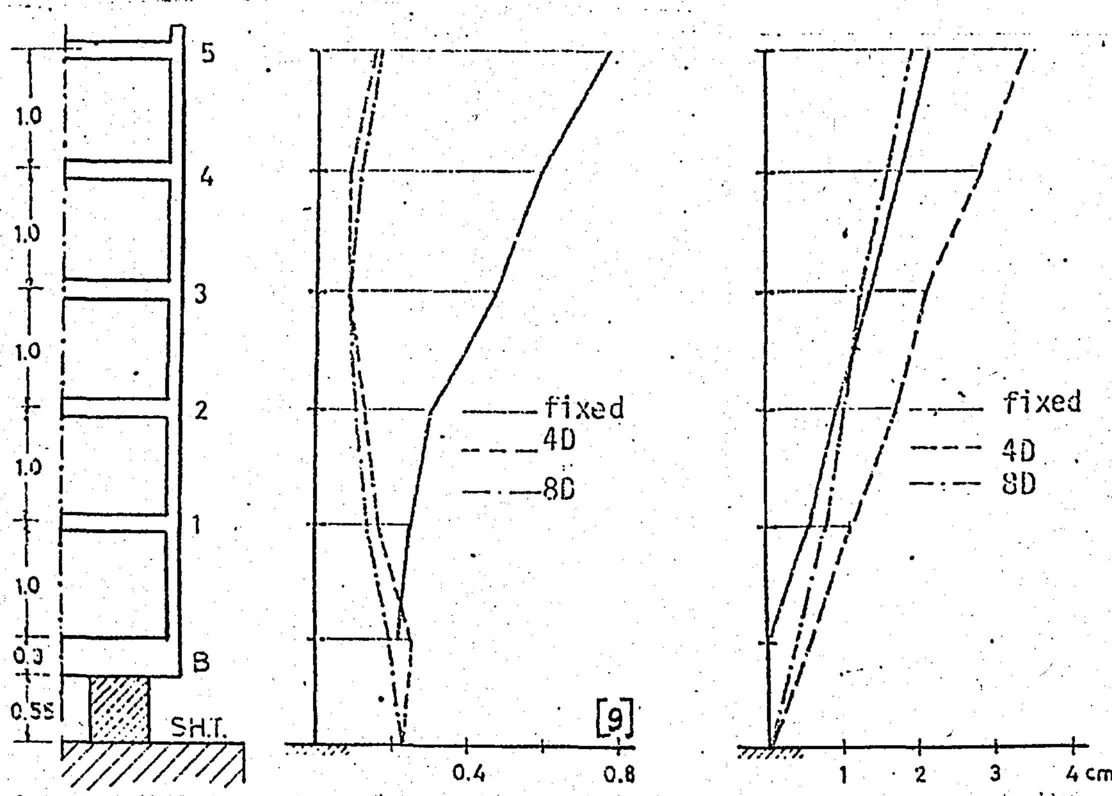


FIG. 4.13 - PEAK ACCELERATIONS AND DISPLACEMENTS - EB 200

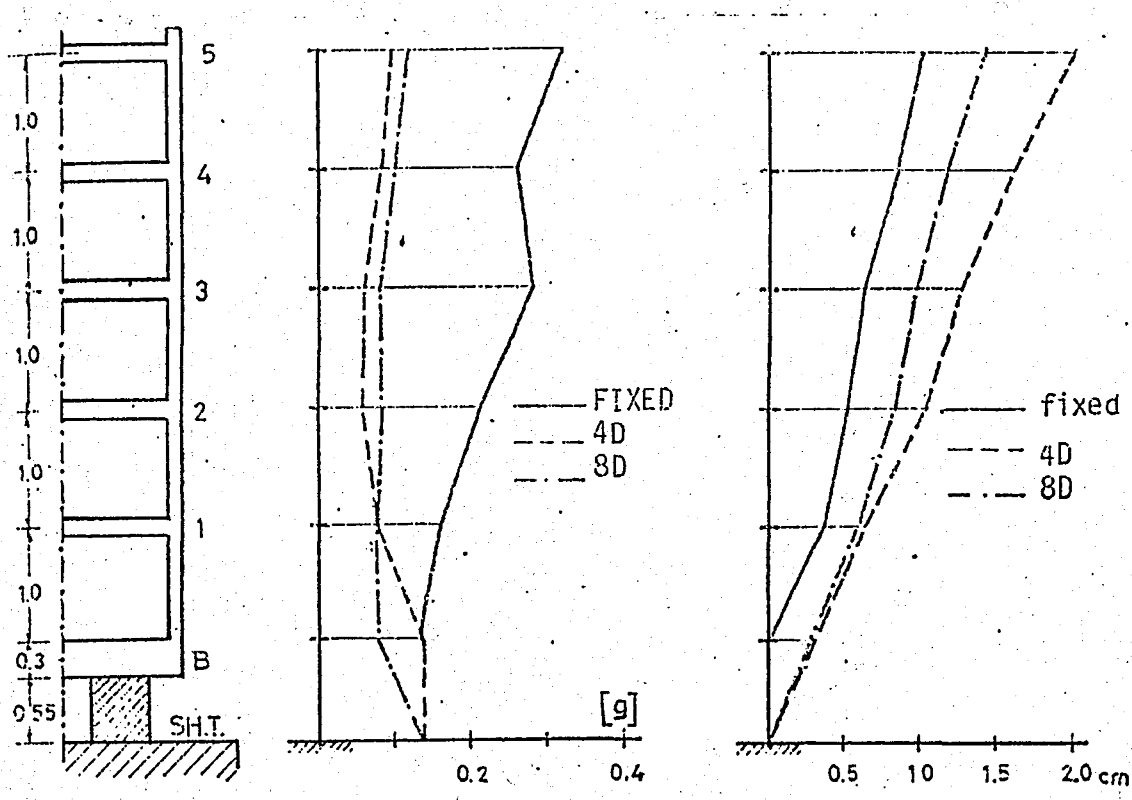


FIG. 4.14 - PEAK ACCELERATIONS AND DISPLACEMENTS - PN 200

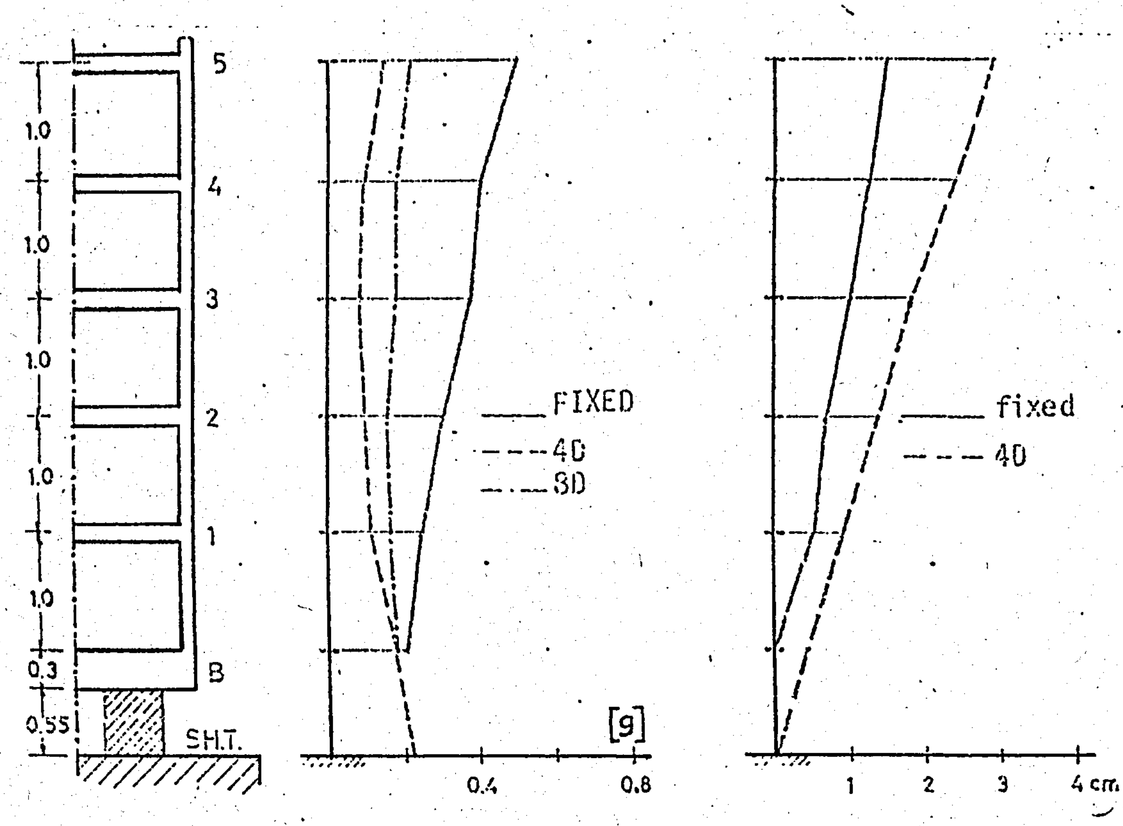


FIG. 4.15 - PEAK ACCELERATIONS AND DISPLACEMENTS - PN 300

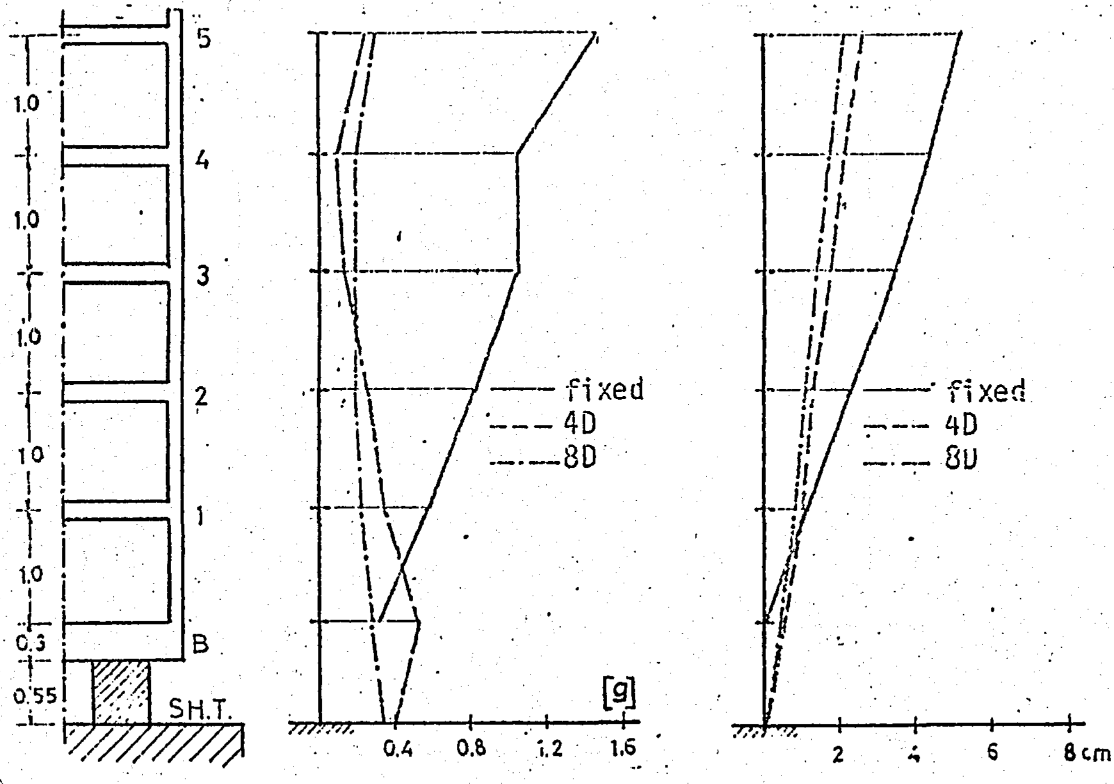


FIG. 4.16 - PEAK ACCELERATIONS AND DISPLACEMENTS - PB 200

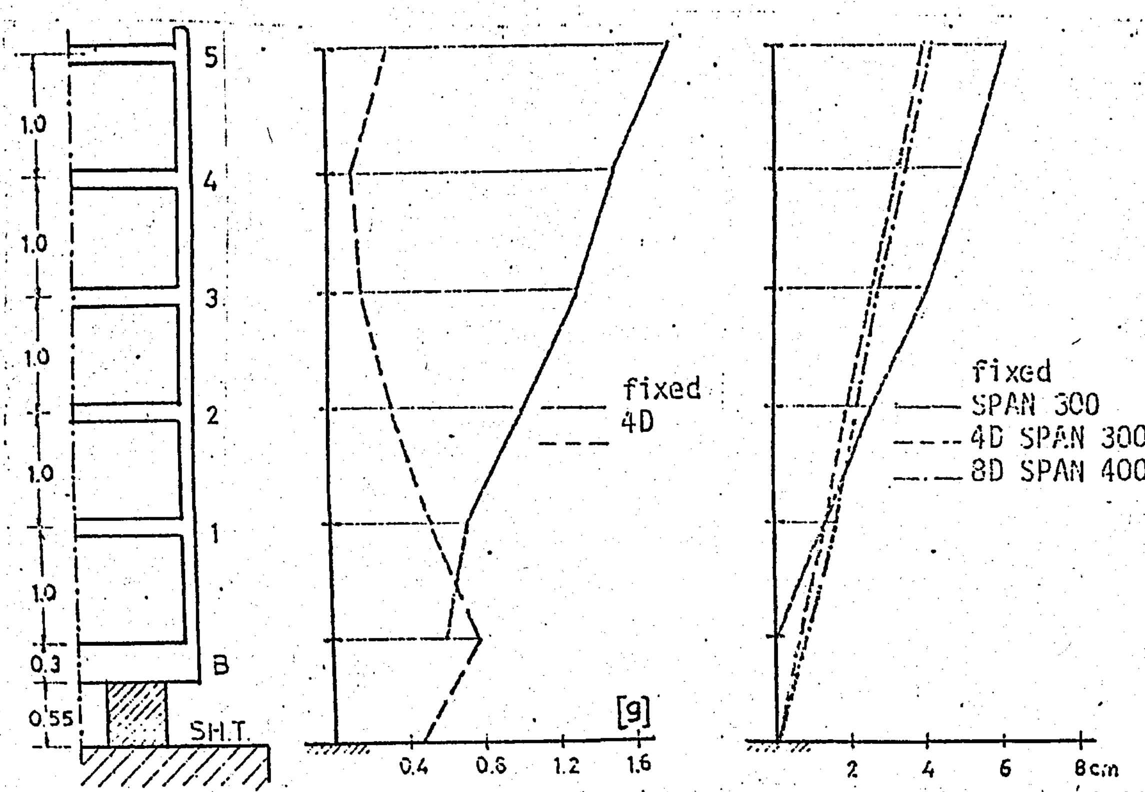


FIG. 4.17 - PEAK ACCELERATIONS AND DISPLACEMENTS - PB 300
PB 400

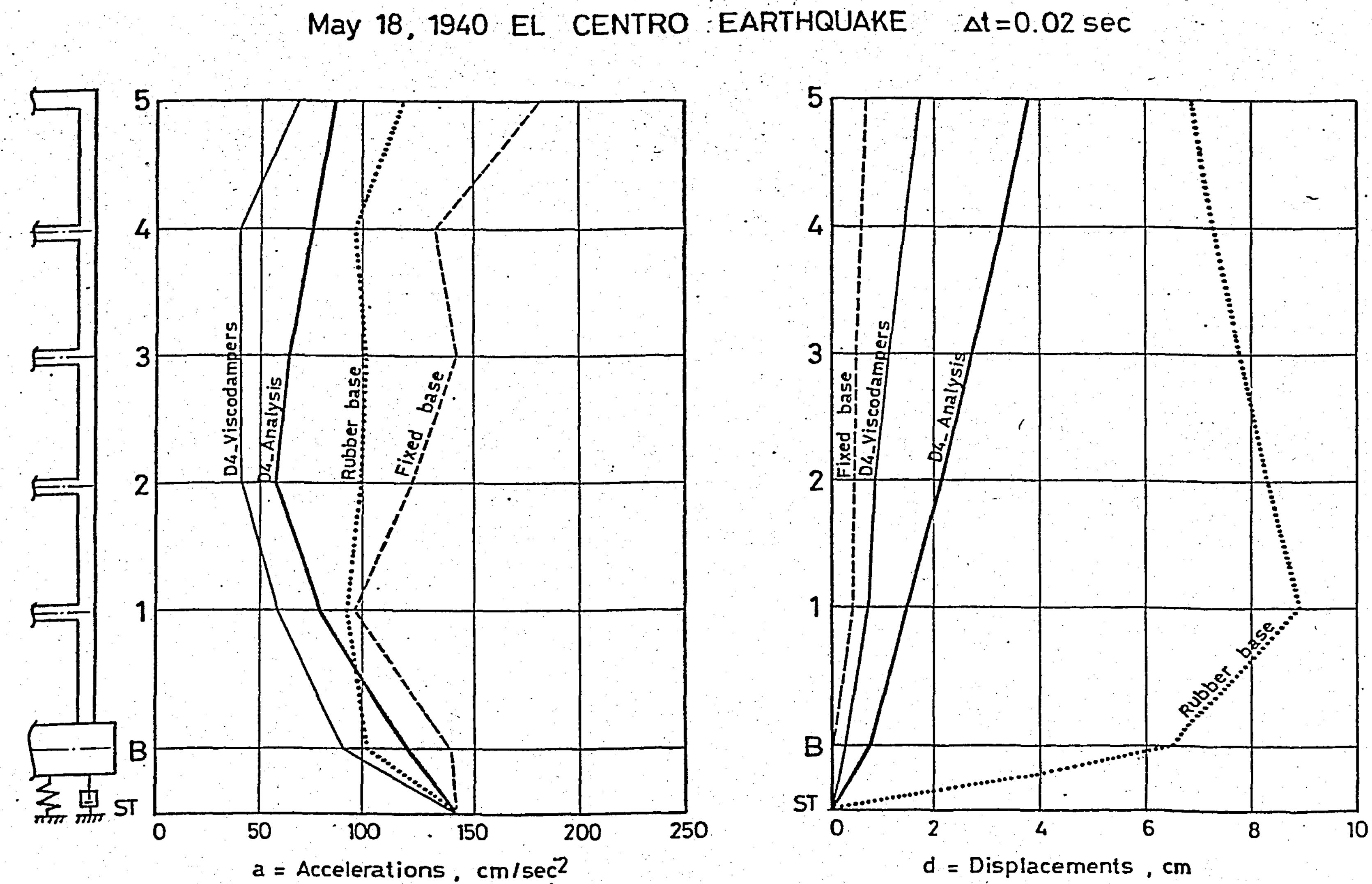


FIG. 4.18 - PEAK RESPONSE VALUES - EN NS 20

April 15, 1979 PETROVAC EARTHQUAKE, $\Delta t = 0.02$ sec

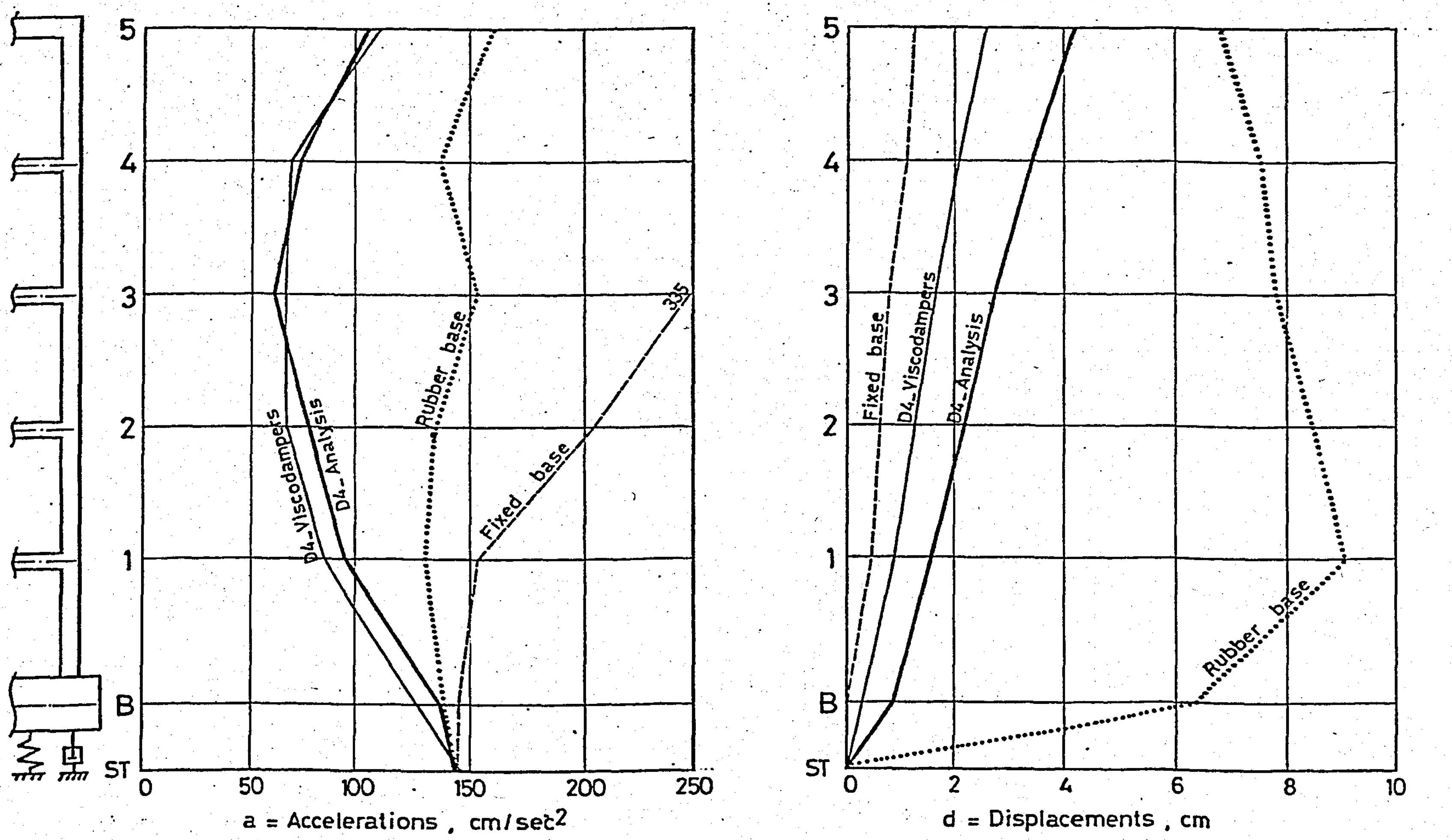


FIG. 4.19 - PEAK RESPONSE VALUES - PN NS 20

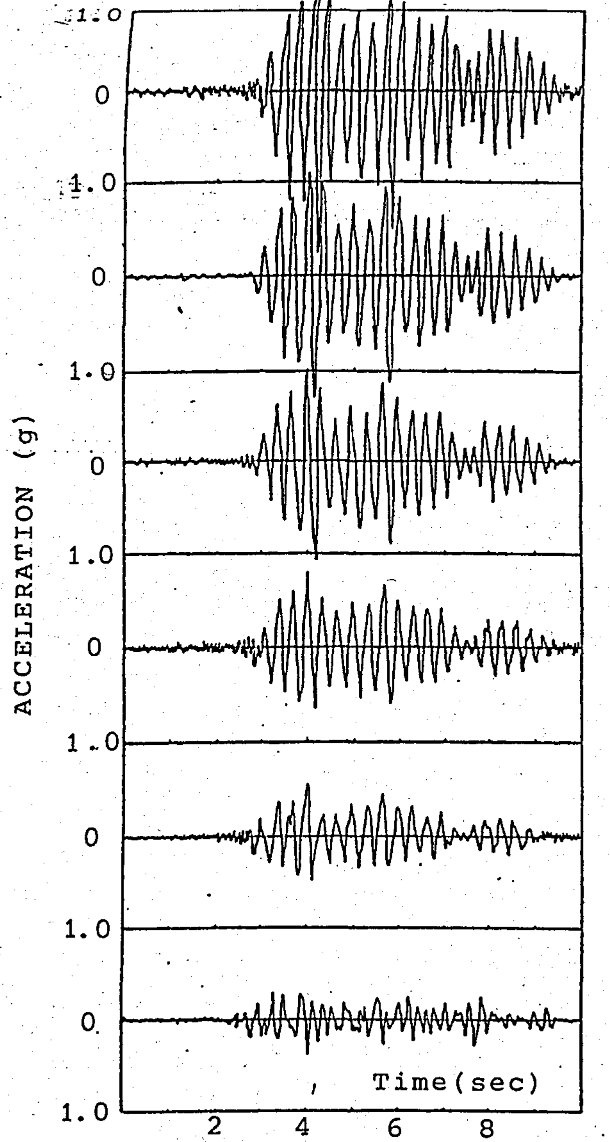


FIG 4.20-ACCELERATION TIME HISTORIES-FBPBNS20

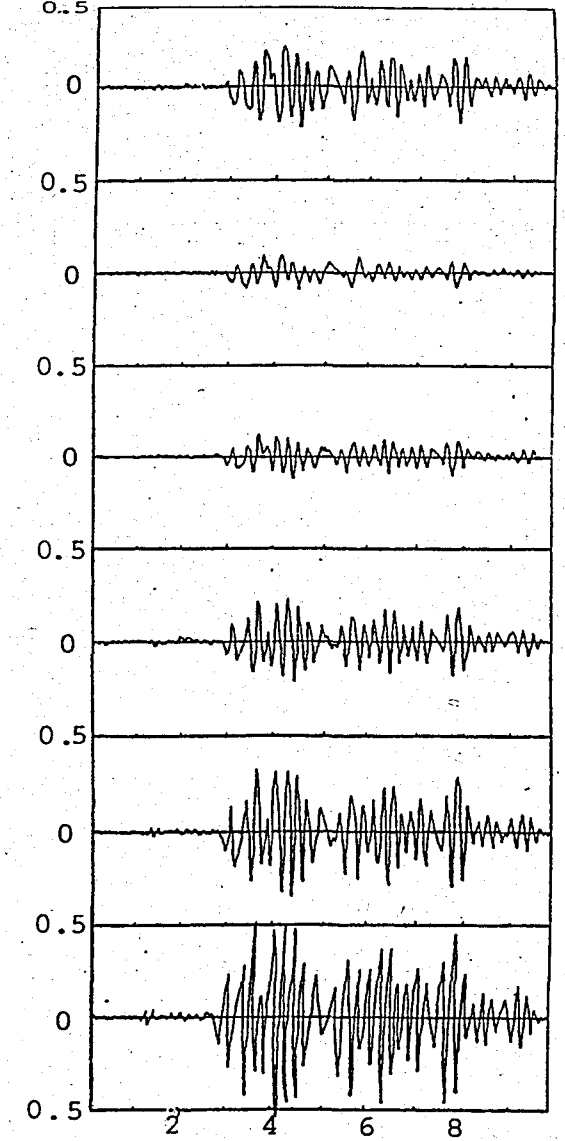


FIG 4.21-ACCELERATION TIME HISTORIES-D4PBNS20

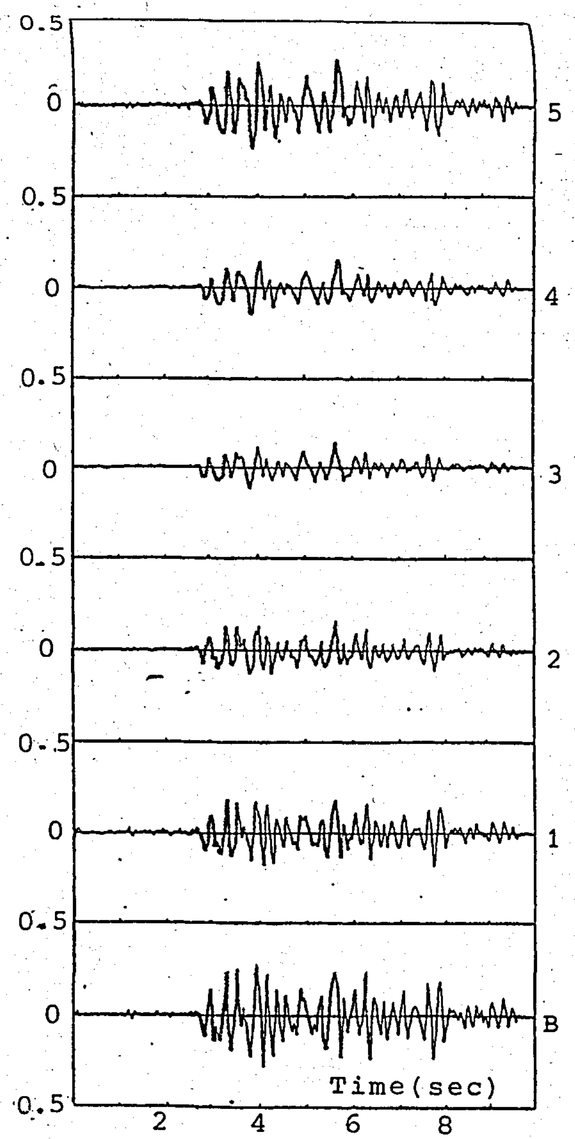


FIG. 4.22-ACCELERATION TIME HISTORIES-D8PBNS20

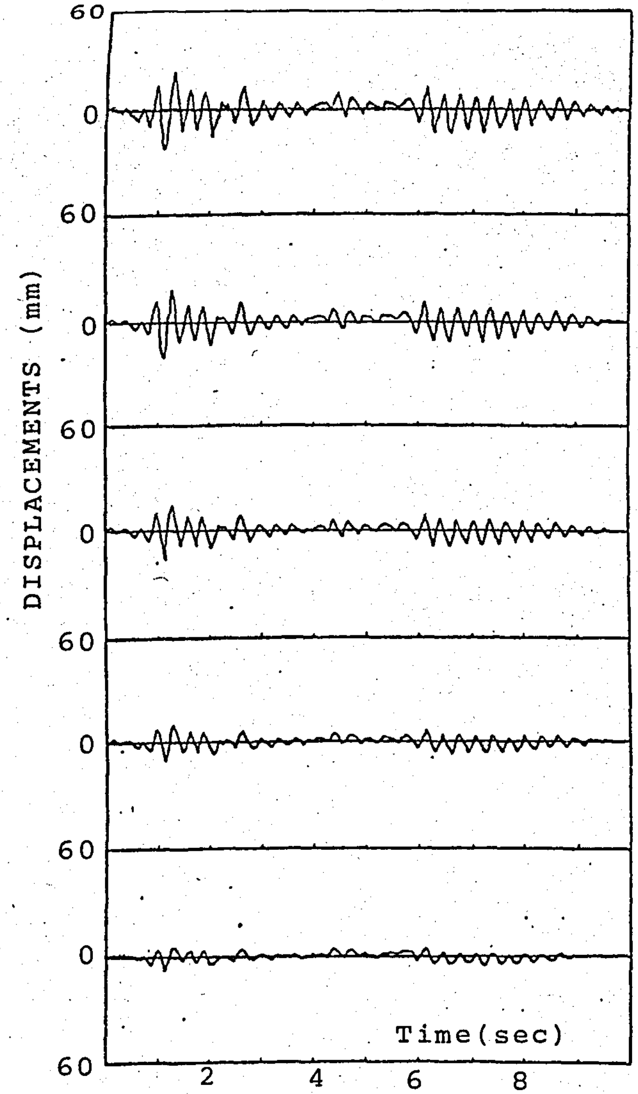


FIG 4.23 DISPLACEMENT TIME HISTORIES-FBEBNS20

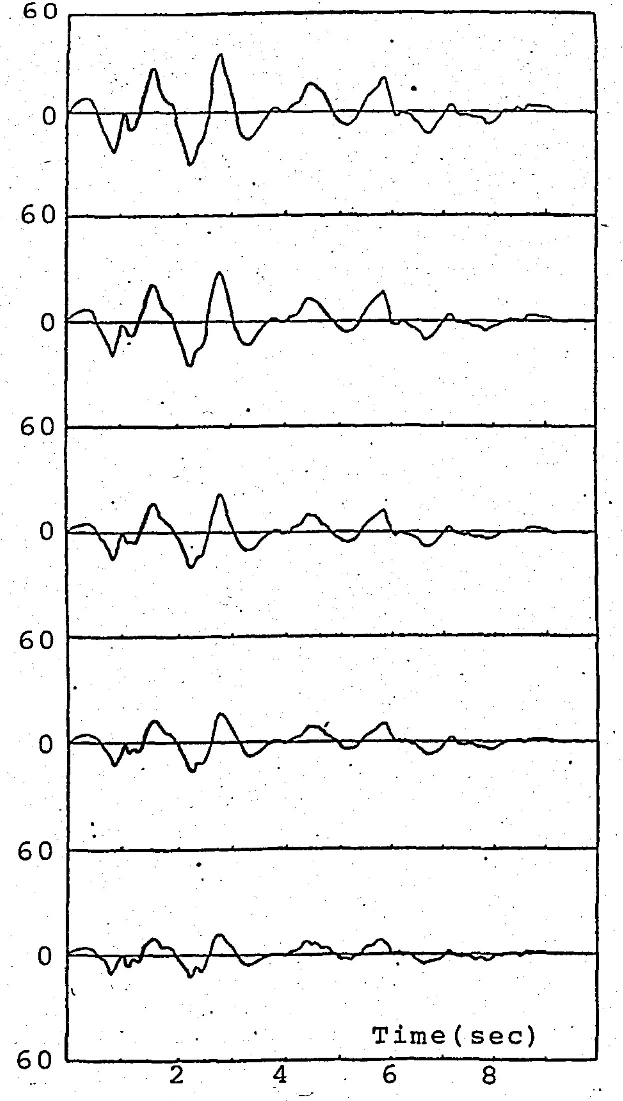


FIG 4.24 DISPLACEMENT TIME HISTORIES-D4EBNS20

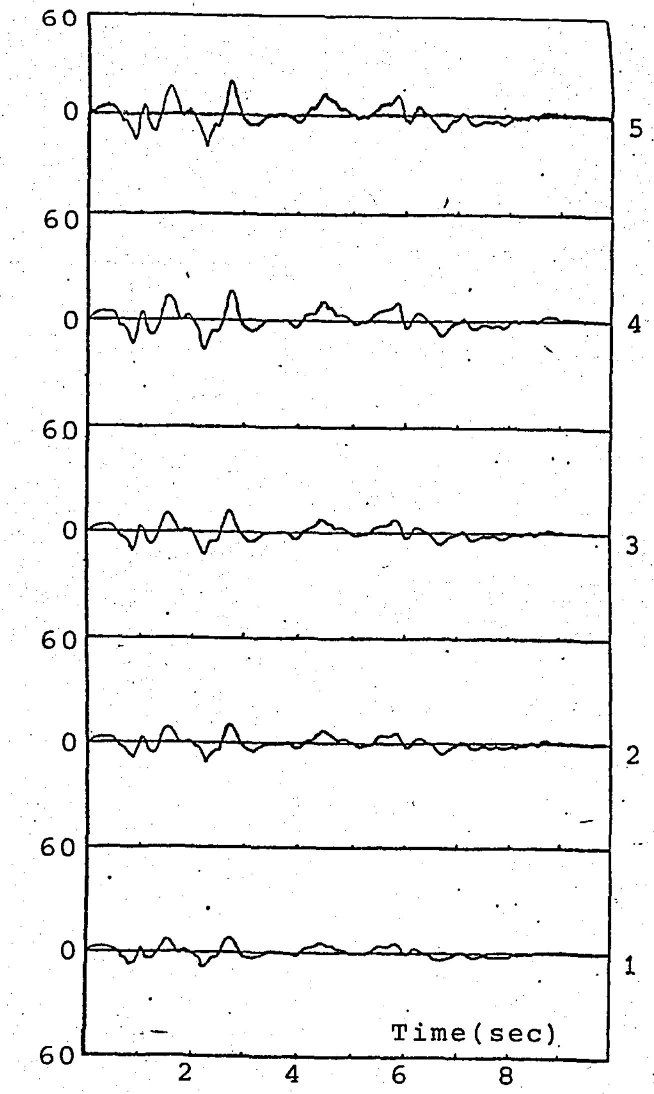


FIG 4.25 DISPLACEMENT TIME HISTORIES-D8EBNS20

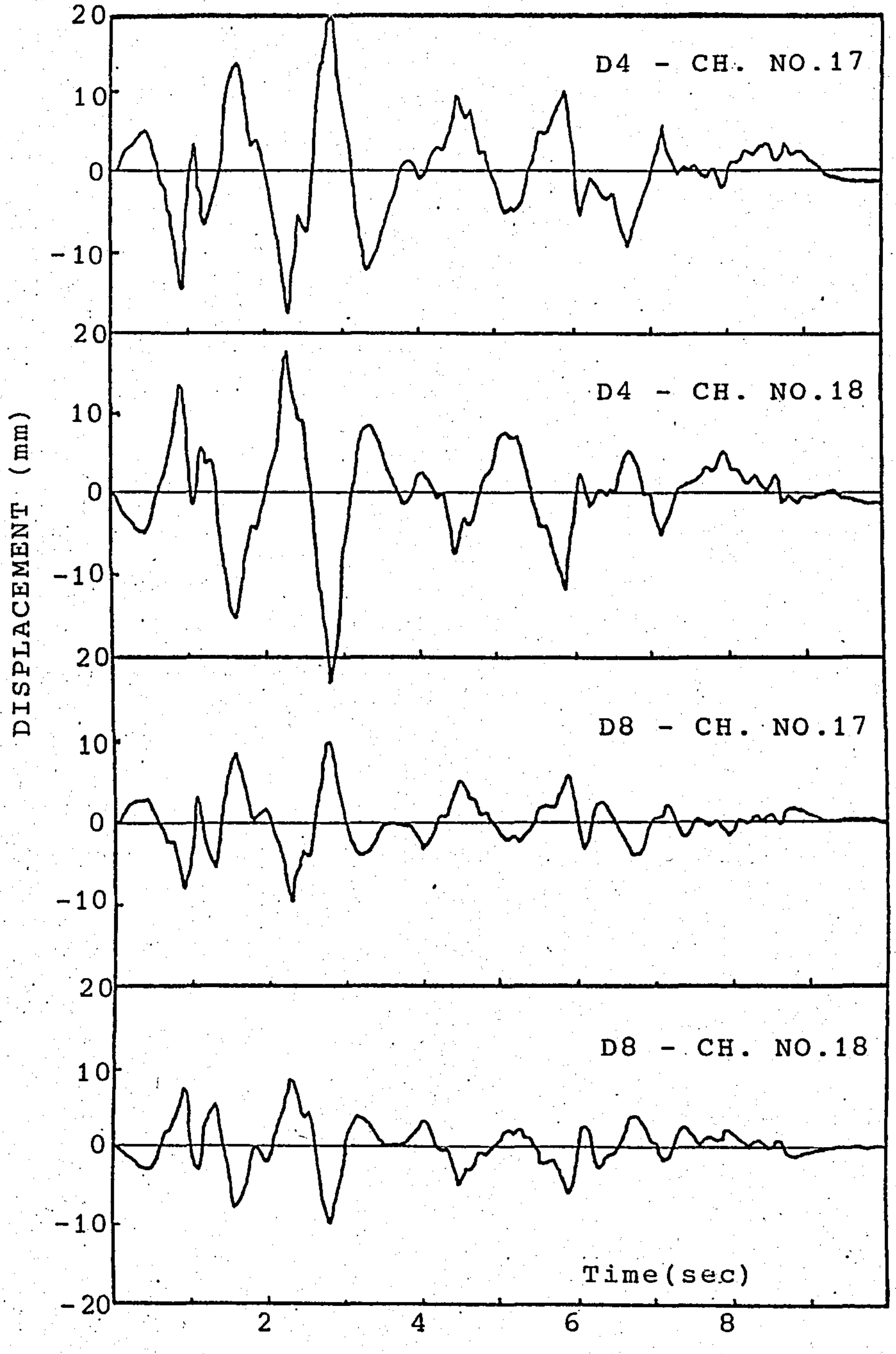


FIG. 4.26 - DISPLACEMENT TIME HISTORIES OF SPRINGS IN LONGITUDINAL DIRECTION (EB NS 40)

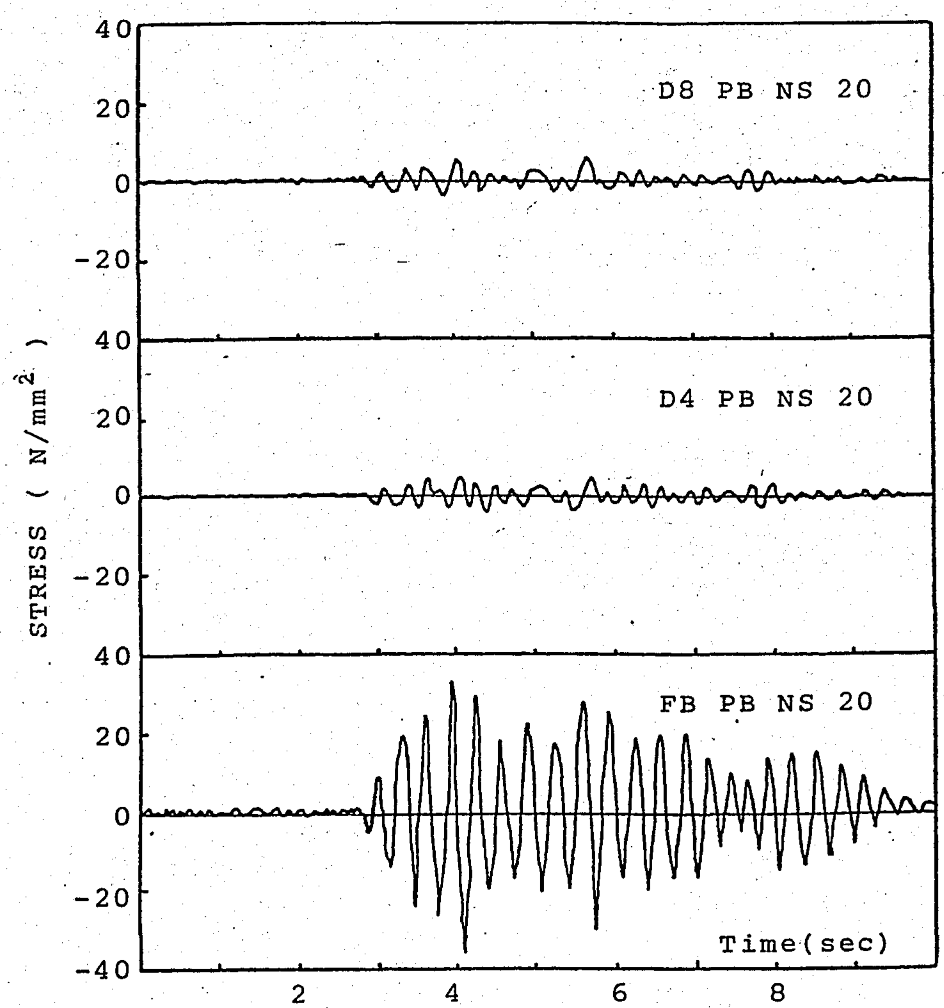


FIG. 4.27 - AXIAL STRESS TIME HISTORIES OF EXTERNAL COLUMN

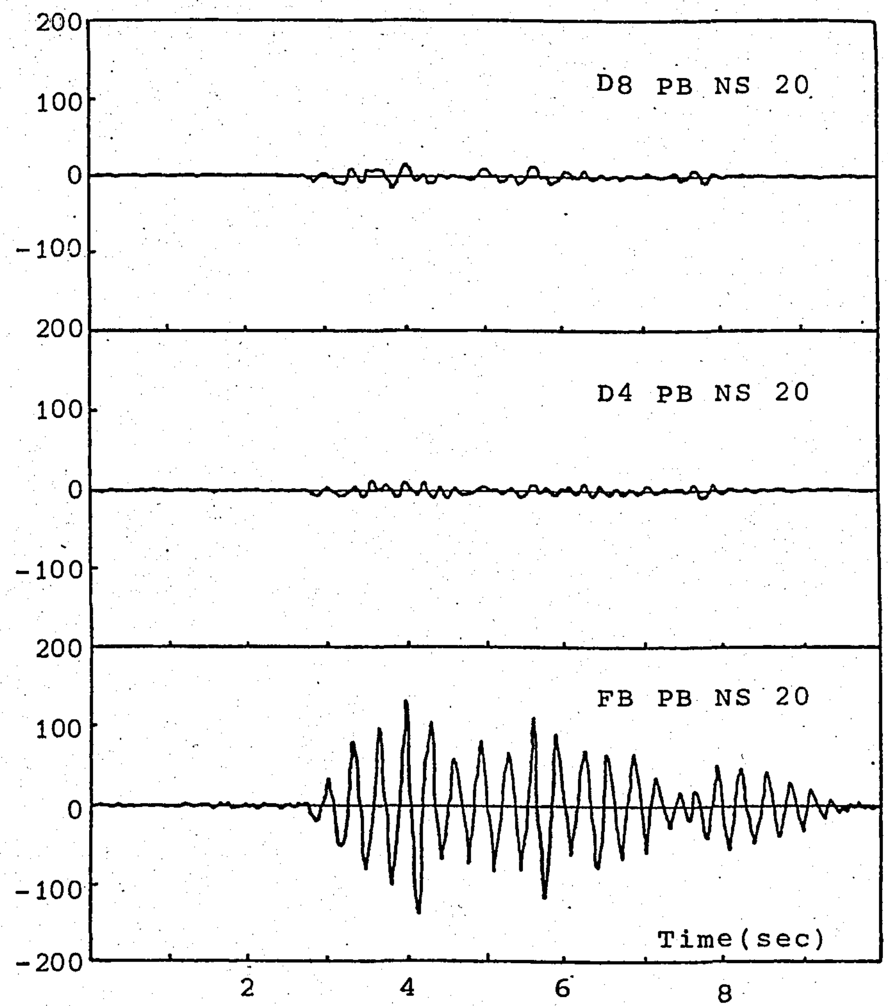


FIG. 4.28 - BENDING STRESS TIME HISTORIES OF EXTERNAL COLUMN

CHAPTER 5

ANALYTICAL STUDIES OF THE TEST FRAME

5.1. GENERAL

For the purpose of demonstrating the feasibility as well as the usefulness of the vibration isolation systems in aseismic design, a series of analytical studies have been performed for the 3-bay and 5-storey test frame.

The geometrical and physical properties of the frame are summarized in Fig. 5.1. The steel frame is tested using basically three different support conditions:

a. The fixed support case
b. The spring support case
c. The rubber support case

In the fixed support case, the base beam, connecting the first storey columns of the frame to each other, is anchored firmly, by means of bolts, onto the shaking table. In the case of vibration isolation, however, the base beam is supported either on helical springs or on rubber elements.

The spring characteristics as well as the height of the rubber elements are selected in such a way that, the fundamental natural frequencies of vibration of the frame in horizontal, vertical and rocking directions are modified to remain considerably outside the range of the predominant frequencies of the possible ground shaking.

When the frame is supported on helical springs, or rubber elements, the acceleration response of the frame is drastically reduced. In fact, the whole frame vibrates practically in the rigid body modes without experiencing large accelerations or any interstorey displacements. Displacements as a whole, are reduced to tolerable limits, by means of special viscodampers, which provide up to 20% to 30% critical damping capabilities in both horizontal and vertical directions. In the case of rubber base isolation however, horizontal displacements become significantly large.

For the purpose of determining the relative influence of viscodampers, the test frame assembly is supplied first, with four viscodampers, one at each corner (4D-Case), and then, with eight viscodampers, two at each corner (8D-Case).

5.2. STRUCTURAL DATA and MATHEMATICAL MODELLING

Basic geometrical and physical data as well as the joint and member labelling for one frame of the testing assembly are shown in Fig. 5.1. Tubular cross-sections are sued for all beams, columns and trasversals. The base beam however, is two U-shaped profile of h= 300 mm.

There are two helical springs supporting one frame, each possessing a vertical spring coefficient of k_v = 0.748 kN/mm. These vertical springs provide also a horizontal spring capability of k_h = 0.395 kN/mm.

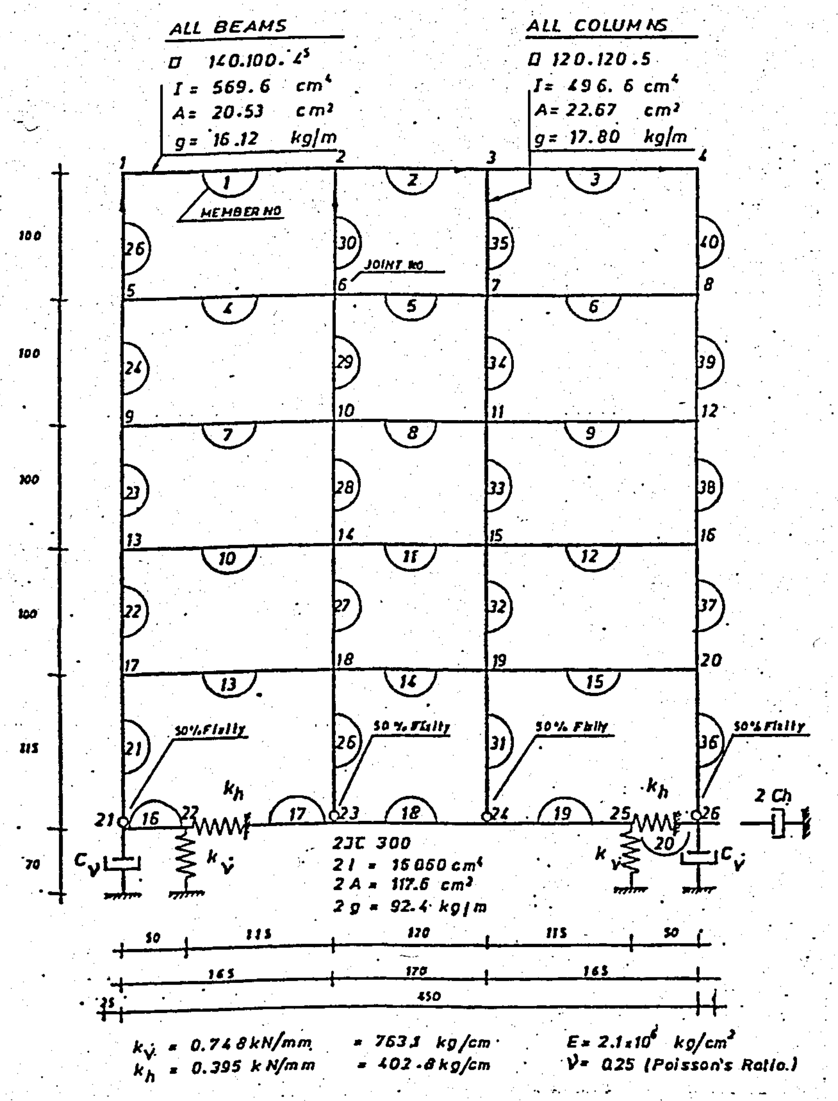


FIG. 5.1 - STRUCTURAL DATA FOR 4D - VISCODAMPERS CASE

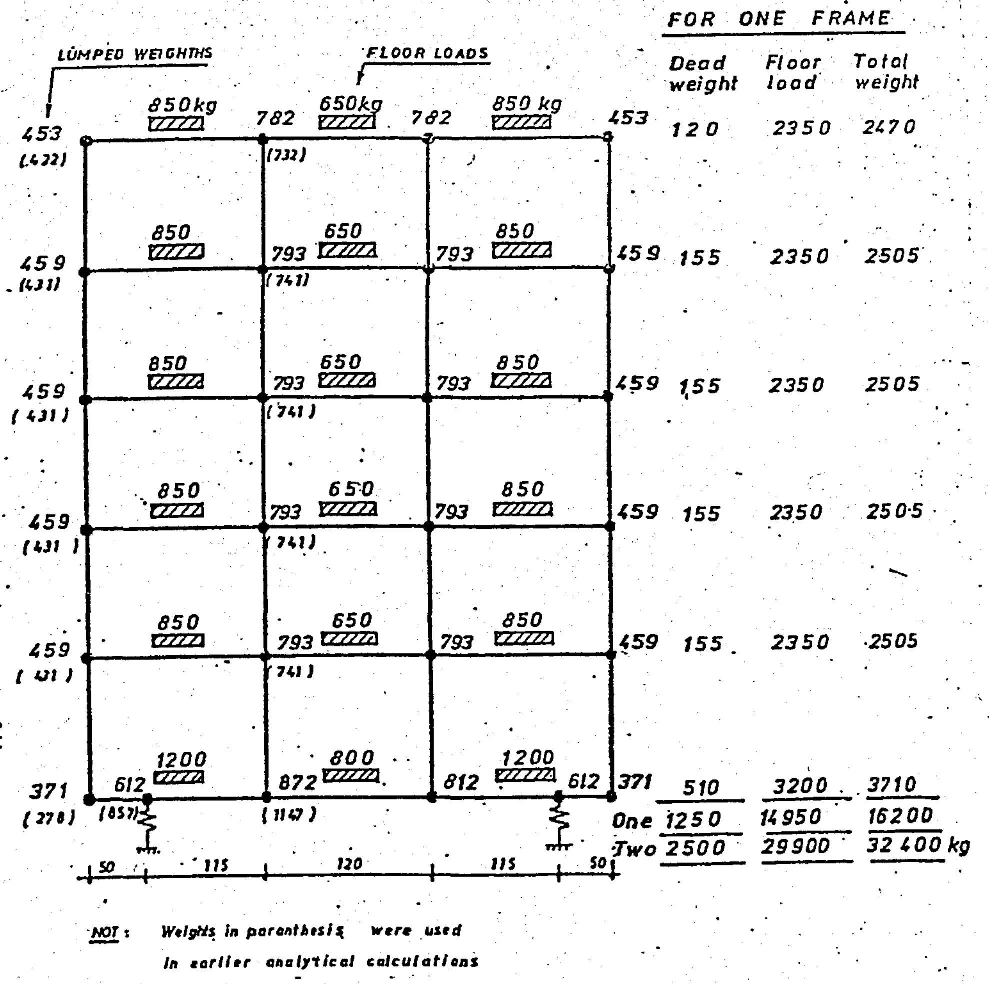


FIG. 5.2 - WEIGHTS FOR ONE FRAME

The total weights and the floor loads for one frame are summarized in Fig. 5.2. For the purpose of analytical investigations, weights are assumed to be lumped at the joints. These weights are calculated on the basis of their tributary distances.

Since, all members are connected to each other by butt welding, they are assumed to be fully fixed at the ends. The first storey columns however, are attached to the base beam by means of four bolts. Therefore, the lower ends of these columns are assumed to be partially fixed. In the absence of any emasured data, the amount of fixity at these bolted connections is assuemd to be 50%, which modify the standard end stiffness influence coefficients of $a_i/b_{ij}/a_j = 4/2/4$ to be $1.72/0.86/3.43$, respectively.

The influence of these partial fixities on the overall dynamic characteristics of the frame is not very significant. For instance, the fundamental natural period of vibration is increased from $T = 1.42$ sec to $T = 1.47$ sec, when the amount of fixity at the lower ends of the first storey columns is reduced from 100% to 50%.

Each lumped weight is assumed to possess three degrees of freedom namely, (a) *Horizontal,* (b) *Vertical* and (c) *Rotational.* Horizontal degrees of freedom at all weights along one floor level are assumed to be equal. That is, the length changes in beams are neglected. Similarly, neglecting the length changes in columns, the vertical degrees of freedom at the weights lying along a particular column line, are assumed to be equal.

No rotary mass moments of inertia is considered to exist for any weight about their own centroids. Consequently, the rotational degrees of freedom of each weight are eliminated from the analysis. For the spring base condition, there are altogether 26 lumped weights in the structure corresponding to a total of 36 degrees of freedom. There are 6 horizontal degrees of freedom (one for each floor) and 6 vertical degrees of freedom (one for each column line and two additional weights at the base beam joints 22 and 25) each vibrating independently. These 12-degrees of freedom are called the "Primary Vibrating Directions". The remaining 26 rotational degrees of freedom are called the "Secondary Vibrating Directions", which are eliminated from the master stiffness matrix of the structure, at the stage of dynamic response analysis calculations.

The primary and the secondary vibrating degrees of freedom of each lumped weight for (a) The Fixed Base Condition, and (b) Base Isolated Condition, are indicated in Figs. 5.3 & 5.4 respectively.

5.3. VISCODAMPERS

For the spring base condition, two alternate sets of viscodampers are used. In the first case, two vertical basic viscodampers are used for each frame, located at the extreme column lines. Since, there are two frames in the testing assembly, altogether 4 basic viscodampers are installed for this first case, which is identified as the "4D-Viscodampers" case.

In the second alternate case however, the number of viscodampers are doubled. That is, there are four vertical viscodampers for each frame. The additional viscodampers are located at the same extreme column centerlines immediately next to the original dampers but at the transversal space between the two frames. In this second alternate case, there are altogether 8 viscodampers, correspondingly this case is identified as the "8D-viscodampers" case.

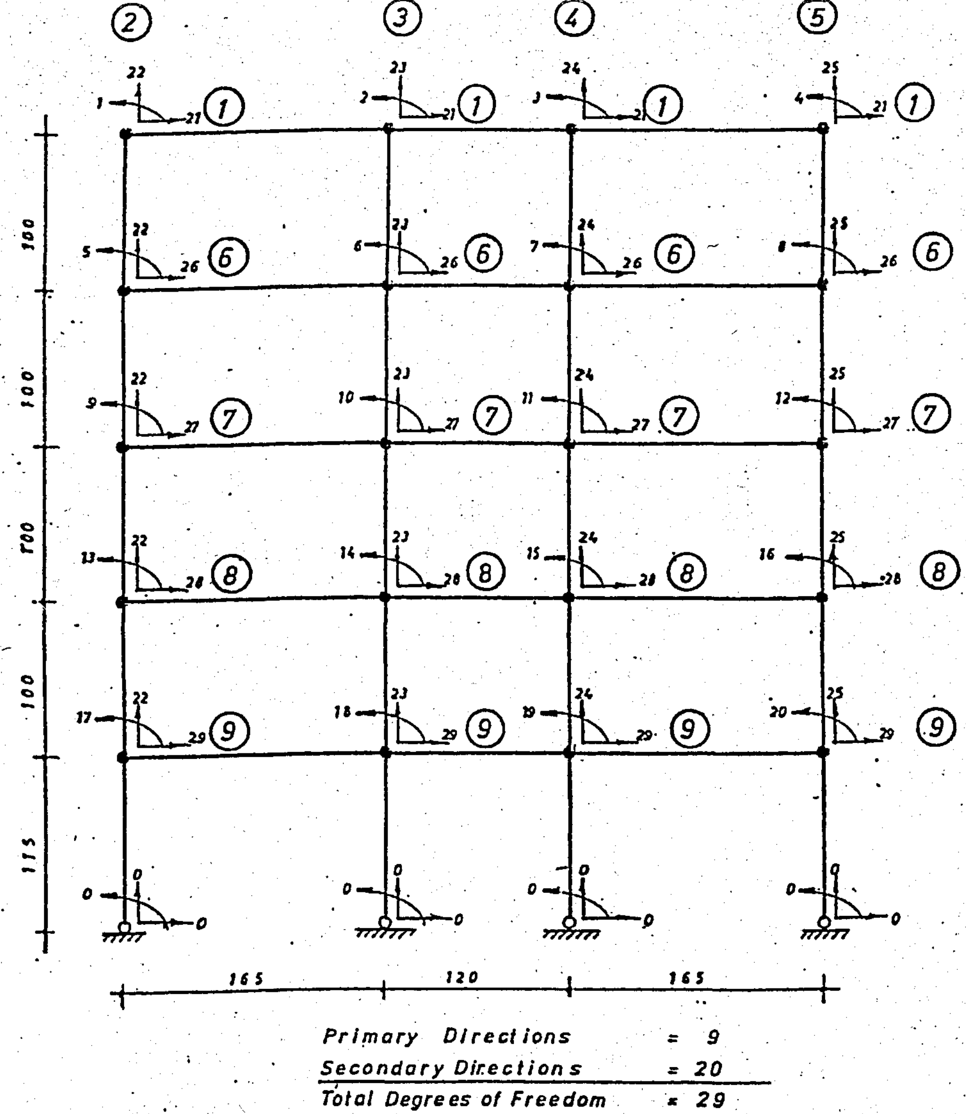


FIG. 5.3 - VIBRATING DIRECTIONS FOR FIXED BASE CONDITION

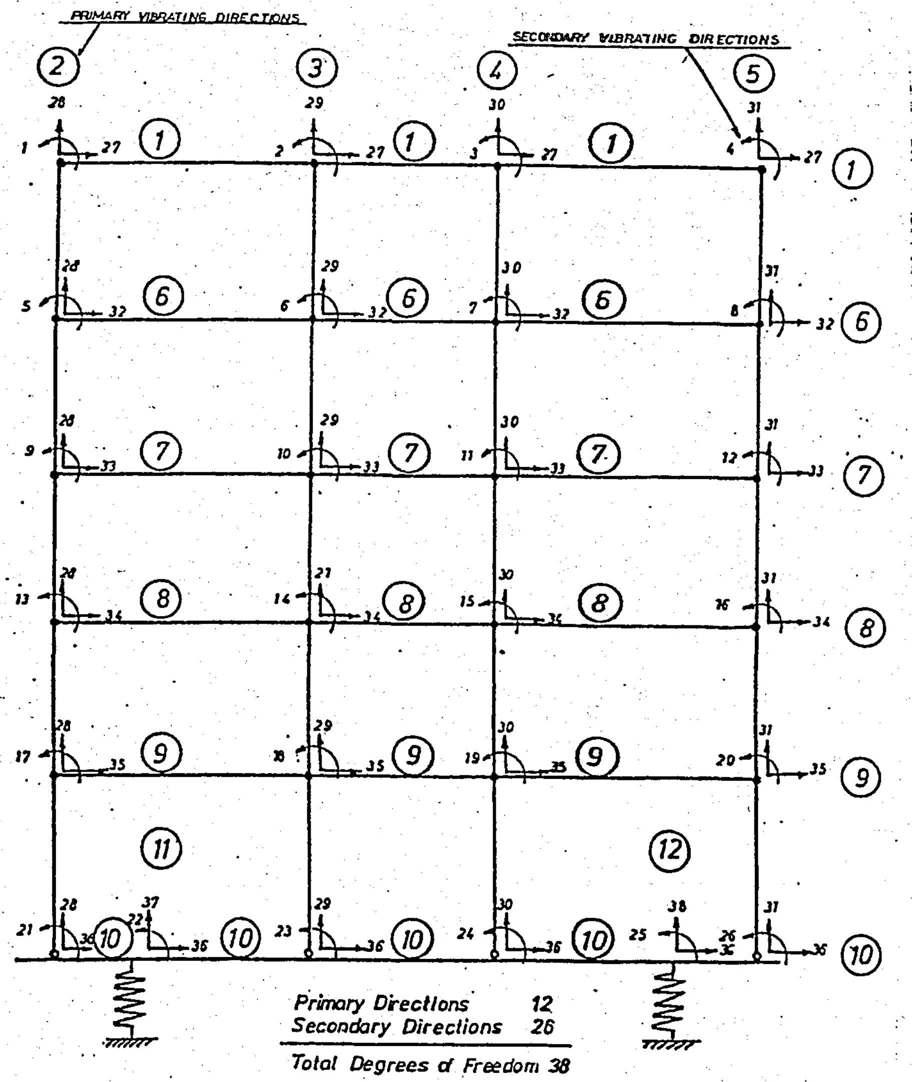


FIG. 5.4 - VIBRATING DIRECTIONS FOR SPRING BASE CONDITION

The c-viscosity coefficients of the viscodampers in the horizontal and vertical directions are both temperature and frequency dependent values, determined at the manufacturing laboratories. The variation of viscosity coefficients of the viscodampers used in the shaking table testing,is given in Fig. 5.5 . It is seen that, the viscosity coefficients are specified normally for 20° temperature and they are very sensitive against temperature changes. They increase by about 15% to 20% for each degree centigrade drop in temperature.

5.4. FREE VIBRATIONS as a RIGID BODY

A) RIGID BODY DISPLACEMENT

For the purpose of determining the fundamental frequencies of vibration of the model frame, a rigid body analysis is performed. One frame of the testing assembly is idealised as a rigid body as shown in Fig. 5.6. The necessary mathematical formulation is derived as follows:

Let us assume that the rigid body moves horizontally by x_1, vertically by y_1, and rotates clockwise by an angle θ.

(a) Any point P moves to a new position P′, due to a pure rotation, as follows (Fig. 5.7):

$(r\theta)\sin\alpha = y\theta$ Horizontal movement (+)

$(r\theta)\cos\alpha = x\theta$ Vertical movement (-)

$P(x;y)$ moves to $P'(x+y\theta\ ;\ y-x\theta)$

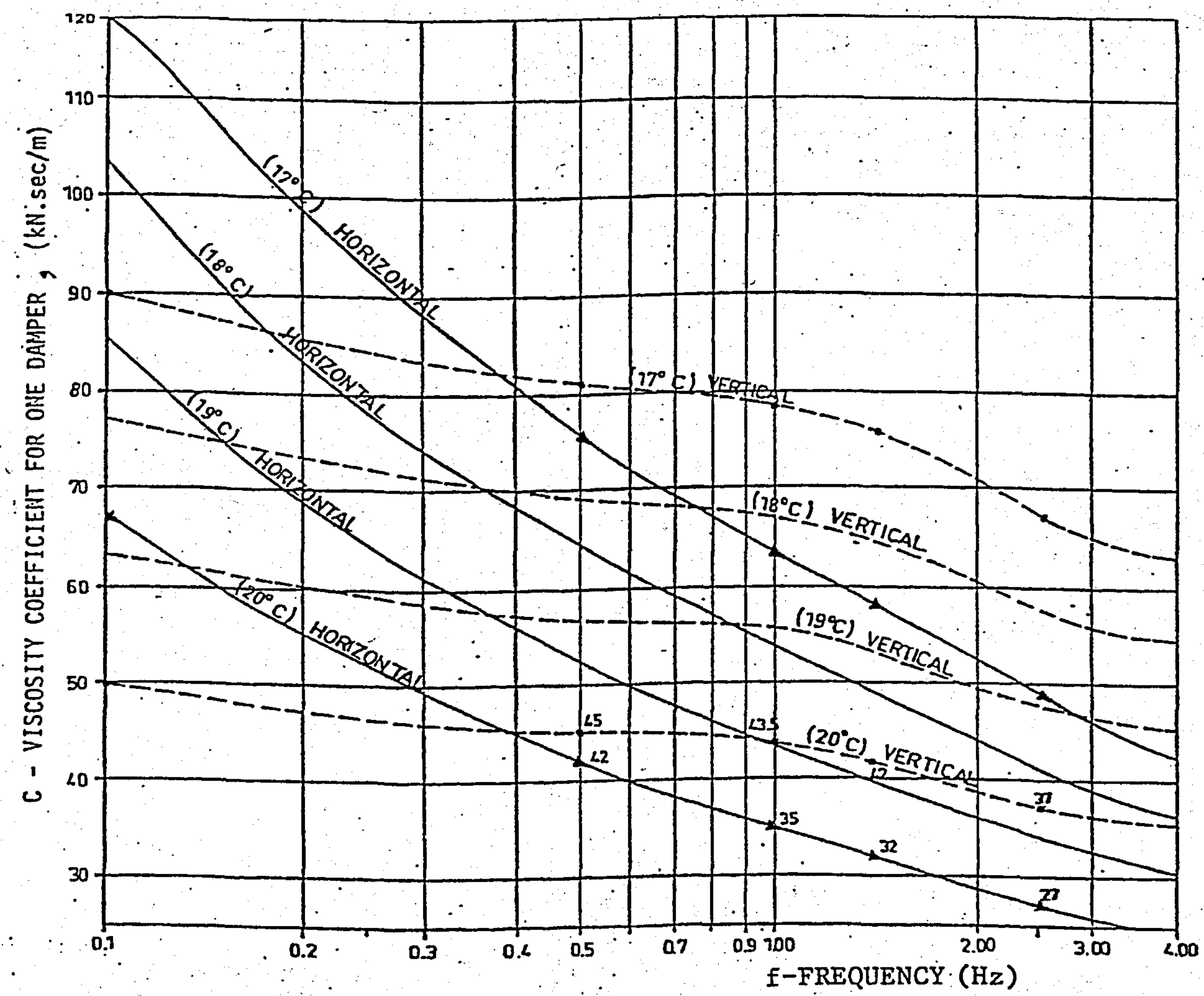


FIG. 5.5 - VISCOSITY COEFFICIENT OF DAMPERS

(b) If, centroid is displaced by (x_g, y_g) and also rotated by an angle θ, P moves to a new position P' as follows:

Old Coordinates	New Coordinates
P_1 (a_1 ; $-h_1$)	P_1' ($a_1 + x_g - h_1\theta$; $-h_1 + y_g - a_1\theta$)
P_2 ($-a_1$; $-h_1$)	P_2' ($-a_1 + x_g - h_1\theta$; $-h_1 + y_g + a_1\theta$)
P_3 (a_2 ; $-h_2$)	P_3' ($a_2 + x_g - h_2\theta$; $-h_2 + y_g - a_2\theta$)
P_4 ($-a_2$; $-h_2$)	P_4' ($-a_2 + x_g - h_2\theta$; $-h_2 + y_g + a_2\theta$)

B) EQUATIONS of MOTION

The total displaced position of the rigid body is shown in Fig. 5.8. Inertia forces and moments are shown purposely in the reverse direction in accordance with *D'Alembert's principle* so that static equilibrium equations can be expressed by direct projections on the respective axes. Thus, the equations of motion become:

$$
\begin{aligned}
& m\ddot{x}_g + 2(x_g - h_2\theta)k_h = 0 \\
& m\ddot{y}_g + (y_g - a_1\theta)k_v + (y_g + a_1\theta)k_v = 0 \qquad \ldots \quad (5.4) \\
& J\ddot{\theta} + (y_g + a_1\theta)(a_1 + h_1\theta)k_v - (y_g - a_1\theta)(a_1 - h_1\theta)k_v \\
& \qquad - (x_g - h_2\theta)(h_2 + a_2\theta)k_h - (x_g - h_2\theta)(h_2 - a_2\theta)k_h = 0
\end{aligned}
$$

where,

$J = \int r^2$

dm = rotational mass moment of inertia

After simplification,

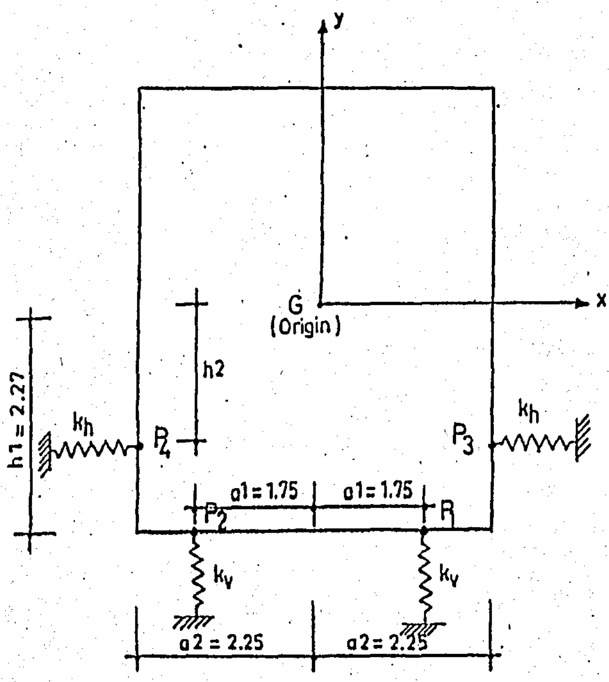


FIG. 5.6 - RIGID BODY IDEALIZATION

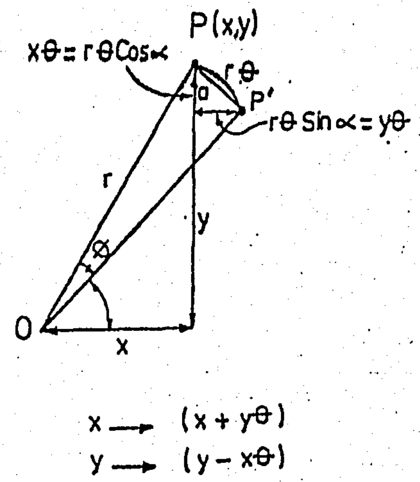


FIG. 5.7 - ROTATION OF THE RIGID BODY

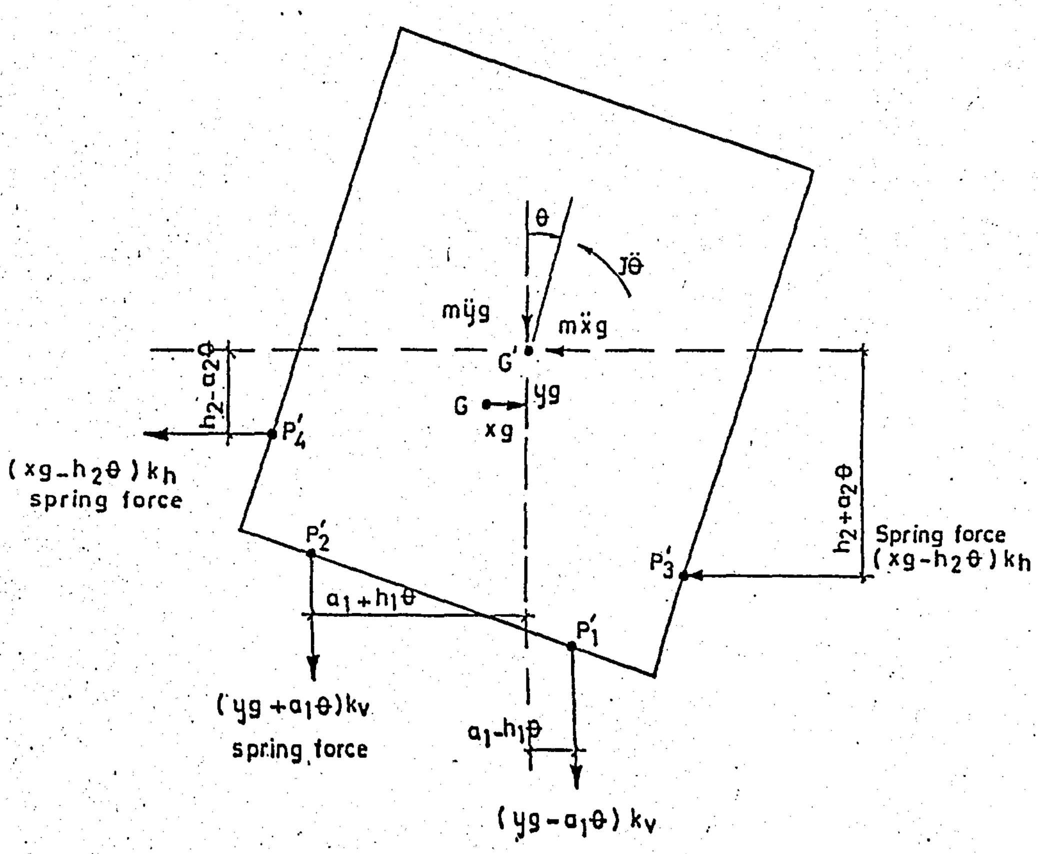


FIG. 5.8 - RIGID BODY DISPLACEMENTS

$$m\ddot{x}_g + 2k_h\,(x_g - h_2\theta) = 0$$
$$m\ddot{y}_g + 2k_v\;y_g = 0 \qquad \ldots \quad (5.2)$$
$$J\ddot{\theta} - 2h_2\,k_h\,x_g + 2h_1\,y_g\theta + 2(a_1^2\,k_v + h_2^2\,k_h)\theta = 0$$

where, the second order term $y_g\theta$ may be neglected.

The second equation is independent of the others. Therefore, in order to allow for easy eigenvalue solution, the first and the third equations may be expressed in matrix notation.

C) EIGEN VALUE PROBLEM

Dynamic equations of motion in matrix notation is,

$$[M]\,\{\ddot{X}\} + [K]\,\{X\} = \{0\} \qquad \ldots \quad (5.3)$$

$$\begin{bmatrix} m & o \\ o & J \end{bmatrix} \begin{Bmatrix} \ddot{x}_g \\ \ddot{\theta} \end{Bmatrix} + \begin{bmatrix} 2k_h & -2h_2k_h \\ -2h_2k_h & 2(a_1^2k_v+h_2^2k_h) \end{bmatrix} \begin{Bmatrix} x_g \\ \theta \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix} \ldots \quad (5.4)$$

After premultyplying by M^{-1}, we obtain the dynamic matrix-U as follows,

$$U = \begin{bmatrix} A = 2k_h/m & B = -2h_2k_h/m \\ C = -2h_2k_h/J & D = 2(a_1^2k_v+h_2^2k_h)/J \end{bmatrix} \qquad \ldots \quad (5.5)$$

introducing, $x = a\sin wt$ as the solution to the above integration,

$$U = \begin{bmatrix} A-\lambda & B \\ C & D-\lambda \end{bmatrix} = (A-\lambda)(D-\lambda) - BC = o \qquad \ldots \quad (5.6)$$

$$\lambda^2 - (A+D)\lambda + (AD-BC) = o \qquad \ldots \quad (5.7)$$

The roots λ are the squares of ω – the angular frequency of motion:

$$\lambda = \omega^2$$

$$T = 2\pi/\omega$$

D) ROTARY MASS MOMENT of INERTIA

By definition,

$$J = \int r^2 dm = \int (x^2 + y^2) dm \qquad \ldots (5.8)$$

Calculations are performed in Table 5.1, for the left-hand side of the frame and then doubled to obtain the complete value of J. The center of gravity is obtained from

$$Y_G = \frac{\Sigma m_i h_i}{\Sigma m_i} \qquad \ldots (5.9)$$

$$m_i = w_i/g$$

where,

w_i = 2.344 ton, i= 1, 4
w_s = 2.308 ton
Y_G = 2.27 m
k_h = 402.8 kg/cm = 40.28 ton/m
k_v = 763.1 kg/cm = 76.31 ton/m

E) ROCKING FREQUENCIES

Substituting these numeirc values in the dynamic matrix,

TABLE 5.1 - ROTARY MASS MOMENT OF INERTIA FOR HALF FRAME

No -	W ton	x m	y m	$(x^2 + y^2)W$
1	0.453	2.25	5.15	14.348
2	0.782	0.60	5.15	21.022
5	0.459	2.25	4.15	10.229
6	0.7935	0.60	4.15	13.952
9	0.459	2.25	3.15	6.878
10	0.7935	0.60	3.15	8.159
13	0.459	2.25	2.15	4.445
14	0.7935	0.60	2.15	3.954
17	0.459	2.25	1.15	2.931
18	0.7935	0.60	1.15	1.335
21	0.371	2.25	0	1.878
22	0.612	1.75	0	1.874
23	0.872	0.60	0	0.523
SUM	8.100	-	-	91.528

$J/2 = 91.528, \quad J = \Sigma W r^2 = 183.056$

TABLE 5.2 - FIXED BASE NATURAL PERIODS

Mode	T = PERIODS, sec		f = FREQUENCIES, Hz	
	CASE 1	CASE 4	CASE 1	CASE 4
1 H	0.2378	0.2879	4.205	3.473
2 H	0.0761	0.0901	13.141	11.099
3 H	0.0429	0.0490	23.310	20.408
4 H	0.0295	0.0324	33.898	30.864
5 H	0.0234	0.0247	42.736	40.485
6 V	0.0175	0.0194	57.143	51.55
7 V	0.0164	0.0180	60.976	55.55
8 V	0.0134	0.0148	74.627	67.57
9 V	0.0133	0.0147	75.188	68.03

$$A = 2k_h/m = 48.709$$

$$B = -2h_2 k_h/m = -110.386$$

$$C = -2h_2 k_h/J = -9.80$$

$$D = 2(a_1^2 k_v + h_2^2 k_h)/J = 47.294$$

$$U = \begin{bmatrix} 48.709 \quad -\lambda & -110.386 \\ -9.80 & 47.294 \quad -\lambda \end{bmatrix} = 0$$

$$x^2 - (A+D)\lambda + (AD-BC) = 0$$

which yields,

$$\lambda^2 - 96.003\lambda + 1221.861 = 0$$

$$\lambda_1 = \omega_1^2 = 15.104$$

$$\lambda_2 = \omega_2^2 = 80.900$$

$$\omega_1 = 3.886 \text{ rad/sec}$$

$$\omega_2 = 8.994 \text{ rad/sec}$$

$$T_1 = 2\pi/\omega_1 = 1.616 \text{ sec}, \quad f_1 = 0.619 \text{ Hz}$$

$$T_2 = 2\pi/\omega_2 = 0.698 \text{ sec}, \quad f_2 = 1.433 \text{ Hz}$$

F) VERTICAL FREQUENCY

$$m\ddot{y}_g + 2k_v y_g = 0$$

$$\omega^2 = 2k_v/m$$

$$\omega^2 = 92.279$$

$$\omega = 9.606 \text{ rad/sec}$$

$$T = 0.654 \text{ sec}$$

$$f = 1.528 \text{ Hz}$$

Considering the whole frame as a rigid body, the first three fundamental frequencies are obtained as follows:

1^{st}	Rocking mode	T= 1.616 sec	f= 0.619 Hz
2^{nd}	Rocking mode	T= 0.698 sec	f= 1.433 Hz
Vertical	Vibration Mode	T= 0.654 sec	f= 1.528 Hz

5.5. FREE VIBRATIONS of the MATHEMATICAL MODEL

Free vibration analyses of the mathematical models given in Figs. 5.3 & 5.4 have been performed using a plane frame dynamic analysis computer program.

The fundamental periods of vibration are summarized in Tables 5.2 to 5.4, for (a) The Fixed Base, and (b) The Spring Base Conditions, and (c) The Rubber Base Conditions, respectively. Free vibration mode shapes for the first two types of support conditions are also illustrated in Figs. 5.9 & 5.10, respectively.

TABLE 5.3 - SPRING BASE NATURAL PERIODS

Mode	CASE 1	COMP.	CASE 2 D8	CASE 2 D4	CASE 3	CASE 4
T = PERIODS, sec						
1	0.9207H	Rocking	1.1373	1.2252	1.4305	1.4712
2	0.6540V	Vertic.	0.6022	0.6268	0.6547	0.6545
3	0.1513H	Rocking	0.4686	0.5091	0.5612	0.5422
4	0.0675H	Tilting	0.0999	0.1004	0.1010	0.0997
5	0.0408H	Horiz.	0.0771	0.0771	0.0772	0.0781
6	0.0289H	Vertic.	0.0522	0.0529	0.0536	0.0548
7	0.0280V	Horiz.	0.0463	0.0463	0.0463	0.0475
8	0.0233H	Horiz.	0.0311	0.0311	0.0311	0.0321
9	0.0159V	Horiz.	0.0239	0.0239	0.0239	0.0247
10	0.0130V	Vertic.	0.0231	0.0232	0.0233	0.0234
11	0.0025V	Vertic.	0.0066	0.0067	0.0067	0.0059
12	0.0025V	Vertic.	0.0061	0.0061	0.0061	0.0054
f = FREQUENCIES, Hz						
1	1.0861H	Rocking	0.8793	0.8162	0.6991	0.6797
2	1.5291V	Vertic.	1.6606	1.5954	1.5274	1.5279
3	6.6094H	Rocking	2.1340	1.9643	1.7819	1.8443
4	14.815 H	Tilting	10.010	9.9602	9.9010	10.030
5	24.510 H	Horiz.	12.970	12.970	12.953	12.804
6	34.602 H	Vertic.	19.157	18.904	18.657	18.248
7	35.714 V	Horiz.	21.598	21.598	21.598	21.053
8	42.918 H	Horiz.	32.154	32.154	32.154	31.153
9	62.893 V	Horiz.	41.841	41.841	41.841	40.486
10	76.923 V	Vertic.	43.290	43.103	42.918	42.735
11	400.00 V	Vertic.	151.52	149.25	149.25	169.49
12	400.00 V	Vertic.	163.93	163.93	163.93	185.19

TABLE 5.4 NEOPRENE BASE NATURAL PERIODS

MODE	COMP.	Periods T, sec.	Frequencies f, Hz
1	Horizontal	1.2121	0.825
2	Rocking	0.1907	5.244
3	Vertical	0.1042	9.597
4	Tilting	0.0798	12.531
5	Vertical	0.0661	15.129
6	Vertical	0.0515	19.417
7	Horizontal	0.0474	21.097
8	Horizontal	0.0321	31.153
9	Horizontal	0.0247	40.486
10	Vertical	0.0234	42.735
11	Vertical	0.0057	175.44
12	Vertical	0.0053	188.68

5.6. RESPONSE ANALYSIS

Response analyses of the frame, to a number of simulated earthquake ground motions, have been also performed using a computer program. A step-by-step integration procedure is used in matrix notation as already explained in Chapter 3. Basically, this procedure is an extension of the method available in the literature for the solution of dynamic analysis of single mass system using linear accelerations in the numerical integration.

The input motion is supplied in the form of digitised data of ground motion accelerations at equal time intervals. The computer program is capable of handling input motions in both horizontal and vertical directions, simultaneously. The structure may be supported externally, in an absolute manner, by any number of helical springs and/or viscodampers in horizontal and/or vertical directions,

The time history of accelerations, velocities and displacements are calculated and printed out at each time station, for each lumped mass point and also in each primary vibrating directions. The proper convergence criterium of the numerical integration required that, normally Δt- the time interval of the input motion ordinates- should be less than $^1/_4$ or $^1/_6$ times of the smallest natural period of vibration of the structure under consideration.

Usually, the smallest natural periods of vibration of the structure are much less than the time interval of the input motion data. In such cases, the computer program, at the opinion of the user, subdivides the input motion data within a particular time interval into newer subdivisions. In the analyses performed, the time interval is divided into 10 to 50 subdivisions as required.

Numerical integration is in fact conducted on all of these internal subdivision points, but the results of the response values are printed out only at the prescribed input data time stations. The computer program selects the absolute maximum acceleration, velocity and displacement value at each prescribed lumped mass point and prints out them together with the time of occurence.

The response analyses of the test frame have been performed for each of the fixed base, the spring base and the rubber base conditions, for a variety of input motion data.

5.7. INPUT MOTIONS

Basically, the ground motion accelerations of two different earthquakes are considered as data for the input motion. The irfst set of data belongs to the May 18, 1940 El Centro, Imperial Valley Earthquake. The horizontal and vertical components of the accelerations are taken from EERL Reports (Anonymous, 1972) and are herewith reproduced in Tables 5.5 & 5.6 . The time history record and also the relative velocity response spectrum curves of the El Centro earthquake are given in Figs. 5.11 to 5.14.

The second set of data belongs to the April 15, 1979 Petrovac, Montenegro, Yugoslavia earthquake, for which the horizontal (N-S) and the vertical components of the accelerations are taken from Naumouski, et. al. (1979) of *The Institute of Earthquake Engineering and Engineering Seismology, IZIIS,* Skopje, Yugoslavia. The corrected accelerations for the first 20 seconds, are given in Tables 5.7 & 5.8. The time history diagram of the data as well as its absolute acceleration, relative velocity, and the relative displacement response spectra for the horizontal component are given in Fig. 5.15 to 5.19.

The time interval for the input motion data points is normally specified to be $\Delta t = 0.02$ sec. (Real Time Interval). In order to investigate the influence of frequency content of the input motion on the response analyses however, the time interval is scaled down, for various cases, by a factor of 2, and it is taken as $\Delta t = 0.01$ seconds (scaled time interval).

The amplitudes of the input motion are also modified by means of a variable factor, which determines the intensity of a particular earthquake. The amplitudes are varied during the shaking table testing by means of the SPAN feature. In the analytical calculations, the peak acceleration value corresponding to a particular SPAN, is used to determine the multiplication factor for the real earthquake data.

5.8. COMPUTER RUNS

For the fixed base condition, the response of the mathematical model to a variety of input ground motion data, is determined by means of numerical integration. A critical damping ratio of $\beta = 1.5\%$ is assumed to exist in the structure.

For the spring base condition however, the response analyses for all types of input motion, have been repeated for four different natural periods of vibration. Since, the exact natural periods of vibration of the testing frame are not accurately measured at the shaking table, the first fundamental period of vibration of the frame is assumed to possess (a) T= 0.91 sec., (b) T= 1.14 sec for D8 case, and T=1.23 sec for D4 case, (c) T=1.43 sec, and (d) T=1.47 sec. The response calculations are repeated for each of these natural period cases.

Structural properties and the viscosity coefficients in these four cases of natural periods are somewhat different from each other. The basic differences in the structural and damping data are summarized in Table 5.9.

The list of "Runs" conducted for the El-Centro and Petrovac earthquakes, for which the laboratory testings are also done, has already been given in Chapter 4. The peak accelerations and displacements calculated at each floor level and also at the joints located along the base beam for some cases of comparative importance are given in Tables 5.10 to 5.19.

5.9. INFLUENCE of VISCODAMPERS

In order to determine the optimum amount of viscodamping, the response analysis for the run D8 EN NS 60, corresponding to the horizontal component of the 1940 El Centro earthquake, is repeated many times by varying only the horizontal and vertical coefficients of viscosity.

The peak values of the horizontal floor accelerations, velocities and displacements are given in Tables 5.20 to 5.22 respectively, for a variety of viscosity coefficients. It is seen that, when there is no viscodamping, all peak response values are maximum. As soon as the viscodamping coeffifients start increasing, the peak response values, especially the displacements, become smaller.

Although, this efficiency of the viscodampers continue to exist for a good range of increase in the viscosity coefficients, there is however, an optimum level of damping, beyond whoch the peak response values no longer become smaller. On the contrary, they start becoming larger and larger due to the analytical complexities existing in the high amount of damping.

The critical damping ratio, β, may be related to the c- viscosity coefficient approximately by the following expression:

$$\beta = \frac{cgT}{4\pi W}$$

in which,

T = fundamental period of vibration,
W = total weight of the structure.

For instance, in D4-case, considering a viscosity coefficient of

$$c_v = 30 \text{ kg. sec./cm}$$

the critical damping ratio becomes

$$\beta = \frac{2 \times 30 \times 981 \times 1.10}{4 \times \pi \times 16200} = 0.32$$

Basically the viscodampers reduce the peak response values. Especially, the displacements are reduced with the increase in magnitude of viscosity coefficient. It is possible however, that the additional amount of damping may cause an undesirable increase in the magnitudes of accelerations. The actual reduction in acceleration occurs as a result of modification of the fundamental period of vibration of the structure.

As an example, the reduction in the horizontal acceleration response of the 5th floor of the test frame on account of vibration isolation is illustrated in Figs. 5.20 to 5.23. It is seen that when no vibration isolation is implemented the peak response is $a = 1185$ cm/sec², and it is subsequently reduced to $a = 224$ cm/sec² (reduction by about five times), when vibration isolation is used in connection with four viscodampers.

5.10. DISCUSSION of ANALYTICAL RESULTS

The acceleration response curves (Fig. 5.20 to Fig. 5.23) show in general that the base isolated systems lead to much reduced horizontal and vertical accelerations compared with the fixed base model. The displacements, on the other hand, are observed to be usually larger in the case of isolated models.

It is interesting to note that the increased damping may sometimes lead to slightly higher accelerations than the lower damping. Starting from relatively smaller critical damping ratio values (on the order of 0% to 5% critical), when there is an increase in the viscosity coefficient there is a parallel decrease in both the peak response accelerations and displacements. A further increase in viscodamping beyond an optimum level however, although effective in reducing displacements, may result in an increasing trend in peak accelerations.

In fact such a trend is clearly observed in the analytical investigations performed. As an example to increase in peak accelerations due to increase in viscodamping, the test Run No. 20 and No. 46 may be compared. The peak roof acceleration is a= 224 cm/sec^2, when 4 viscodampers are used. This response is increased to a= 286 cm/sec^2, when the viscodampers are doubled in number.

It is also observed that the level of reduction of the acceleration response is much higher when the input motion time interval is scaled down for both 1940 El Centro and Petrovac earthquakes. This may be observed clearly in Figs. 5.20 and 5.21 for the El Centro earthquakes, one with real t²me and the other with scaled time. In this way, the effect of the frequency shift is analyzed and it is concluded that the reduction in acceleration response is very much related to the natural frequencies of the structure and to the frequency content of the input motion.

Pure vertical earthquake inputs produced negligible horizontal accelerations and displacements in the spring isolated model. It is evident that, in this case only the vertical mode of vibration contributed to the response. The input accelerations is not amplified, on the contrary a reduction by a factor of two generally is observed for both El Centro and Petrovac earthquakes.

In the case of spring isolated system the horizontal displacements grow from base to top of the model similar to the fixed base model with very small horizontal deflections at the base level. The only difference however is that in phase with the horizontal deflections, the test model is also vertically displaced. A tilting motion, coupling horizontal translation and pure rocking with the rocking center near to the base level is the reason for horizontal amplitudes growing with height. Hence, the structure is displaced almost as a rigid body with little or no relative storey displacements thus creating negligible internal stresses.

It is finally observed that the results for the concurrent actions of the horizontal and vertical earthquakes may be derived simply through linear superposition of the independent results of separate excitations.

TABLE 5.5 - EL CENTRO SOOE COMPONENT HORIZONTAL ACCELERATIONS

IIA001 40.001.0 IMPERIAL VALLEY EARTHQUAKE MAY 18, 1940 - 2037 PST EPICENTER 32 44 00N,115 27 00W
STATION NO. 117 EL CENTRO SITE IMPERIAL VALLEY IRRIGATION DISTRICT COMP SOOE 32 47 43N,115 32 55W
INSTR PERIOD = 0.099 SEC DAMPING = 0.552 ACCELEROGRAM IS BAND-PASS FILTERED BETWEEN 0.070 AND 25 CYC/SEC.

PEAK VALS ACLN = 341.7 CM/SEC/SEC AT 2.12 SEC VELO = 33.4 CM/SEC AT 2.18 SEC DISP = 10.9 CM AT 8.58 SEC

INITIAL VELO = -4.66421 CM/SEC INITIAL DISP = 2.15852 CM HORIZONTAL

2688 INSTRUMENT AND BASELINE CORRECTED DATA IN MM/SEC/SEC AT EQUALLY-SPACED INTERVALS OF 0.02 SEC

-14	-108	-101	-88	-95	-120	-142	-128	-110	-85	-85	-131	-176	-194	-162	-144	-108	-82	-42	-66
-131	-190	-196	-66	30	141	-49	-128	-144	-203	-260	-325	-306	-172	-197	-163	-164	-67	25	150
236	252	336	463	492	419	359	271	235	339	412	530	639	732	652	599	400	400	63	-515
-787	-603	-484	-250	-59	134	308	499	710	995	1219	1529	1449	1155	935	892	926	839	901	993
1209	328	-1475	-2066	-1989	-2034	-1816	-1725	-1752	-1753	-1805	-1630	-1347	-1087	-782	-429	-17	360	785	1164
1598	1960	2412	2729	3036	3200	3417	2821	2324	-1198	-2373	-1640	-1865	-1095	-753	-173	113	533	895	1186
1757	576	-2631	-1547	-1729	-1012	-579	237	-670	-1980	-1641	-1685	-1481	-1231	-1001	-751	-523	-271	-44	188
-95	-433	-838	-951	-716	-599	-334	-108	185	420	673	-97	-372	-40	11	344	565	883	1130	1363
219	241	683	689	1318	1353	2040	-931	-1308	-692	-546	72	675	-1067	-1488	-1071	-1162	-762	-559	-215
-126	-674	-324	-337	-109	17	299	488	608	222	-32	-245	77	211	568	926	1206	1478	1737	421
29	259	293	-55	-147	143	206	499	645	957	1128	1447	1629	1945	1856	1984	1769	1250	-1207	-542
-384	-311	-1118	-1661	-2464	-2025	-1835	-1317	-960	-325	154	816	1319	1818	-58	-169	285	-447	983	1424
1853	2456	1685	-1380	-999	-1289	-907	-469	-1250	-2111	-1617	-1692	-1306	-1111	-773	-510	-544	-1200	-1209	-1158
-1145	-717	-546	64	-804	-1634	-859	-961	-396	-147	319	648	876	472	198	-27	292	445	785	1033
1352	1606	1861	1281	640	204	314	373	496	235	-84	-168	-113	-229	-248	-157	-69	147	379	579
255	-41	-428	-133	95	230	-129	-50	80	210	380	510	157	-32	-111	5	76	35	-95	-36
-16	38	85	-56	-304	-421	-244	-236	-177	-129	-18	203	-108	-91	-34	-106	-111	-99	-2	73
235	355	705	779	184	-263	-124	-42	159	48	-219	-467	-428	-216	-43	159	320	419	123	-160
-204	-82	-206	-137	-55	53	134	266	232	79	-8	200	435	492	191	92	-22	-21	52	93
255	368	525	541	425	398	559	756	365	411	98	-204	-249	-405	-413	-471	-433	-458	-57	178
-208	-492	-530	-362	-405	-303	-316	-265	-265	-269	-345	-309	-217	-78	87	281	310	358	341	358
287	305	112	214	136	384	-861	-1349	-1342	-1354	-1193	-1042	-829	-651	-444	-258	-60	-91	-182	-147
85	163	50	264	582	867	1200	1695	1111	-1100	-366	-445	-236	-960	-656	-597	-670	-552	-27	378
1072	1669	947	408	667	132	-95	-520	-927	-1152	-1150	-803	-369	29	545	1178	1610	-270	34	-56
20	146	537	798	-205	-593	-169	-175	-28	74	382	567	753	801	592	304	23	64	-406	-451
-79	168	567	93	-55	44	-123	-282	-437	-352	-255	-111	205	519	854	1144	733	237	-368	-271
-217	-873	-973	-589	-336	77	-259	508	361	81	-56	-209	-317	-238	-376	-550	-722	-803	-523	-340
-11	65	-37	-5	-168	-419	-80	79	374	615	665	254	-57	-474	-356	-243	-48	126	379	241
-227	-428	-679	-661	-590	-513	-408	-309	-266	-541	-628	-908	-1107	-881	-770	-582	-473	-333	-199	20
211	432	613	767	933	1066	1130	1187	1247	1334	1594	1797	2037	1236	442	-140	-666	-555	-693	-984
-1246	-1179	-1050	-920	-743	-809	-850	-860	-863	-873	-868	-885	-537	52	215	245	580	314	236	485
589	525	355	197	199	492	343	288	432	239	88	77	-148	-77	-19	75	44	-145	-316	-241
-28	182	426	439	512	466	479	193	222	274	393	504	577	588	822	797	949	-345	45	-123
-347	-426	-416	-275	-270	74	428	-231	-387	-83	139	445	27	-697	-796	-251	-135	79	-115	-251
-333	-269	-301	-200	-67	-38	105	296	344	957	898	179	-362	-994	-807	-744	-539	-330	-128	31
148	508	-22	-489	-358	-691	-516	-371	88	632	841	1276	1388	1193	751	225	-88	-227	74	181
544	399	45	-82	-185	-20	6	-117	-210	-303	-512	-727	-579	-266	-178	40	98	137	221	437
91	-548	-555	-243	-81	250	410	182	-27	-243	-15	247	482	783	622	331	-14	-195	-247	-212
-110	50	241	-34	-216	-471	-363	-195	-18	170	-80	5	230	374	601	516	432	344	505	653
683	172	-170	-527	-664	-387	-222	-33	119	-128	-351	-514	-335	-218	-12	142	70	-63	-120	-322

TABLE 5.6 - EL CENTRO VERTICAL COMPONENT ACCELERATIONS

IIA001 40.001.0 IMPERIAL VALLEY EARTHQUAKE MAY 18, 1940 - 2037 PST EPICENTER 32 44 00N,115 27 00W
STATION NO. 117 EL CENTRO SITE IMPERIAL VALLEY IRRIGATION DISTRICT. COMP VERT 32 47 43N,115 32 55W
INSTR PERIOD = 0.0950 SEC DAMPING = 0.574 ACCELEROGRAM IS BAND-PASS FILTERED BETWEEN 0.070 AND 25 CYC/SEC.

PEAK VALS ACLN = -206.3 CM/SEC/SEC AT 0.98 SEC VELO = -10.8 CM/SEC AT 3.26 SEC DISP = -5.6 CM AT 3.42 SEC

INITIAL VELO = 2.97210 CM/SEC INITIAL DISP = 1.03927 CM VERTICAL

2690 INSTRUMENT AND BASELINE CORRECTED DATA IN MM/SEC/SEC AT EQUALLY-SPACED INTERVALS OF 0.02 SEC

24	-230	-275	-397	-390	-60	409	209	-683	-647	91	134	317	738	600	-531	-261	-130	-435	-690
-117	260	840	1224	-232	-1371	-727	-381	394	771	356	179	-171	-58	-194	229	903	-80	-1112	-699
-188	577	1077	1724	-36	-968	666	1173	-1728	$\boxed{-2063}$	-555	-431	936	220	-1836	-1837	-419	606	1722	-1410
-440	-267	519	875	-35	-809	-98	278	1693	421	-791	-664	-279	-423	-261	204	521	-399	-56	236
781	455	-589	-315	215	743	1242	-527	-133	-398	-641	18	946	52	-1522	-489	-99	543	-66	-219
-10	-121	-529	-426	-73	227	314	-606	-171	149	241	157	509	103	-289	-144	-241	-211	239	-867
-780	-263	22	599	-705	-898	-227	-66	785	31	-366	-214	-807	-106	302	1060	679	-1136	-493	441
1126	285	135	28	-244	-186	81	74	32	120	-187	65	347	419	216	-26	-531	-675	-242	-34
-230	-1021	-656	28	748	1645	1904	-1163	136	935	1347	772	515	-49	130	-215	-202	-369	-114	-48
156	270	78	-188	-9	-681	-425	143	406	-474	-761	-324	87	623	1019	-84	-521	-235	347	328
55	367	218	-404	-413	-21	241	-223	-5	609	-65	-556	19	495	23	-740	-149	-54	178	-165
-159	86	442	57	-260	113	437	285	34	-134	-60	37	53	-112	-118	-6	-43	-161	151	464
415	10	-308	-69	185	355	165	204	-109	-43	432	423	-36	-154	55	-79	-374	175	509	-831
-672	-233	142	596	-74	85	476	56	-683	-234	361	130	-13	-98	166	184	178	-124	-328	54
495	363	193	139	-460	-450	383	103	-820	-86	63	154	289	97	-448	-201	341	677	2	-774
-584	-157	471	750	-683	-660	153	278	447	173	-590	-739	-218	471	809	144	-637	-304	43	733
548	-619	-341	-6	457	802	387	-94	366	45	-866	14	736	470	-237	-426	-86	388	-307	-1233
-348	341	1100	-388	-1067	-520	-183	539	425	-304	-281	-22	548	271	75	-313	15	-348	-311	46
294	86	492	156	-505	198	399	220	-52	-632	-210	-19	-228	-585	-593	-75	-211	-117	115	651
221	-758	-760	-116	573	548	-665	-57	495	162	-72	-486	-40	183	27	146	-153	-388	-742	-540
410	306	-80	37	-197	-13	254	113	56	-52	-413	-161	245	-185	188	388	500	-20	-189	90
498	464	-40	-434	-210	50	398	588	85	-351	-206	90	259	153	45	-92	-21	87	51	69
-237	-34	577	612	51	-290	-564	-348	174	72	-522	187	77	-483	-102	514	-957	674	546	-68
9	-180	-86	239	-20	-50	-98	244	172	265	188	127	0	-78	-161	87	-213	-41	336	230
202	147	88	-55	-147	-36	-323	-274	196	288	-81	-231	158	197	-435	-740	-625	-491	-136	313
323	-39	-147	-39	149	125	-111	-325	-345	-214	-70	15	76	-47	-239	-182	-46	157	14	122
219	562	-100	-459	262	19	-314	12	242	98	-236	-192	129	318	-318	-343	84	93	303	249
-126	-149	81	53	22	26	16	25	312	8	-264	-109	-328	-178	127	403	177	-153	-456	-268
112	105	-118	77	122	115	-107	-58	252	378	32	-162	410	245	-257	973	491	217	-109	-487
-615	-421	-349	-377	-452	-415	-415	-363	-356	-276	-272	-56	165	21	522	246	-9	-88	28	30
18	193	327	528	515	253	-66	-64	69	61	106	50	25	32	-53	88	153	-232	-15	209
185	36	338	266	88	-160	0	324	-75	-105	226	184	-354	-210	155	242	-157	-117	-109	-196
-228	52	241	104	167	211	252	208	289	-151	-168	410	2	-159	-242	-119	-149	-149	-140	-5
33	64	51	-106	-337	-122	73	464	387	-103	-158	-175	-162	-197	91	325	505	-379	-98	428
386	-143	-114	-13	108	67	-31	-74	-213	-247	-226	-185	120	-13	-107	-41	-11	18	-12	-311
-367	-451	-454	-554	-119	192	-7	70	-104	-133	-42	-140	-302	-217	-91	-41	162	337	82	-167
-216	79	67	48	87	102	91	135	27	-26	113	213	180	-57	33	229	237	112	-139	-275
-22	42	186	223	210	87	177	128	-61	-171	-105	49	-25	-107	-56	-119	-30	107	103	-16
-144	-79	245	191	-87	-241	-156	-122	221	90	143	193	194	11	-77	-142	-130	-59	5	-77
-15	-32	139	95	-173	63	317	138	153	103	157	165	70	35	-168	-163	-179	109	206	123

TABLE 5.7 - PETROVAC EARTHQUAKE HORIZONTAL ACCELERATIONS

RECORD NO IIIE58,N-S
PETROVAC, 1979-04-15,06-20
NUMBER OF DATA = 2413

MAX. ACCEL. = -4273 (8.04 SEC)
ACCEL. UNIT = MM/SEC**2
TIME INTERVAL = 0.02 SEC

TIME	0.00	0.02	0.04	0.06	0.08	0.10	0.12	0.14	0.16	0.18
0.0	34	49	31	38	-13	-42	-56	-1	-17	-62
0.2	-11	21	-58	-53	-8	-46	-82	-47	16	34
0.4	-11	-78	-91	-46	-4	-4	-66	-21	47	7
0.6	37	55	-4	-33	-39	-59	-23	28	-8	-61
0.8	-116	-109	4	40	-5	4	-44	-48	-37	13
1.0	45	31	31	-9	-73	-59	-33	-52	-23	-8
1.2	-51	-31	-13	-67	-24	-51	-59	23	36	29
1.4	37	-4	-30	-31	-24	-119	-149	-111	-26	79
1.6	95	16	-27	-95	-96	-41	6	20	41	16
1.8	-2	-26	-2	-8	16	21	4	52	43	-102
2.0	-169	-207	-200	-97	-41	-94	-31	-30	-36	75
2.2	137	226	296	243	191	225	61	-144	-287	-432
2.4	-341	-200	-157	-38	31	86	107	125	102	161
2.6	261	192	52	-35	-23	-124	-135	-73	-71	75
2.8	72	84	112	-30	-64	1	75	56	30	59
3.0	-14	-35	-93	-60	22	138	16	66	248	241
3.2	106	-29	-103	-150	-159	-192	-63	95	146	99
3.4	103	191	144	141	209	154	145	212	126	2
3.6	-101	-159	-66	27	-1	39	148	110	-11	-48
3.8	-1	111	114	115	198	186	156	19	-37	21
4.0	-83	-255	-278	-161	-36	10	58	121	284	311
4.2	128	-54	-103	-86	27	180	180	32	-215	-322
4.4	-247	-31	175	280	201	54	48	135	101	-31
4.6	-104	-73	40	30	52	178	221	128	-79	-240
4.8	-214	-6	229	272	67	-154	-393	-196	56	199
5.0	423	474	219	-53	-119	-187	-117	231	330	139
5.2	163	113	78	219	146	-10	-30	34	334	123
5.4	-206	-304	-345	-172	-92	-604	-1252	-1669	-1674	-1596
5.6	-1407	-869	-443	-250	75	50	-14	364	926	1603
5.8	1893	1929	2108	2070	1653	942	213	-458	-1240	-1780
6.0	-1951	-1613	-1039	-727	-863	-1100	-1198	-1335	-1713	-2109
6.2	-2367	-2090	-1196	-209	222	176	-198	-748	-1015	-328
6.4	782	1582	1955	2647	3310	3733	3985	3353	2037	588
6.6	-945	-2033	-2401	-1958	-1955	-1917	-2037	-2584	-2639	-2339
6.8	-1999	-1243	-82	961	1972	2699	3004	3264	3286	2973
7.0	2652	2241	1584	498	-450	-681	-373	-75	174	130
7.2	-91	-171	-119	-325	-612	-777	-347	-603	-239	-246
7.4	-816	-1462	-1954	-2527	-2936	-3045	-2769	-2367	-1906	-1421
7.6	-655	337	1501	2634	3389	3506	3460	3454	3459	3303
7.8	2911	2586	2288	2110	1905	1470	793	73	-824	-1912
8.0	-3219	-3968	$\boxed{-4273}$	-4206	-3343	-2406	-966	201	1122	1809
8.2	2363	2385	1971	1745	1743	1623	1308	795	-192	-1185
8.4	-1636	-2195	-2929	-2813	-3093	-2836	-2090	-884	-85	513
8.6	1407	1755	1811	1861	1701	1470	1581	1644	1198	907
8.8	231	-558	-1087	-1485	-1962	-2199	-1744	-758	8	778
9.0	1151	1256	1188	905	426	89	100	219	98	-6
9.2	29	-273	-820	-1123	-1201	-1173	-891	-629	-514	-501
9.4	-659	-905	-1392	-1671	-1882	-1486	-254	886	1462	1931
9.6	2062	1871	1582	1668	1495	1296	1322	1330	1252	1152
9.8	1228	1500	1040	465	35	-401	-537	-836	-886	-947

TIME	0.00	0.02	0.04	0.06	0.08	0.10	0.12	0.14	0.16	0.18
10.0	-877	-761	-843	-1144	-1331	-1546	-1405	-1184	-982	-805
10.2	-882	-749	-1056	-1311	-1105	-844	-412	-155	190	540
10.4	871	1148	1344	1413	1781	2015	1837	1090	-77	-914
10.6	-1424	-1871	-2040	-2222	-2335	-2368	-1840	-1426	-1137	-697
10.8	-224	149	465	765	871	1086	1444	1800	2233	2359
11.0	2296	2414	2628	2925	3087	3109	2712	1738	570	-362
11.2	-1132	-2055	-3100	-4050	-4231	-3039	-2264	-1539	-664	118
11.4	201	569	929	536	-108	-602	-852	-1015	-964	-667
11.6	-655	-632	-136	190	80	-150	-302	-338	59	364
11.8	651	798	816	948	1564	2134	2514	2494	2068	1451
12.0	743	64	-555	-1224	-1554	-1411	-823	-273	-554	-1108
12.2	-1277	-1389	-1429	-1293	-725	-202	473	1354	2434	3388
12.4	3755	3142	2079	765	-385	-1176	-1663	-2060	-2760	-3545
12.6	-3054	-2132	-1042	-305	249	676	814	757	705	937
12.8	1297	1535	1654	1398	959	763	417	-5	-454	-822
13.0	-1147	-1326	-1232	-1003	-392	188	715	1064	1110	906
13.2	459	-79	-422	-623	-1028	-1407	-1731	-1452	-943	-667
13.4	-243	-7	285	579	774	1000	1072	964	932	1073
13.6	1077	836	383	-108	-481	-684	-973	-1382	-1555	-1296
13.8	-881	-771	-699	-294	139	374	635	990	1482	1859
14.0	1907	1861	1624	892	120	-355	-578	-822	-1257	-1526
14.2	-1168	-972	-1116	-832	-642	-652	-542	-108	128	174
14.4	-34	9	347	706	698	746	921	753	602	504
14.6	425	457	501	[illegible]	538	564	429	148	-207	-625
14.8	-994	-1180	-1440	-1732	-2059	-1742	-1515	-1322	-643	95
15.0	766	1173	1477	1841	2179	2466	2645	2524	2197	1582
15.2	655	-144	-886	-1423	-1983	-2927	-3062	-3565	-2977	-2565
15.4	-1842	-1064	-460	-220	-8	479	1049	1608	2102	2446
15.6	2851	3168	2944	2308	1440	595	-123	-594	-1015	-1386
15.8	-1578	-1612	-1538	-1375	-1074	-899	-800	-448	-238	-138
16.0	7	64	332	529	574	457	381	411	435	333
16.2	234	265	361	503	566	312	28	-91	-163	-244
16.4	-270	-258	-75	217	443	497	467	109	-259	-332
16.6	-408	-524	-637	-719	-629	-568	-483	-457	-532	-340
16.8	97	418	554	706	[illegible]	929	861	702	508	705
17.0	-130	-407	-505	-635	-779	-797	-645	-282	78	247
17.2	424	600	701	626	428	113	-183	-263	-125	62
17.4	219	290	288	154	13	-151	-329	-438	-404	-155
17.6	-7	-91	-219	-211	-241	-347	-503	-636	-738	-784
17.8	-565	-56	392	710	1003	1093	915	818	715	578
18.0	411	304	142	-45	-227	-342	-398	-495	-656	-794
18.2	-929	-989	-854	-692	-575	-425	-113	324	-379	769
18.4	947	1209	1322	1278	1041	595	186	-10	-114	-198
18.6	-315	-462	-599	-748	-893	-946	-811	-468	-161	94
18.8	362	532	637	565	339	206	62	-39	-104	-79
19.0	19	184	268	199	61	-83	-203	-266	-111	189
19.2	291	137	-134	-288	-344	-319	-261	-316	-332	-152
19.4	57	96	43	-13	-131	-227	-322	-227	-71	64
19.6	402	121	167	98	47	139	264	310	403	[illegible]
19.8	508	344	387	118	-301	-486	-608	-700	-794	-868

TABLE 5.8 - PETROVAC EARTHQUAKE VERTICAL ACCELERATIONS

RECORD NO. IIIE58.VERT.
PETROVAC.1979-04-15,06-20
NUMBER OF DATA = 2413

MAX. ACCEL. -1976 6.75 SEC
ACCEL. UNIT = MM/SEC**2
TIME INTERVAL = 0.02 SEC

TIME	0.00	0.02	0.04	0.06	0.08	0.10	0.12	0.14	0.16	0.18
0.0	-27	-58	35	26	-4	78	3	-75	-12	25
0.2	17	70	30	-4	-10	28	-3	-81	-7	6
0.4	46	-1	-47	25	117	28	-37	-14	-37	51
0.6	33	-89	-41	70	21	-48	-9	-48	-6	49
0.8	-48	-52	32	25	45	-42	-56	45	126	-55
1.0	-18	36	18	-108	-69	-23	27	62	-42	-59
1.2	-123	-122	84	-7	-22	137	21	-168	-43	102
1.4	22	53	11	-120	-9	-48	-204	2	78	-53
1.6	-57	18	-16	26	-26	19	87	36	-9	-30
1.8	-22	-174	-75	33	-116	-30	-19	-10	-12	-19
2.0	-204	86	89	-144	-112	-112	37	183	-4	-273
2.2	85	118	30	-45	-112	69	105	3	-70	-77
2.4	15	-109	-149	-36	34	223	-100	-144	161	142
2.6	-119	84	199	-122	-53	88	-130	90	66	-77
2.8	-41	43	-78	-92	78	75	174	131	112	76
3.0	12	-143	-197	-35	88	-61	119	166	6	17
3.2	94	-53	-182	44	24	-202	-134	15	35	182
3.4	113	48	204	138	-106	-6	33	-100	-83	-1
3.6	-1	154	143	-130	15	94	46	-127	-67	-10
3.8	-81	29	-87	33	167	154	25	-65	-10	406
4.0	-243	-177	-181	-156	109	146	210	305	116	-139
4.2	-166	86	255	205	388	384	-65	-360	-103	-35
4.4	-197	-293	61	159	-71	-232	91	254	-7	-156
4.6	90	72	89	-32	-390	-400	-477	-401	-471	62
4.8	384	4	-88	174	-121	83	229	8	-48	53
5.0	131	-274	-189	-3	-248	-99	262	210	34	83
5.2	-89	-640	-252	518	78	-157	207	158	177	-243
5.4	-306	23	311	482	175	327	-74	-593	-422	-468
5.6	-469	38	257	-235	1	299	-120	-448	323	905
5.8	382	307	420	-27	-216	-102	56	569	418	-37
6.0	-220	-269	-699	-454	-343	-178	-90	-533	-555	-289
6.2	-111	38	216	556	473	101	-113	126	320	50
6.4	211	469	423	872	413	-67	254	223	-269	-169
6.6	-537	613	208	-287	-1084	-1217	-961	-1622	-1978	-1801
6.8	-1098	-518	310	521	699	874	1109	651	1400	1319
7.0	-667	742	778	552	380	176	81	-347	-119	242
7.2	-18	-526	-273	9	-357	-317	335	-9	-411	-697
7.4	-722	-311	-300	-119	-395	-231	71	-267	-738	-532
7.6	-269	153	232	-417	-287	390	108	-31	-133	314
7.8	704	693	1119	785	446	239	130	-106	-365	-728
8.0	-426	15	126	-201	84	245	229	-337	-781	-150
8.2	1035	1336	1021	659	301	-524	-828	-18	217	-170
8.4	81	-207	-533	-1135	-483	-235	539	-32	-640	-452
8.6	-307	-819	-444	295	251	-394	591	975	583	124
8.8	-193	-323	287	-115	-272	-774	-167	-182	-472	-672
9.0	-720	-470	-108	499	742	427	496	634	818	615
9.2	89	-60	-557	-837	-548	285	553	610	1141	338
9.4	-548	-382	32	37	307	708	597	101	-44	-94
9.6	-333	-79	233	136	52	-102	-451	-633	-548	-619
9.8	-584	-18	372	90	-73	441	117	11	-29	195

TIME	0.00	0.02	0.04	0.06	0.08	0.10	0.12	0.14	0.16	0.18
10.0	479	103	-451	105	1013	795	75	-456	-700	-876
10.2	-497	-14	109	705	540	813	576	204	-606	-368
10.4	-222	194	-186	177	769	1032	1334	925	-268	-521
10.6	34	51	-983	-773	-261	261	-63	-508	-878	-979
10.8	-221	-235	-179	102	110	-60	-607	43	701	1126
11.0	814	-156	132	488	667	211	-225	-301	153	394
11.2	139	210	108	197	-254	-916	-1076	-1152	-879	-573
11.4	-435	-507	-400	-271	82	791	474	505	203	302
11.6	275	-32	205	166	-241	-788	-799	-289	15	82
11.8	-117	-277	-49	213	-401	-698	-226	632	1180	198
12.0	-14	166	240	243	132	-163	128	-24	-382	-653
12.2	-715	-1136	-1254	-991	-209	807	1397	1159	865	1095
12.4	732	-137	-822	-228	229	-92	-80	480	880	677
12.6	159	-281	-627	-1179	-1335	-1079	-521	-113	626	1152
12.8	1094	1095	1083	440	-380	-419	-131	-68	-237	-216
13.0	56	-170	-300	-121	68	201	94	158	-53	-24
13.2	47	-189	-80	-20	549	439	-133	-338	-172	-401
13.4	-529	-360	446	687	494	477	-344	285	375	510
13.6	183	-235	-545	-355	-51	-284	-399	-30	196	156
13.8	361	416	-37	-560	-623	-178	-140	-140	398	880
14.0	441	68	184	150	-142	-482	-11	379	126	-403
14.2	-427	-229	-294	-650	-738	-722	-336	-197	-214	-274
14.4	-129	74	519	895	1074	621	449	357	70	-495
14.6	-748	-488	-134	-128	-326	-81	370	312	-65	-6
14.8	39	186	268	-97	-155	-114	-393	-264	135	-66
15.0	-397	-565	-281	66	36	123	455	710	627	467
15.2	213	257	314	-24	-228	-469	-635	-766	-732	-411
15.4	-198	-367	-437	-298	-194	11	558	744	620	318
15.6	124	-83	-4	249	678	935	843	371	-4	-173
15.8	-475	-703	-680	-474	-583	-749	-195	356	683	793
16.0	592	493	296	13	-399	-478	-447	-65	37	-45
16.2	292	426	163	-109	-146	127	64	-83	2	28
16.4	162	228	62	-77	-116	-26	-94	-96	32	268
16.6	247	237	88	125	232	188	-229	-393	-312	-351
16.8	-272	-283	-119	-84	115	298	267	73	-58	-44
17.0	-67	-165	-279	57	310	163	-298	-718	-651	-320
17.2	-35	237	434	474	382	322	204	98	213	424
17.4	221	-135	-254	-308	-393	-263	139	244	228	101
17.6	131	-3	-170	-277	-312	-263	-201	-228	-71	-23
17.8	87	157	226	254	142	151	94	32	140	-31
18.0	-139	-224	-176	-83	-58	31	179	157	13	-86
18.2	-64	-44	44	-67	-174	-146	-59	-138	-139	-39
18.4	-50	-46	168	178	62	254	448	349	219	237
18.6	124	-167	-560	-546	-295	-171	-124	64	323	272
18.8	54	-277	-320	-50	124	116	190	212	317	271
19.0	110	-50	-237	-330	-395	-390	-203	-120	15	201
19.2	276	147	-43	-77	-25	13	62	41	-163	-207
19.4	-163	176	175	-62	-105	-40	-10	-107	-50	92
19.6	1	-147	-152	-50	-96	-67	75	400	578	313
19.8	222	82	-186	-291	-218	-88	69	96	180	196

TABLE 5.10 | EN NS 20 | EL CENTRO, Real Time

$a_{max,h} = 342$ cm/sec^2
$a_{max,v} = 206$ cm/sec^2

ACCELERATIONS, cm/sec^2

		FIXED BASE	SPRING BASE - D4				SPRING BASE - D8				RUBBER BASE
		$a_h = 137$	$a_h = 137$, $a_v = 0$				$a_h = 128$, $a_v = 0$				$a_h = 137$
		T = 0.29	T = 0.92	T = 1.23	T = 1.43	T = 1.47	T = 0.92	T = 1.14	T = 1.43	T = 1.47	T = 1.21
HORIZONTAL	5	444	134	124	89	87	137	159	66	97	113
	4	412	125	108	70	73	135	128	55	77	108
	3	412	117	93	59	64	125	108	52	66	105
	2	353	104	82	57	55	117	93	57	62	98
	1	223	97	72	80	79	102	86	77	77	99
	B	-	98	103	118	120	97	92	100	109	99
VERTICAL	21			62	84	53		59	60	45	7
	22			48	65	40		50	47	37	5
	23			44	22	41		55	16	45	2

DISPLACEMENTS, cm

		FIXED BASE T = 0.29	D4 T = 0.92	D4 T = 1.23	D4 T = 1.43	D4 T = 1.47	D8 T = 0.92	D8 T = 1.14	D8 T = 1.43	D8 T = 1.47	RUBBER T = 1.21
HORIZONTAL	5	0.85	1.81	3.56	4.11	3.80	1.07	2.66	2.89	3.11	3.71
	4	0.78	1.79	3.12	3.50	3.28	1.05	2.28	2.49	2.63	3.68
	3	0.67	1.77	2.66	2.89	2.70	1.03	1.90	2.10	2.15	3.63
	2	0.52	1.74	2.20	2.26	2.11	1.00	1.51	1.69	1.66	3.56
	1	0.32	1.70	1.73	1.63	1.52	0.98	1.17	1.28	1.17	3.48
	B	0	1.67	1.19	0.87	0.85	0.96	0.87	0.75	0.64	3.36
VERTICAL	21			1.06	1.34	1.29		0.58	0.89	0.90	0.05
	22			0.87	1.04	1.04		0.51	0.69	0.74	0.03
	23			0.50	0.38	0.53		0.50	0.24	0.51	0.01

TABLE 5.11 | EB NS 20 | EL CENTRO, Scaled Time | $a_{max,h} = 342$ cm/sec^2, $a_{max,v} = 206$ cm/sec^2

ACCELERATIONS, cm/sec^2

		FIXED BASE	SPRING BASE - D4				SPRING BASE - D8				RUBBER BASE
		$a_h = 204$	$a_h = 203$, $a_v = 0$				$a_h = 221$, $a_v = 0$				$a_h = 203$
		T = 0.29	T = 0.92	T = 1.23	T = 1.43	T = 1.47	T = 0.92	T = 1.14	T = 1.43	T = 1.47	T = 1.21
HORIZONTAL	5	927	115	130	96	106	173	142	91	123	130
	4	836	94	97	68	73	135	96	74	79	115
	3	782	89	86	66	73	117	95	80	86	107
	2	654	92	86	77	86	113	98	104	122	104
	1	427	87	102	87	110	97	139	124	154	93
	B	-	90	130	119	192	136	164	130	157	106
VERTICAL	21			60	72	57		101	82	101	13
	22			48	56	44		88	64	85	10
	23			33	19	27		61	22	54	3

DISPLACEMENTS, cm

		FIXED BASE T = 0.29	D4 T = 0.92	D4 T = 1.23	D4 T = 1.43	D4 T = 1.47	D8 T = 0.92	D8 T = 1.14	D8 T = 1.43	D8 T = 1.47	RUBBER T = 1.21
HORIZONTAL	5	1.59	1.41	3.04	3.34	2.96	1.03	2.03	2.38	2.65	3.51
	4	1.47	1.41	2.62	2.86	2.54	1.01	1.76	2.04	2.20	3.48
	3	1.26	1.37	2.18	2.37	2.10	0.99	1.47	1.71	1.74	3.43
	2	0.97	1.34	1.74	1.87	1.67	0.96	1.18	1.38	1.28	3.37
	1	0.58	1.30	1.30	1.37	1.23	0.93	0.88	1.03	0.90	3.30
	B	0	1.26	0.86	0.84	0.70	0.91	0.54	0.62	0.52	3.19
VERTICAL	21		0.73	1.18	0.92	0.91		0.39	0.85	0.66	0.05
	22		0.53	0.91	0.71	0.71		0.30	0.66	0.52	0.04
	23		0.35	0.31	0.44	0.44		0.32	0.23	0.49	0.01

TABLE 5.12. | EN VK 20 | EL CENTRO, Real Time | $a_{max,h}$ = 342 cm/sec² ; $a_{max,v}$ = 206 cm/sec²

ACCELERATIONS, cm/sec²

		FIXED BASE	SPRING BASE - D4				SPRING BASE - D8				RUBBER BASE
		a_v= 87	a_h = 0 , a_v = 86				a_h = 0 , a_v = 66				a_v=86
		T = 0.29	T = 0.92	T = 1.23	T = 1.43	T = 1.47	T = 0.92	T = 1.14	T = 1.43	T = 1.47	T = 1.21
HORIZONTAL	5										
	4										
	3										
	2										
	1										
	B										
VERTICAL	21	31	36	44	39	43	51	34	29	31	746
	22	-	36	43	37	39	51	31	27	27	765
	23	25	36	48	36	41	52	30	27	27	1053

DISPLACEMENTS, cm

		FIXED BASE	SPRING BASE - D4				SPRING BASE - D8				RUBBER BASE
HORIZONTAL	5										
	4										
	3										
	2										
	1										
	B										
VERTICAL	21	0.0006	0.33	0.34	0.32	0.34	0.34	0.19	0.19	0.19	0.17
	22	-	0.33	0.36	0.32	0.36	0.34	0.19	0.19	0.19	0.19
	23	0.0008	0.33	0.41	0.34	0.39	0.34	0.23	0.19	0.23	0.28

TABLE 5.13 | EN 20 10 | EL CENTRO, Real Time | $a_{max,h} = 342$ cm/sec^2, $a_{max,v} = 206$ cm/sec^2

ACCELERATIONS, cm/sec^2

		FIXED BASE	SPRING BASE - D4				SPRING BASE - D8				RUBBER BASE
		a =	$a_h = 221$, $a_v = 44$				$a_h = 247$, $a_v = 36$				$a_h = 221$, $a_v = 44$
		T = 0.29	T = 0.92	T = 1.23	T = 1.43	T = 1.47	T = 0.92	T = 1.14	T = 1.43	T = 1.47	T = 1.21
HORIZONTAL	5		218	198	144	142	264	297	128	189	183
	4			177	114	121		240	107	150	175
	3		189	151	95	102	241	203	101	127	170
	2			133	92	90		178	109	118	158
	1			114	130	128	198	167	149	147	161
	B		160	166	191	194	190	179	194	211	161
VERTICAL	21		20	96	141	82	14	106	121	88	383
	22		20	78	110	66	14	87	93	73	392
	23		10	68	43	65	13	104	36	84	536

DISPLACEMENTS, cm

		FIXED BASE	D4 T = 0.92	D4 T = 1.23	D4 T = 1.43	D4 T = 1.47	D8 T = 0.92	D8 T = 1.14	D8 T = 1.43	D8 T = 1.47	RUBBER T = 1.21
HORIZONTAL	5		2.92	5.83	6.66	6.25	2.06	5.12	5.59	6.07	6.02
	4			5.09	5.67	5.32		4.40	4.82	5.15	5.96
	3		2.87	4.34	4.67	4.37	2.00	3.65	4.06	4.20	5.88
	2			3.58	3.66	3.42		2.92	3.28	3.24	5.77
	1			2.81	2.64	2.46	1.89	2.24	2.48	2.28	5.64
	B		2.70	1.92	1.41	1.39	1.86	1.66	1.45	1.25	5.44
VERTICAL	21		0.17	1.66	2.15	2.07	0.10	1.08	1.72	1.75	0.13
	22		0.17	1.36	1.66	1.67	0.10	0.95	1.34	1.43	0.13
	23		0.17	0.77	0.60	0.86	0.10	0.92	0.46	1.00	0.15

TABLE 5.14 | EN 40 20 | EL CENTRO, Real Time

$a_{max,h}$ = 342 cm/sec^2
$a_{max,v}$ = 206 cm/sec^2

ACCELERATIONS, cm/sec^2

		FIXED BASE	SPRING BASE - D4				SPRING BASE - D8				RUBBER BASE
		a =	a_h = 416 ,		a_v = 81		a_h = 370 ,		a_v = 62		
		T = 0.29	T = 0.92	T = 1.23	T = 1.43	T = 1.47	T = 0.92	T = 1.14	T = 1.43	T = 1.47	T = 1.21
HORIZONTAL	5		409	371	269	267	264	442	191	282	
	4			332	214	227		357	159	224	
	3		355	283	179	192	241	302	150	190	
	2			249	173	169		265	163	176	
	1		297	215	244	240	198	249	223	218	
	B		301	312	359	365	189	268	290	315	
VERTICAL	21		38	180	265	153	14	155	181	132	
	22		37	145	207	123	14	134	140	110	
	23		36	128	80	121	13	154	54	125	

DISPLACEMENTS, cm

HORIZONTAL	5		2.92	10.94	12.49	11.73	2.06	7.63	8.35	9.09	
	4			9.56	10.65	9.99		6.56	7.20	7.69	
	3		2.87	8.14	8.77	8.21	2.00	5.45	6.06	6.28	
	2			6.72	6.88	6.42		4.35	4.89	4.85	
	1			5.28	4.95	4.62	1.89	3.35	3.70	3.42	
	B		2.70	3.61	2.64	2.60	1.86	2.50	2.16	1.86	
VERTICAL	21		0.17	3.11	4.04	3.89	0.10	1.60	2.57	2.61	
	22		0.17	2.56	3.12	3.14	0.10	1.41	2.00	2.13	
	23		0.17	1.45	1.12	1.61	0.10	1.36	0.69	1.51	

TABLE 5.15 | EB 20 10 | EL CENTRO, Scaled Time

$a_{max,h}$ = 342 cm/sec^2
$a_{max,v}$ = 206 cm/sec^2

ACCELERATIONS, cm/sec^2

		FIXED BASE	SPRING BASE - D4				SPRING BASE - D8				RUBBER BASE
		a =	a_h = 306 , a_v = 87				a_h = 312 , a_v = 80				a_h=196 a_v= 56
		T = 0.29	T = 0.92	T = 1.23	T = 1.43	T = 1.47	T = 0.92	T = 1.14	T = 1.43	T = 1.47	T = 1.21
HORIZONTAL	5		167	192	145	157	241	195	128	172	125
	4			144	103	111		138	105	115	112
	3		140	127	100	113	167	130	113	123	103
	2			134	116	130		140	146	172	100
	1			159	132	168	135	199	175	221	90
	B		136	200	132	170	188	232	183	222	102
VERTICAL	21		17	104	112	97	19	138	142	138	223
	22		17	73	87	74	18	120	104	116	177
	23		16	50	39	45	18	89	45	76	226

DISPLACEMENTS, cm

HORIZONTAL	5		2.13	4.58	5.04	4.53	1.45	2.83	3.36	3.76	3.39
	4			3.95	4.31	3.89		2.45	2.88	3.12	3.36
	3		2.08	3.30	3.58	3.23	1.40	2.06	2.42	2.48	3.32
	2			2.64	2.83	2.56		1.66	1.94	1.83	3.26
	1			1.97	2.06	1.88	1.31	1.24	1.45	1.28	3.19
	B		1.92	1.32	1.27	1.09	1.28	0.78	0.87	0.77	3.09
VERTICAL	21		0.13	0.13	1.79	1.47	0.09	0.57	1.19	0.98	0.06
	22		0.13	0.83	1.39	1.17	0.09	0.42	0.92	0.81	0.05
	23		0.13	0.52	0.48	0.67	0.09	0.48	0.31	0.71	0.06

TABLE 5.16

EB 30 15	EL CENTRO, Scaled Time

$a_{max,h} = 342$ cm/sec^2
$a_{max,v} = 206$ cm/sec^2

ACCELERATIONS, cm/sec^2

		FIXED BASE	SPRING BASE - D4				SPRING BASE - D8				RUBBER BASE
		a =	$a_h = 411$, $a_v = 147$				$a_h = 430$, $a_v = 114$				
		T = 0.29	T = 0.92	T = 1.23	T = 1.43	T = 1.47	T = 0.92	T = 1.14	T = 1.43	T = 1.47	T = 1.21
HORIZONTAL	5		227	256	194	210	333	268	176	237	
	4			192	138	150		190	145	158	
	3		190	171	134	152	229	179	156	169	
	2			180	156	175		193	202	237	
	1		176	214	177	226	185	275	241	305	
	B		182	270	177	245	261	320	252	306	
VERTICAL	21		28	145	156	134	26	190	197	190	
	22		28	101	117	102	26	165	144	160	
	23		26	75	58	67	27	123	62	105	

DISPLACEMENTS, cm

HORIZONTAL	5		2.87	6.15	6.77	6.10	2.00	3.90	4.63	5.18	
	4			5.30	5.79	5.23		3.38	3.96	4.31	
	3		2.79	4.43	4.80	4.35	1.93	2.83	3.33	3.42	
	2			3.54	3.80	3.45		2.28	2.68	2.52	
	1		2.64	2.66	2.77	2.53	1.80	1.72	2.00	1.77	
	B		2.57	1.78	1.71	1.48	1.76	1.07	1.20	1.06	
VERTICAL	21		0.21	1.52	2.41	2.00	0.13	0.79	1.63	1.36	
	22		0.21	1.12	1.87	1.60	0.13	0.58	1.27	1.12	
	23		0.21	0.70	0.66	0.89	0.13	0.66	0.43	0.98	

TABLE 5.17 | PN NS 20 | PETROVAC, Real Time | $a_{max,h} = 427$ cm/sec^2, $a_{max,v} = 198$ cm/sec^2

ACCELERATIONS, cm/sec^2

		FIXED BASE	SPRING BASE - D4				SPRING BASE - D8				RUBBER BASE
		$a_h = 141$	$a_h = 142$, $a_v = 0$				$a_h = 138$, $a_v = 0$				$a_h = 142$
		T = 0.29	T = 0.92	T = 1.23	T = 1.43	T = 1.47	T = 0.92	T = 1.14	T = 1.43	T = 1.47	T = 1.21
HORIZONTAL	5	626	152	162	122	105	125	214	96	88	106
	4	573	149	112	86	69	125	163	74	69	102
	3	481	143	84	73	61	123	116	72	69	97
	2	386	139	94	82	73	220	100	77	76	90
	1	258	131	100	94	95	116	114	84	90	88
	B	-	127	120	119	133	114	123	102	123	84
VERTICAL	21			76	101	67		59	83	53	6
	22			68	78	63		61	64	51	4
	23			64	27	61		81	22	55	1

DISPLACEMENTS, cm

		FIXED BASE	D4 T = 0.92	D4 T = 1.23	D4 T = 1.43	D4 T = 1.47	D8 T = 0.92	D8 T = 1.14	D8 T = 1.43	D8 T = 1.47	RUBBER BASE
HORIZONTAL	5	1.35	2.53	3.70	4.69	4.18	1.57	3.74	3.29	3.08	3.35
	4	1.25	2.50	3.08	3.98	3.49	1.55	3.16	2.81	2.57	3.32
	3	1.06	2.46	2.48	3.25	2.81	1.53	2.55	2.32	2.06	3.28
	2	0.81	2.40	1.98	2.59	2.16	1.49	1.92	1.84	1.57	3.22
	1	0.49	2.34	1.50	1.96	1.54	1.45	1.27	1.37	1.10	3.15
	B	0	2.30	1.02	1.25	0.93	1.41	0.74	0.81	0.61	3.04
VERTICAL	21			0.96	1.70	1.19		0.59	1.13	0.73	0.05
	22			0.84	1.31	0.97		0.59	0.88	0.67	0.03
	23			0.74	0.44	0.71		0.83	0.30	0.65	0.01

TABLE 5.18: PB NS 20 — PETROVAC, Time Scaled

$a_{max,h} = 427$ cm/sec^2
$a_{max,v} = 198$ cm/sec^2

ACCELERATIONS, cm/sec^2

		FIXED BASE	SPRING BASE - D4				SPRING BASE - D8				RUBBER BASE
		a_h=345	a_h = 354 , a_v = 0				a_h = 344 , a_v = 0				a_h=354
		T = 0.29	T = 0.92	T = 1.23	T = 1.43	T = 1.47	T = 0.92	T = 1.14	T = 1.43	T = 1.47	T = 1.21
HORIZONTAL	5	2222	193	130	84	102	243	148	90	112	139
	4	2152	174	78	59	67	202	93	72	73	110
	3	1819	139	87	80	100	179	137	98	133	79
	2	1309	106	136	121	154	164	180	137	153	74
	1	865	108	178	160	193	155	198	174	190	98
	B	-	120	215	201	207	164	239	196	237	134
VERTICAL	21			142	113	141		147	148	165	33
	22			114	88	116		122	115	133	23
	23			62	30	160		82	40	71	7

DISPLACEMENTS, cm

		FIXED BASE T = 0.29	D4 T = 0.92	D4 T = 1.23	D4 T = 1.43	D4 T = 1.47	D8 T = 0.92	D8 T = 1.14	D8 T = 1.43	D8 T = 1.47	RUBBER T = 1.21
HORIZONTAL	5	4.39	1.43	2.55	2.77	2.63	1.22	1.86	2.21	2.08	2.44
	4	4.05	1.40	2.18	2.54	2.37	1.18	1.63	1.97	1.78	2.40
	3	3.44	1.38	1.81	2.32	2.12	1.12	1.37	1.74	1.47	2.37
	2	2.60	1.35	1.44	2.08	1.86	1.06	1.12	1.51	1.25	2.31
	1	1.56	1.31	1.16	1.83	1.63	1.00	0.86	1.29	1.08	2.26
	B	0	1.29	1.15	1.46	1.31	0.96	0.59	0.96	0.86	2.19
VERTICAL	21			0.60	0.99	0.97		0.37	0.71	0.78	0.05
	22			0.44	0.77	0.81		0.16	0.55	0.67	0.04
	23			0.38	0.26	0.45		0.47	0.19	0.19	0.02

TABLE 5.19 | PN VK 40 | PETROVAC, Real Time

$a_{max,h} = 427$ cm/sec^2
$a_{max,v} = 198$ cm/sec^2

ACCELERATIONS, cm/sec^2

		FIXED BASE	SPRING BASE - D4				SPRING BASE - D8				RUBBER BASE
		$a_v = 138$	$a_h = 0$, $a_v = 146$				$a_h = 0$, $a_v = 108$				$a_v = 146$
		T = 0.29	T = 0.92	T = 1.23	T = 1.43	T = 1.47	T = 0.92	T = 1.14	T = 1.43	T = 1.47	T = 1.21
HORIZONTAL	5										
	4										
	3										
	2										
	1										
	B										
VERTICAL	21	25	127	131	125	130	87	88	130	84	735
	22		127	138	125	132	87	89	130	81	770
	23	14	127	155	129	144	87	96	133	84	1007

DISPLACEMENTS, cm

HORIZONTAL	5										
	4										
	3										
	2										
	1										
	B										
VERTICAL	21	0.0001	1.16	1.20	1.16	1.29	0.57	0.54	0.84	0.60	0.18
	22		1.16	1.27	1.16	1.36	0.57	0.60	0.84	0.65	0.20
	23	0.0001	1.16	1.49	1.18	1.53	0.57	0.76	0.87	0.73	0.27

D8 EN NS 60

TABLE 5.20 - CHANGE IN ACCELERATIONS (cm/sec^2) DUE TO VISCOSITY

FLOOR	C_V = VERTICAL VISCOSITY (kg.sec/cm) C_H = HORIZONTAL VISCOSITY										
	C_V=0	10	20	30	40	50	60	70	90	110	LAB
	C_H=0	5	10	20	30	40	50	60	80	100	
5H	406	197	164	147	151	158	168	179	206	239	266
4	182	122	119	118	117	119	135	153	186	215	177
3	244	100	99	109	121	136	146	161	193	221	161
2	427	121	93	94	109	127	142	154	173	187	151
1	615	165	132	151	172	192	209	223	247	267	177
B	843	260	204	178	192	206	218	230	248	262	213
21V	480	166	118	99	89	84	81	82	79	73	115
22	373	121	79	63	65	75	78	79	75	68	-
23	128	59	58	59	67	71	72	74	74	78	-

TABLE 5.21 - CHANGE IN VELOCITIES (cm/sec) DUE TO VISCOSITY

FLOOR	C_V = VERTICAL VISCOSITY (kg.sec/cm) C_H = HORIZONTAL VISCOSITY										
	C_V=0	10	20	30	40	50	60	70	90	110	LAB
	C_H=0	5	10	20	30	40	50	60	80	100	
5H	53	38	34	31	28	26	25	25	26	28	-
4	36	31	29	26	24	22	21	21	22	23	-
3	35	25	23	21	19	18	16	17	18	19	-
2	46	21	19	16	15	13	13	13	13.5	14	-
1	56	18	15	12	11	9.5	8.9	8.7	8.8	8.8	-
B	70	20	13	9.5	7.4	6.1	5.6	5.4	5.0	4.7	-
21V	47	16	11	9.3	7.8	7.0	6.6	6.2	6.0	5.9	-
22	36	12	9.0	7.5	6.6	6.2	5.8	5.6	5.3	4.9	-
23	12	6.7	6.4	6.0	5.7	6.2	6.6	7.0	7.6	8.0	-

TABLE 5.22 - CHANGE IN DISPLACEMENTS (cm) DUE TO VISCOCITY

FLOOR	C_V = VERTICAL VISCOSITY (kg.sec/cm) C_H = HORIZONTAL VISCOSITY										
	C_V=0	10	20	30	40	50	60	70	90	110	LAB
	C_H=0	5	10	20	30	40	50	60	80	100	
5H	8.53	6.73	6.33	5.89	5.39	4.90	4.71	4.80	4.95	5.11	3.54
4	7.12	5.75	5.36	5.00	4.59	4.16	4.00	4.06	4.17	4.28	2.88
3	7.21	4.75	4.37	4.11	3.77	3.41	3.27	3.30	3.38	3.44	2.34
2	7.27	3.74	3.40	3.21	2.93	2.65	2.54	2.56	2.58	2.59	2.00
1	7.26	2.72	2.44	2.33	2.12	1.89	1.82	1.80	1.76	1.73	1.62
B	7.27	2.12	1.53	1.33	1.19	1.04	0.98	0.93	0.82	0.72	?
21V	5.43	2.30	1.74	1.59	1.50	1.42	1.35	1.28	1.16	1.05	0.80
22	4.22	1.69	1.52	1.28	1.21	1.15	1.10	1.05	0.96	0.89	-
23	1.44	0.78	0.75	0.70	0.73	0.74	0.76	0.80	0.94	1.07	-

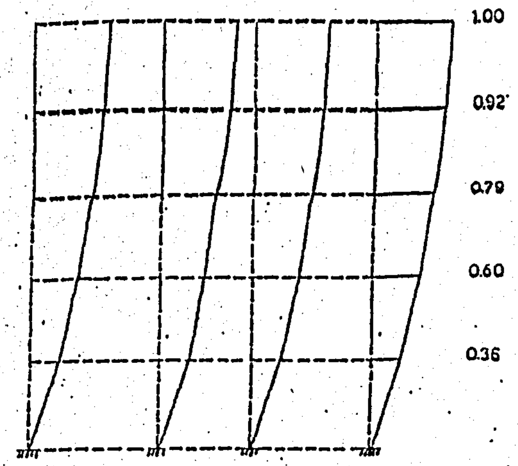


FIG 5.9.1 - MODE 1, FIXED BASE
T = 0.2879 sec

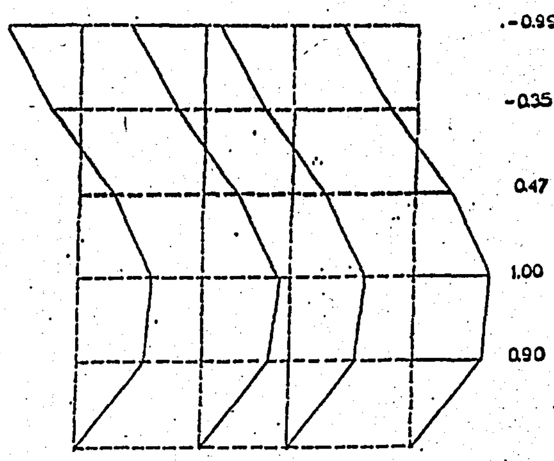


FIG 5.9.2 - MODE 2, FIXED BASE
T = 0.0901 sec

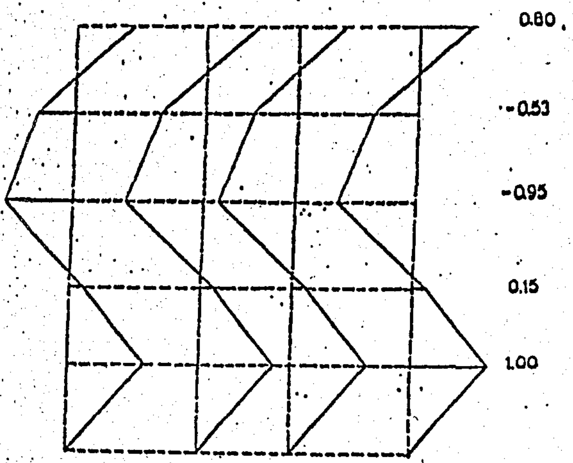


FIG 5.9.3 - MODE 3, FIXED BASE
T = 0.049 sec

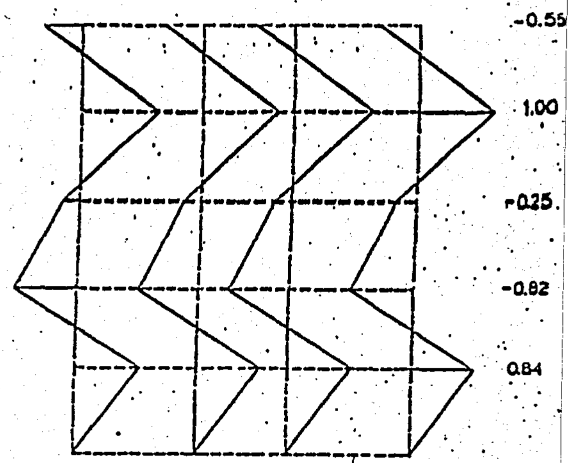


FIG 5.9.4 - MODE 4, FIXED BASE
T = 0.0324 sec

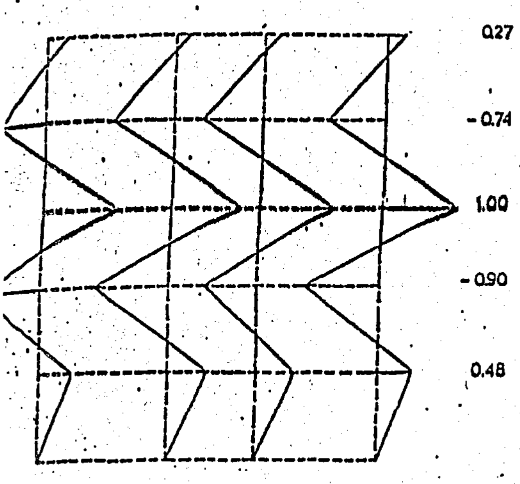


FIG 5.9.5 - MODE 5, FIXED BASE
T = 0.0247 sec

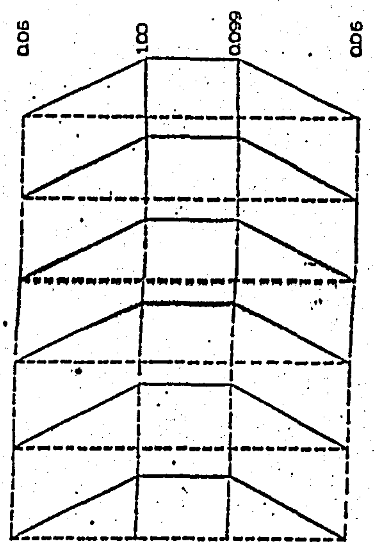


FIG 5.9.6 - MODE 6, FIXED BASE
T = 0.0194 sec

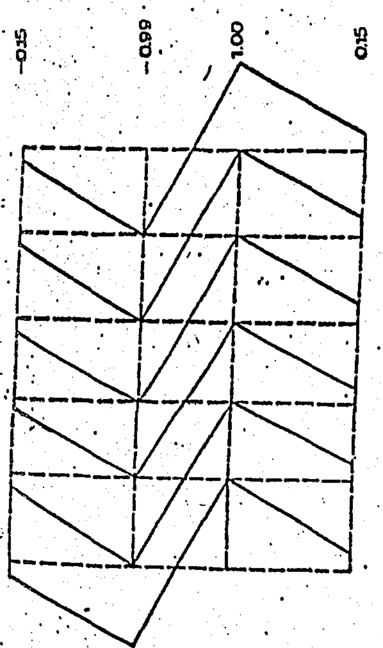


FIG 5.9.7 - MODE 7, FIXED BASE
T = 0.0180 sec

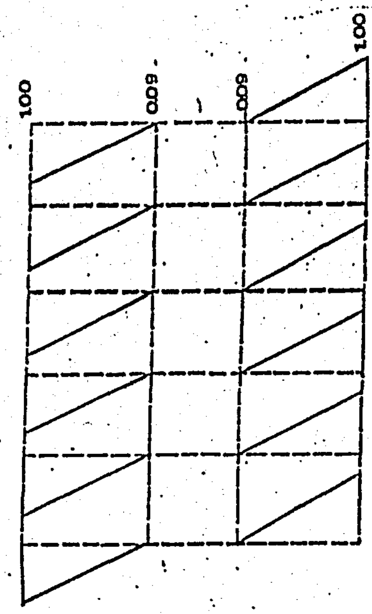


FIG 5.9.8 - MODE 8, FIXED BASE
T = 0.0148 sec

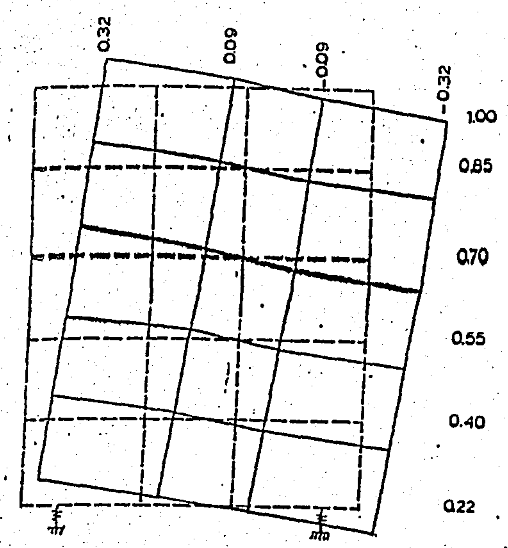


FIG 5.10.1 - MODE 1, SPRING BASE
T = 1.471 sec

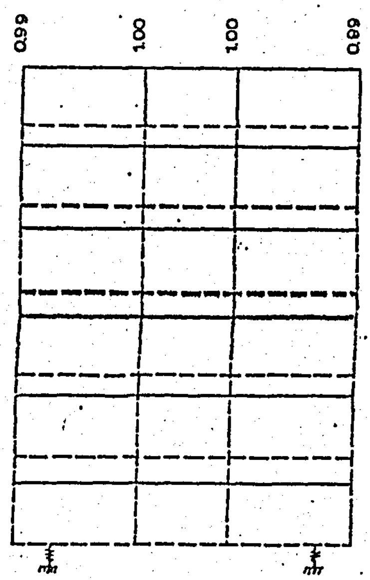


FIG 5.10.2 - MODE 2, SPRING BASE
T = 0.655 sec

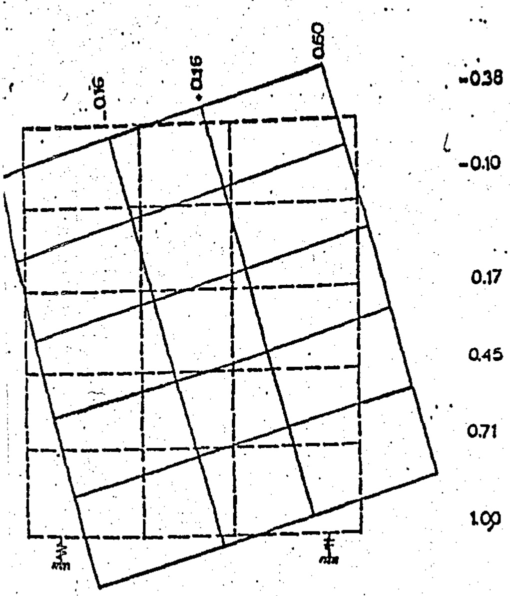


FIG 5.10.3 - MODE 3, SPRING BASE
T = 0.542 sec

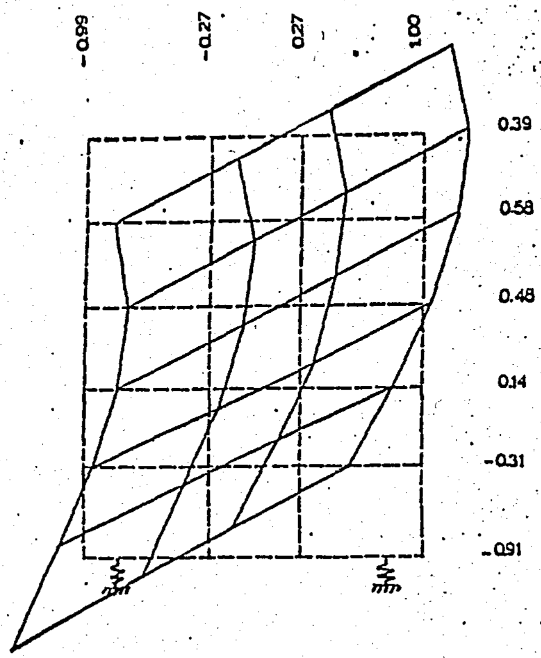


FIG 5.10.4 - MODE 4, SPRING BASE
T = 0.0997 sec

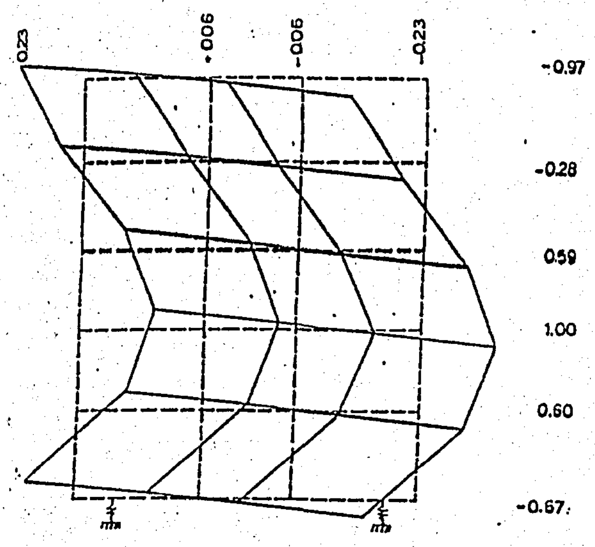


FIG 5.10.5 - MODE 5, SPRING BASE
T = 0.0781 sec

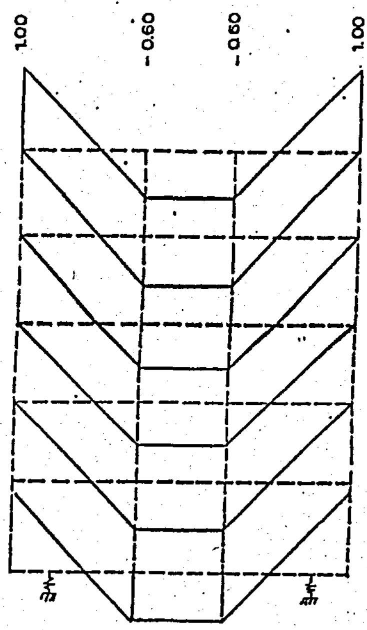


FIG 5.10.6 - MODE 6, SPRING BASE
T = 0.0548 sec

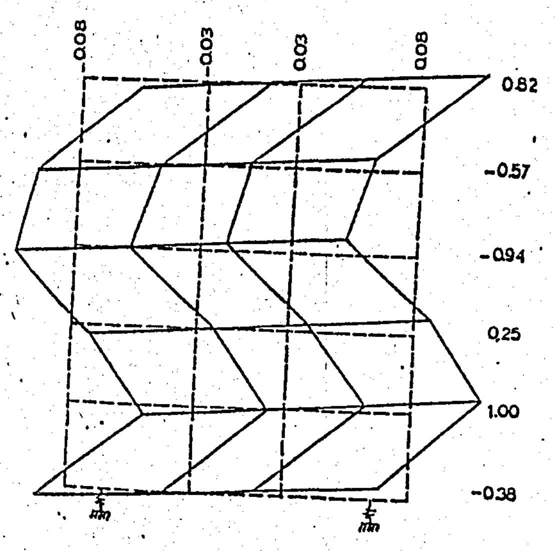


FIG 5.10.7 - MODE 7, SPRING BASE
T = 0.0475 sec

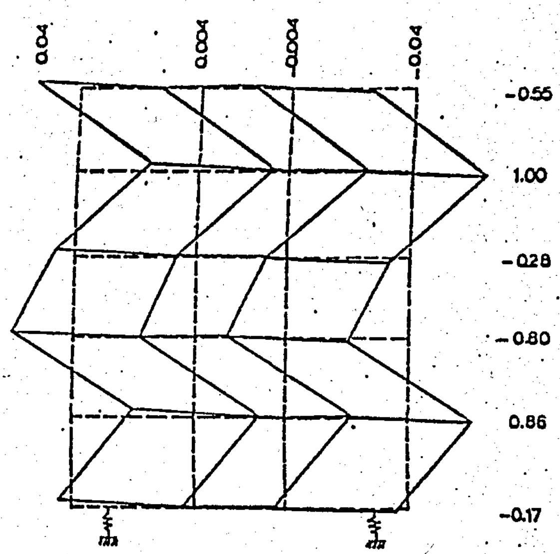


FIG 5.10.8 - MODE 8, SPRING BASE
T = 0.0321 sec

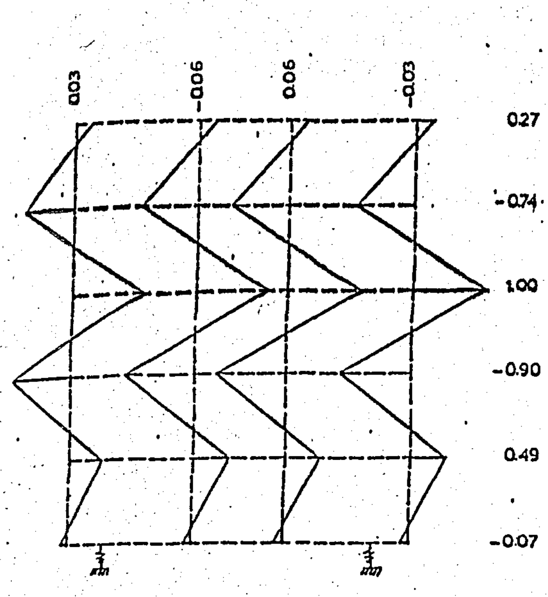


FIG 5.10.9 - MODE 9, SPRING BASE
T = 0.0247 sec

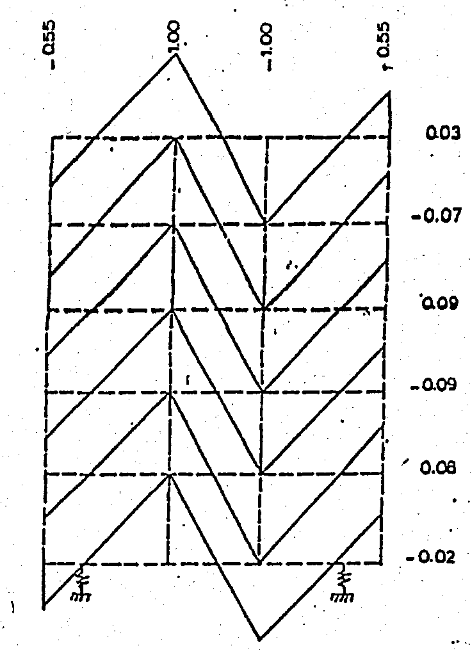


FIG 5.10.10 - MODE 10, SPRING BAS
T = 0.0234 sec

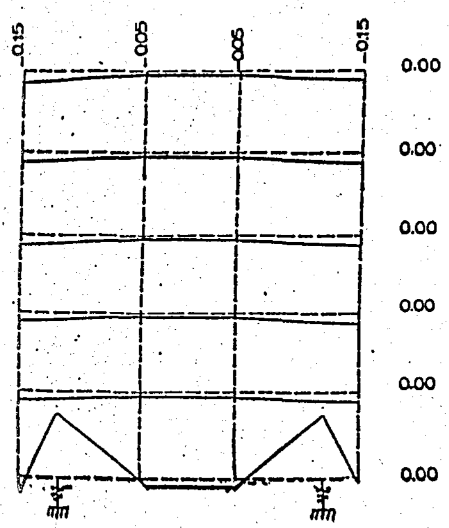


FIG 5.10.12 - MODE 12, SPRING BASE
T = 0.0054 sec

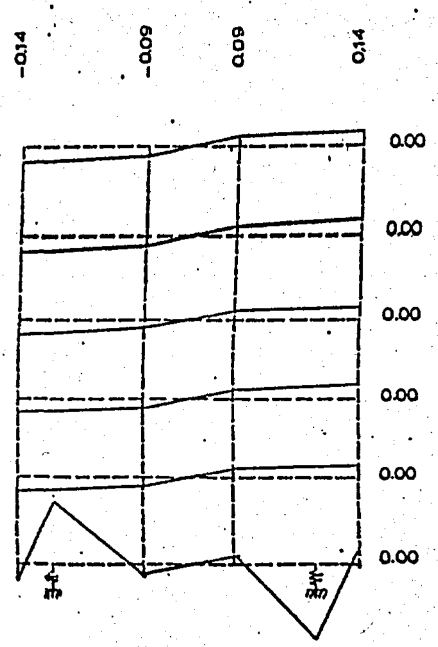


FIG 5.10.11 - MODE 11, SPRING BA
T = 0.0059 sec

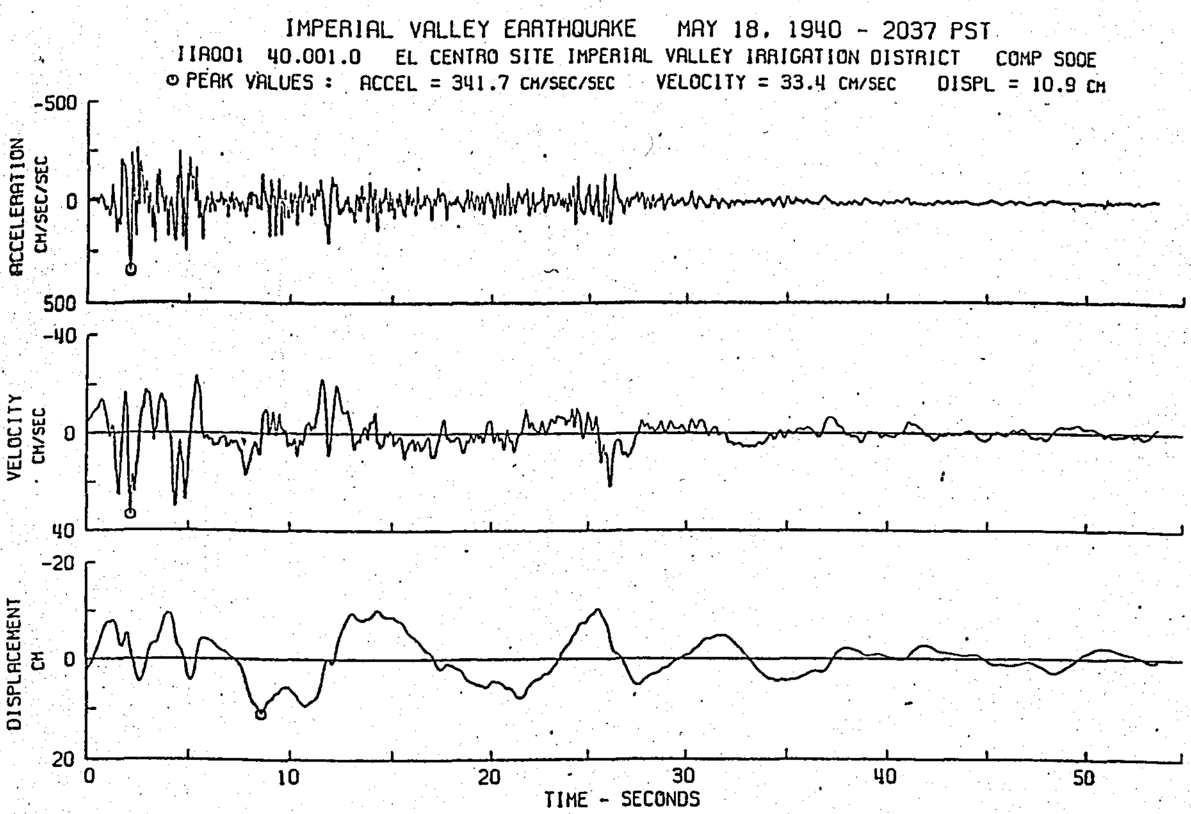


FIG. 5.11

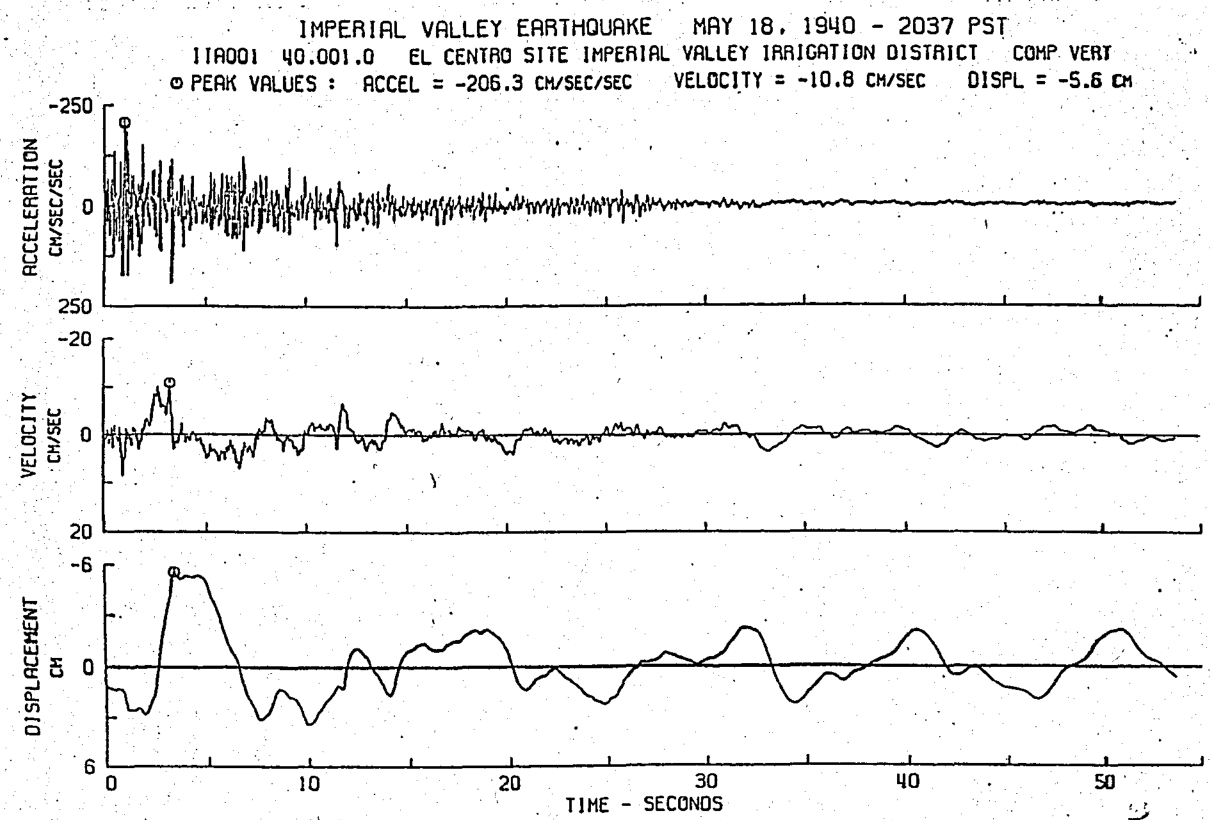


FIG. 5.12

RELATIVE VELOCITY RESPONSE SPECTRUM
IMPERIAL VALLEY EARTHQUAKE MAY 18, 1940 - 2037 PST
IIIA001 40.001.0 EL CENTRO SITE IMPERIAL VALLEY IRRIGATION DISTRICT COMP S00E
DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

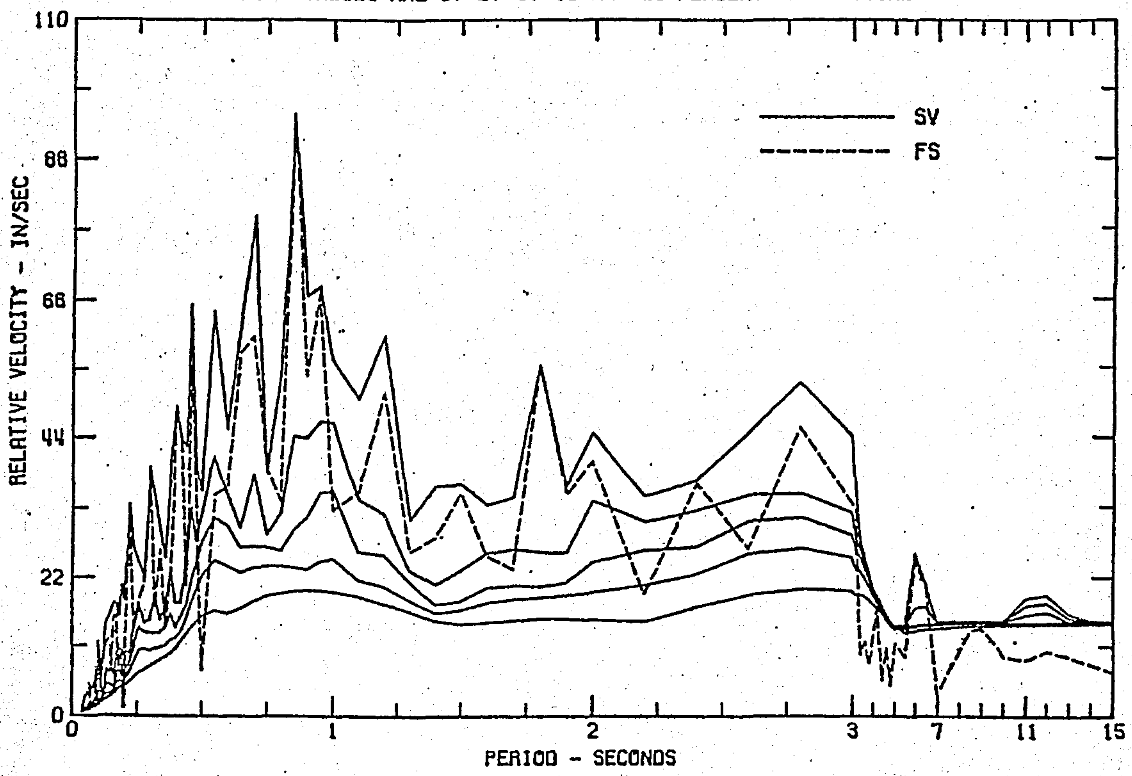


FIG. 5.13

RELATIVE VELOCITY RESPONSE SPECTRUM
IMPERIAL VALLEY EARTHQUAKE MAY 18, 1940 - 2037 PST
IIIA001 40.001.0 EL CENTRO VALLEY IRRIGATION DISTRICT COMP VERT
DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

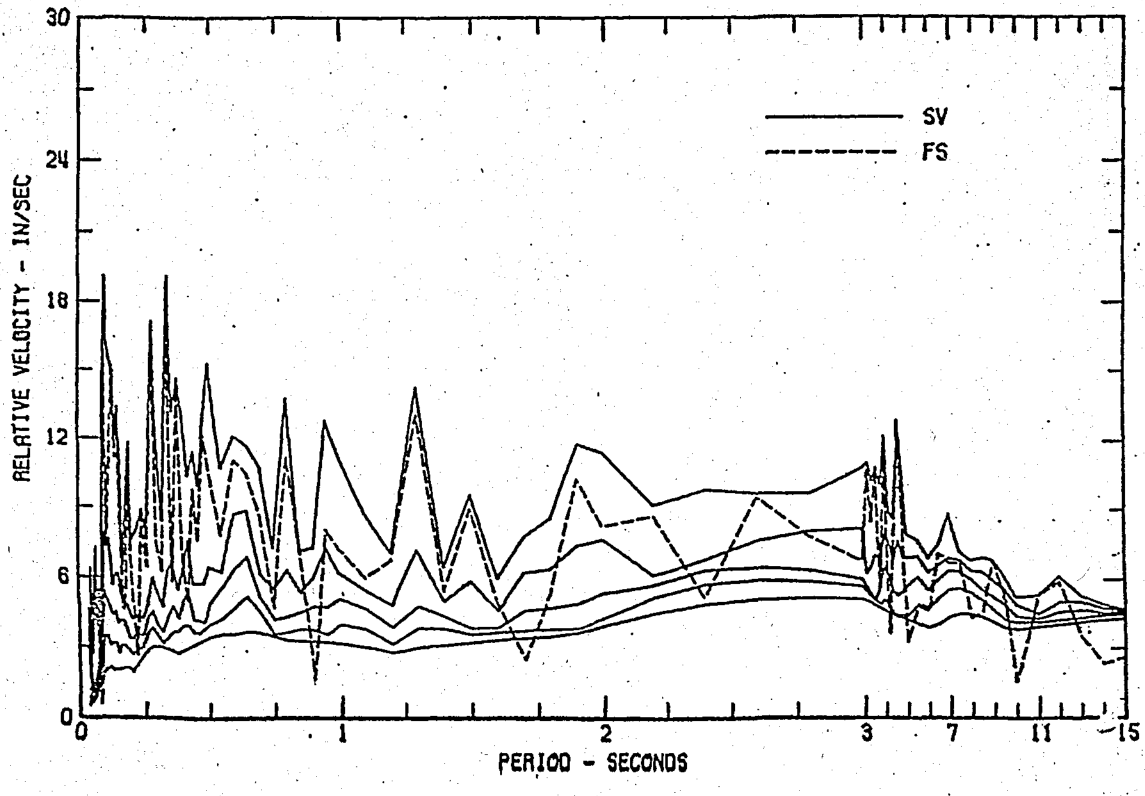


FIG. 5.14

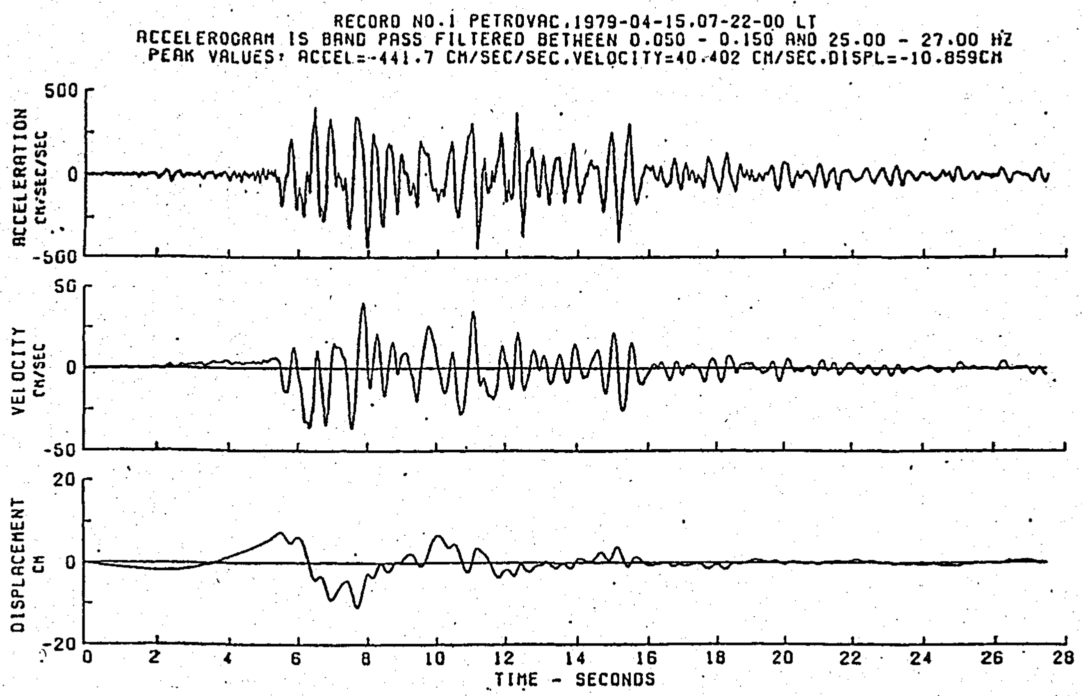


FIG. 5.15

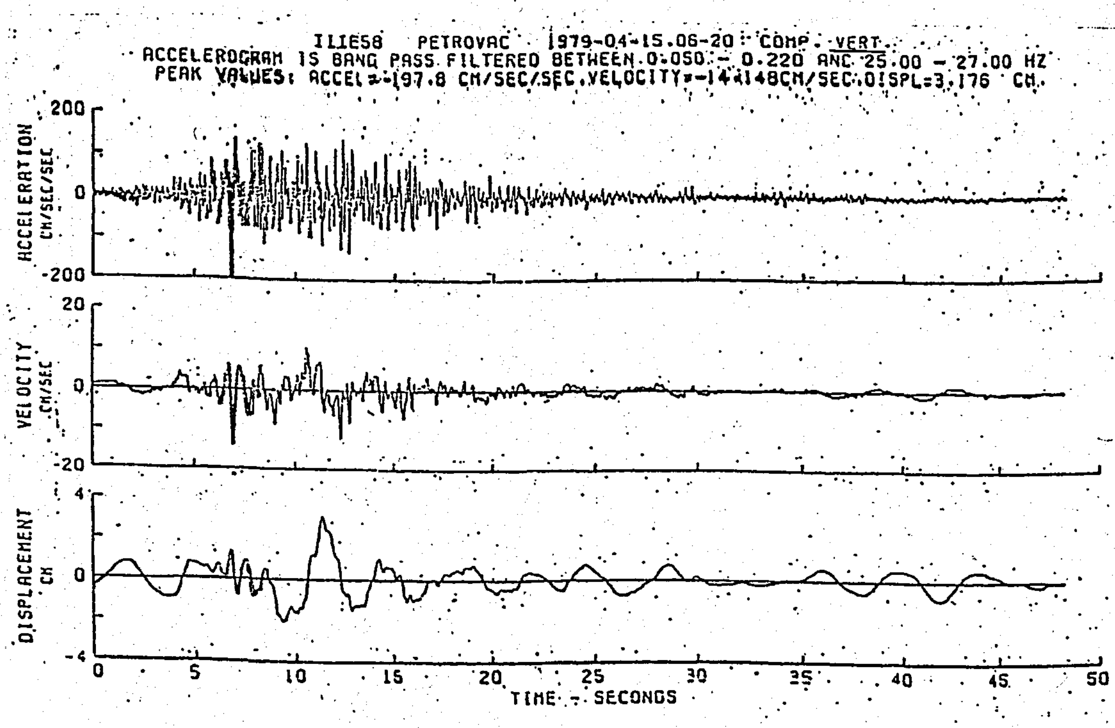


FIG. 5.16

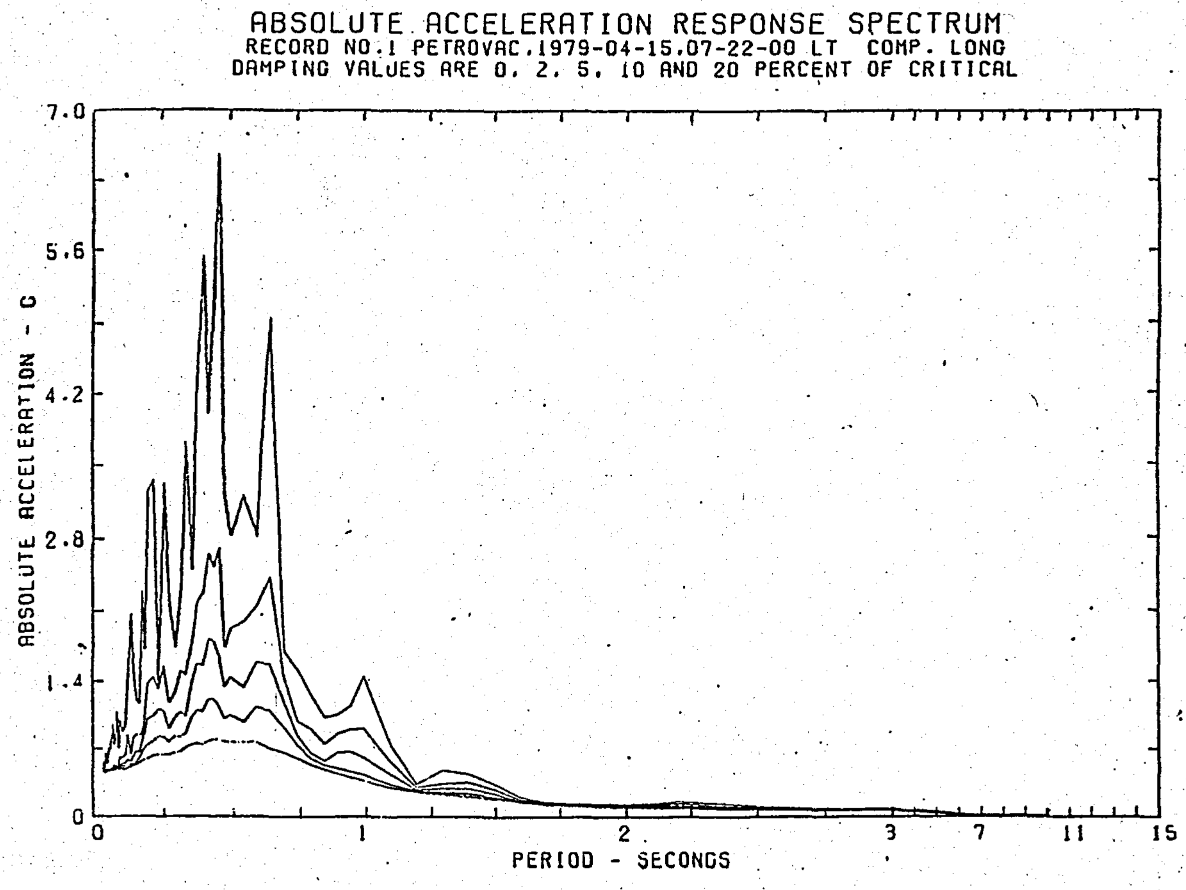


FIG. 5.17

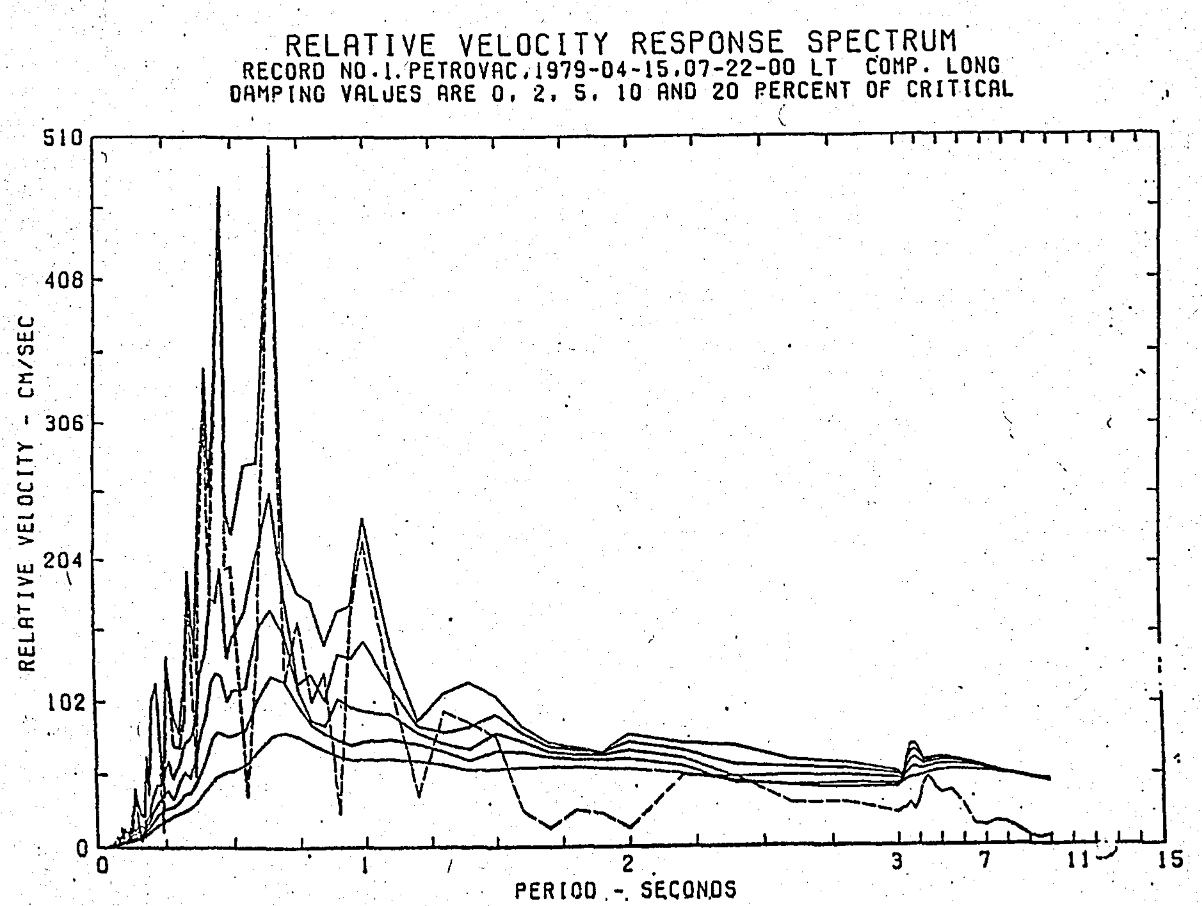


FIG. 5.18

RELATIVE DISPLACEMENT RESPONSE SPECTRUM
RECORD NO.1 PETROVAC,1979-04-15,07-22-00 LT COMP. LONG
DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

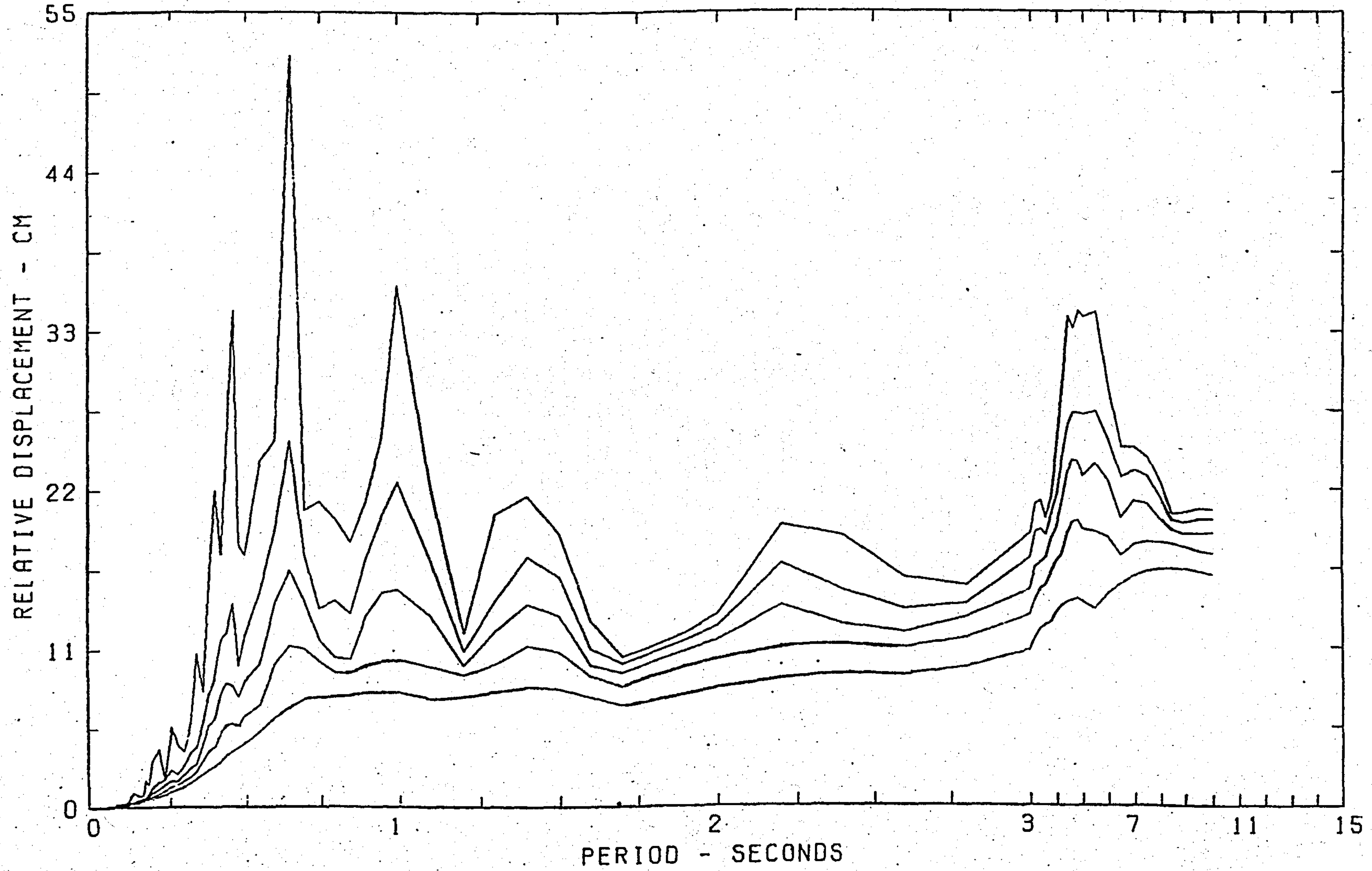


FIG. 5.19

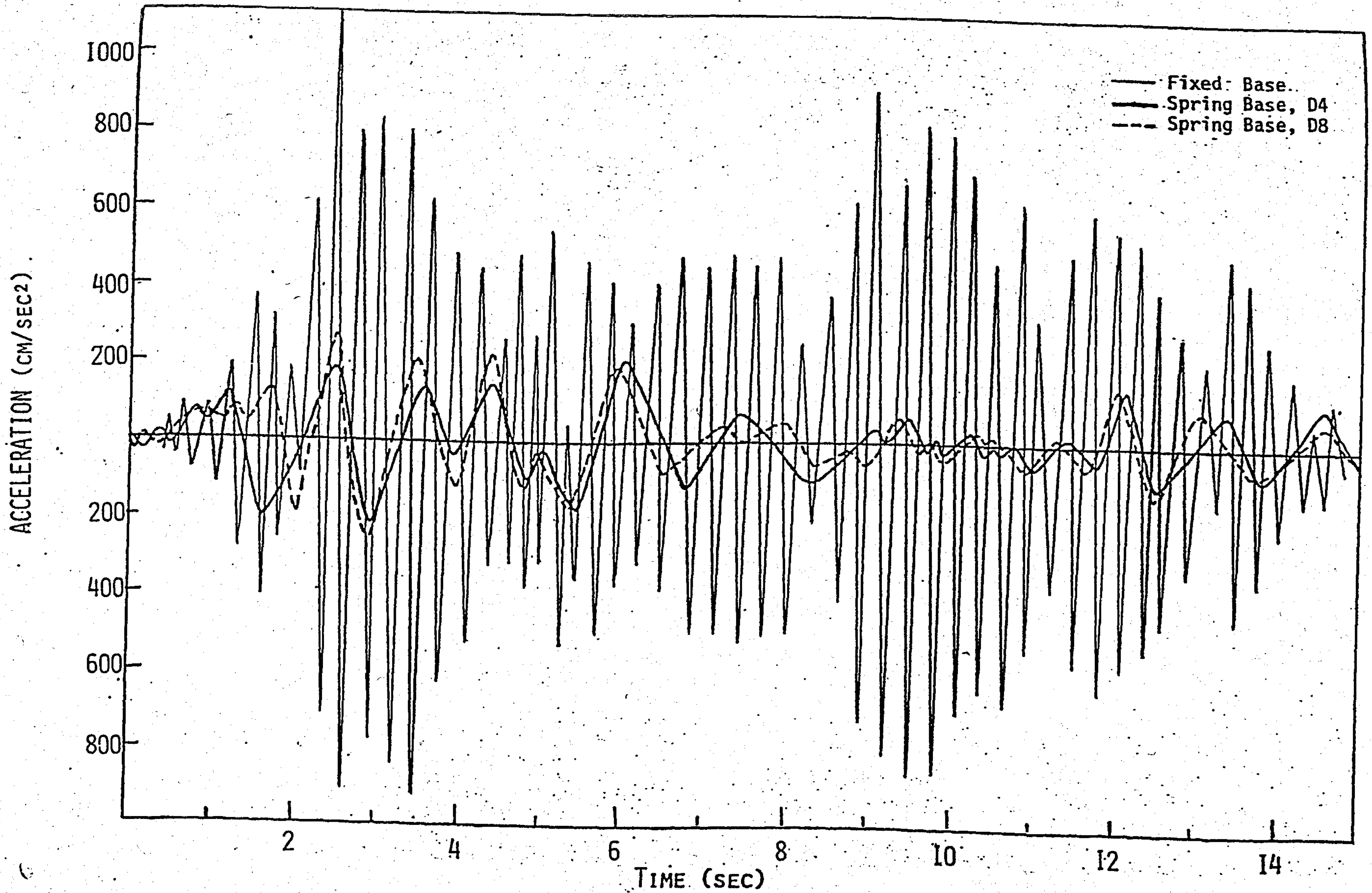


FIG. 5.20 - ACCELERATION RESPONSE OF ROOF TO EL CENTRO, REAL TIME - EN NS 60

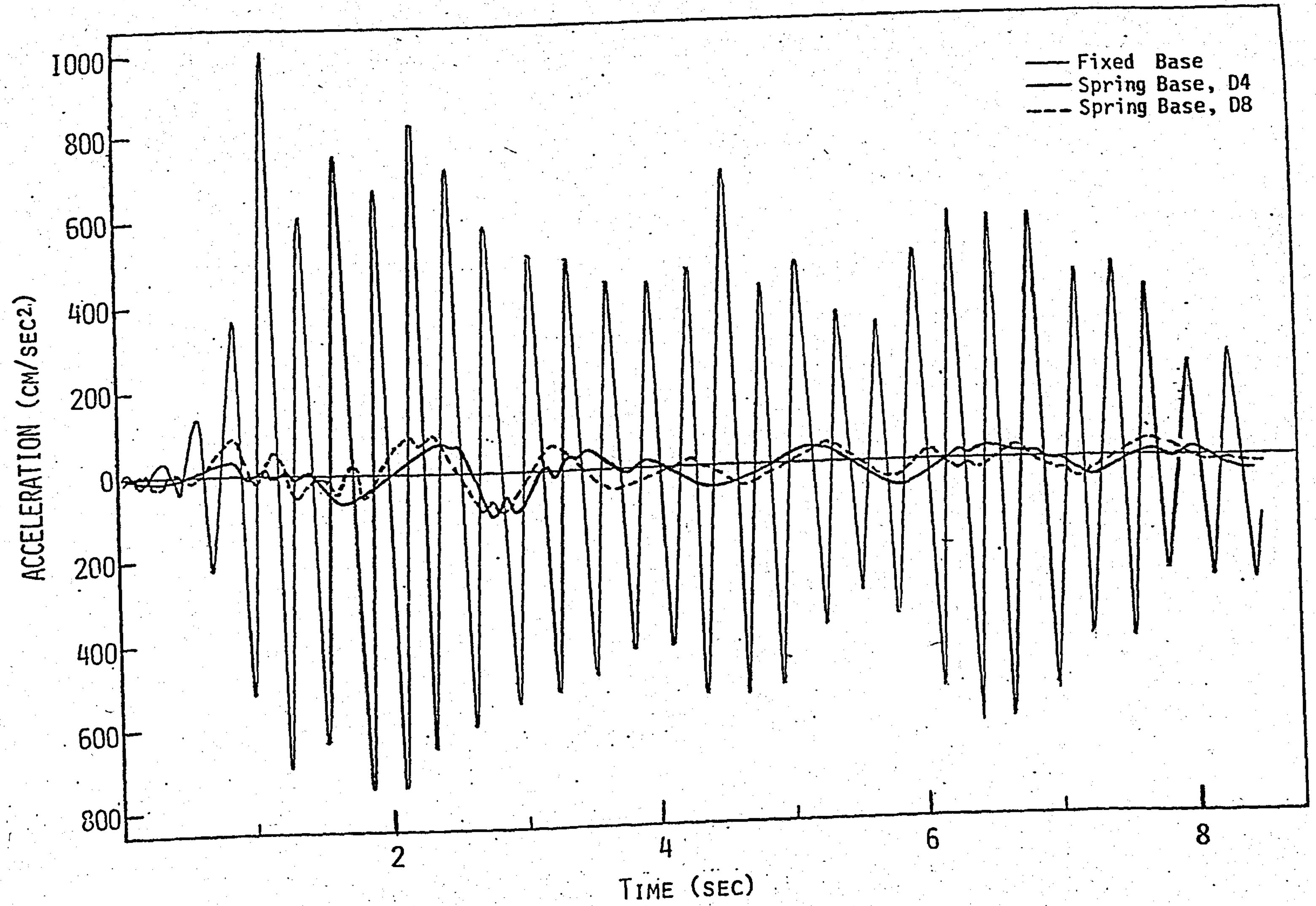


FIG. 5.21 - ACCELERATION RESPONSE OF ROOF TO EL CENTRO SCALED TIME - EB NS 20

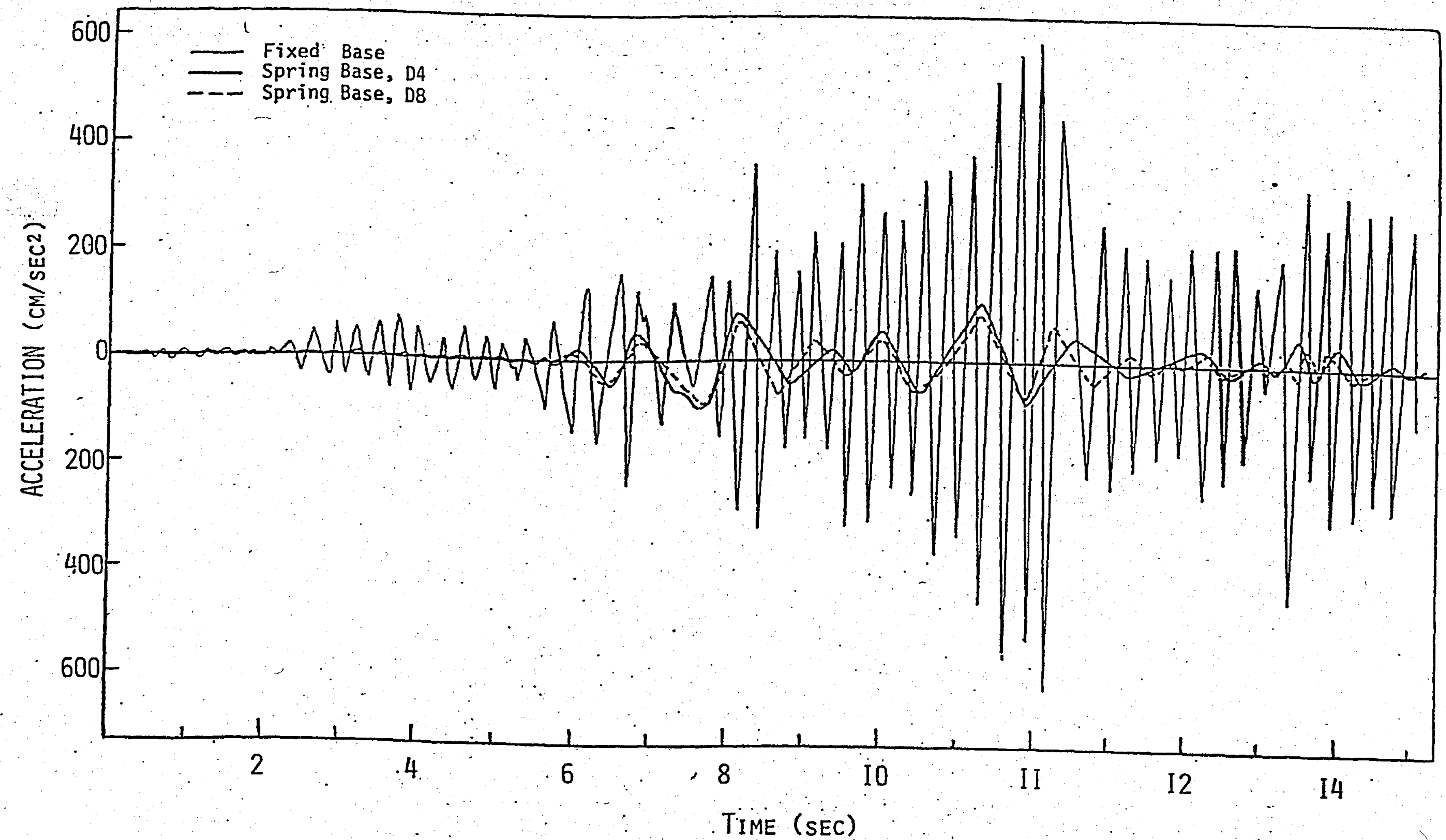


FIG. 5.22 - ACCELERATION RESPONSE OF ROOF TO PETROVAC REAL TIME - PN NS 20

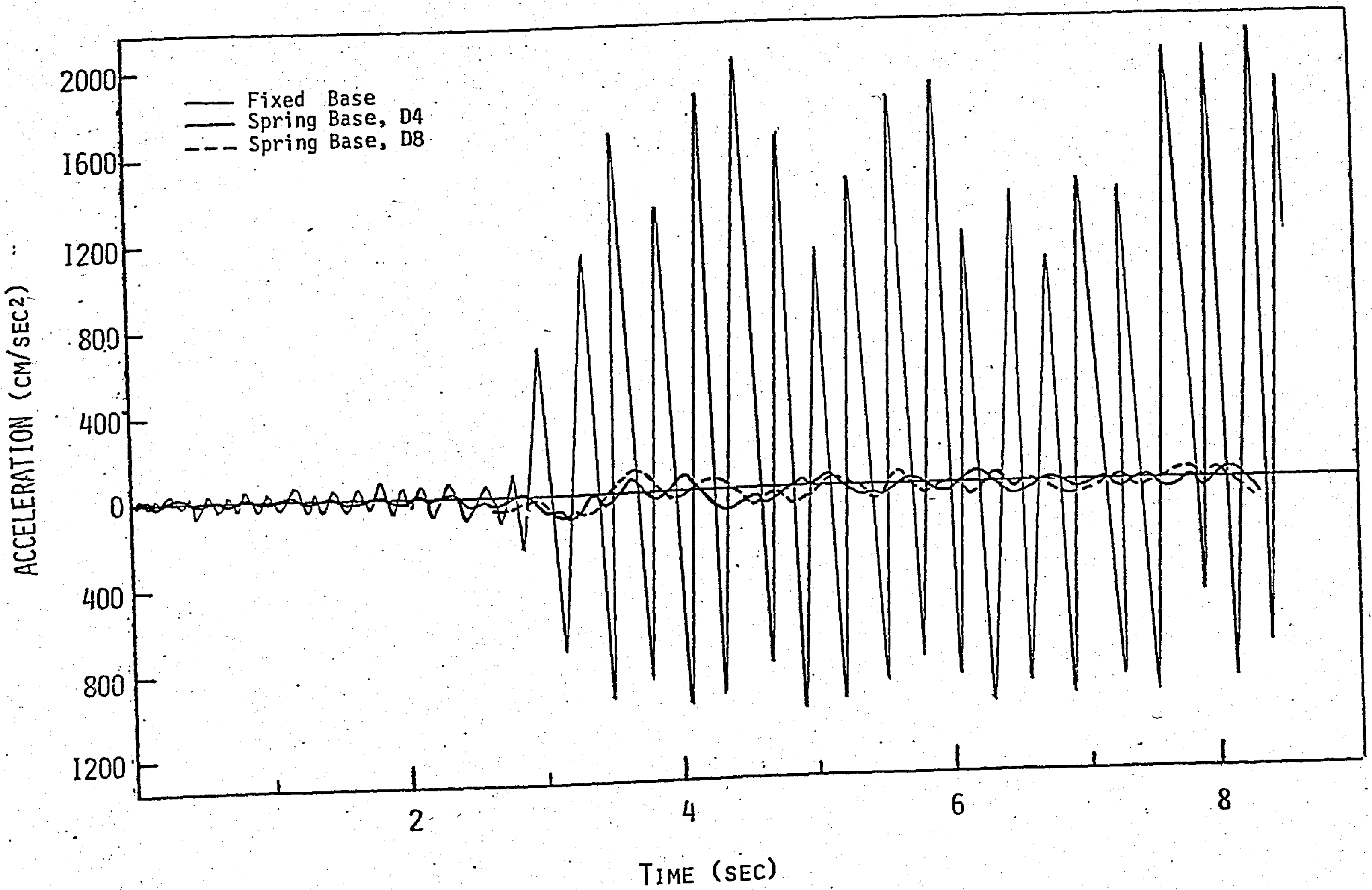


FIG. 5.23 - ACCELERATION RESPONSE OF ROOF TO PETROVAC SCALED TIME - PB NS 20

CHAPTER 6

CORRELATION OF TEST RESULTS WITH ANALYTICAL STUDIES

6.1. GENERAL

It is highly desirable to have sufficient experimental data preferably from shaking table tests for the verification of analytical studies. In the laboratory, structural models may be tested at shaking tables capable of simulating specified earthquake motions. Although the geometrical scale of the model and the characteristics of the simulated earthquake may not necessarily represent the actual conditons, the data obtained from the experimental testing provide fairly sufficient means of better understanding of dynamic behavior of the structural systems. On the other hand, these results are compared with those of the analytical works to estimate the level of correctness of the assumptions made in the analytical models. Consequently, this sets up an effective feedback mechanism to lead the analytical studies to more realistic assumptions.

Vibration isolation of structures against earthquake motions is a relatively new concept which is primarily at the stage of laboratory testing and analytical investigations. There is still much to do both analytically and experimentally in order to determine the behavior of various isolation schemes.

It is evident that a properly designed isolation scheme may substantially reduce the structural damage for a wide range of earthquakes. However, laboratory testing of the model structures with isolation elements is time consuming and very expensive. These laboratory investigations will, on the other hand, produce more reliable data about the exact behavior of the system under earthquake loads compared with the results of the analytical works, which are based on numerous simplifying assumptions.

It is expected that careful interpretation of the discrepancies between the measured and analytical results will pave the way for the development of more realistic analysis of structures.

In previous chapters, the experimental and the analytical studies of a 3-bay and 5-storey steel frame with and without vibration isolation have been discussed. The objective of this chapter is to compare and critisize the results obtained from the laboratory work and the analytical investigations. In the following sections probable causes of differences in the results will be discussed.

6.2. COMPARISON OF NATURAL FREQUENCIES OF VIBRATION

Free vibration analyses of the test model for various base conditions have been performed in order to determine the natural frequencies and the mode shapes. The results are already summarized in Chapter 5.

In the laboratory, a series of tests is carried out on the test model by simulating both impulse and sinusoidal displacement inputs. Impulse excited fixed base model showed a first natural frequency of vibration at 3.38 Hz (T = 0.30 sec). Sinusoidal input tests on the other hand, produced natural frequencies which are about ten percent higher.

The frequencies defined by the simulation of sinusoidal input show better correlation with those of the analytical results. The reason for the higher frequencies in the impulse tests is that a certain coupling exists between the model and the shaking table.

The sinusoidal input tests on the shaking table are also carried out for the model with spring-dashpot elements installed at the base. The results for both four and eight dashpots are already given in Chapter 4.

The results of the analytical investigations in the form of free vibration analysis were given in Chapter 5. The first natural period of vibration corresponding to the rocking mode ranges from T = 0.92 sec to T = 1.47 sec for various base conditions. This range covers the test results of T = 1.15 sec and T = 1.08 sec for the cases with four and eight dashpots, respectively. The calculated natural periods for the vertical direction range from T = 0.65 sec to T = 0.60 sec. These results are in good agreement with the measured results of T = 0.59 sec and T = 0.56 sec.

The discrepancies in the measured and calculated natural periods of vibration, may be attributed mainly to inadequate mathematical modellind and unintentional irregularities of the test structure. First of all, the additional tensional rigidities existing in the test model due to the existence of out-of-plane members are not accounted for in the analytical studies. Moreover, the analytical model does not consider the partially hinged connection at the base level in the fixed base case. Also, slight deviations from the expected natural frequencies of the test model may be caused by probable non-uniform distribution of the gravity loads due to small differecnes in the masses of some blocks.

6.3. DAMPING PROPERTIES

A) MEASURED DAMPING RATIOS

Equivalent viscous damping in the test structure is estimated from the results of impulse and sinusoidal tests using the zell known half-power band method. In this method the damping ratio β is determined from the frequencies at which the response is reduced to $\sqrt{2}/2$ times the amplitude at resonance, as

$$\beta = \frac{f_2 - f_1}{2f_0} \qquad \dots (6.1)$$

in which f_0 = resonance frequency, f_1 and f_2 = frequencies at $\sqrt{2}/2$ times the maximum amplitudes.

The value of percentage of critical damping determined for the fixed base case is $\beta = 0.01$ (Fig. 4.7). However, this value is not very reliable, since the impulse test is performed by considering very small displacements when compared with displacements during the actual earthquake simulation tests.

In fact, in some cases the test structure is subjected to very severe base motions pushind the stresses into the inelastic range. Apparently, the capacity of the model to absorb energy is significantly larger during the earthquake simulation tests. It is thus evident that the actual damping is larger than the measured value obtained through the half-power method. Therefore, in the analytical investigations a damping value of $\beta = 0.0i5$ is used.

In the vibration isolated cases, however, the damping values are calculated by two different techniques in connection with impulse and sinusoidal tests. The logarithmic decrement method produced damping values for different modes of vibration by utilizing the impulse excited displacement response curves shown in Fig. 6.1. In this method, the reduction in amplitude for the free vibration response of the structure, is used to determine the damping ratio as follows:

$$\beta = \frac{\mathrm{Ln}(x_1/x_2)}{2\pi} \qquad \ldots \quad (6.2)$$

in which, x_1 and x_2 are the two successive displacement amplitudes, a full period apart, as measured from the impulsive vibration response curve.

The critical damping ratios of $\beta = 0.16$ and $\beta = 0.35$ are obtained in the vertical direction for the four visco dampers and eight visco dampers cases, respectively (Fig. 6.1). The first natural frequency of vibration of the test model with base isolation springs and dashpots corresponds to the rocking mode and mainly activated by the

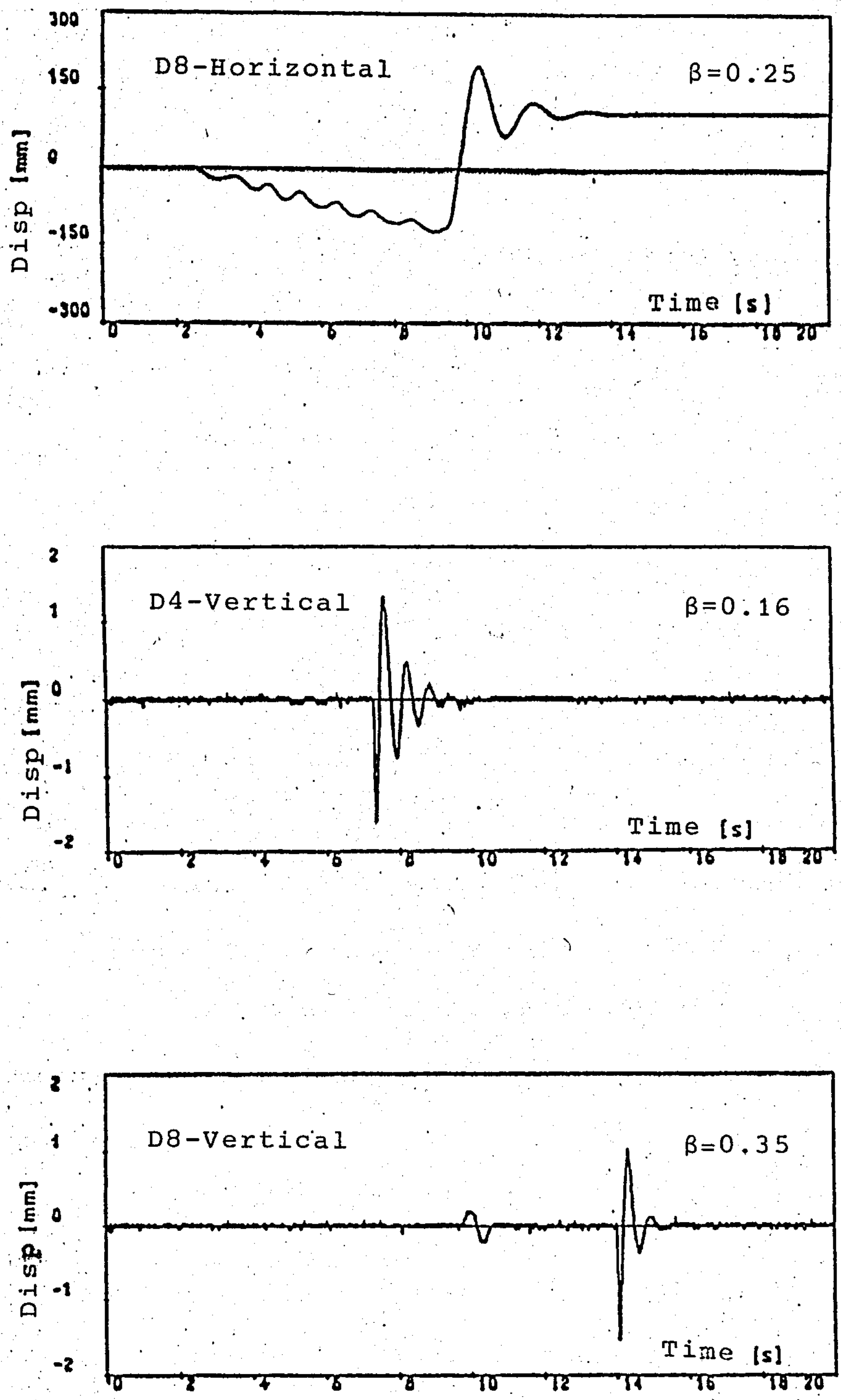


FIG. 6.1 - DISPLACEMENT RESPONSE CURVES TO IMPULSE EXCITATION

horizontal displacements. The response curves obtained from the horizontal displacement impulse tests shows a critical damping ratio of $\beta = 0.25$ for the eight visco dampers case.

The frequency response curves obtained from the sinusoidal input tests yield damping ratios of $\beta = 0.28$ and $\beta = 0.33$ for the four visco dampers and eight visco dampers cases, respectively (Fig. 4.8). However, it is evident from the wide arch chape of the spectrum curves that the use of half-power method will give only an estimated damping capacity much higher than the true values. This is due to the coupling of the two frequencies which are rather close and their response curves are not distinctly separated. The response of the system between frequencies of 0.90 Hz and 1.70 Hz is the sum of the responses of the two resonant curves (those corresponding to the uncoupled systems). Therefore, the damping obtained in this way is considered to be overestimated.

B) DAMPING VALUES IN THE ANALYTICAL STUDIES

The uncertainties encountered in damping measurements using the impulse and the sinusoidal tests are mostly due to the complex dynamic behavior of the structural model. In addition, further difficulties exist in measuring the damping characteristics of the visco-dampers.

It is already mentioned in the previous chapter that the viscodampers used in shaking table tests are very sensitive to both frequency and temperature changes. The variation of viscosity coefficients of the viscodampers with respect to frequency and temperature, is given in Fig. 5.5. It is apparent that 15% to 20% increase in the value of viscocity is possible for each degree centigrade drop in temperature. In order to investigate the effects of changes in temperature, three different sets of damping values were considered in the analytical studies as follows:

CASE NO.	C_v kg-sec/cm	C_h kg-sec/cm
Case 1 and 3	41	34
Case 2	68	57
Case 4	60	50

The viscosity coefficients are approximately proportional to the velocity at certain frequencies of the input motion. At higher frequencies however, the amplitudes are so small that the dampers provide only velocity indenpendent material damping. Due to this fact damping should be treated as a variable parameter rather than constant during the analytical investigations.

Further, in the base isolated case, only the effect of the actually existing viscodampers is taken into account in the form of vilcous damping. However, some additional material damping inherent to the structure also exists. No structural damping is considered to exist in the analytical investigations. It is assumed that the relatively high values of damping coefficients assigned to the viscodampers may readily and easily accomodate the small percentages of structural damping, which may exist in a steel frame.

6.4. MEASURED AND CALCULATED RESPONSE

Parallel to the tests conducted on shaking table, the test model is extensively analyzed under the action of the same earthquake loads in order to correlate the results of tests and analyses with each other. For purposes of simplicity, only the maximum amplitudes of the analytical and tests results are compared.

It is clearly demonstrated by both analytical and experimental studies that when vibration isolation system in the form of helical springs and viscodampers are used, both the vertical and horizontal components of acceleration response of the structure to any given ground excitation is significantly reduced.

In general, the peak response values determined by analytical studies are in acceptable ranges with those obtained from the shaking table tests. In the case of the 1940 El Centro real time earthquakes however, the correlation is not satisfactory regarding the peak accelerations of the fixed base model. The analytical results are higher almost by a factor of two with respect to the experimental results. This is particularly attributable to the nonlinear behavior of the frame during the tests, which is considered to be linear in the analytical investigations. The peak accelerations of the analyses and tests for the vibration isolated models, however, are in very good agreement with each other.

Similarly, the Petrovac earthquake results indicate that accelerations of the base isolated system are in very good agreement for both low and high damping values. For the fixed base model however, the correlation is not satisfactory for the reason of linear assumption of behavior in the analysis.

The measured and calculated displacements for the fixed base case are consistently in good agreement at all earthquake input motions. In the spring supported conditions, however, the peak displacement values of the analysis for the real time scale of the El Centro earthquake are somewhat larger than those of the test results. This discrepency may be due to the low frequency content of the motion altering the behavior of the frame and the dampers differently than those assumed in the analysis. The peak displacements of the test results and the analyses are in good correlation for almost all cases of scaled time input motions.

The peak accelerations and displacements calculated at each floor level and also at the joints located along the base beam are given in Tables 6.1 to 6.10. The laboratory testing results are also tabulated for comparison purposes. Although, the analytical studies have been performed for four different structural and damping conditions, only the peak response values of the "CASE 4" with T 1.47 sec natural period is used in constructing the comparative tables. The peak floor response values of the analytical studies are also compared illustratively with those of the shaking table test results for some of the input motion data in Figs. 6.2 and 6.13.

The anomolies between the experimental and the analytized results may be attributed to various differences existing in the real test structure and its corresponding mathematical model.

First of all, it should be pointed out that, in the analytical studies, the input motion is assumed to be given to the structure at the level of the centerline of the base girders. The real location of the input motion is actually at the centerline of the shaking table actualers. Therefore, any possible modification of the input motion from the level of actuators to the centerline of base girders, while transferred through the height of the elastic springs, has not been accounted for in the analyses.

Secondly, the input motion ordinates are displacements in simulating the earthquake motion at the shaking table. In fact, the actuators are programmed to apply only the time history ordinates of displacements. In the analytical investigations however, only the time history of acceleration ordinates are used as input data. Therefore, there is a basic difference in the characteristics of the input motion data used in the experiments and the analytical studies.

The accelerations actually recorded at the shaking table are not necessarily the same as those of the real earthquake data given in the literature. The differences of the time history accelerations recorded at the shaking table and of the real earthquake are illustrated in Figs. 6.14 and 6.15 for the horizontal components of the 1940 El Centro and 1979 Petrovae earthquakes respectively. Also, the acceleration Fourier Transforms of the shaking table and the real earthquake records are evaluated for the horizontal and vertical components of the Petrovae earthquake. The spectrum curves are plotted in Figs. 6.16 and 6.17 for horizontal and vertical components, respectively. The main frequency content of the horizontal excitation is below 3 Hz, whereas of the vertical excitation between 0 Hz and 5 Hz. In both horizontal and vertical cases of the shaking table there exists some additional peaks which are not noticed in the real earthquake record.

In conclusion, it is observed in all cases that the frequency contents of the accelerations recorded at the shaking table are larger than those of the accelerations given in the literature. This fact is important in the sense that the discrepancies in the acceleration response values of the fixed base models are likely due to this frequency content change. This agreement is strengthened by the fact that considerably good correlations existed in the acceleration responses of the spring supported models with lower governing frequencies. Further, response values calculated on the basis of recorded shaking table input accelerations, correlated better with the test results.

It is obvious that the displacements are very much controlled by the amount of damping existing in the structure. The differences in the displacements therefore may be thought of being caused by the discrepencies of damping assumptions not necessarily corresponding to the real values existing in the tests.

Finally, certain other differences may exist between the measurement and the analysis, especially when the coupling of the shaking table with the test model is considered. Considerable coupling of the mass of the shaking table (32 tons) may exist with that of the model frame, (also 32 tons), possibly altering the predictable peak response values. In fact, a clear whipping action (a local increase in the peak response) is observed in both the top floor and base beam horizontal accelerations, when the scaled time ($t = 0.01$ sec) interval is used in conjunction with either the 1940 El Centro or the 1979 Petrovac, Yugoslavia earthquakes. It appears that, this phenomenum resembles, to a certain extent, to the presence of an appendix or a set-back superstructure on top of a tall building.

TABLE 6.1

EN NS 20	EL CENTRO, Real Time
$\Delta t = 0.02$ sec	

Peak Accel.	
a_h	a_v
342	206

ACCELERATIONS, cm/sec²

	LOCATION	FIXED BASE		SPRING BASE-D4		SPRING BASE-D8		NEOPRENE BASE	
		a_h	a_v	a_h	a_v	a_h	a_v	a_h	a_v
		137	–	137	–	128	–	137	–
		TEST No:1	ANALYS[1] T=0.29	TEST No:18	ANALYS T=1.47	TEST No: 44	ANALYS T=1.47	TEST No:	ANALYS T = 1.21
HORIZONTAL	5	181	444	14?	87	85	97	119	113
	4	129	412	14?	73	48	77	97	108
	3	141	412	17?	64	48	66	102	105
	2	122	353	14?	55	46	62	100	98
	1	93	223	14?	79	58	77	90	99
	B	–	–	14?	120	72	109	102	99
VERTICAL	21				53		45		7.4
	22			17	40	38	37		5.4
	23				41		45		1.8

DISPLACEMENTS, cm

	LOCATION	FIXED BASE		SPRING BASE-D4		SPRING BASE-D8		NEOPRENE BASE	
HORIZONTAL	5	0.60	0.85	1.73	3.80	1.29	3.11	6.95	3.71
	4	0.53	0.78	1.37	3.28	1.03	2.63	7.20	3.68
	3	0.43	0.67	1.07	2.70	0.79	2.15	7.70	3.63
	2	0.35	0.52	0.84	2.11	0.71	1.66	8.30	3.56
	1	0.31	0.32	0.65	1.52	0.55	1.17	8.70	3.48
	B	–	–	?	0.85	?	0.64	6.40	3.36
VERTICAL	21				1.29		0.90		0.05
	22			0.03?	1.04	0.29	0.74		0.03
	23				0.53		0.51		0.01

[1] *Damping ratio* $\beta = 1.5\%$

TABLE 6.2

EN NS 40	EL CENTRO, Real Time
Δt= 0.02sec	

Peak Accel.	
a_h	a_v
342	206

ACCELERATIONS, cm/sec^2

	LOCATION	FIXED BASE		SPRING BASE-D4		SPRING BASE-D8		NEOPRENE BASE	
		a_h	a_v	a_h	a_v	a_h	a_v	a_h	a_v
		257	-	256	-	252	-	256	-
		TEST No: 2	ANALYS[1] T= 0.29	TEST No: 19	ANALYS T=1.47	TEST No: 45	ANALYS T= 1.47	TEST No:	ANALYS T = 1.21
HORIZONTAL	5	338	835	10?	163	176	191		211
	4	256	775	65	137	117	152		202
	3	244	775	67	120	103	130		196
	2	208	664	72	103	103	122		183
	1	180	419	101	148	127	152		185
	B	-	-	168	224	146	215		185
VERTICAL	21				99		89		14
	22			101	75	72	73		10
	23				77		89		3.4

DISPLACEMENTS, cm

HORIZONTAL	5	1.30	1.60	3.59	7.22	2.28	6.13		6.93
	4	1.04	1.47	2.91	6.13	1.85	5.18		6.88
	3	0.81	1.26	2.27	5.05	1.53	4.24		6.78
	2	0.70	0.98	1.81	3.95	1.30	3.27		6.65
	1	0.62	0.60	1.33	2.84	1.05	2.30		6.50
	B	-	-	?	1.59	?	1.26		6.28
VERTICAL	21				2.41		1.77		0.09
	22			1.00	1.94	0.54	1.46		0.06
	23				0.99		1.00		0.02

[1] *Damping ratio β= 1.5%*

TABLE 6.4

EB NS 20	EL CENTRO, Scaled Time
Δt= 0.01 sec	

Peak Accel.	
a_h	a_v
342	206

ACCELERATIONS, cm/sec^2

	LOCATION	FIXED BASE		SPRING BASE-D4		SPRING BASE-D8		NEOPRENE BASE	
		a_h	a_v	a_h	a_v	a_h	a_v	a_h	a_v
		204	–	203	–	221	–	203	–
		TEST No: 5	ANALYS[1] T= 0.29	TEST No: 23	ANALYS T= 1.47	TEST No: 50	ANALYS T= 0.29	TEST No: 50	ANALYS T = 1.21
HORIZONTAL	5	759	927	146	106	178	123		130
	4	575	836	77?	73	91	79		115
	3	467	782	82	73	89	86		107
	2	307	654	117	86	98	122		104
	1	259	427	161	110	129	154		93
	B	–	–	249	192	192	157		106
VERTICAL	21				57		101		13
	22			129	44	101	85		9.5
	23				27		54		3.2

DISPLACEMENTS, cm

	LOCATION	FIXED BASE		SPRING BASE-D4		SPRING BASE-D8		NEOPRENE BASE	
HORIZONTAL	5	2.34	1.59	3.41	2.96	1.92	2.65		3.51
	4	1.96	1.47	2.79	2.54	1.58	2.20		3.48
	3	1.54	1.26	2.13	2.10	1.25	1.74		3.43
	2	1.11	0.97	1.67	1.67	1.03	1.28		3.37
	1	0.66	0.58	1.24	1.23	0.85	0.90		3.30
	B	–		?	0.70	?	0.52		3.19
VERTICAL	21						0.66		0.05
	22			1.00		0.48	0.52		0.04
	23						0.49		0.10

[1] *Damping ratio β= 1.5%*

TABLE 6.5

EN VK 20	EL CENTRO, Real Time
Δt= 0.02 sec	

Peak Accel.	
a_h	a_v
342	206

ACCELERATIONS, cm/sec^2

	LOCATION	FIXED BASE		SPRING BASE-D4		SPRING BASE-D8		NEOPRENE BASE	
		a_h	a_v	a_h	a_v	a_h	a_v	a_h	a_v
		-	45	-	50	-	34	-	50
		TEST No:	ANALYS[1] T= 0.29	TEST No: 26	ANALYS T= 1.47	TEST No: 54	ANALYS T= 1.47	TEST No:	ANALYS T = 1.21
HORIZONTAL	5	11		5		8			0.0006
	4	10		7		7			
	3	19		7		10			0.0006
	2	19		12		17			
	1	5		7		7			
	B	-		5		7			0.0006
VERTICAL	21		16		25		16		434
	22		-	22	23	22	14		445
	23		13		24		14		612

DISPLACEMENTS, cm

	LOCATION	FIXED BASE a_h	FIXED BASE a_v	SPRING BASE-D4 a_h	SPRING BASE-D4 a_v	SPRING BASE-D8 a_h	SPRING BASE-D8 a_v	NEOPRENE BASE a_h	NEOPRENE BASE a_v
HORIZONTAL	5	0.11		0.10		0.10			0
	4	0.06		0.03		0.08			
	3	0.06		0.04		0.06			0
	2	0.04		0.04		0.07			
	1	0.03		0.06		0.07			
	B	-		?		?			0
VERTICAL	21		0.0003		0.20		0.10		0.10
	22			0.09	0.21	0.05	0.10		0.11
	23		0.0004		0.23		0.12		0.16

[1] *Damping ratio β= 1.5%*

TABLE 6.6

EN 40 20	EL CENTRO, Real Time
Δt= 0.02 sec	

Peak Accel.	
a_h	a_v
342	206

ACCELERATIONS, cm/sec²

	LOCATION	FIXED BASE		SPRING BASE-D4		SPRING BASE-D8		NEOPRENE BASE	
		a_h	a_v	a_h	a_v	a_h	a_v	a_h	a_v
				416	81	370	62	416	83
		TEST No:	ANALYS[1] T=	TEST No:31	ANALYS T= 1.47	TEST No:58	ANALYS T= 1.47	TEST No:	ANALYS T = 1.21
HORIZONTAL	5			120	267	162	282		344
	4			69	227	103	224		329
	3			74	192	144	190		320
	2			91	169	149	176		297
	1			110	240	115	218		303
	B			182	365	180	315		303
VERTICAL	21				153		132		721
	22			110	123	285	110		738
	23				121		125		1009

DISPLACEMENTS, cm

HORIZONTAL	5			3.94	11.73	3.17	9.09		11.33
	4			3.19	9.99	2.50	7.69		11.22
	3			2.54	8.21	1.81	6.28		11.07
	2			2.05	6.42	1.89	4.85		10.86
	1			1.51	4.62	1.48	3.42		10.62
	B			?	2.60	?	1.86		10.24
VERTICAL	21				3.89		2.61		0.24
	22			1.10	3.14	2.17	2.13		0.24
	23				1.61		1.51		0.28

[1] *Damping ratio β= 1.5%*

TABLE 6.7

EB 30 15	EL CENTRO, Scaled Time
Δt= 0.01 sec	

Peak Accel.	
a_h	a_v
342	206

ACCELERATIONS, cm/sec^2

	LOCATION	FIXED BASE		SPRING BASE-D4		SPRING BASE-D8		NEOPRENE BASE	
		a_h	a_v	a_h	a_v	a_h	a_v	a_h	a_v
				411	147	430	114	411	117
		TEST No:	ANALYS[1] T=	TEST No:33	ANALYS T=1.47	TEST No:61	ANALYS T=1.47	TEST No:	ANALYS T= 1.21
HORIZONTAL	5			205	210	256	237		262
	4			117	150	125	158		235
	3			137	152	129	169		216
	2			149	175	153	237		210
	1			218	226	182	305		189
	B			347	245	244	306		214
VERTICAL	21				134		190		468
	22			192	102	129	160		371
	23				67		105		474

DISPLACEMENTS, cm

	LOCATION	FIXED BASE TEST	FIXED BASE ANALYS	SPRING BASE-D4 TEST	SPRING BASE-D4 ANALYS	SPRING BASE-D8 TEST	SPRING BASE-D8 ANALYS	NEOPRENE BASE TEST	NEOPRENE BASE ANALYS
HORIZONTAL	5			5.54	6.10	2.99	5.18		7.11
	4			4.48	5.23	2.47	4.31		7.05
	3			3.51	4.35	1.93	3.42		6.96
	2			2.73	3.45	1.55	2.52		6.84
	1			1.94	2.53	1.24	1.77		6.69
	B			?	1.48	?	1.06		6.48
VERTICAL	21				2.00		1.36		0.13
	22			1.71	1.60	0.78	1.12		0.10
	23				0.89		0.98		0.13

[1] *Damping ratio β= 1.5%*

TABLE 6.8

PN NS 20	PETROVAC, Real Time
Δt= 0.02 sec	

Peak Accel.	
a_h	a_v
427	198

ACCELERATIONS, cm/sec^2

	LOCATION	FIXED BASE		SPRING BASE-D4		SPRING BASE-D8		NEOPRENE BASE	
		a_h	a_v	a_h	a_v	a_h	a_v	a_h	a_v
		141	-	142	-	138	-	142	-
		TEST No: 11	ANALYS[1] T= 0.29	TEST No: 35	ANALYS T= 1.47	TEST No: 64	ANALYS T= 1.47	TEST No:	ANALYS T= 1.21
HORIZONTAL	5	335	626	106	105	120	88	163	106
	4	261	573	67	69	89	69	135	102
	3	249	481	65	61	81	69	153	97
	2	204	386	65	73	84	76	136	90
	1	153	258	84	95	79	90	128	88
	B	-	-	125	133	93	123	138	84
VERTICAL	21				67		53		5.8
	22			77	63	58	51		4.2
	23				61		55		1.4

DISPLACEMENTS, cm

	LOCATION	FIXED BASE TEST	FIXED BASE ANALYS	SPRING BASE-D4 TEST	SPRING BASE-D4 ANALYS	SPRING BASE-D8 TEST	SPRING BASE-D8 ANALYS	NEOPRENE BASE TEST	NEOPRENE BASE ANALYS
HORIZONTAL	5	1.26	1.35	2.56	4.18	2.16	3.08	6.85	3.35
	4	1.05	1.25	2.09	3.49	1.75	2.57	7.50	3.32
	3	0.84	1.06	1.63	2.81	1.38	2.06	7.80	3.28
	2	0.63	0.81	1.26	2.16	1.12	1.57	8.40	3.22
	1	0.44	0.49	0.89	1.54	0.88	1.10	9.05	3.15
	B	-	-	?	0.93	?	0.61	6.35	3.04
VERTICAL	21				1.19		0.73		0.05
	22			0.81	0.97	0.53	0.67		0.03
	23				0.71		0.65		0.01

[1] *Damping ratio β= 1.5%*

TABLE 6.9

PB NS 20	PETROVAC, Scaled Time
Δt= 0.01 sec	

Peak Accel.	
a_h	a_v
427	198

ACCELERATIONS, cm/sec^2

	LOCATION	FIXED BASE		SPRING BASE-D4		SPRING BASE-D8		NEOPRENE BASE	
		a_h	a_v	a_h	a_v	a_h	a_v	a_h	a_v
		345	-	354	-	344	-	354	-
		TEST No: 14	ANALYS[1] T= 0.29	TEST No: 39	ANALYS T= 1.47	TEST No: 68	ANALYS T= 1.47	TEST No:	ANALYS T = 1.21
HORIZONTAL	5	1763	2222	226	102	245	112		139
	4	1268	2152	98	67	151	73		110
	3	1050	1819	113	100	141	133		79
	2	791	1309	216	154	151	153		74
	1	558	865	331	193	182	190		98
	B	-	-	546	207	276	237		134
VERTICAL	21				141		165		33
	22			268	116	158	133		23
	23				160		71		7.2

DISPLACEMENTS, cm

	LOCATION	FIXED BASE TEST	FIXED BASE ANALYS	SPRING BASE-D4 TEST	SPRING BASE-D4 ANALYS	SPRING BASE-D8 TEST	SPRING BASE-D8 ANALYS	NEOPRENE BASE TEST	NEOPRENE BASE ANALYS
HORIZONTAL	5	5.18	4.39	2.67	2.63	2.07	2.08		2.44
	4	4.38	4.05	2.18	2.37	1.71	1.78		2.40
	3	3.42	3.44	1.69	2.12	1.35	1.47		2.37
	2	2.45	2.60	1.31	1.86	1.11	1.25		2.31
	1	1.44	1.56	1.08	1.63	0.89	1.08		2.26
	B	-	-	?	1.31	?	0.86		2.19
VERTICAL	21				0.97		0.78		0.05
	22			0.89	0.81	0.52	0.67		0.04
	23				0.45		0.43		0.02

[1] *Damping ratio β= 1.5%*

TABLE 6.10

PN VK 40	PETROVAC, Real Time
$\Delta t = 0.02$ sec	

Peak Accel.	
a_h	a_v
427	198

ACCELERATIONS, cm/sec^2

	LOCATION	FIXED BASE		SPRING BASE-D4		SPRING BASE-D8		NEOPRENE BASE	
		a_h	a_v	a_h	a_v	a_h	a_v	a_h	a_v
		-	138	-	146	-	108	-	146
		TEST No: 17	ANALYS[1] T=0.29	TEST No: 42	ANALYS T=1.47	TEST No: 72	ANALYS T=1.47	TEST No:	ANALYS T=1.21
HORIZONTAL	5	40		2		40			0.0009
	4	29		10		22			
	3	34		12		26			0.0013
	2	29		14		24			
	1	12		19		19			
	B	-		81		38			0.0009
VERTICAL	21		25		130		84		735
	22		-	141	132	113	81		770
	23		14		144		84		1007

DISPLACEMENTS, cm

	LOCATION	FIXED BASE		SPRING BASE-D4		SPRING BASE-D8		NEOPRENE BASE	
HORIZONTAL	5	0.21		0.10		0.42			0
	4	0.09		0.06		0.35			
	3	0.08		0.06		0.23			0
	2	0.07		0.05		0.22			
	1	0.06		0.06		0.21			
	B	-		?		?			0
VERTICAL	21		0.0001		1.29		0.60		0.18
	22		-	0.62	1.36	0.30	0.65		0.20
	23		0.0001		1.53		0.73		0.27

[1] *Damping ratio β= 1.5%*

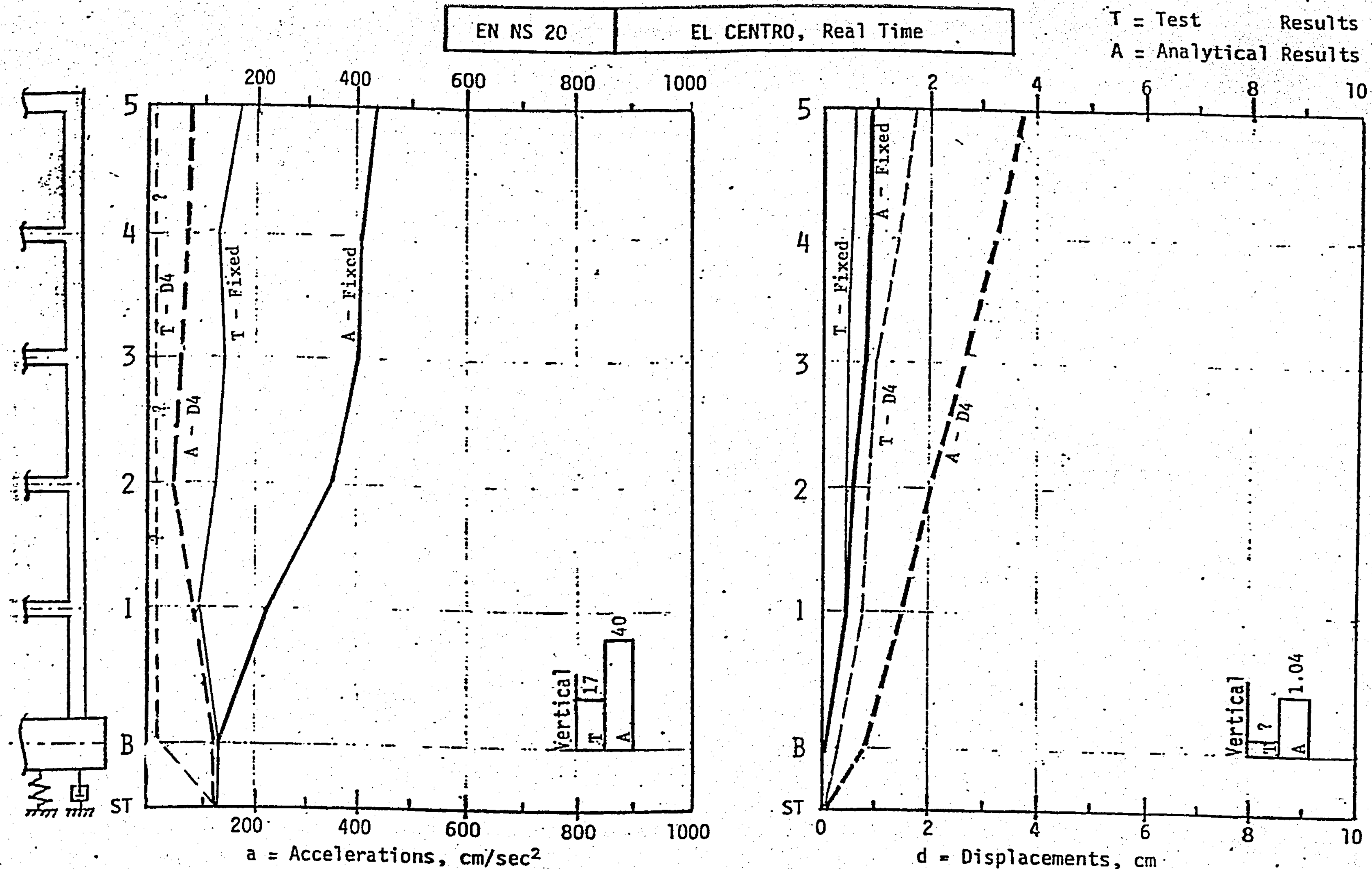


FIG. 6.2 - RESPONSE COMPARISON, D4 vs FB

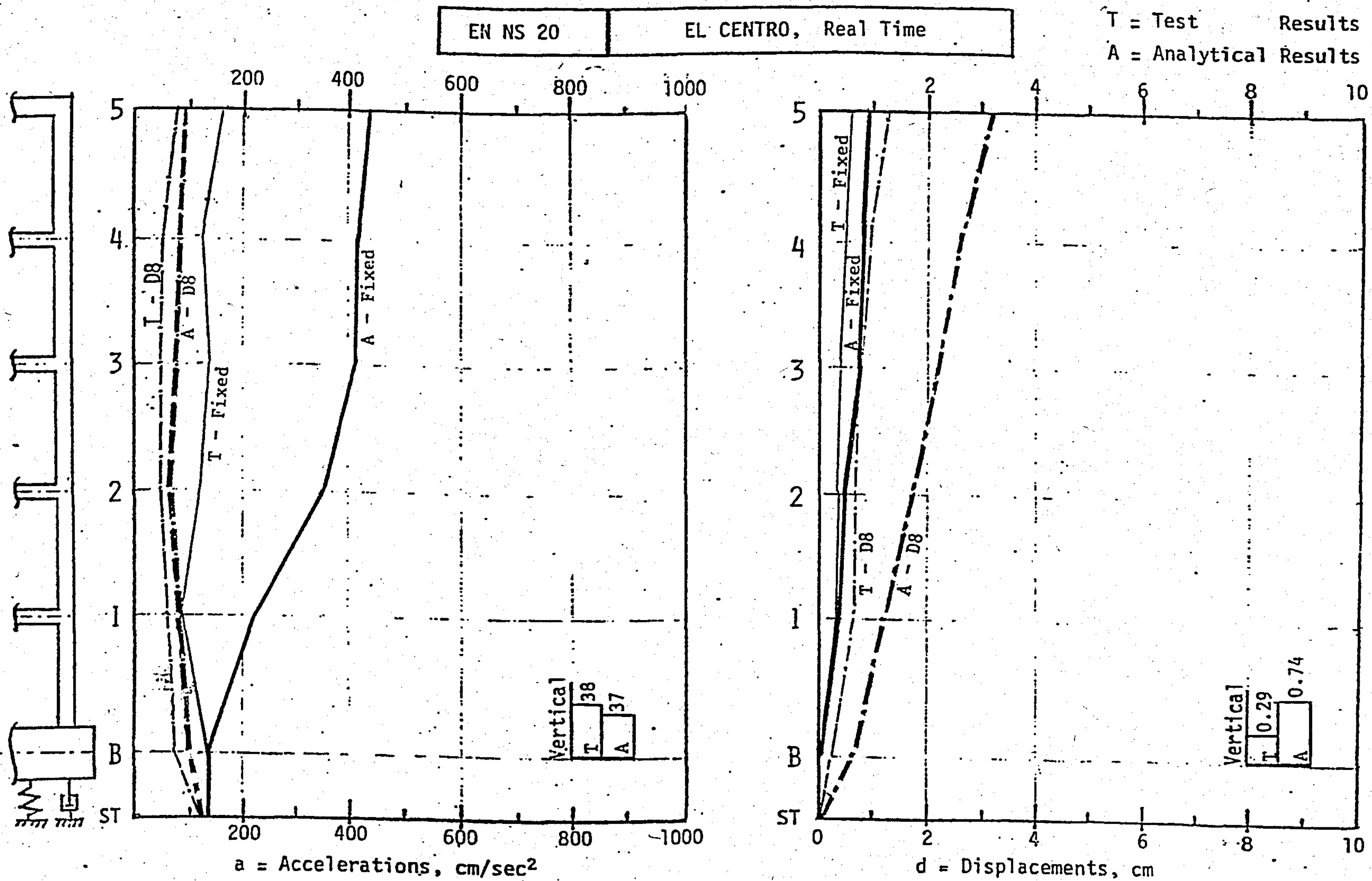


FIG. 6.3 - RESPONSE COMPARISON, D8 vs FB

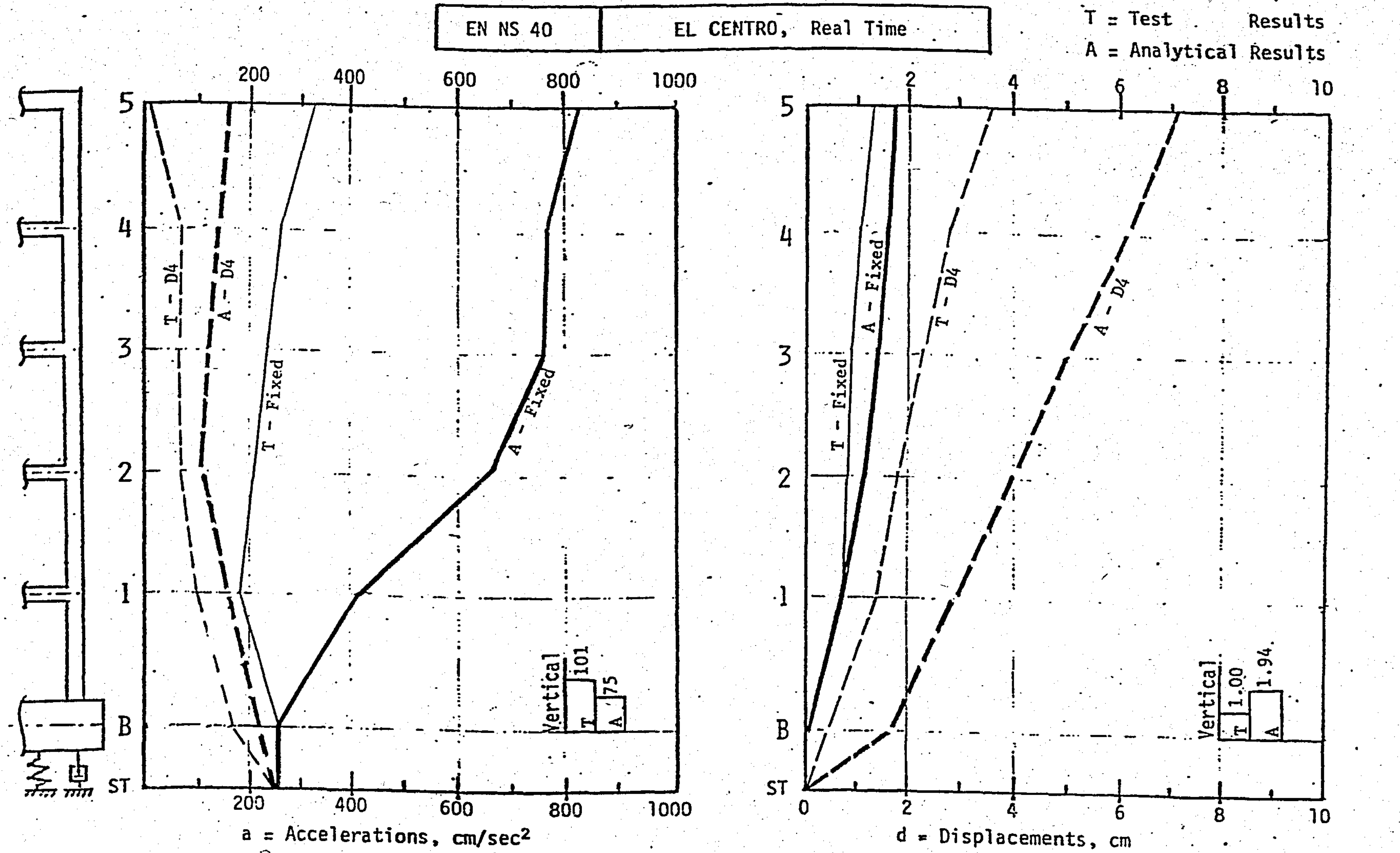


FIG. 6.4.- RESPONSE COMPARISON, D4 vs FB

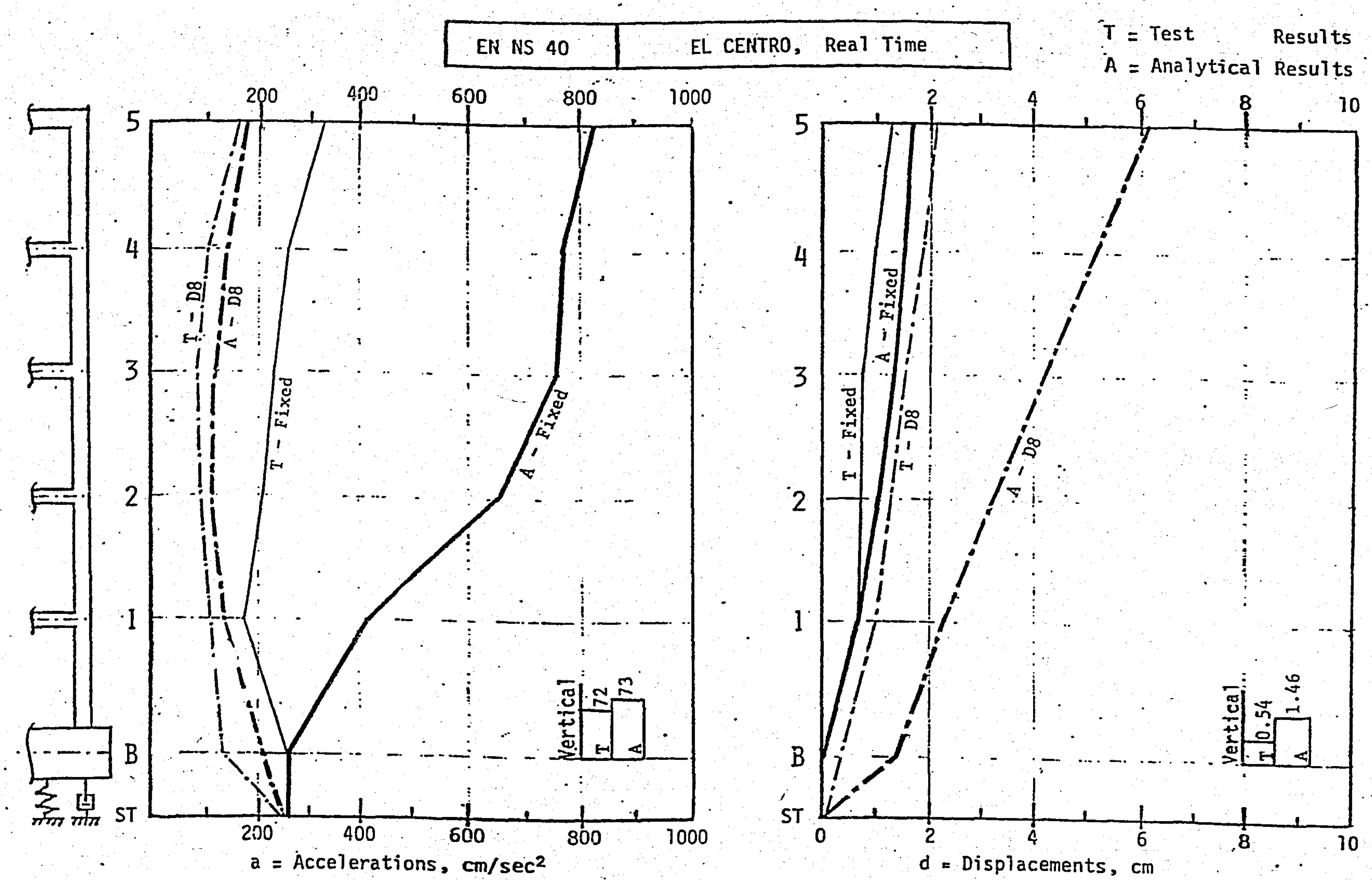


FIG. 6.5 - RESPONSE COMPARISON, D8 vs FB

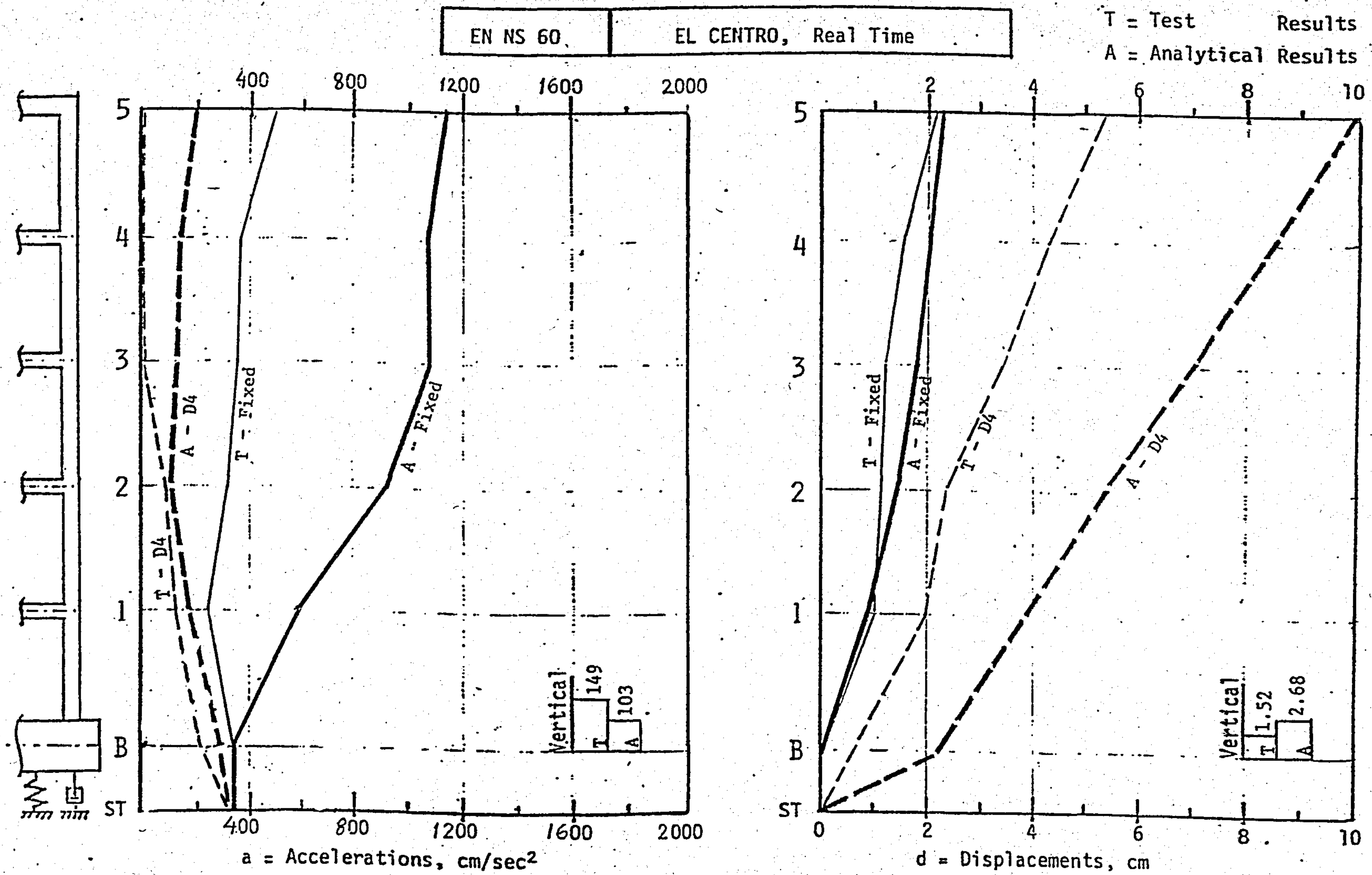


FIG. 6.6 - RESPONSE COMPARISON, D4 vs FB

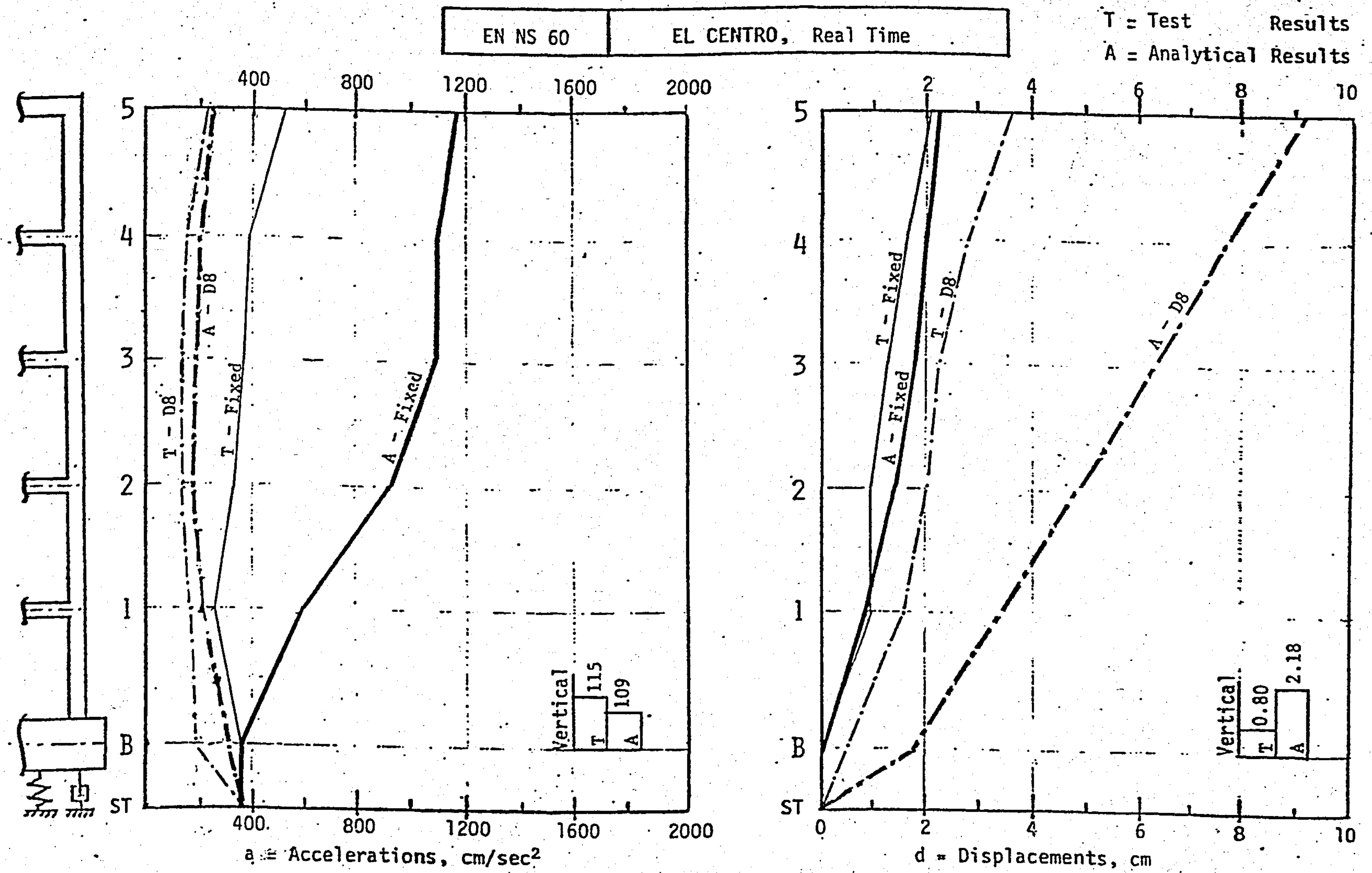


FIG. 6.7 - RESPONSE COMPARISON, D8 vs FB

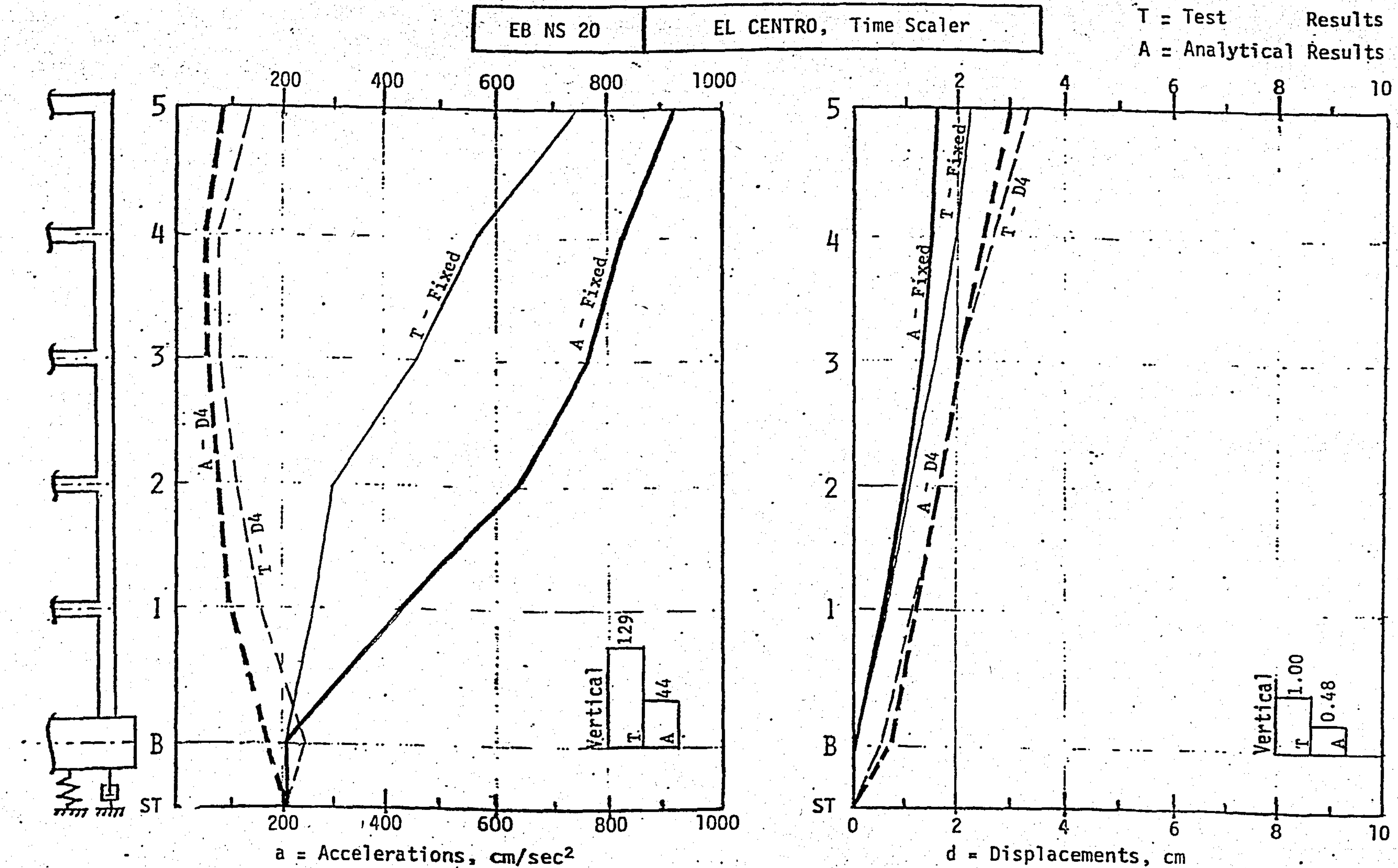


FIG. 6.8 - RESPONSE COMPARISON, D4 vs FB

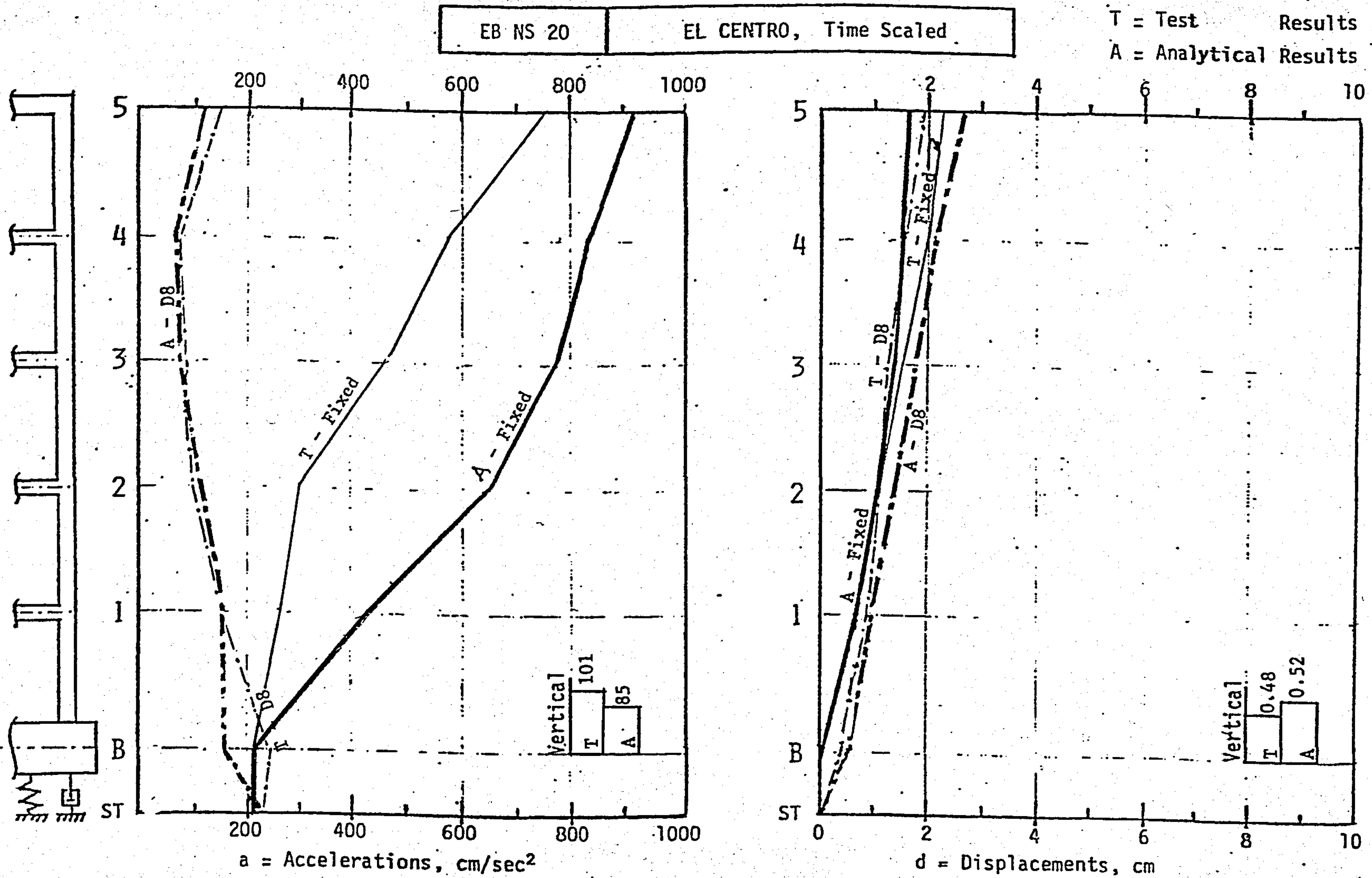


FIG. 6.9 - RESPONSE COMPARISON, D8 vs FB

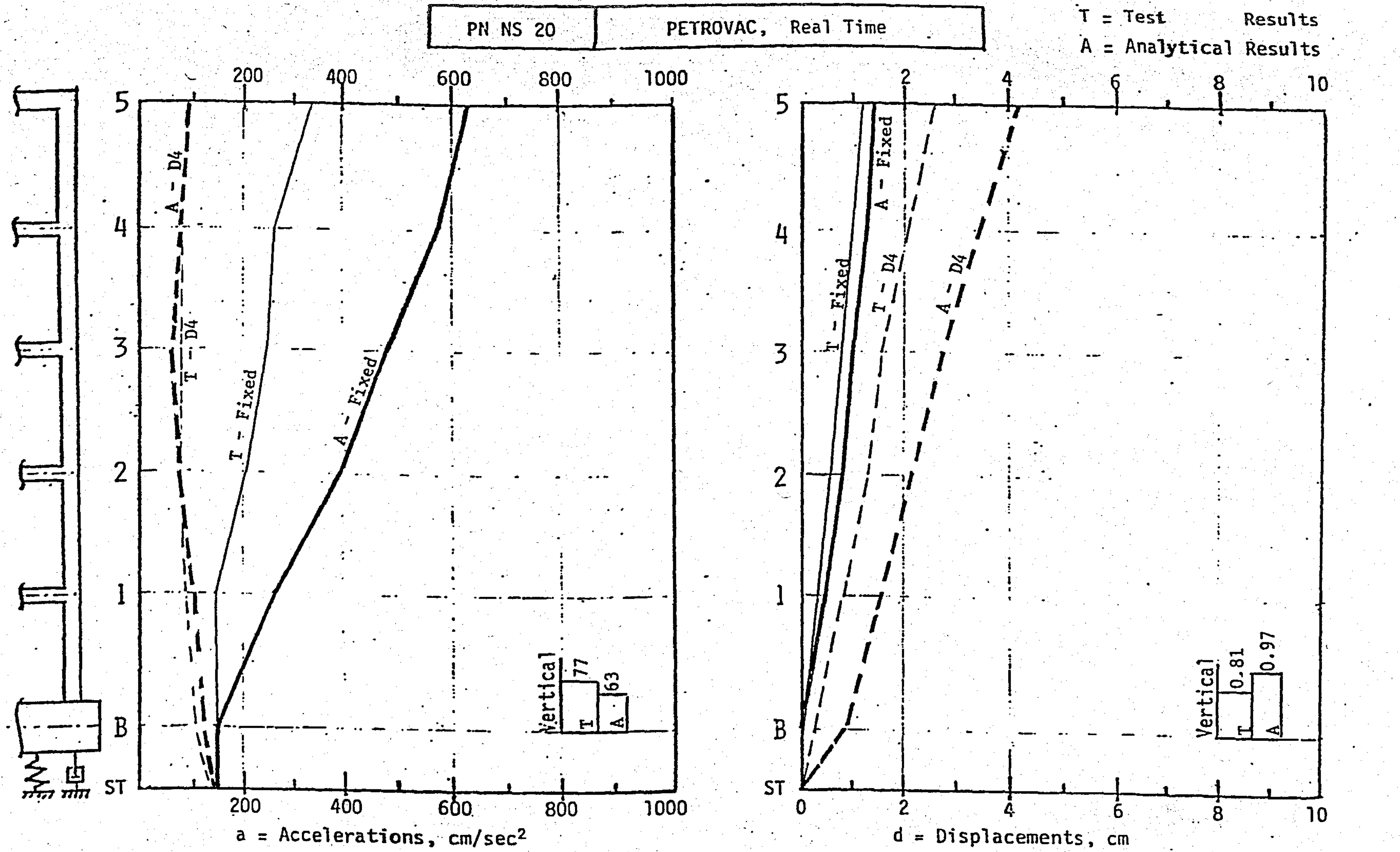


FIG. 6.10 - RESPONSE COMPARISON, D4 vs FB

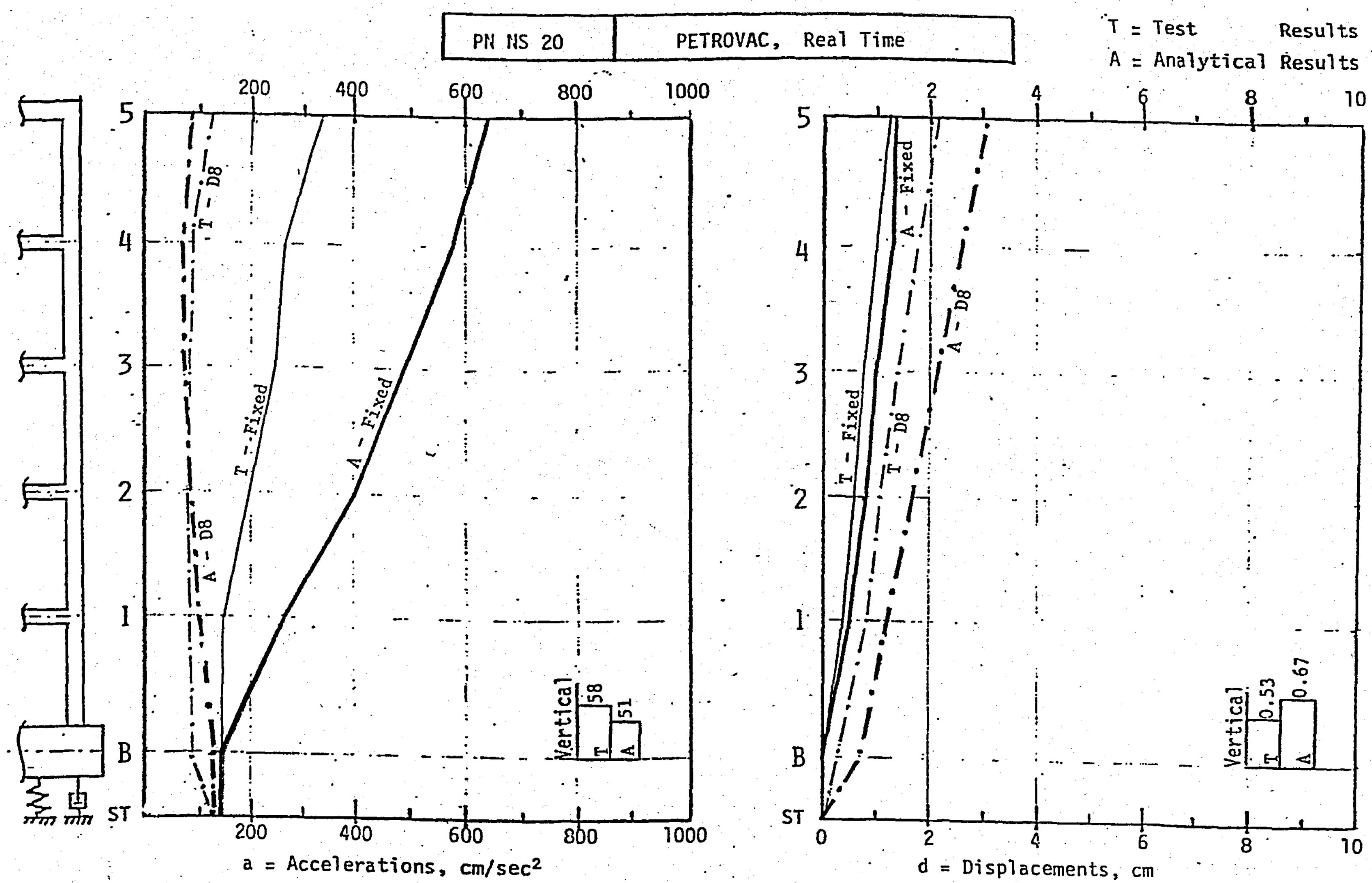


FIG. 6.11 — RESPONSE COMPARISON, D8 vs FB

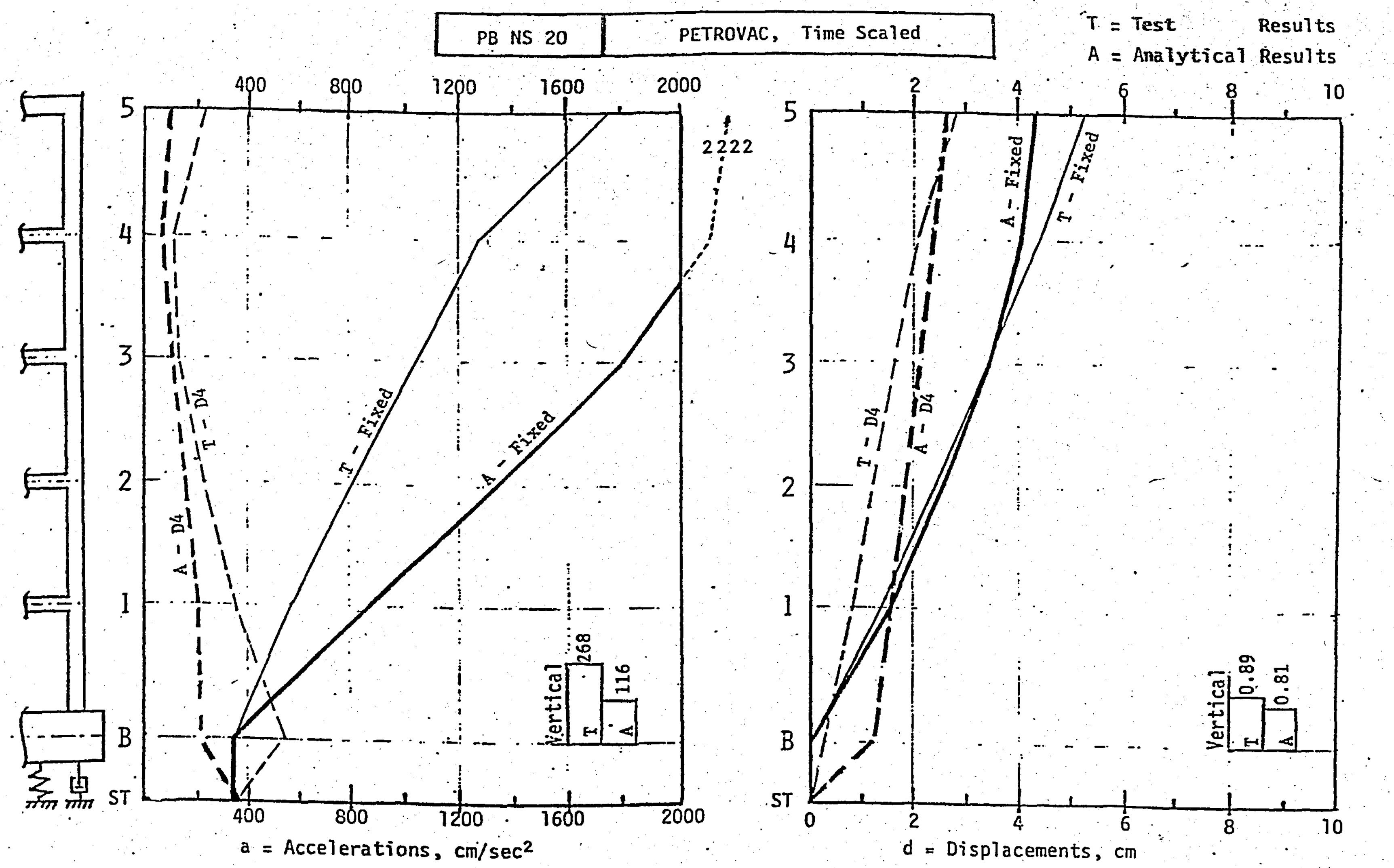


FIG. 6.12 - RESPONSE COMPARISON, D4 vs FB

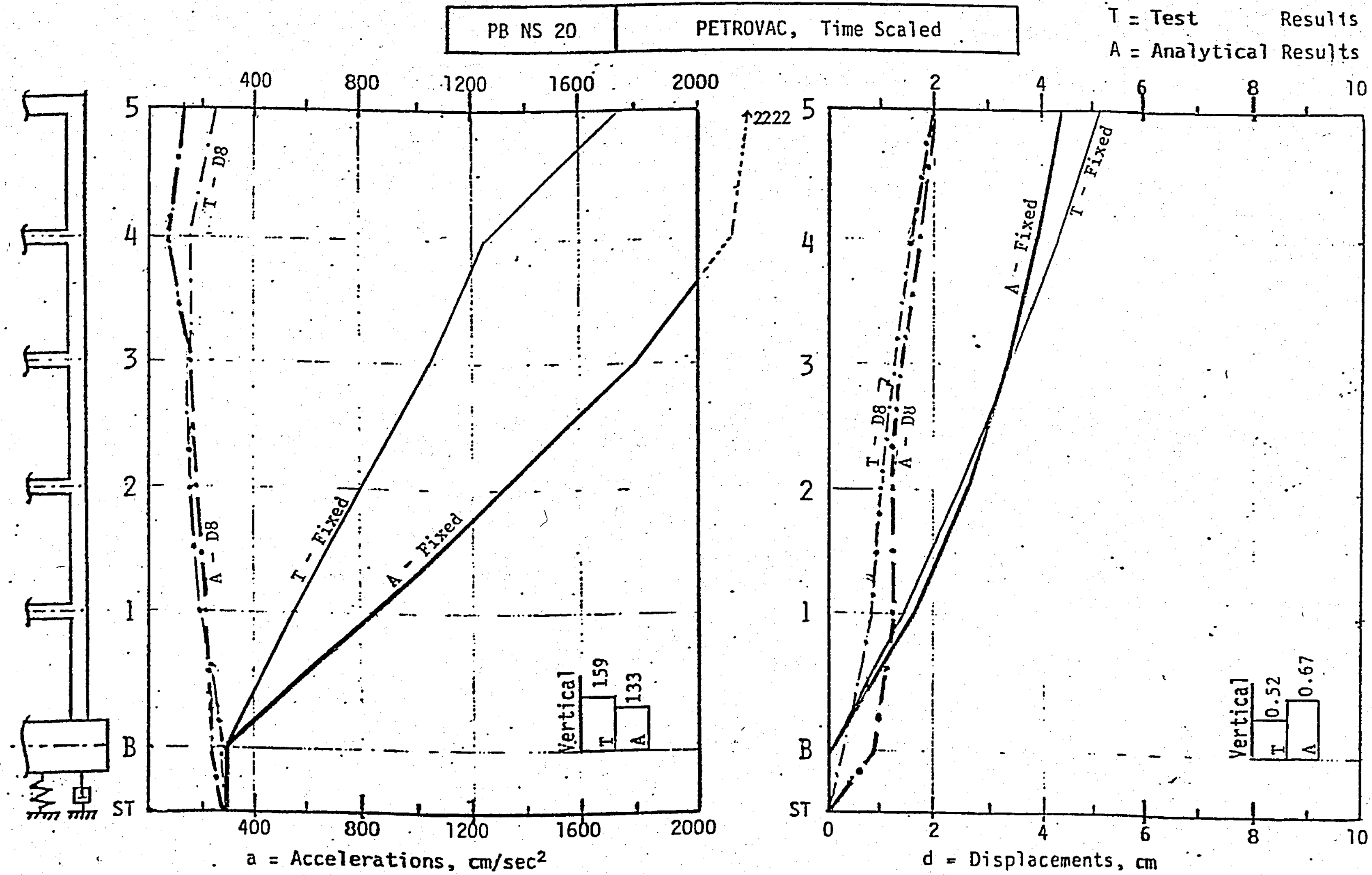


FIG. 6.13 — RESPONSE COMPARISON, D8 vs FB

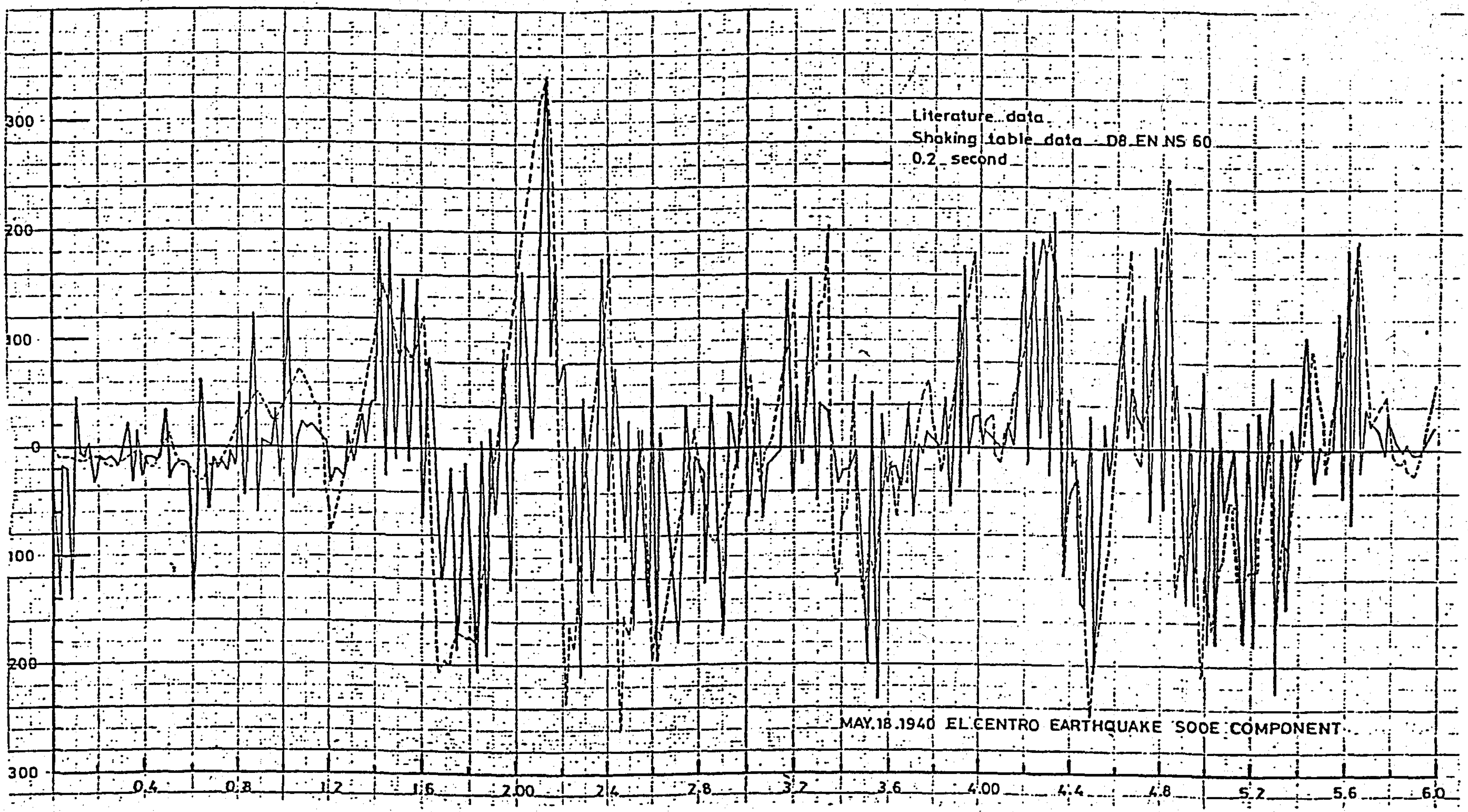


FIG. 6.14 - REAL ACCELERATIONS vs. SHAKING TABLE DATA
(EL CENTRO, 1940 EARTHQUAKE - HORIZONTAL COMPONENT)

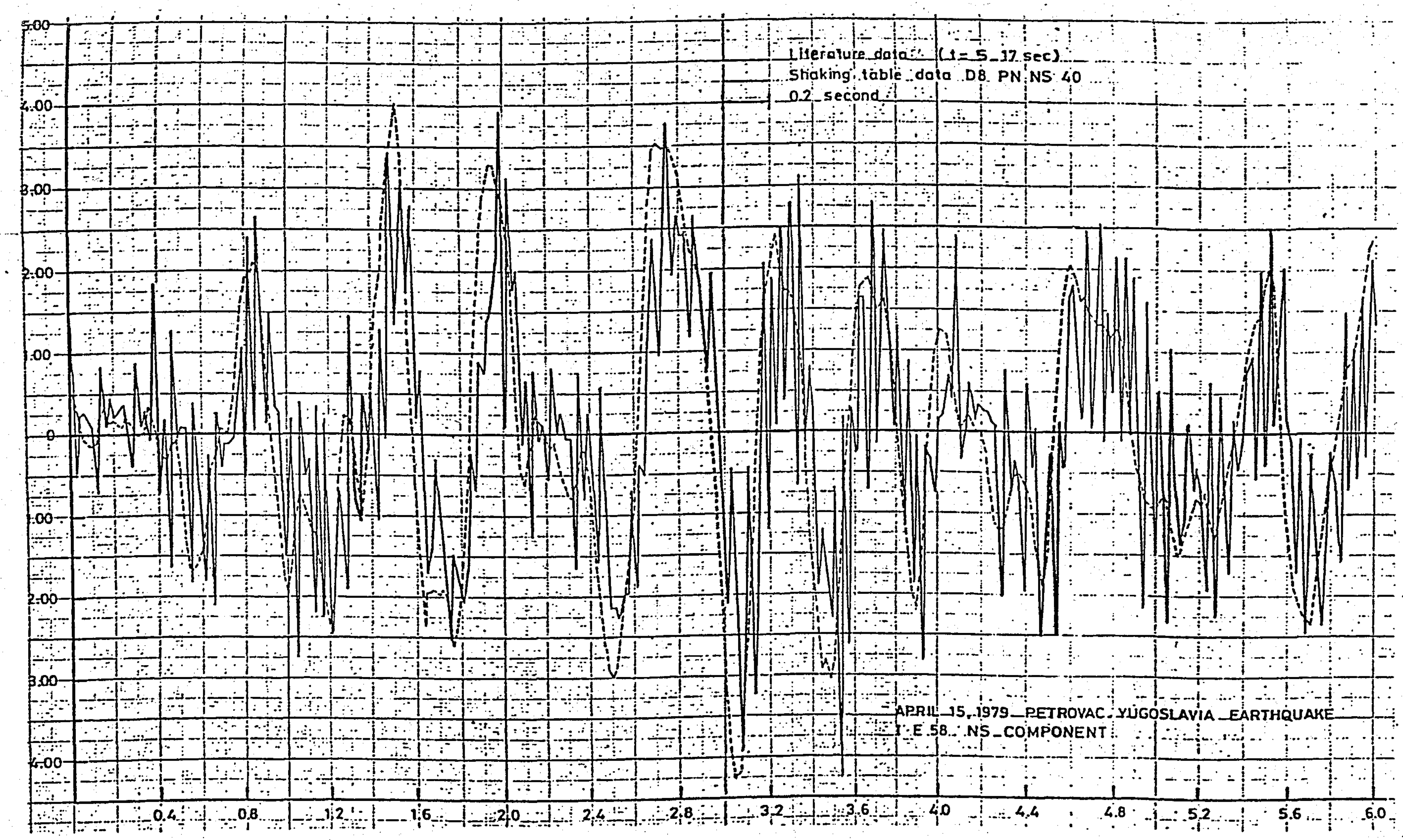


FIG. 6.15 - REAL ACCELERATIONS vs. SHAKING TABLE DATA
(PETROVAC. 1979 EARTHQUAKE - HORIZONTAL COMPONENT)

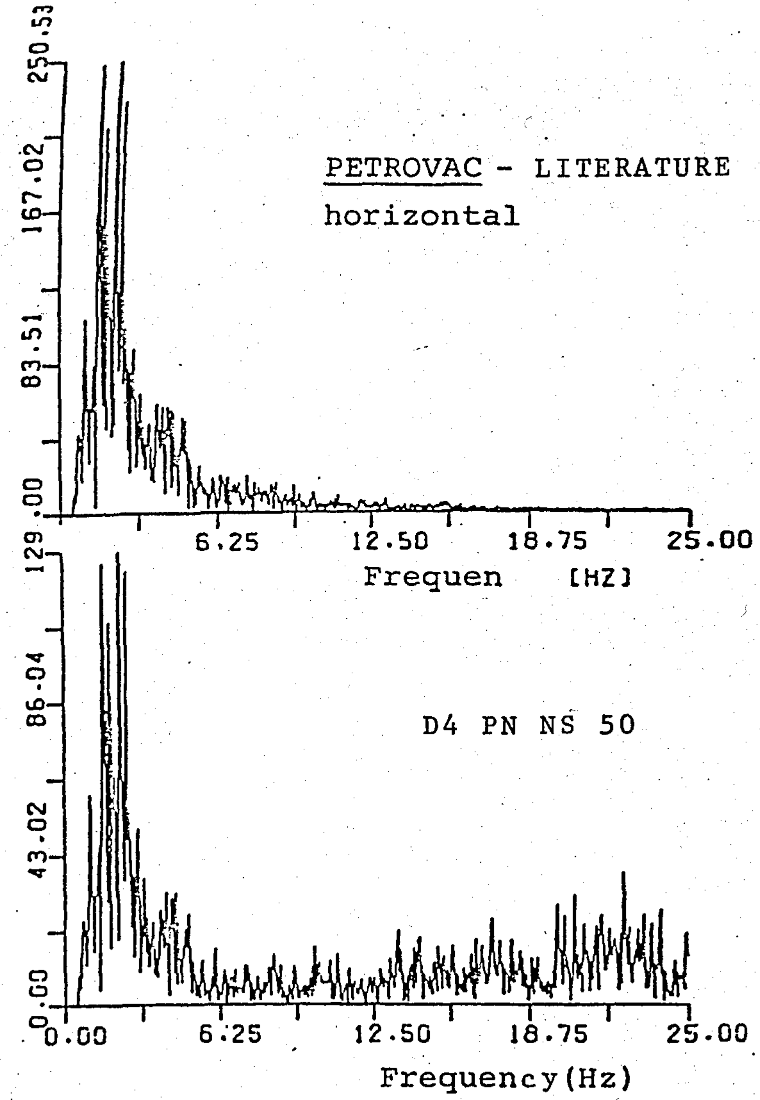


FIG. 6.16 - FOURIER TRANSFORMS OF PETROVAC EQ.(ACCEL-HOR)

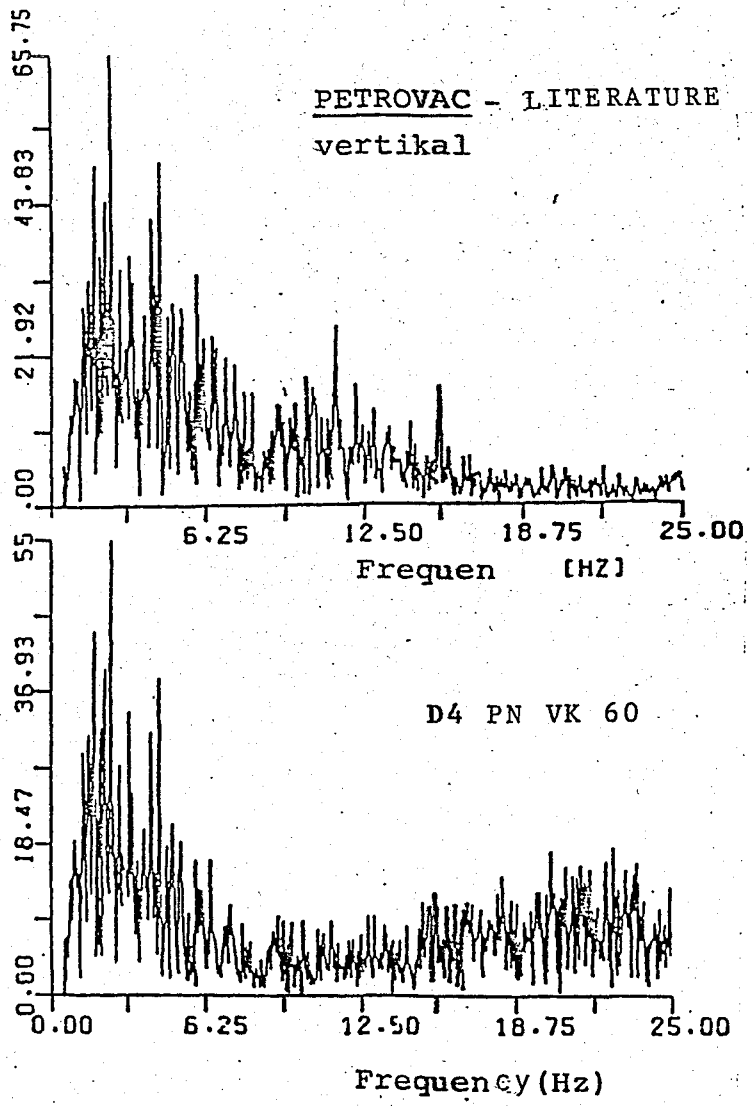


FIG. 6.17 - FOURIER TRANSFORMS OF PETROVAC EQ.(ACCEL-VER)

CHAPTER 7

OTHER ASPECTS OF VIBRATION ISOLATION

7.1. COMPARISON OF RUBBER AND SPRING ISOLATION SYSTEMS

It is already mentioned that a variety of shock absorbing devices and base isolation systems, such as mechanical restrainers, neoprene pads and spring-dashpot systems, also available in literature. The behavior of the spring-dashpot isolation system and the rubber elements have been already discussed, both experimentally and analytically in Chapters 4 and 5 in connection with a five storey steel frame. In this chapter, a more detailed comparative study will be presented in order to determine the relative efficiencies of the neoprene pads and the spring-dashpot isolation systems.

A) ANALYTICAL MODEL AND INPUT MOTIONS

For the purpose of comparing the behavior of the spring-dashpot vibration isolators with that of the neoprene pads, especially in the vertical direction a simple two-story test frame is selected as shown in Fig 7.1.

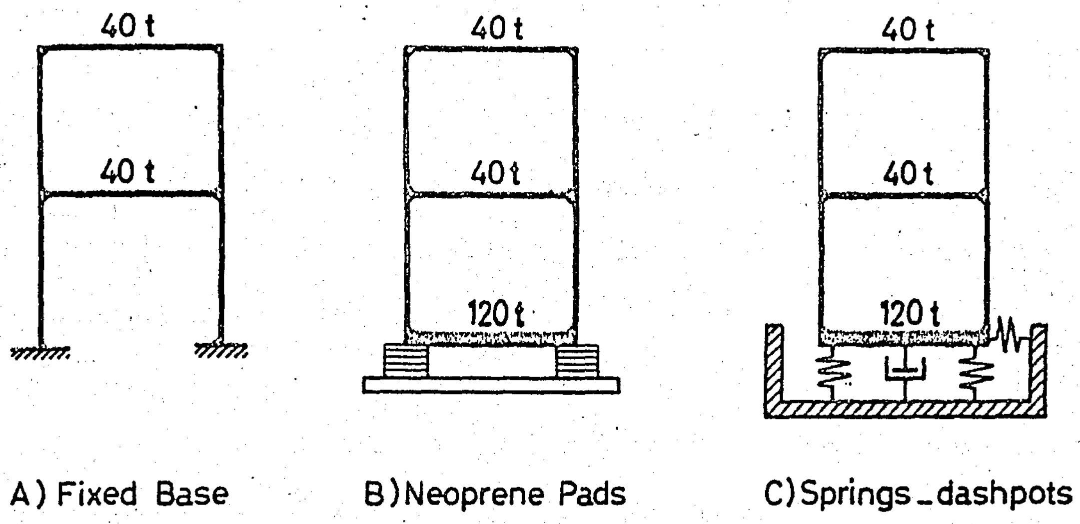


FIG. 7.1 - TWO-STOREY TEST FRAME

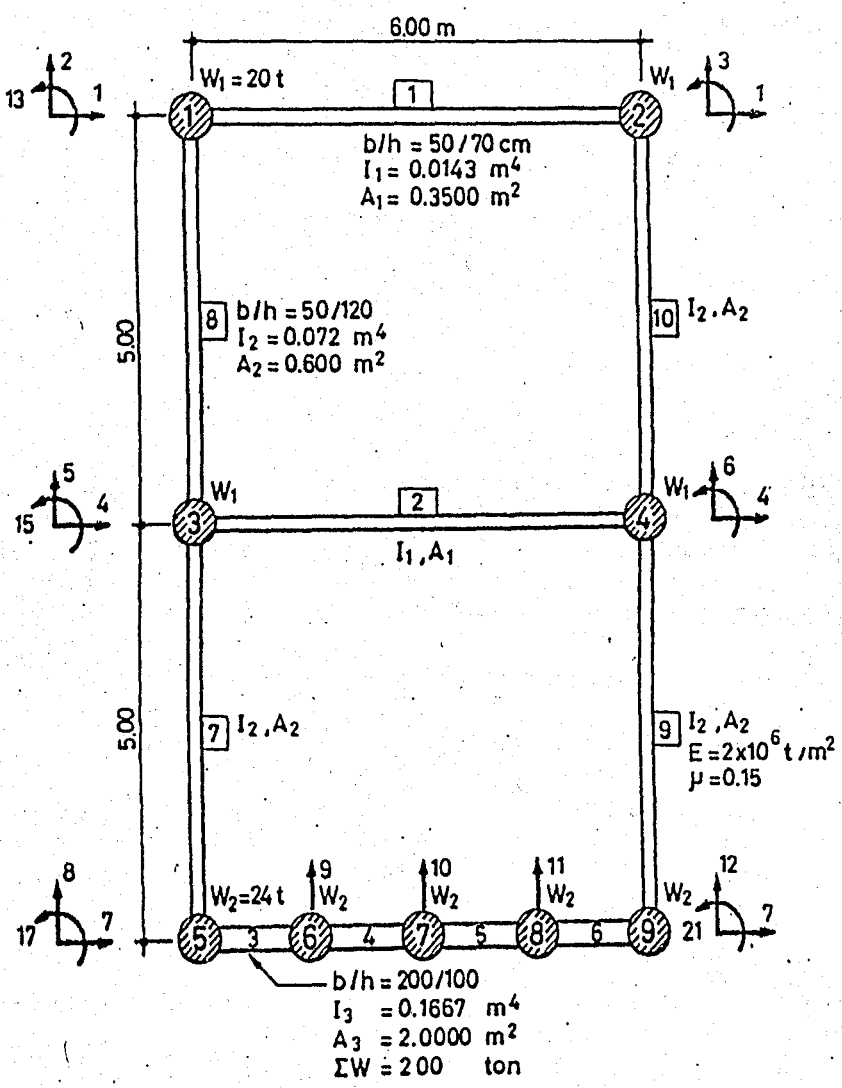


FIG. 7.2 - MATHEMATICAL MODEL OF THE TEST FRAME

The basic data and the mathematical model of the test frame is given in Fig. 7.2. A total of nine lumped masses are considered, each capable of vibrating in both horizontal and vertical directions.

The base of the model was considered to be supported in a number of different ways as shown in Fig. 7.3, so as to represent (a) the fixed base case, (b) the isolation by neoprene pads, and (c) the isolation by springs and dashpots. In order to arrive at the best possible vibration isolation, to discover the relative efficiency of the locations of the dashpots, and also to find out the most suitable orientation of springs and viscous dampers, a parametric study has been conducted also as summarized in Table 7.1.

The two-storey test frame with different support conditions is subjected to the N-S and vertical components of the 1940 El Centro earthquake. For the purpose of investigating the importance of vertical isolation, the vertical and horizontal components of the input earthquake motion have been applied first separately and then simultaneously.

The peak ground acceleration is 0.35 g (3.45 m/sec^2) in the horizontal and 0.21 g (2.06 m/sec^2) in the vertical directions. The reason for this earthquake to be selected as the input ground motion is that it causes a significant

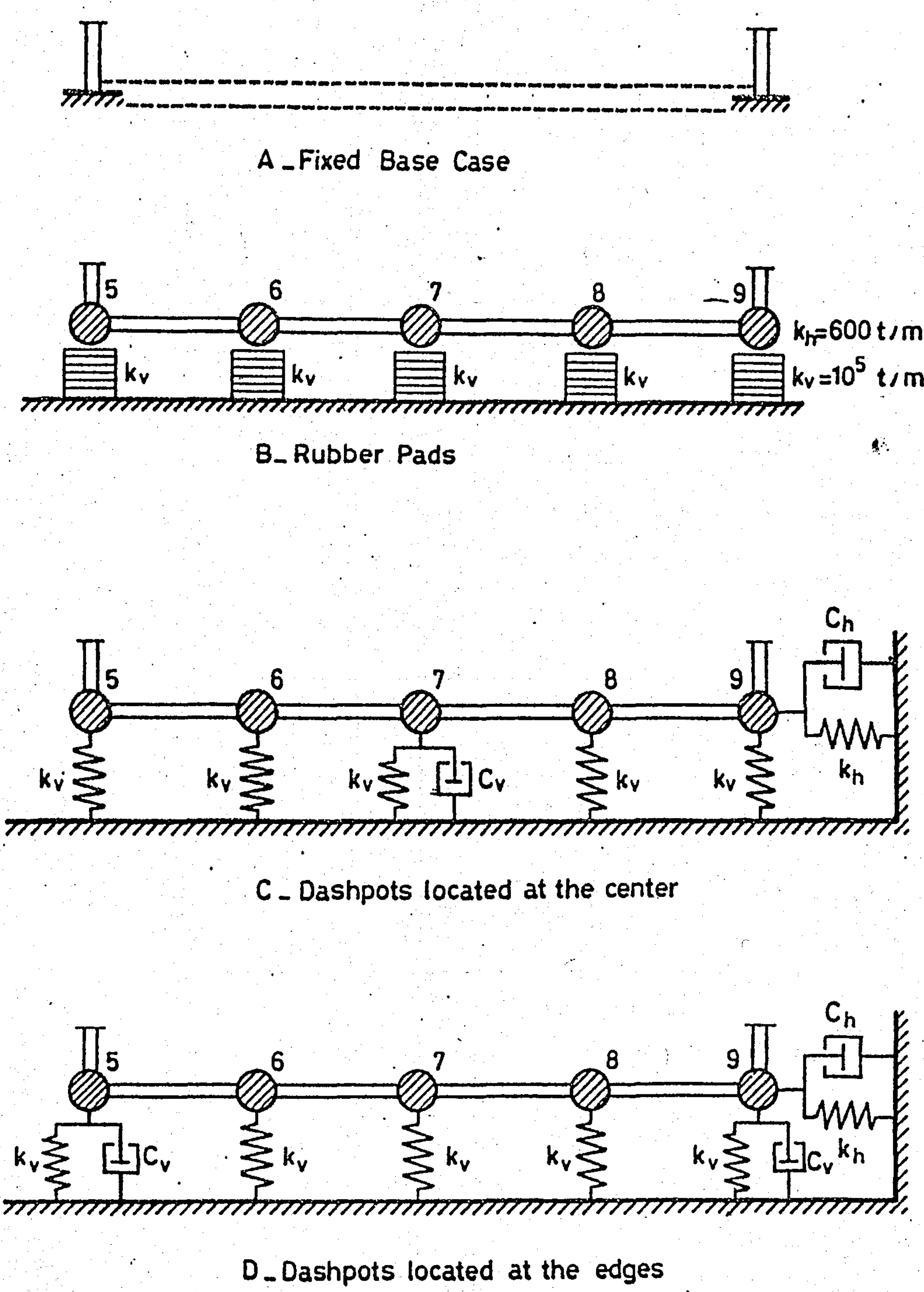


FIG. 7.3 - VIBRATION ISOLATION ARRANGEMENTS

TABLE 7.1 PARAMETERS OF VIBRATION ISOLATION OF THE TWO-STOREY FRAME

FOUNDATION ISOLATION CASE	SPRINGS (ton/m)		DAMPING (-)		VISCOSITY (t-sec/m)		T = PERIODS (sec)		f = FREQUENCY (Hz)	
	k_v	k_h	β_v	β_h	C_v	$C_h^{(1)}$	Vertical	Rocking	Vertical	Rocking
A. FIXED BASE CASE	∞	∞	0	0	0	0	0.03	0.28	33.33	3.57
B. NEOPRENE PADS	10^5	600	0	7%	0	15.3	0.05	1.17	20.00	0.85
C. SPRING-DASHPOT SYSTEM										
1. Springs only, No dashpots	660	1200	0	0	0	0	0.50	1.42	2.00	0.70
2. Dashpots at the center	330	600	20%	34%(2)	73.2	43.9	0.70	1.99	1.43	0.50
3. Dashpots at the edges (a)	330	600	20%	34%(2)	36.6	43.9	0.70	1.99	1.43	0.50
(b)	660	1200	20%	34%(2)	52.5	63.0	0.50	1.42	2.00	0.70

(1) Viscosity coefficient in the horizontal direction is taken as 60% that of the vertical direction.
(2) Horizontal critical damping ratio is calculated on the basis of the rocking natural period.

disturbance on relatively rigid structures like two-storey test frame.

B) ENERGY ABSORPTION BY VISCODAMPERS AND NEOPRENE PADS

The major handicap in vibration isolation is that the rigid body displacements of the structure may be excessively large. The use of viscous dampers however, is an indispensible tool in eliminating these large displacements. The viscous dampers provide sufficient amount of damping, up to the value of 20% to 30% of critical damping, in all three directions. The successful application of viscous dampers and the performance of spring-dashpot systems are presented by Huffmann (1980).

Based on experimental evidence, the upper limit of the amount of damping supplied by the viscous dampers in the vertical direction is taken as 20% of the critical value. Once the critical damping ratio is known, the coefficient of viscosity C is determined, for the first mode of vibration, from

$$\beta = \frac{C}{C_{cr}} = \frac{C}{2mw} , \quad (w = 2\pi/T)$$

$$C = \frac{4\ W}{gT} \tag{7.1}$$

in which, W = total weight of the structure, and T = natural period of vibration of the structure. The coefficient of viscosity in the horizontal direction is assumed to be 60% that of vertical direction.

In the case of neoprene pads, the critical damping ratio in the horizontal direction is assumed as $\beta_h = 0.07$, which corresponds, for the natural period of $T = 1.17$ seconds in the rocking motion, to the coefficient of viscosity of $C_h = 15.3$ ton-sec/m. No viscous damping is assumed to be present in the vertical direction when neoprene pads are used.

C) DISCUSSION OF RESULTS

For the test frame shown in Fig. 7.2 and for all cases of vibration isolation given in Table 7.1, the time history of displacements, velocities and accelerations at each node have been calculated. In this way, it has been possible to determine the axial forces, shears and moments in each element of the structure due to the single or combined action of the horizontal and vertical components of the earthquake.

The main emphasis in the calculations has been to demonstrate the significance of the vertical and rocking motions. It has been shown that in the case of neoprene pads, the vertical accelerations in the structure are prohibitively large, since these pads are unable to provide vibration isolation in the vertical direction. The spring-dashpot system however, is a three dimensional arrangement and it is capable of providing vibration isolation in all three directions. Therefore, the response of the structure in vertical and rocking motions also becomes very small.

Although the complete time history response of all the nodes of the two-storey test frame has been obtained, only the summary of the results for Nodes 1 and 5 are given in Table 7.2. Comparative results are also summarized in Table 7.3. It is seen that in the case of neoprene pads the vertical acceleration at the top storey at Node 1 is magnified more than six times. But, in the case of springs with dashpots the vertical acceleration is about the same as that of the ground motion.

Maximum horizontal absolute accelerations and relative displacements of the test structure, due to the combined action of the horizontal and vertical components of the earthquake, are illustrated comparatively in Fig. 7.4. Maximum vertical absolute accelerations and relative displacements, due to the same input motion, are illustrated comparatively in Fig. 7.5. In these figures, the results for the viscodampers case correspond to the natural periods of $T_r = 1.42$ sec and $T_v = 0.50$ sec. It is seen that the best isolation is achieved, especially in the vertical direction, by means of the spring and dashpot system. Isolation by means of neoprene pads in vertical motion is even worse than the case with no isolation.

TABLE 7.2 - MAXIMUM RESPONSE OF NODE 1 AND 5 OF THE TEST FRAME

INPUT MOTIONS (1940 El Centro Earthquake)		NODE 1				NODE 5			
		FIXED BASE	NEOPRENE PADS	SPRINGS ONLY	SPRINGS AND DASHPOTS	FIXED BASE	NEOPRENE PADS	SPRINGS ONLY	SPRINGS AND DASHPOTS
NATURAL PERIODS (sec)									
Horiz./Rocking		0.28	1.17	1.42	1.42	0.28	1.17	1.42	1.42
Vertical		0.03	0.05	0.50	0.50	0.03	0.05	0.50	0.50
HORIZONTAL EARTHQUAKE ($3.45 m/sec^2$)									
Acceleration (m/sec^2)	Horiz.	16.90	3.54	4.06	2.18	3.45	3.06	9.46	3.88
	Vert.	0.31	0.07	3.65	1.63	0	0.05	3.63	1.63
Displacement (cm)	Horiz.	3.14	10.72	12.67	9.88	–	9.93	10.31	3.18
	Vert.	0.04	0.04	5.57	2.63	0	0.03	5.55	2.63
VERTICAL EARTHQUAKE ($2.06 m/sec^2$)									
Acceleration (m/sec^2)	Horiz.	0	0	0	0	0	0	0	0
	Vert.	6.57	12.72	2.61	1.21	2.06	7.95	2.59	1.18
Displacement (cm)	Horiz.	0	0	0	0	0	0	0	0
	Vert.	0.02	0.08	1.61	0.69	–	0.05	1.60	0.68
(HOR+VERT) EARTHQUAKE									
Acceleration (m/sec^2)	Horiz.	16.90	3.54	4.06	2.18	3.45	3.06	9.46	3.88
	Vert.	6.72	12.73	5.62	1.77	2.06	7.95	5.61	1.73
Displacement (cm)	Horiz.	3.13	10.72	12.67	9.88	–	9.93	10.31	3.18
	Vert.	0.04	0.10	6.32	2.64	–	0.06	6.30	2.64

Notes: 1. Neoprene pads are assumed to provide 7% critical damping in the horizontal direction.
2. Viscodampers are assumed to provide 20% critical damping in the vertical direction.
3. Viscodampers are placed at the extreme edges of the foundation.

TABLE 7.3 RESPONSE VALUES AT ROOF LEVEL

INPUT MOTION (1940 El Centro N-S)	DIRECTION	FIXED BASE	RUBBER PADS	SPRING BASE	
				Without Dampers	With Visco-dampers
T = NATURAL PERIODS	Hor Ver	0.28 sec 0.03 sec	1.17 sec 0.05 sec	1.42 sec 0.50 sec	1.42 sec 0.50 sec
(HOR + VERT) EARTHQUAKE					
Accelerations (a_g = 0.35 g)	Hor	4.90 a_g	slides	1.18 a_g	0.63 a_g
(a_g = 0.20 g)	Hor	4.90 a_g	1.03 a_g	1.18 a_g	0.63 a_g
(a_g = 0.20 g)	Ver	3.19 a_g	6.18 a_g	2.73 a_g	0.86 a_g
Displacements (a_g = 0.35 g)	Hor	3.13 cm	slides	12.67 cm	9.88 cm
(a_g = 0.20 g)	Hor	1.79 cm	6.13 cm	7.24 cm	6.65 cm
(a_g = 0.20 g)	Ver	0.04 cm	0.10 cm	6.32 cm	2.64 cm

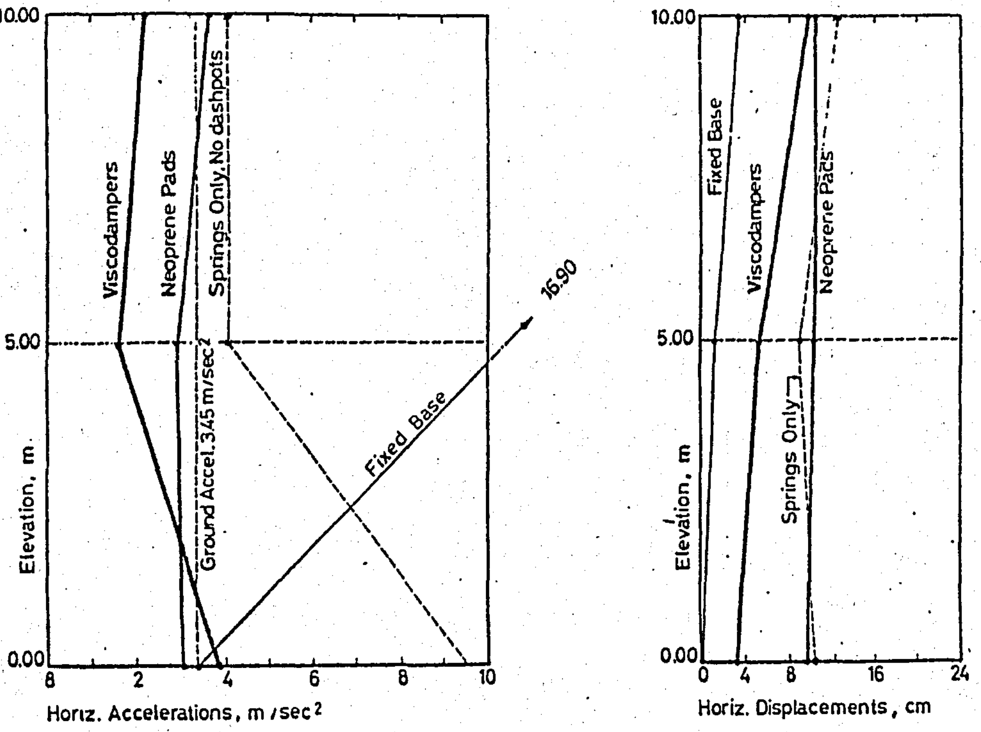


FIG. 7.4 - MAXIMUM HORIZONTAL RESPONSE DUE TO THE COMBINED ACTION OF (HOR+VERT) EARTHQUAKE

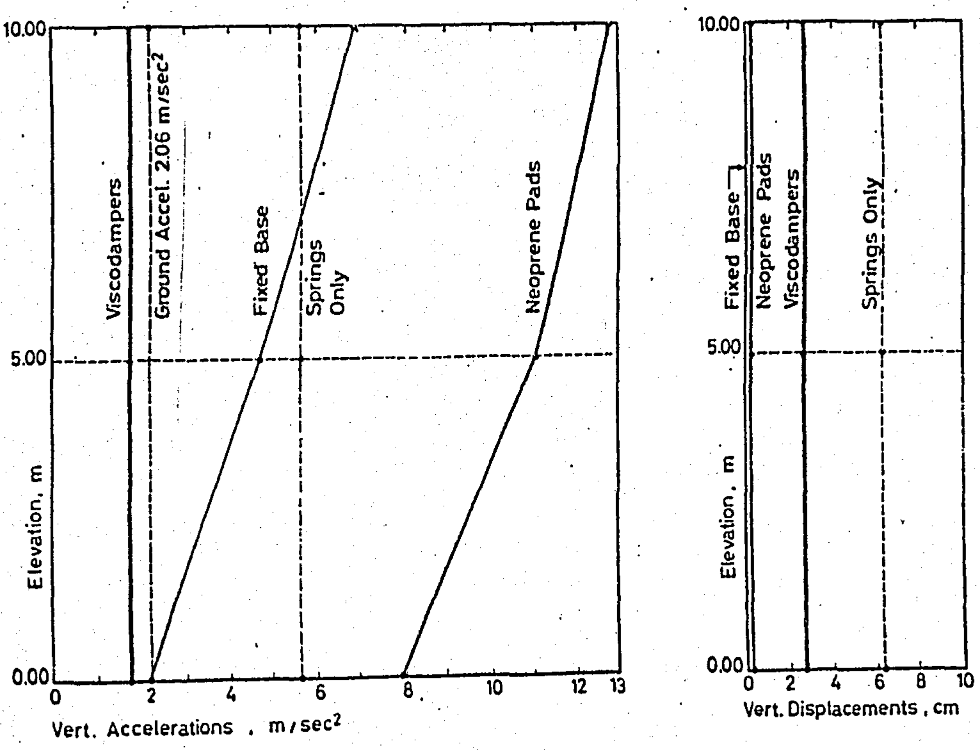


FIG. 7.5 - MAXIMUM VERTICAL RESPONSE DUE TO THE COMBINED ACTION OF (HOR+VERT) EARTHQUAKE

From the discussion of the results it appears that the spring-dashpot of base isolation is more effective than the rubber pads, in reducing the acceleration responses especially in the vertical direction. The neoprene pads are equivalent to elastic springs only in the horizontal direction. Their capacity of providing flexibility in the vertical direction is very small. Therefore, the natural periods of vibration of the structure in the vertical motion will remain almost unchanged when neoprene pads are used. Consequently, the structure and its components will not be safeguarded against vertical earthquake motions. Spring and dashpot systems however, can provide any desirable degree of flexibility in all three directions of motion. Therefore, complete vibration isolation is achieved for all types of motions including the vertical and rocking motions, thus resulting in safer and more reliable design against earthquakes.

D) OPTIMUM NATURAL PERIODS AND CRITICAL DAMPING

The optimum values of spring coefficients are those corresponding to the natural periods of $T_r = 1.42$ sec in the rocking motion and $T_v = 0.50$ sec in the vertical motion. Shorter natural period systems suffer higher accelerations while the longer natural periods cause excessively large deflections requiring unnecessarily costly damping facilities. The best arrangement of viscodapers is achieved, if the critical damping ratio is selected in the order of 20% in the vertical direction.

E) TIME HISTORY RESPONSE COMPARISONS

The time history variation on the acceleration and displacement responses of Node 1, due to the combined action of both the horizontal and vertical earthquake input, are illustrated in Fig. 7.6, 7.7 and 7.8. The efficiency of the spring-dashpot system over the neoprene pads is clearly observed in these figures.

F) LOCATIONS OF VISCODAMPERS

In order to determine the relative efficiency of the locations of the viscodampers, identical analyses have been performed once placing the viscodampers at the center, then placing them at the extreme edges of the foundation. Results of these two sets of analyses are compared in Figs. 7.9 and 7.10. Response is more efficienty reduced if the dampers are located at the edges.

G) INFLUENCE ON AXIAL FORCES OF COLUMNS

In ordinary structures, displacements of the underlying soil produce large relative storey displacements which develop elongation and shortening of the vertical members at opposite sides of the structure simultaneously, causing the rocking of the system. The step-by-step direct integration displayed that in the spring isolated cases although horizontal displacements being larger than the systems with no isolation, the structure sways as a rigid

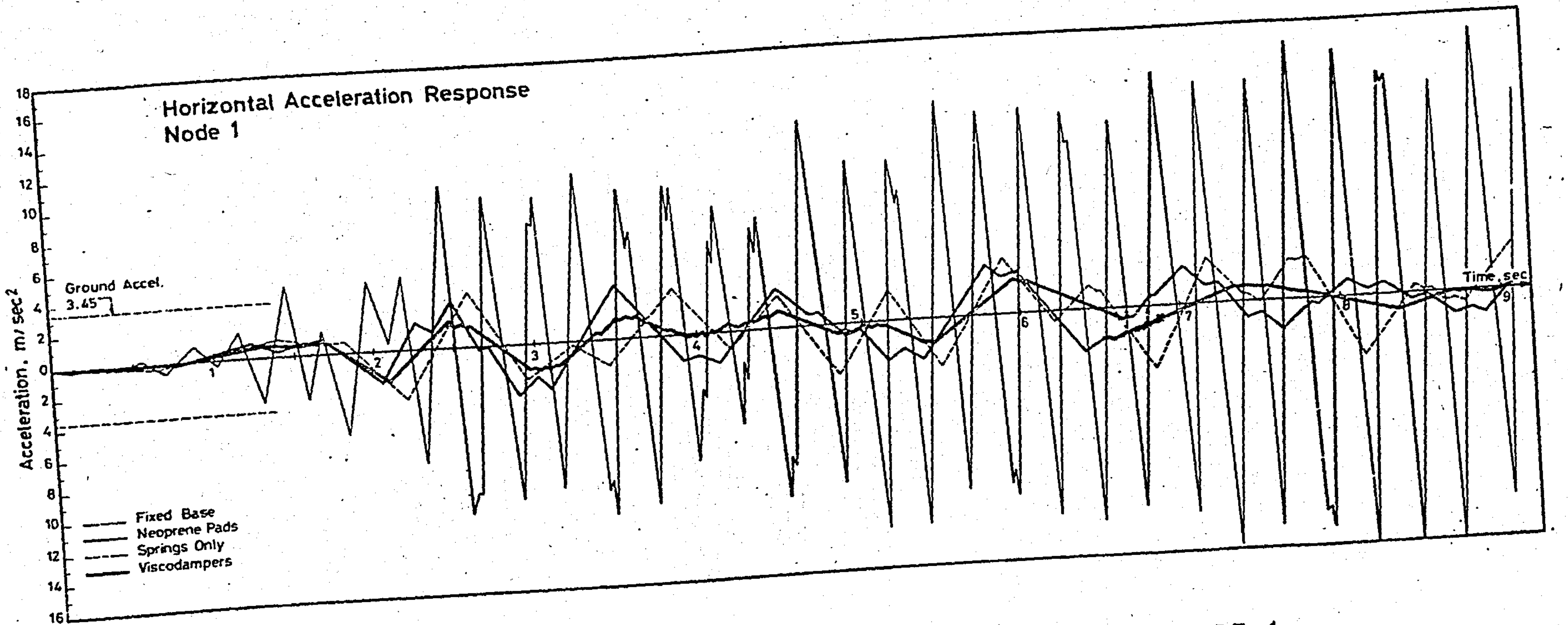


FIG. 7.6 - HORIZONTAL ACCELERATION RESPONSE , NODE 1

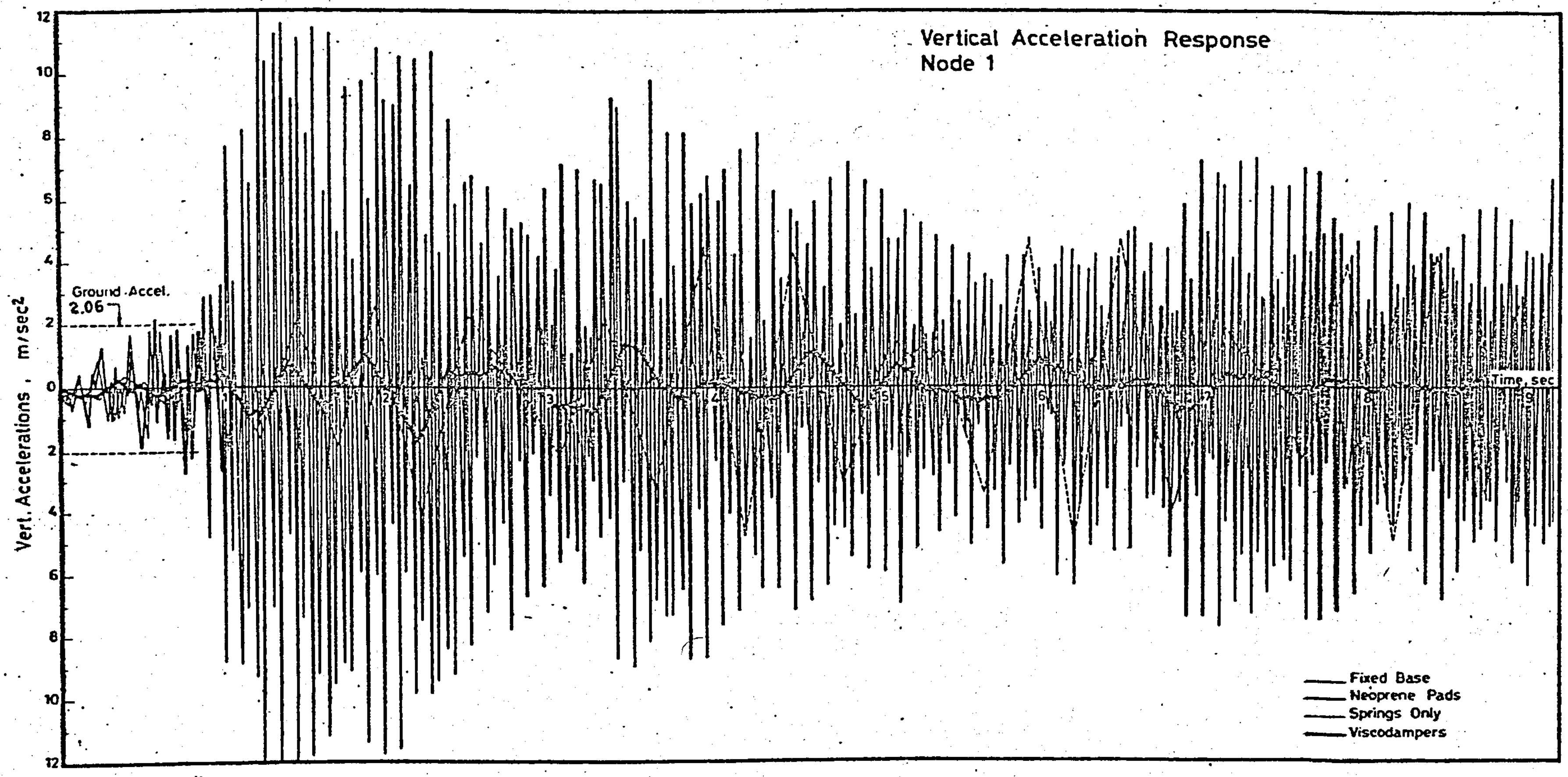


FIG. 7.7 - VERTICAL ACCELERATION RESPONSE , NODE 1

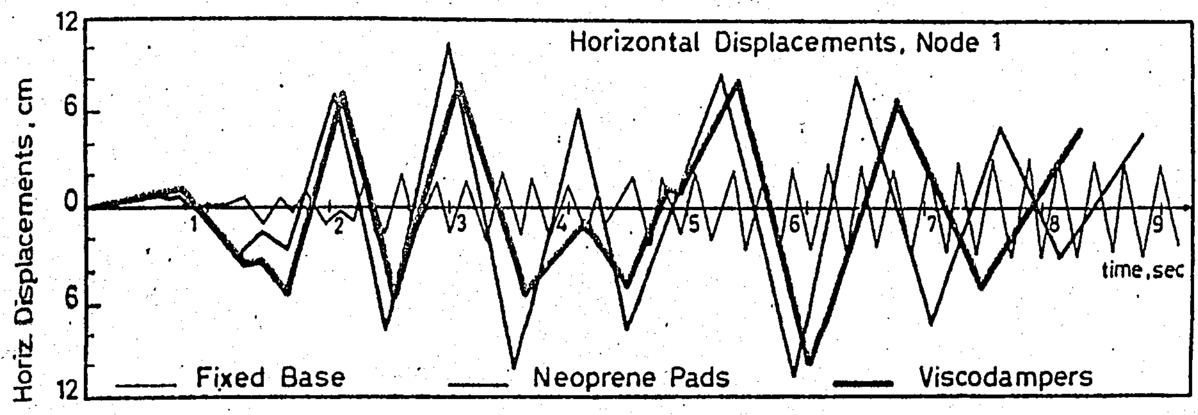
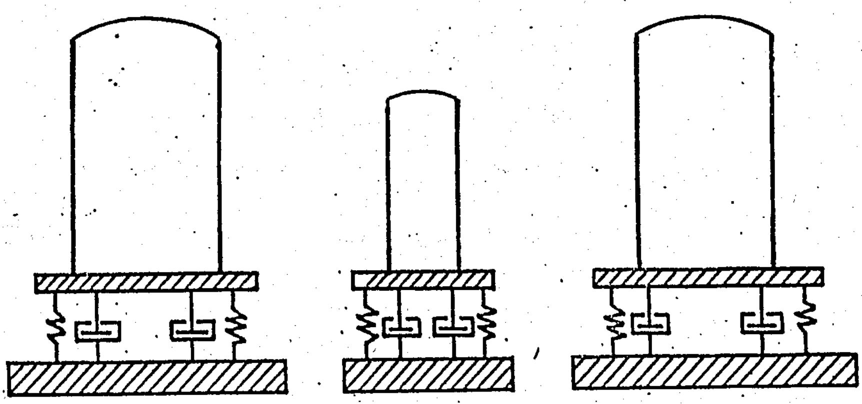
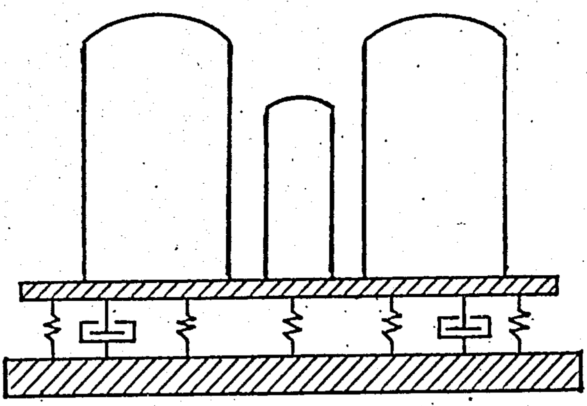


FIG. 7.8 - HORIZONTAL DISPLACEMENTS , NODE 1



a) Independent isolation (Incorrect)



b) Complete isolation (Correct)

FIG. 7.11 - VIBRATION ISOLATION OF THE NUCLEAR ISLAND

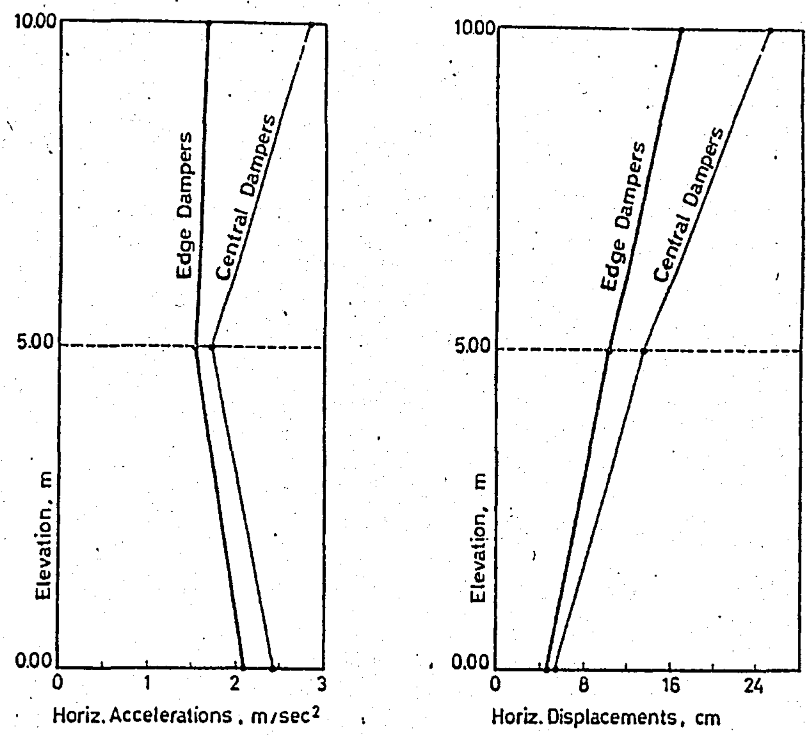


FIG. 7.9 - HORIZONTAL RESPONSE COMPARISIONS (T_r = 1.99sec)

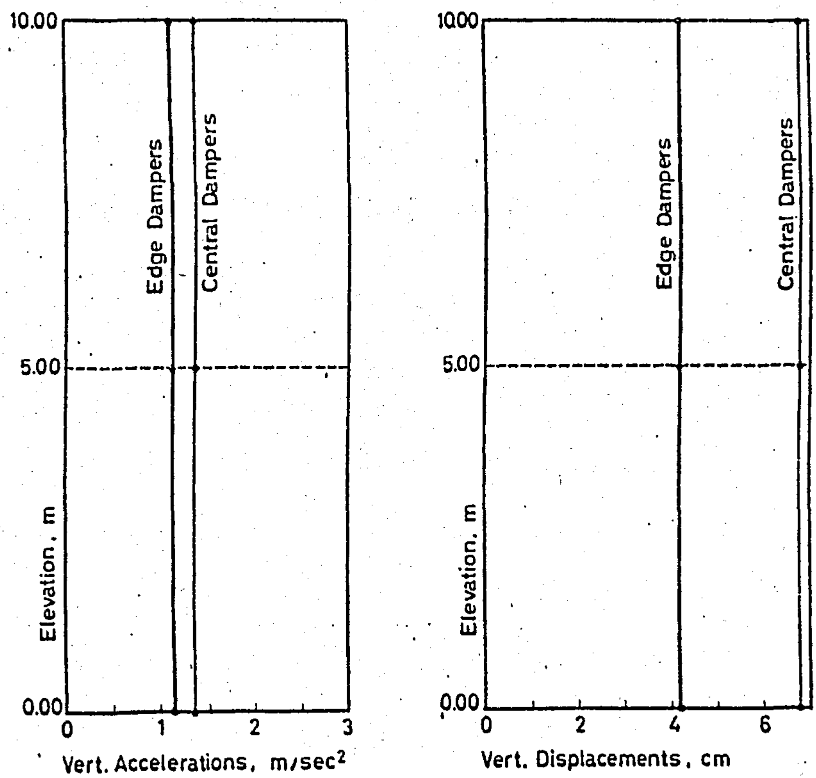


FIG. 7.10 - VERTICAL RESPONSE COMPARISIONS (T_r = 1.99sec)

body at any instant of time during the earthquake. Hence, the relative storey displacements are almost nil permitting no stresses and strains in the structural members.

It is computed that the axial forces produced in the columns of the lower storey of the model are 62.4 tons and 12 tons in the neoprene and spring isolated cases, respectively. The adverse effect of the neoprene pads in vertical vibration becomes once again obvious because of the fact that the axial forces in the columns are more than five times larger than those of the spring base case. The situation in the fixed base case is even worse. Shears and moments in the isolated systems are very small, since the structure behaves almost as a rigid body not permitting any nodal rotations.

7.2. ADDITIONAL SAFETY PRECAUTIONS

It is already shown that the response of structures to earthquake induced motions is greatly reduced by means of vibration isolators placed under the foundation. The energy of the earthquake motion is absorbed primarily by means of the shock absorbing devices resulting in reduction of accelerations. The structure behaves completely in the elastic range and does not require any plastic deformations for the purpose of absorbing energy. However, it is unavoidable with the vibration isolation systems that the displacements may become larger as the accelerations are reduced. The dampers which are used together with the springs reduce the amplitudes of displacements also qnd prevent any occurrence of a probable quasi-resonant state of vibration.

In addition to the fundamental elements of vibration isolation, such as springs or neoprene pads, some other auxiliary safety precautions should be incorporated into the system, depending on the needs of the intensity of vibrations. Some of these additional features and the safety precautions are explained in the following paragraphs.

A) SAFETY FUSES

One of the problems associated with the use of base isolation is that the structure may undesirably oscillate even during the small excitations of wind forces or low amplitude earthquakes. Especially, the predominent frequency of the wind force is likely in the same order of magnitude as the isolation frequency of the structures. Therefore, the oscillations of the structure due to the wind load may be at a considerable level.

While there is no intrinsic danger associated with such small oscillations, they would be clearly disturbing to the occupants and would not be acceptable. Therefore, for the comfort of the occupants in residential buildings and also to prevent the cyclic motion of the service lines in other structures, it is advisable to install mechanical fuses in the form of shear pins between the two foundation rafts, such that for small disturbances the vibration isolqtors do not participate. When the level of disturbances reach to a base shear value of about 10% of the weight however, the shear pins acting like mechanical fuses, fracture and the structure is coupled with the vibration isolators as described by Derham, et al (1977) and Delfosse (1977). These safety pins should be supplied in both horizontal and vertical direction^s^ and should be easily replacable after a strong earthquake.

B) EMERGENCY SUPPORTS

In order to prevent excessive displacements of the foundation during an unexpectedly high earthquake or in the very unlikely situation of springs and dashpots being damaged, emergency supports should be designed in both horizontal and vertical directions. These emergency supports should be of shock-absorbing type rather than of a mechanical type. For purposes of maintenance and servicing, the spacing between the upper and lower rafts should be in the order of 1.80 m, the base isolation elements are normally mounted on appropriate pedestals. These isolation elements should be easily accessible for inspection purposes. They should also be removed and replaced at any time, if necessary, without interferring with the main function of the building.

C) FLEXIBLE COUPLING OF PIPES

Differences of displacements between the lower and upper foundation rafts should be accommodated particularly for service lines by means of flexible couplings and fail-safe mechanical devices. Such couplings are necessary only at the interface of foundation and building, since the relative displqcements at upper elevations are negligible.

7.3. INFLUENCE OF SOIL CONDITIONS

A) GENERAL

In order to demonstrate the influence of local soil conditions on the vibration isolation, especially in the vertical direction, a simple numerical example is selected. The mathematical model of a typical nuclear reactor building by Plithon and Jelivet (1978) is idealized into a two-mass assembly of structure and soil as shown in Fig. 3.3.

The subsoil medium may be represented by means of either an assemblage of finite elements or a series of linear elastic springs. For reasons of simplicity in determining the effects of local soil conditisons, the subsoil is represented by means of equivalent springs. In the numerical example a single spring is selected to represent the vertical action of the soil_ because only the tendency of the influence of the soil condition is investigated.

Similarly, the superstructure is reduced to a single mass, because the whole structure moves like a rigid body causing almost no stresses or strains in the superstructure. Further, the purpose of this particular analysis is only to investigate the changes of natural periods of vibration due to different soil conditions.

B) STRUCTURAL AND SOIL PROPERTIES

The total weight of the reactor building is given as W_2 = 45 000 ton by Plichon and Jolivet (1978). The total weight of the lower foundation, combined with the phase in mass of the soil, is assumed as W_1 = 10 000 ton. The total horizontal stiffness of the reinforced elastomer bearing pads is calculated from equation (1) of the reference as $K = 1.12 \times 10^5$ ton/m. Assuming the vertical stiffness to be 800 times greater than the horizontal stiffness, the total vertical stiffness becomes $k_2 = 9 \times 10^7$ ton/m. Two different soil conditions will be considered as follows:

i) "Soft Soil" representing fine dense sand with an allowable bearing stress of 25 ton/m^2 requiring a mat foundation area of about 1800 m^2. Assuming the modulus of subgrade as $k = 5$ kg/cm^3 the total vertical stiffness becomes k_1=0.9×10^7 ton/m.

ii) "Hard Soil" representing dense gravel or rock-like soil with an allowable bearing stress of 35 ton/m^2 requiring a mat foundation area of about 1300 m^2. Assuming the modulus of subgrade as k = 22 kg/cm^3, the total vertical stiffness becomes $k_1 = 2.84 \times 10^7$ ton/m.

C) CHANGES IN NATURAL PERIOD OF VIBRATION

The natural period of vibration of a two-spring model is

$$T = 2\pi/\omega \quad \text{and} \quad \left(\frac{B \pm \sqrt{B^2 - 4A}}{2}\right)^{\frac{1}{2}} \qquad \ldots (7.2)$$

where

$$A = \frac{k_1 k_2}{m_1 m_2} \qquad \ldots (7.3)$$

$$B = \frac{k_1 + k_2}{m_1} + \frac{k_2}{m_2} \qquad \ldots (7.4)$$

Incorporating the above mentioned numerical values in these expressions, the periods of vibration are obtained for "soft" and "hard" soil conditions as, T = 0.162 sec, and T = 0.098 sec, respectively. For infinitely rigid subsoil condition, the natural period of vibration in the vertical direction is T = 0.045 sec. It is seen that the influence of soft soil condition on the change of natural period of vibration is very insignificant.

D) DISCUSSION OF RESULTS

The soil under most nuclear power plants is hard soil. Even in the case of soft soil, its contribution to the vibration isol tion is practically nil. In the case of neoprene pads, no vibration isolation is supplied for vertical and rocking motions of the structure, since neoprene pads are very stiff in the vertical direction. The claim that the subsoil conditions may assist the neoprene pads in the isolation of vertical motions is thus unquestionably disproved by the above numerical example.

Earthquakes generate three dimensional moitions which may contain, close to the epicentral region, vertical accelerations as high as those in the horizontal direction. The structures possess inherently greater strength in the vertical direction thus being sensitive to higher frequency motions. Since the vertical components of earthquake motion contain relatively higher dominant frequencies, in order to prevent any quasi-resonance condition, the nuclear power plants must be appropriately isolated also in the vertical direction.

Helical springs and dashpots are ideally suitable for this purpose, since they may provide any desired amount of flexibility and damping in all three directions. Usually, the vertical stiffness of helical springs is in the range of 2 to 5 times that in the horizontal direction.

The dashpots mya supply damping values as high as 20% to 30% of that of the critical damping ratio. The coefficient of viscosity of the dashpots in the horizontal direction is normally 60% of that in the vertical direction as given by Huffmann (1980).

7.4. BASE ISOLATION FOR NUCLEAR POWER PLANTS

Due to very serious consequences of earthquake induced damages, it is necessary to make provisions in the aseismic design of nuclear power plants, for very severe earthquakes with recurrence intervals of many thousands of years. It is essential that nuclear power stations which operate in seismically active areas have the ability to shut down safely and the important components such as full element, control nod, piping system, etc. must all operate with high reliability when attacked by an earthquake.

The siting of nuclear power plants in regions of high seismicity may not be desirable but may sometimes be necessary. Since it is very difficult to provide adequate earthquake resistance with conventional design principles, other techniques such as vibration isolation is indispensable in order to increase safety and reliability. With vibration isolation systems a low probability of failure of the important plant components is achieved mainly by excluding large applied forces rather than designing the system to withstand such forces. The energy of the earthquake motion is absorbed primarily by means of the isolation devices installed at the bottom of the foundation and the structure behaves only in the elastic range. Thus vibration isolation systems offer a very practical solution as they can reduce the seismically induced forces in the structures to very low tolerable levels.

The aseismic design of structures requires a proper estimate of earthquake parameters, such as peak amplitude, frequency content and the duration of the ground motion. In general, because of the lack of adequate data and the complexity of the problem, the determination of these quantities introduces many uncertainties. The use of base isolation however, reduces the significancy of the above uncertainties since the natural frequency of vibration of the system is pushed very far away from the governing frequencies of the probable earthquakes. Therefore, owing to the reduction of the importance of the parameters of earthquakes which the structure will be exposed to during its lifetime, the use of vibration isolation concept in the aseismic design of nuclear structures becomes a very attractive solution.

For the time being, because of importance of safety the studies have been concentrated mainly on the isolation of nuclear power plants. Only a few ordinary buildings have been vibration isolated so far. As reported by Roth et al (1970), the Heinrich Pestalozzi School in Skopje, Yugoslavia, built on vulcanized rubber cushions placed between the strip foundation and the first floor slab of the structure was constructed in 1969. The building was designed by Swiss engineers and was a gift to the rebuilding of Skopje after the 1963 earthquake. The synamic characteristics of this school building and also the assessment of its aseismic design are later studied by Petrovski and Simovski (1982). In that presentation, the natural periods of vibration of the building are reported to be T = 0.71 sec and T = 0.44 sec in two horizontal directions.

Nuclear power plant structures on the other hand are expensive structures where unquestionable protection from any possible damage is absolutely necessary. The placement of the isolation elements leads to feasible results compared to the questionable solutions through classical design principles. The isolation system brings with advantages like safety and reliability during the life span of the plant due to the efficiency in elimination of excessive earthquake accelerations. The reduced designed time needed because of the readily predictable dynamic behavior of the structure depending only on the known characteristics of the isolation elements is another advantage. In addition to the safety, reliability and the reduced design effort, the possibiliıty for the reproduction of proven standard design schemes for the plant and its components as developed for a series of plants can be used independent of the seismic environment.

While the isolation permits the construction of standardized structures in regions of high seismicity there exist certain fundamental requirements such as the reduction in acceleration response, controlled displacements, etc, which have to be satisfied by the isolation system, if it is to be considered practical. In the case of nuclear structures these requirements are particularly severe because of the extremely stringent safety standards that must be fulfilled. A considerably important design requirement for a seismic isolation system is that it must still be in operation for aftershocks without necessitating any repairs after the main shock. A further requirement is that after a major earthquake, it must be possible to restore the full effectiveness of the isolation system by replacing devices where and if necessary.

It is evident that base isolation provides an efficient means for ensuring a high degree of earthquake safety. However, it is usually preferable that a site with adequately hard rock should be chosen to minimize the effects of longer period ground accelerations, since these are very difficult to attenuate with a base isolation system.

A nuclear power plant consists of various important structures like the reactor building, the fuel building, safeguard building, and the turbine building, etc on the "nuclear island". These buildings adjacent to each other are interconnected by a network of electrical and mechanical installations and also by heating and other piping systems. If these buildings are vibration isolated independently, there may be undesirable relative displacements between the adjacent buildin~s resulting in a possible damage of the install^tions. If the complete foundation raft of the whole nuclear island is vibration isolated however, as shown in Fig 7.12, the whole island behaves like a rigid body during a strong fround motion and practically no relative displacements occur among the individual structures, thus the installations remain undamaged. In any case, the critical installations and piping elements, especially those extending from an isolated to a non-isolated building, should be supported and detailed by means of special energy-absorbing restrainers as described by Skinner et al (1976) and Powell et al (1980).

The idea of vibration isolation is widely utilized for a very long time in connection with machine and turbine foundations. The implementation of this principle against earthquake motions in ordinary buildings and especially in nuclear power plants however, is relatively new. Although the use of vibration isolators against earthquakes is in its developing stage, there are various structures and nuclear power plants already utilizing this concept as a means of protection against earthquakes.

By far the most successful examples of vibration isolation systems applied to nuclear power plants are the Electicite de France (E.D.F.) system as described by Jolivet and Richli (1977) and Plichon and Jolivet (1979), and Plichon et al (1980). The system has been already applied to nuclear power plants in (a) Koeberg, South Africa, (b) Cruas, France, and (c) Karun, France. In this isolation system, combination of neoprene rubber pads and the friction plates of stainless steel are used. The friction plates are normally added to the isolation system in order to decouple the superstructure from the base by sliding and limiting the shear forces transmitted. With the use of neoprene pads the fundamental frequency of the system is generally reduced to 1 Hz and thus moving it out of the range of 2 Hz to 8 Hz, where the earthquake excitation is highest.

For a seismic isolation system to be feasible, it must be cheaper to construct, monitor and maintain during the life of the structure than a conventional structure constructed to withstand the seismic loads.

When conventional earthquake resistant design principles are followed, the additional cost of construction is normally in the order of 5% to 10% of the overall cost. The additional cost however, is significantly less is vibration isolation system is used. In fact, based on the preliminary cost-benefit analysis performed on a typical reactor building, the total additional cost of the vibration isolation system and the special second layer foundations, etc., is found to be not greater than 2% to 3% of the overall cost of the reactor building, as reported by Kunan and Maini (1979). In addition, the facility brought with the vibration isolation system so as to allow the use of a standard design, further increases the potential use of the vibration isolation concept.

7.5. CRITERIA FOR IDEAL SEISMIC ISOLATION

Theoretically speaking, an ideal seismic isolation scheme should possess all of the criteria, which may be required to satisfy the conditions of maximum safety, economy, functionality and durability. A set of such ideal criteria reproduced from Tezcan (1982) is listed below for reference purposes.

It is quite natural to expect that a particular isolation scheme may satisfy some of these requirements, while another isolation scheme may satisfy still others. Before adopting any particular seismic isolation scheme, one should be aware of its advantages and limitations. The basic requirements expected from a seismic isolation system are as follows:

1. Seismic isolation should be achieved in all four possible modes of vibration: horizontal, vertical, rocking and torsional,
2. Damping should also be effective in all of these four possible modes of vibration,
3. The damping mechanism should be capable of reducing the excessive displacements to any tolerable limits,
4. There should be no need for a mechanism in the form of shock absorbers or stoppers to prevent large displacements in the event of an unexpected severe shock,
5. Isolation elements should be capable of self-centering after an earthquake. They should possess restoring capacity allowing for no permanent dislocation.
6. Isolation elements should be in working condition immediately after a major earthquake, no replacement of the entire system should be necessary.
7. Damping elements should be in working condition immediately after a major earthquake. No permanent off-set or plastic deformations should exist,
8. Seismic isolation should not be sensitive to travelling waves,
9. Mathematical analysis and also the implementation in the field should be as simple as possi-

ble to convince and to obtain the confidence of both the designing and the licesing persons. Simple systems are easy to analyse and also easy to test in the laboratories or in the field.

10. Isolation elements should be easily replaced when worn out or defective.
11. Isolation elements should be less sensitive to changes in temperature. They should maintain the desired levels of stiffness and damping at all practical ranges of temperature,
12. Isolation elements should not be very sensitive to small deflections, imperfections or differential settlements, no construction or manufactured material provides ideal theoretical conditions,
13. Isolation elements should provide sufficient protection against external impact loads such as aircraft collision and blast.
14. There should be some mechanism to prevent undue vibrations and displacements during wind loads and minor earthquakes.
15. Seismic isolation should be capble of being implmented in any location or site.

Basic characteristics of various seismic isolation schemes are listed in Table 7.4 on a comparative basis, considering the ideal criteria mentioned earlier. There is no doubt that, before deciding on any particular seismic

TABLE 7.4 - COMPARATIVE ASSESSMENT OF VARIOUS SCHEMES

IDEAL REQUIREMENTS	FLOATING PLATFORM	ROCKING BEARING BALLS + DAMPERS	NEOPRENE PADS + FRICTION PLATES	HELICAL SPRINGS + VISCO-DAMPERS
REE DIMENSIONAL ISOLATION				
Horizontal	✓	✓	✓	✓
Vertical	✓	NO	NO	✓
Rocking	✓	NO	NO	✓
Torsional	✓	✓	✓	✓
REE DIMENSIONAL DAMPING				
Horizontal	NO	?	✓	✓
Vertical	NO	NO	NO	✓
Rocking	NO	NO	NO	✓
Torsional	NO	?	✓	✓
FFICIENT DAMPING CAPACITY EXISTS				
Horizontal	NO	-	7%	20%
Vertical	NO	-	2%	25%
N WORK WITHOUT A SHOCK ABSORBER/STOPPER	✓	NO	NO	✓
SELF-CENTERING POSSIBLE?	✓	✓	NO	✓
IT REUSABLE AFTER AN EARTHQUAKE?	✓	✓	✓	✓
DAMPING DEVICE REUSABLE?	-	NO	✓	✓
IT INSENSITIVE TO TRAVELLING WAVES?	✓	✓	NO	✓
IT A SIMPLE SYSTEM? (imple to analyse? Simple to implement?)	NO	NO	NO	✓
IT EASY TO REPLACE?	NO	NO	NO	✓
IT INSENSITIVE TO TEMPERATURE CHANGES?	✓	✓	✓	NO
IT INSENSITIVE TO IMPERFECTIONS?	✓	NO	NO	✓
FFICIENTLY PROTECTED AGAINST AIR HAZARDS	NO	✓	✓	✓
ND LOADS TAKEN CARE OF?	NO	✓	✓	✓
SY TO APPLY AT ANY LOCATION	NO	✓	✓	✓

isolation scheme, a cost-performance analysis should be carried out.

It is seen that helical springs and viscodampers are quite superior to the reinforced neoprene pads and friction plates. The superiority stems mainly from the fact that the neoprene pads are unable to provide vibration isolation in the vertical direction. Further, the capability of damping resistance of viscodampers is much greater than that of the neoprene pads.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1. GENERAL

a) The use of vibration isolators in nuclear power plants as well as in ordinary structures as a protection against earthquakes is a revolutionary technique in earthquake engineering. With the wide scale commercial production of the vibration isolators there will be a major break-through and the conventional earthquake resistant design principles based on reserve energy and large plastic deformations will be obsolete.

b) Vibration isolators consist of elastic or elasto-plastic rubber pads or springs and dampers. The basic idea of installing elastic devices at the foundation level is to increase the natural period of vibration of the structure to a level much greater than the dominant periods of the earthquake ground motion so that the response is drastically reduced.

c) Vibration isolators reduce the earthquake loads on a structure by more than five to ten times with respect to those of a fixed base strcture. Therefore, the increase in cost due to isolating devices and double layer foundations is much less than the extra cost of aseismic design requirements of the conventional approach. In fact, the additional cost of incorporating earthquake resistance to a nuclear power plant by the conventional principle may increase the overall cost by as much as 40% to 60%. The double mat foundations, vibration isolation devices, however, may increase the cost by only 3% to 6%.

d) The overall response of a structure to a given earthquake ground motion is considerably reduced by means of vibration isolation at its base, since the structure behaves mainly as a rigid body. Consequently, the stresses in structural elements remain in the elastic range providing confidence in design, simplicity in detailing, and economy in overall cost.

e) It is a general tendency with vibration isolators that the displacements become progressively larger as the accelerations are reduced. Therefore, the amount of springs and dampers must be selected so appropriately that an optimum situation is developed in which the accelerations are reduced to a satisfactory level, while the displacements remain within tolerable limits. Reduction in acceleration response always results in increase of displacements. With vibration isolation, maximum displacements occur at the base of the structure. Relative storey displacements at superstructure are negligible.

f) In order to prevent the presence of undue rigid body oscillations, at small wind loads for instance, the flexible starter pins or couplings should be installed at the base, such that the vibration isolation is triggered only after the ground motion exceeds a certain allowable value. Similarly, to prevent excessive deflections of the spring system, a series of large capacity safety pins should be also installed to discontinue the vibration isolation after a certain maximum amount of relative base displacement.

g) In general, the peak response values determined by analytical studies, all in good agreement with those measured at the shaking table. Therefore, it would be sufficient to perform analysis of structures in order to predict their response to any given ground motion. Expensive laboratory testing would thus be unncessary, since the theoretical studies produce reliable results.

8.2. SUPERIORITY OF SPRING-DASHPOT SYSTEMS

a) Spring-dashpot arrangement is found to be the most suitable and safest system for the vibration isolation of structures especially against the vertical motions of earthquakes.

b) The use of viscodampers provides significant energy absorption capacity and reduces the response of the structure considerably. The efficiency is further increased if the viscodampers are placed at the exterior ends of the base. Any degree of damping may be achieved simply by installing the necessary amount of viscodampers at the base. The major handicap in vibration isolation is that the rigid body displacements of the structure may be excessively large. The use of viscous dampers however is an indispensible tool in eliminating these large displacements. The viscous dampers provide sufficient amount of damping, up to the value of 20% to 30% of critical damping, in all three directions.

c) As far as damping is concerned neoprene has a critical damping ratio of 7% in the horizontal direction only. Because of high rigidity and no damping in the vertical direction, neoprene isolation is not considered an ideal solution. Since a system vertically flexible and horizontally rigid would be even less satisfactory, a spring-support system combining high flexibility in both vertical and horizontal direction seem to be a reasonable solution.

d) A greater degree of safety and assurance is incoporated into the design, with spring-dashpot isolation system than, the conventional principle or the neoprene pads. There is no complex nonlinear behavior, no possibility of slippage of pads even at accelerations as high as 1.0 g to 1.5 g, no possibility of magnified response in vertical and rocking motions.

e) The number of shut-downs in nuclear power plants will be much less, since the acceleration response in all three directions is greatly reduced. Further, a prompt and immediate restarting of the facilities becomes possible after a major earthquake, since neither the vibration isolation system nor the installations are expected to be damaged. In the case of neoprene pads, a difficult and elaborate adjustment procedure is necessary after a major earthquake.

f) Helical springs and dashpots are durable and less sensitive to support settlements and to physical conditions. It is very easy to replace any defective helical spring or dashpot.

g) Stiffness of the natural rubber bearings in the vertical direction is about four hundred times greater than that in the horizontal direction. This is a serious restriction in isolating the vertical vibrations. The spring constants of mechanical springs however, may be adjusted to any desired value for achieving the best efficiency in both horizontal and vertical directions.

h) The rubber element behaves non-linearly under higher loads, becomes stiffer. This increases the stiffness causing the whole system to have higher natural frequencies. Due to this fact isolation efficiency decreases as the stiffness of the isolation element increases. However, displacements of the helical springs are linear up to the block height of the spring. Therefore, the efficiency of vibration isolation does not decrease in helical springs, as the displacements become larger.

8.3. RECOMMENDATIONS

a) Obviously, the real test for the efficiency and reliability of vibration isolation as an indispensible method of aseismic design, would be to observe their performance in the field under real structures subjected to real strong ground motions. Therefore, it would be very much desirable to install more and more vibration isolators under as many structures as possible in the areas of high potential earthquake zones. Only after satisfactory performance of vibration isolators observed under real strong earthquake conditions, satisfactory level of confidence will be established for their undeniable acceptance by the engineering profession.

b) Usually, the rubber elements are used without damping elements. It is considered to be sufficient with the material damping inherent in the rubber. But in most applcications, this material damping is not sufficient. Therefore, in order to reduce the excessive lateral displacements, rubber elements should be preferably accompanied by suitable damping devices.

c) With vibration isolation the foundation is usually heavier than that of conventional types of structures. This very heavy mass at the base of the structure sometimes causes rocking motion about the upper rolling center which is undesirable and should be avoided. A parametric study should be undertaken in order to determine the influence of various parameters affecting this behavior.

d) Viscodampers work approximately proportional to the velocity in the vicinity of the resonance. However, at higher frequencies the amplitudes are usually so small that the viscodampers provide only velocity independent material damping, hence the isolation efficiency is practically not affected by the damping. Due to this fact damping should be considered as a frequency dependent parameter rather than constant during the calculations.

e) Viscodampers are also very much dependent on the variations in temperature. Damping is drastically reduced with increase in temperature beyond the prescribed levels. It may therefore be necessary to arrange a suitable environmental mechanism around the foundations, so that the temperature may be controlled to remain between desired levels.

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APPENDIX

1-CODE NUMBERS

By definition, a code number expresses the directions of deformations of the ends of a structure's member, with reference to the directions of deformations of the entire structure.

The code numbers technique, which is described below, permits an easy generation of the main stiffness matrix as well as of load matrix of any structure under consideration.

This method is valid for all types of structures: trusses, frames, grids, plates and shells, in which "members" may be bars or finite plate elements, as well.

A code number furnishes the following information:

a. It locates each member within the structure.
b. It indicates the place in the main stiffness matrix, where the elements of a member's stiffness matrix should be transferred.
c. It indicates the place in the main load matrix, where the member fixed end bending and torsional moments, shear and axial forces should be transferred.
d. From the mathematical standpoint, it represents in a short hand form the displacement transformation matrix of a member. (A displacement transformation matrix indicates the coefficients of equations of displacement compatibility, which transform member deformations to system deformations, and insure, in this way, the continuity of the structure's mathematical model.)

Graphically, a code number is a series of locations which correspond to the sequential order assumed for numbering of the end deformations of a typical member. The quantity of locations in a code number is equal to the quantity of specified end deformations (degrees of freedom) of the corresponding member. For instance, there are twelve locations in the code number of a space frame member. For each end deformation of the member, a location is reserved inside the code number. When written, a comma or a reasonable space separates these locations from each other. Because each member is assumed to span from its first end (i) to its second end (j), and these directions are indicated by arrows at all members, on the structural scheme of the examined structure, it is always easy to match any direction of system deformation to the given member end deformation. Thus, the consecutive number of the deformation of the system coinciding with that particular member end deformation is entered into the location in the code number. If there is no system deformation with which to coincide, a zero is inserted. If any particular system deformation is equal for a series of members, the same values are placed in the corresponding locations of the code numbers.

However, it is indispensable that common reference axes must be used for the deformations of the system and the individual members. Hence, when the members do not intersect at right angles, in a structure referenced to a system of orthogonal axes, the code number method cannot be applied unless the directions of the member deformation are transformed from the system of member axes to the system of common axes.

The code numbers are generated automatically by the computer. The following example illustrates the procedure. Given a plane frame, as shown on Fig. A.1, members of which are assumed not to have any axial deformability, the possible directions of deformations are indicated in the sequential order by numbered arrows.

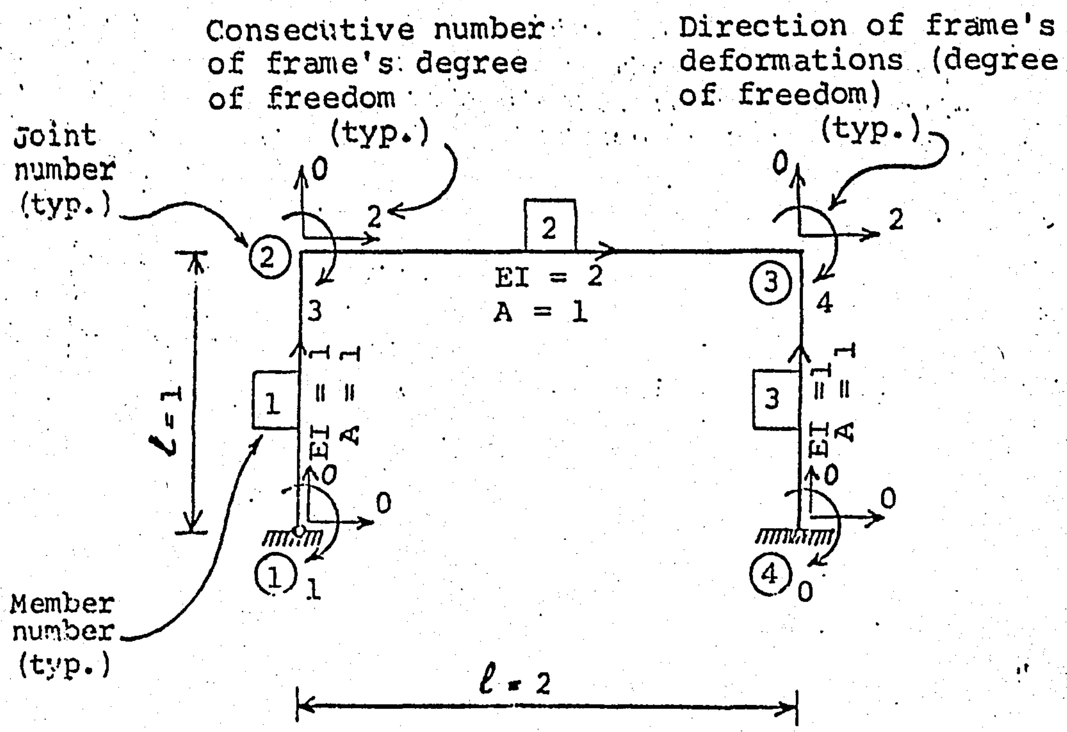

FIG. A.1 - STRUCTURAL SCHEME : RIGID FRAME WITHOUT AXIAL DEFORMABILITY.

Figure A.2 shows a typical member "n" of this frame and it visualizes the sequential numbering system of member end deformations

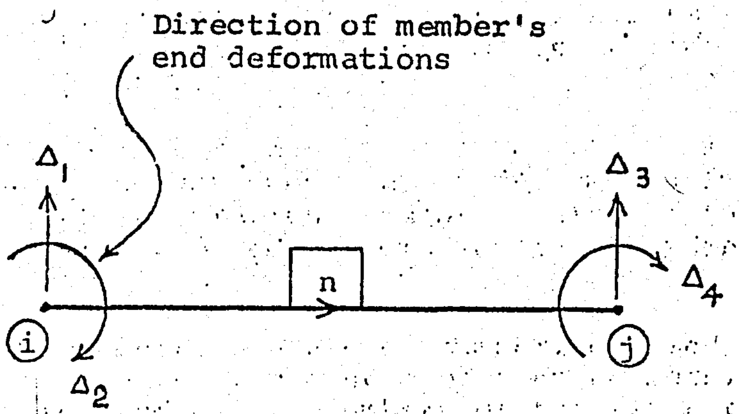

FIG. A.2 - TYPICAL MEMBER

The code numbers of the three members of the frame are listed in Fig. A.3. As may be seen, these code numbers can be expressed readily at a glance, requiring no calculation. They are, anyhow, generated by the computer, automatically.

Member No.	CODE NUMBERS			
	Δ_1	Δ_2	Δ_3	Δ_4
Member 1	0	1	2	3
Member 2	0	3	0	4
Member 3	0	0	2	4

Direction of member end deformations

Direction of frame deformations which corresponds to a given member's end deformation (typ.)

FIG. A.3 - CODE NUMBER TABLE

A.2-GENERATION OF THE MAIN STIFFNESS MATRIX

On the basis of the supplied joint coordinates, member incidences and member angle β, as well as member elastic data, the computer calculates automatically member stiffness matrices, transforms them into the system of common axes, and after generating code numbers, builds up the main stiffness matrix.

Since the location of an element in a matrix is described by its row and column count, and code numbers furnish precisely that information, the generation of the main stiffness matrix proceeds automatically according to the following rule:

"Each location number in a code number, coupled with itself and with all the other location numbers, constitutes the row and column numbers of the elements to be taken from the individual stiffness matrix, whereas the numbers in the corresponding locations coupled with each other in the same manner indicate the row and column numbers of the general stiffness matrix into which these elements must be placed."

Locations containing zeros are ignored. The same operation must be repeated with the code numbers of all frame members, in order to obtain the main stiffness matrix.

Fig.A.4 indicates the synthesis of the main stiffness matrix of the frame shown on Fig A.1

In order to show the procedure, the member stiffness matrices are bordered by the corresponding code numbers taken from Fig. A.3. For clarity, the cross-reference between the elements of the main stiffness matrix and the elements of the member stiffness matrices is indicated by roman numerals, in parenthesis.

Frame's degrees of freedom (typ.)

$$[K]_{xyz}=\begin{array}{c|c}
 & \begin{array}{cccc}1 & 2 & 3 & 4\end{array} \\
\begin{array}{c}1\\2\\3\\4\end{array} &
\left[\begin{array}{c:c:c:c}
4^{(I)} & 0^{(II)} & 2^{(III)} & \\ \hdashline
0^{(IV)} & 1^{(V)} \quad 1^{(XIV)} & 0^{(VI)} & 0^{(XV)} \\ \hdashline
2^{(VII)} & 0^{(VIII)} & 4^{(IX)} \quad 4^{(X)} & 2^{(XI)} \\ \hdashline
 & 0^{(XVI)} & 2^{(XII)} & 4^{(XIII)} \quad 4^{(XVII)}
\end{array}\right]
\end{array}
=\begin{bmatrix}4 & 0 & 2 & 0\\0 & 2 & 0 & 0\\2 & 0 & 8 & 2\\0 & 0 & 2 & 8\end{bmatrix}$$

Main stiffness matrix

member number (typ.)

code number (typ.)

$$[k_1]_{xyz}=\begin{array}{c|c}
 & \boxed{\begin{array}{cccc}0 & 1 & 2 & 3\end{array}} \\
\boxed{\begin{array}{c}0\\1\\2\\3\end{array}} &
\left[\begin{array}{c:c:c:c}
1 & 0 & -1 & 0 \\ \hdashline
0 & 4^{(I)} & 0^{(II)} & 2^{(III)} \\ \hdashline
-1 & 0^{(IV)} & 1^{(V)} & 0^{(VI)} \\ \hdashline
0 & 2^{(VII)} & 0^{(VIII)} & 4^{(IX)}
\end{array}\right]
\end{array};
\quad [k_2]_{xyz}=\begin{array}{c|c}
 & \boxed{\begin{array}{cccc}0 & 3 & 0 & 4\end{array}} \\
\boxed{\begin{array}{c}0\\3\\0\\4\end{array}} &
\left[\begin{array}{c:c:c:c}
12 & 6 & -12 & 6 \\ \hdashline
6 & 4^{(X)} & -6 & 2^{(XI)} \\ \hdashline
-12 & -6 & 12 & -6 \\ \hdashline
6 & 2^{(XII)} & -6 & 4^{(XIII)}
\end{array}\right]
\end{array};
\quad [k_3]_{xyz}=\begin{array}{c|c}
 & \boxed{\begin{array}{cccc}0 & 0 & 2 & 4\end{array}} \\
\boxed{\begin{array}{c}0\\0\\2\\4\end{array}} &
\left[\begin{array}{c:c:c:c}
1 & 0 & -1 & 0 \\ \hdashline
0 & 4 & 0 & 2 \\ \hdashline
-1 & 0 & 1^{(XIV)} & 0^{(XV)} \\ \hdashline
0 & 2 & 0^{(XVI)} & 4^{(XVII)}
\end{array}\right]
\end{array}$$

Member stiffness matrix (typ.)

FIG. A.4 - SYNTHESIS OF MAIN STIFFNESS MATRIX