

University of London
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DESIGN OF ECONOMICAL MEANS
OF ENHANCING THE ENERGY-ABSORPTION CAPABILITY
OF BRACED FRAME STRUCTURES

by

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To my mother

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ABSTRACT

This thesis is focused on improving the safety and economy of steel braced frame structures in resistance to the effects of earthquakes by enhancement of the capability of the structure to dissipate the energy of oscillation that might be built up by the response to ground motion.

The main effort of this work is concentrated in the development of linear-sliding friction, or rotational friction devices, which would generally be placed at the bracing connections. Slip at these devices imposes a limit on the load that can be transmitted to the bracings, giving protection against the progressive deterioration of strength which occurs in the event of cyclic deformation that causes buckling or yield in tension of the bracing members. The associated energy dissipation is typically large, becoming the dominant dissipative mechanism in the frame.

The development of these devices was primarily experimental by general investigation of material characteristics, mostly with constant-amplitude low-frequency cyclic motion, at scales approaching practical full-size. A dynamic model of a single-storey frame with such a device, at about one-third linear scale relative to the full-size application, was also examined on the Imperial College shake table. Parallel extensive analytical studies were carried out on the response of braced frames incorporating such devices. Design criteria related to these devices are also given.

This work is accompanied by studies of energy dissipation by plastic deformation on primary structural elements. The frictional devices have the advantage, by comparison with established energy-dissipative systems based on plastic deformation in shear or bending of the primary frame members, that the damage to the primary members resisting loading is completely prevented.

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NOTATION

In this thesis, mathematical formulae taken from other investigators are commonly given in their original notation. The following notation has been adopted for the studies specifically developed for this thesis.

C	= Viscous damping matrix
E	= Young's modulus
G	= Shear modulus
$H_x(\omega)$	= Complex frequency response function of x
K	= Stiffness matrix
L	= Length
L/r	= Slenderness ratio
M	= Mass matrix
M_p, V_p	= Plastic bending moment and the shear yielding load of a cross-section
P_{cr}	= Critical buckling load
P_y, Δ_y	= Yield load and the yield displacement in tension of a strut
$S(\omega)$	= Power spectrum
T	= Natural period of vibration of the structure
U	= Displacement (in Chapter 2: generated random numbers)
$\ddot{U}_g, \ddot{X}_g$	= Earthquake acceleration at ground surface
x	= Displacement
x_f	= Output of filter
Y, F_y	= Yield displacement and yield force of a hysteretic damper
Z	= Dimensionless hysteretic parameter of a hysteretic damper
Δ	= Differences of displacements, velocities and accelerations
Δt	= Time step
Δ_y, P_y	= Yield displacement in tension of a strut and yield load
μ	= Ductility
σ_x^2	= Variance of x
σ_y	= Yield stress of the material
ζ	= Viscous damping coefficients, fraction of critical damping
$\phi(t)$	= Variance intensity function
ω, f	= Frequencies in rad/sec and Hz respectively
ω_g, ζ_g	= Effective frequency and damping of the soil

CHAPTER 1

INTRODUCTION.

1.1. Role of energy absorption capability in design

1.1.1 General discussion on structural response to earthquake loading

The governing target in the design of buildings to resist the effects of earthquakes is most commonly the prevention of total collapse, which would lead to severe loss of life and exacerbate economic disruption of the community. In seismically active regions it is commonly not economically possible to ensure that the majority of buildings resist the effects of the 'design' earthquake without significant permanent deformation. The permanent deformation of the structure is accompanied by energy dissipation which is in many ways analogous to a major increase in the damping of the dynamic response of the structure to the ground motion.

The safe utilisation of the design philosophy by which permanent deformation is permitted in the event of a severe earthquake, in order to reap the benefits of energy absorption as discussed above, clearly requires adequate ductility of the structure. Steel structures are commonly favourable in this property. This is especially true of frame (as opposed to truss-braced) structures, and this is commonly recognised by design codes, which specify equivalent static horizontal loads which vary according to the structural form.

1.1.2 Minimum design earthquake forces in structures

Semi-empirical procedures for design of structures to resist earthquakes have been in existence for many years, typified by the Unified Building Code (UBC) widely used in the USA, which is based largely on practice built up in California. The design values thus specified are generally significantly smaller than the actual motions generated by an earthquake. Typically, the base shear of the 1940 NS El Centro earthquake, based upon the elastic response of a single degree of freedom system with realistic values of viscous damping, is 4-5 times bigger than the design base shear. The justification for permitting lower values for design are manifold and include: damping contributed by all the building elements, an increase in ductility by the ability of members to yield beyond elastic limits, and other redundant contributions.

The factors used in such codification for determination of the design seismic base shear include: the seismicity of the area (Z), the importance of the structure (I), the type of lateral resisting system (K), the response of the structure (C) relating to its fundamental period of vibration (T) ($C=1/15\sqrt{T}$), the soil-structure interaction (S), and the effective inertia mass of the structure (M).

The design seismic base shear is distributed in the various floors assuming a straight line deflection curve for the fundamental mode of vibration. The influence of the higher modes of vibration is usually taken into account by assigning a constant fraction of the total force to the top floor of the building.

The design K-coefficient has a substantial influence reflecting the type of the structural system and the nature of the structure itself. Types of construction which are recognised to have inherent resistance (or higher ductility) to earthquakes, and which have performed well in earthquakes, are assigned lower values of K. Conversely, structures which have not performed well, and are inherently weak in resisting the dynamic action of the earthquake ground motion, are assigned higher values of K.

1.1.3 Braced frames

A load-carrying frame can be designed to resist horizontal loads entirely by shear forces in the columns with associated high bending moments. This is generally uneconomic where the lateral forces are large, and this thesis considers the frame created by addition of relatively slender diagonal braces to create two or more braced frames.

The lateral force resisting system of a braced frame is composed of two basic systems: one, the braced system; and two, the moment-resisting frame consisting of the beam and columns. Since the rigidity usually resides in the braced system, it would be expected that in the middle stages of an earthquake the shears imposed would be resisted primarily, if not entirely, by this system. For earthquakes of higher intensity or duration, the brace members start either yielding in tension or buckling in compression. At this stage some incremental shear is

transferred to the more flexible moment-resisting space frame, necessitating a degree of ductility to be maintained by the bracing elements.

In the final stages of the earthquake it is conceivable that the integrity and safety of the structure and its components would be relying primarily upon the action of the moment-resisting frame. In a braced frame the moment-resisting frame consisting of beam and columns usually operates in the elastic range, but in a major earthquake it is likely that it would operate in the inelastic range.

Moment-resisting frames of ductile materials have shown particularly good earthquake resistant characteristics. For this reason the UBC specifies the relatively low design value of $K=0.67$ when the entire lateral force is resisted by ductile moment-resisting space frames. On the contrary, higher values of $K=0.80$ or $K=1$ are assigned to braced frames depending upon the amount of lateral stiffness provided by the moment resisting frame and by the braces. The higher values of the K -coefficient for braced frames reflects the limited amount of energy dissipation of the bracing elements.

1.2 Scope of the present work.

The present work is to study energy absorption devices which, suitably placed, can completely change the hysteretic response of a braced frame. These devices protect the braces against the progressive deterioration of stiffness and strength producing hysteresis loops similar to those of the ductile moment-resisting frame.

The major part of this work is concentrated on the development of friction devices. These devices tend to absorb almost all the imparted earthquake energy. If necessary, they can easily be replaced afterwards. Economy can also be achieved, using these devices, by conventional design basis by allowing a reduction in the seismic K -coefficient to a value of $K=0.67$.

The development of the devices was primarily experimental. Analytical studies were carried out in parallel into the response of braced frames incorporating such devices. Two different approaches were followed in the analytical studies. In the first, the response of the structure was examined in the time domain, performing a step-by-step solution, while in the second the response of the structure was examined in the frequency domain following the approach of probabilistic dynamics.

Section 1.3 of this chapter gives a summary of the computer programs developed to examine the response of the structure in the time domain performing a step-by-step solution.

Chapter 2 describes a way of finding the response of structures in the frequency domain. This chapter also describes different ways of generating artificial earthquakes. The idea of examining the response of a structure subjected to many artificial earthquakes was extensively used in the analytical and experimental work of this thesis.

Chapter 3 describes the hysteretic response of braces and braced frames. The poor hysteretic response of braced frames, resulting from the buckling or yield in tension of the braces, is clearly shown in this chapter.

Chapter 4 shows the configuration of various devices which can be used to enhance the energy absorption of braced frames. These can either be devices based upon shear or bending deformation, or devices based upon friction. Friction devices were found to be the best since, suitably designed, they are able to deform a large number of cycles keeping always the same basic characteristics.

The development of the friction devices is described in Chapter 5.

Chapter 6 examines the dynamic response on the shake table of an X-braced frame incorporating a friction device.

Final remarks concerning the response of braced frames incorporating friction devices are given in Chapter 7.

1.3 Dynamic non-linear analysis

1.3.1 General

The satisfactory performance of a multi-storey steel building frame subjected to earthquake can be examined by evaluation by computer in the time domain of the response of the structural model. A brief description is given here of computer programs developed and the models used to describe the hysteretic response of each structural element in order to find the response of multi-storey steel building frames to earthquake loading.

1.3.2 Description of the analysis procedure.

1.3.2.1 General discussion of the dynamic analysis performed.

Two computer programs were developed. The first is a relatively simple program of the dynamic response of a shear building frame. The experience gained from this program was used to construct the second, more general, program describing the dynamic response of any building frame.

In the shear building program all the mass is assumed to be concentrated on the floors, which are themselves rigid. Only one degree-of-freedom per floor is allowed in this program, i.e. the displacement of that floor relative to the ground. The lateral resistance at every storey can be provided either by column elements or by bracing elements. For the column elements plastic deformation is assumed to be concentrated at the two ends of the element as plastic bending moments. When this happens, the shear force supported by this storey is constant and in the text this shear is frequently referred to as the ultimate shear supported by this storey, which is not to be confused with the shear strength of the elements.

The second program is more general, incorporating beam, column, and bracing elements, in this way taking into account the influence of the beam flexibility on the dynamic response of the frame. Three degrees-of-freedom for every node are allowed in this program, i.e. two

displacements and one rotation. For the beam and the column elements all the plastic deformation is assumed to be concentrated at the ends of the elements as plastic bending moments. The bi-linear moment-rotation relationship developed by Clough is used here and found with the same suitable formulations in ref. (1). For braces the model shown in section 3.3 is used.

The bi-linear moment-rotation model assumes that a plastic bending moment develops at the end of a beam or a column element when the bending moment at this point exceeds the plastic bending moment capacity of the cross-section. The plastic bending moment at this point continues to exist only as far as the rotation at this point is of the same sign as the existing plastic bending moment. This analysis is clearly only approximate, since the bending moment at a point is affected not only by the rotation at this end of the element but also by the rotation at the other end, as well as by the transverse displacements at the two ends of the element. This analysis is only exact for some simple cases. Consider for example the single storey frame shown in fig. 1.1. Assume that the columns are elastic and that all the inelastic deformation is concentrated at the two ends of the beam element. When the frame is subjected to earthquake loading then, because of the symmetry of the frame, the two rotations and the two bending moments at the two ends of the beam are always equal, so the bending moment at the one end is always related to the rotation at this end. When the frame is not symmetric then the assumed bi-linear moment-rotation model is not so accurate.

1.3.2.2 Solution of the equations of motion.

The equations of motion are written in incremental form to account for the non-linear response, i.e.

$$M\Delta\ddot{U} + C\Delta\dot{U} + K_t\Delta U = -M\Delta\ddot{U}_g$$

where

K_t = the instantaneous global stiffness matrix which depends upon the amount of non-linearity experienced by each structural element

- M = the global mass matrix which is kept constant
 C = the viscous damping matrix
 $U, \dot{U}, \ddot{U}$ = the displacement, velocity and acceleration vector respectively
 $\ddot{U}_g$ = the acceleration of the ground
 Δ = symbol indicating differences of displacements, velocities and accelerations respectively.

Three different methods were followed in solving the previous equations. The first is the β -Newmark method which is described in Newmark's (2) classical paper with linear variation of the acceleration within the time step. An improvement of β -Newmark method is a method developed by Chaudhury (3) and (4). This method was used in those references for an elastic and a geometrically non-linear structure. Here, it is extended to include material non-linearities as well.

As well as the models for beams, columns, and braces discussed so far, another model which is frequently used in this project is a hysteretic model described in detail in section 2.3.2. The equations of motion of the structural system incorporating this model are solved by the fourth order Runge-Kutta method.

1.3.2.3 Shear deformation and panel zone distortions.

Shear deformation for beams and columns is included in the programs. The inclusion of shear deformation and, in particular, the hysteretic response of short structural elements which yield in shear, is described in detail in section 4.2.2.

The effect of joint panel zone distortion on the storey drift and on the total structural response is ignored here. In practice the panel zone distortion is minimised by the use of doubler plates and then calculation of member stiffnesses with centre-to-centre dimensions.

1.3.2.4 Correction of the out-of-balance forces in the dynamic response.

The solution of the equations of motion at any time t are based upon the response values at time $(t-\Delta t)$ which are used as initial values, and upon the stiffness matrix at the time $(t-\Delta t)$. In this form total

equilibrium is satisfied at any time t only as far as the stiffness matrix within the time step Δt remains the same. When the stiffness matrix changes within the time step Δt this results in out-of-balance forces as shown in fig. 1.2. As shown in this figure, using the stiffness matrix at time $(t-\Delta t)$ results in restoring forces F_t at time t . This produces the out-of-balance forces ΔF_u . The out-of-balance forces are corrected by writing the equation of motion in incremental form in the next time step as:

$$M\Delta\ddot{U} + C\Delta\dot{U} + K_t\Delta U - \Delta F_u = -M\Delta\ddot{U}_g$$

The same procedure to correct the out-of-balance forces is also used in the Drain-2D computer program (5)

1.3.2.5. Geometric non-linearities.

The influence of the axial load on the stiffness matrix of a column element is obtained by expressing the consistent geometric stiffness matrix for each member, as given by Meyer (6)

The reduction of the plastic bending moment for the beam and column elements because of the axial load is ignored in the present programs.

1.3.2.6 Energy expressions.

Energy equilibrium at any time step requires the satisfaction of the following equation:

$$IE = ST + KE + DV + DE$$

where IE = input energy

ST = strain energy

KE = kinetic energy

DV = energy dissipated through viscous damping

DE = energy dissipated through inelastic deformation.

Analytic expressions for each of these components are given in many textbooks and papers, as for example in ref (1), so they are not repeated here.

In the present dynamic analyses the energy imbalance is kept within 1%.

1.3.2.7 Introduction of damping.

According to Caughey (7), a damping matrix can be expressed as:

$$|C| = \sum_{i=1}^N \alpha_i |M| (|M|^{-1} |K|)^{i-1}$$

where the coefficients α_i are determined by specifying damping to each mode of vibration and solving the set of orthogonality conditions.

A simpler expression for damping is to take the damping proportional to the mass and stiffness matrices through the coefficients α and β , i.e. $C = \alpha M + \beta K$. Using this expression we succeed in the simultaneous diagonalisation of the M , K , C matrices. If mass dependant damping only is assumed, damping equal to λ_i in a mode with period T_i can be obtained by specifying $\alpha = 4\pi\lambda_i/T_i$. Similarly, if stiffness dependant damping only is assumed, $\beta = \lambda_i T_i / \pi$. In both cases, the damping for all values of T_i is determined once either α or β has been specified. If both α and β are specified, then the amount of critical damping can be defined at two different periods. That is:

$$\lambda_i = \frac{\alpha T_i}{4\pi} + \frac{\beta\pi}{T_i}, \quad \lambda_j = \frac{\alpha T_j}{4\pi} + \frac{\beta\pi}{T_j}$$

hence

$$\alpha = \frac{4\pi(T_j\lambda_j - T_i\lambda_i)}{T_j^2 - T_i^2}$$

$$\beta = \frac{T_i T_j (T_j\lambda_i - T_i\lambda_j)}{T_j^2 - T_i^2}$$

Stiffness dependant damping would appear to be the most reasonable, as it implies progressively higher damping in the higher modes. It also distributes the damping effect throughout the structure and not just at mass points.

When the structure yields, damping is increased quite materially by the frictional energy generated by movement of the surfaces along the plane of cracks. It is thought that original stiffness proportional damping is more indicative of the true conditions, since it results in

a substantial increase in damping after yielding. Additional damping may also be expected by the cracking of the non-structural components.

Mainly original-stiffness proportional damping is used throughout this research, unless otherwise specified.

1.3.3 Examples.

1.3.3.1 Static solution through dynamic analysis

The elastic static solution to a problem can also be obtained through dynamic analysis as is also mentioned in ref(3). A typical example is presented in fig. 1.3 for the static solution through dynamic analysis of the example shown in fig. 1.4. The two degrees-of-freedom system has frequencies of 1 Hz and 2.4 Hz. 60% damping is introduced in the two modes for higher convergence. The force is assumed to vary as shown in fig. 1.5.

Shortly after 1 second the dynamic analysis convergence to the static solution occurs which is due to the very high damping introduced in the dynamic response.

1.3.3.2 Dynamic response of a multi-storey building frame

The dynamic response of a multi-storey braced frame is considered here. Multi-storey braced frames usually involve a large number of unknown displacements and their dynamic analysis clearly requires a large amount of computer time. Accordingly, for simplicity, a shear building frame is considered.

The frame is a ten-storey structure as shown in fig. 1.6. The storey height is 3.7m and the span is 6.2m. The design of this frame is based on the procedures reported by Goel (8) and is in accordance with the UBC code. The design earthquake is based upon a dead load per storey per bay of 197 kN and the frame is considered to carry lateral forces from three bays of the building. The design C coefficient for the UBC design code was taken as $C=0.05$ and the other design values as unity. This is a somewhat crude but conservative estimate of the lateral forces.

Mild steel cross-sections were assumed for all the members. Two design criteria: member stresses and a lateral deflection of 0.35% of the storey height at design earthquake, were used. The moment resisting frame is designed for 25% of the total lateral load and the complete system for the full loads. The live load was taken as 143 kN per floor. The total mass in the response is taken as the mass of the dead load plus 25% of the live load of the frame. In the design of the moment resisting frame, the ultimate shear supported by each storey was taken as 1.2 times the storey design shear.

The diagonal braces were designed as tension members using an allowable stress of 153 N/mm². A minimum area of 1865 mm² (2.88 in²) was used for these members and the brace slenderness ratio was taken as 300.

The ultimate shear supported by the columns at each storey is shown in fig. 1.7(b) together with the proportion of the lateral stiffness carried by the columns and the braces. It is assumed that only the tensile braces contribute to the stiffness. The fundamental period of this frame, assuming again that only the tensile braces contribute to the stiffness, is 1.58 sec.

The frame is subjected to the first 20 seconds of the 1940 NS El Centro earthquake. The maximum deflections achieved by each floor are shown in fig. 1.8(a) and the maximum storey deflections in fig. 1.8(b). Since this frame requires approximately 1 cm storey deflection for the braces to yield, the storey deflections shown in fig. 1.8(b) introduce significant inelastic effects on the braces.

This relatively simple multi-storey frame was also used in the studies described in subsequent chapters.

CHAPTER 2

PROBABILISTIC BEHAVIOUR OF STRUCTURES DURING EARTHQUAKES

2.1 Introduction

The satisfactory earthquake resistance of conventional structures is commonly largely due to their ability to absorb the earthquake energy imparted by the ground motion. This is achieved by allowing inelastic deformation which is usually accompanied by some structural damage. The knowledge of the energy dissipation by each structural element, as well as the maximum inter-storey drifts during an earthquake are useful information to the designer wishing to have an optimum solution.

This may be achieved by examining in the time domain the response of the structural model using a step-by-step solution and modifying the design if necessary. Past earthquakes are usually used as the excitation, although, especially in the last two decades, there has been an increasing interest in using artificial earthquakes. The artificial earthquakes generated usually match a specified spectrum, typical of an average spectrum of previous earthquakes in the area under consideration. The advantage of this approach is that a large number of earthquakes can be used and the statistics of the response can be considered.

An alternative way of finding the variance of the response is through random vibration using the approach of probabilistic dynamics. In this approach excitation and response are described in the frequency domain; a power spectrum of an expected earthquake, together with an intensity function, are sufficient to determine the statistics of the response.

Three methods of generating artificial earthquakes are described. The first two were developed by other investigators, while the third is an extension of the existing procedure which has been developed for this study. This is followed by a description of the non-stationary response of a single-degree-of-freedom elastic system subjected to a non-stationary excitation. A method of finding the stationary response of an inelastic system to a stationary excitation is also presented.

Comparisons are then made between frequency domain and time domain solutions, and the errors involved in analysis based entirely upon the frequency domain solution are summarised.

Finally, applications are presented of the various analyses introduced in this chapter.

2.2 Generation of artificial earthquakes

2.2.1 Artificial earthquakes having an exact response spectrum.

A very simple way of generating artificial earthquakes with a specified acceleration spectrum is given by Levy (9). In this method an initial earthquake record is first generated which is then continuously modified until finally it matches a specific acceleration spectrum. The earthquake record is expressed as:

$$\ddot{x}_g(t) = F(t) \sum_{i=1}^N (-1)^i A_i \sin(2\pi f_i t)$$

where: A_i = coefficients to be determined

f_i = frequencies in Hz

$F(t)$ = an envelope function taking account of the expected form of build up and decay of ground motion.

The frequencies f_i are selected in such a way as to ensure full coverage of the frequency range. For the example presented here 2% critical damping is used and 90 frequency components within the frequency range 0.2 Hz to 30 Hz, satisfying the condition $\frac{\Delta f}{f} < 2\zeta$, where Δf is the frequency separation at frequency f , and ζ is the damping ratio. This choice is such that the half-power points of adjacent frequencies overlap.

The shaping function $F(t)$ used for the illustrative example is selected to give an approximation to the 1940 NS El Centro earthquake, as given in ref (9).

Initially random numbers are chosen for the coefficients A_i . In subsequent iterations the new values of A_i are found by multiplying the previous values of A_i by the ratio of the desired response spectrum at frequency f_i to that computed in the previous iteration. This

method quickly converges to a specific earthquake. Fig. 2.1 shows the target acceleration spectrum, the spectrum achieved in the first and in the fourth (final) iteration, and the corresponding ground motion. In this method, as also in the other two methods described, a base line correction is introduced to compensate for the fact that the displacements and velocities are not zero at the end of the earthquake. This is achieved by adding the parabolic term at^2+bt+c to the ground motion (10). The base line correction changes the displacements and velocities of the record but does not significantly affect the accelerations.

This method of generating artificial earthquakes lacks in comparison with the other two methods next described, which are based upon a power spectrum, since a power spectrum is a direct measure of the variance of the response, which does not, however, imply a single deterministic history of ground motion. } Confused!

2.2.2 A random approach based on a second order filter

This method was developed at the University of California, Berkeley (refs. 10 and 11). Non-stationary shot noise is first generated to represent the bedrock acceleration, which is then passed through a second order filter, representing the soil, to generate the ground acceleration.

Shot noise is usually called the non-stationary counterpart of white noise and is fully described by the covariance function $\text{Cov}(t, t+\tau) = \phi(t)\delta(\tau)$, in which $\phi(t)$ is the variance intensity function of the process and $\delta(\tau)$ is the Dirac δ -function. The shot noise process can be described mathematically as the product of white noise and a deterministic function of time $p(t)$. The variance intensity function of the shot noise process then becomes

$$\phi(t) = \pi S_0 p(t)^2 \quad (2.1)$$

where S_0 is the intensity of the white noise.

Initially, a sequence of independent random numbers U_j with uniform distribution in the interval (0,1) is obtained, using a standard subroutine on the computer. A new sequence of independent random numbers W_j with a Gaussian distribution having zero mean and unit variance is obtained by the transformation:

$$\begin{aligned} W_j &= (-2\ln U_j)^{\frac{1}{2}} \cos(2\pi U_{j+1}) \\ W_{j+1} &= (-2\ln U_j)^{\frac{1}{2}} \sin(2\pi U_{j+1}) \end{aligned} \quad (j \text{ odd}) \quad (2.2)$$

An example of the distribution of 2000 random numbers generated by this algorithm is shown in Fig. 2.2, together with the target Gaussian distribution.

If the numbers thus generated are deemed to represent the bedrock ground motion at successive time steps $\Delta\tau$, the power spectral density function of the ensemble of such wave forms takes the form:

$$S(\omega) = S_0 \frac{6-8 \cos(\omega\Delta\tau) + 2 \cos(2\omega\Delta\tau)}{(\omega\Delta\tau)^4} \quad (2.3)$$

where

$$S_0 = \frac{\Delta\tau}{\pi}$$

To model an earthquake of realistic finite duration, this time series W_j is subjected to the shaping function $p(t)$, which is given in terms of the variance intensity function by equation (2.1). The variance intensity function used here is

$$\begin{aligned} \phi(t) &= \pi S_0 & \text{for } t < 11.5 \text{ sec} \\ \phi(t) &= \pi S_0 e^{-0.155(t-11.5)} & \text{for } t \geq 11.5 \text{ sec} \end{aligned} \quad (2.4)$$

This function was put forward in ref. (10) to represent an average of previous American earthquakes.

Denoting the bedrock acceleration as A_f , the ground acceleration is found by passing A_f through a second order filter to represent the soil, i.e. in terms of the ground motion relative to bedrock, z :

$$\ddot{z} + 2\zeta_g \omega_g \dot{z} + \omega_g^2 z = -A_f(t) \quad (2.5a)$$

whence the absolute motion of the ground;

$$\ddot{x}_g(t) = -2\zeta_g \omega_g \dot{z} - \omega_g^2 z \quad (2.5b)$$

The values of the effective "frequency" and "damping" of the soil used here are $\omega_g = 5\pi$ rad/sec and $\zeta_g = 0.6$, as suggested by Tajimi (12). Equations (2.5) are solved by numerical integration using a time step of $\Delta t = 0.03$ sec.

The power spectral density (p.s.d.) of the output process then becomes:

$$S_g(\omega) = S_o \frac{1 + 4\zeta_g^2 \left(\frac{\omega}{\omega_g}\right)^2}{\left(1 - \frac{\omega^2}{\omega_g^2}\right)^2 + 4\zeta_g^2 \frac{\omega^2}{\omega_g^2}} \quad (2.6)$$

This is shown in Fig. 2.3.

The previous p.s.d. is only approximate, since there is a reduction of the p.s.d. of the white noise, as given by equation (2.3). Combination of equations (2.3) and (2.6) suggests a 10% reduction of the standard deviation of the ground acceleration compared with that obtained using only equation (2.6). This result was verified by finding the standard deviation of 30 earthquake records each having a total duration of 100 seconds.

A typical earthquake record with a total duration of 30 seconds using the variance intensity function given by equation (2.4) is shown in Fig. 2.4(a) and its acceleration spectrum for zero damping in Fig. 2.4(b). The visual resemblance of this record to actual earthquake records is also satisfactory. Earthquakes with bigger or smaller intensities can easily be obtained by multiplying the basic ordinates by an appropriate factor.

2.2.3. A random approach based on an auto-regressive filter.

May (13) generates wind records which specify a particular power spectrum using an auto-regressive filter. This approach is used here and extended to be suitable for earthquake records. The main advantage of this method compared with that described above is that any power spectrum may be considered.

Consider the random process z_q formed by a moving linear combination of another random process x_q :

$$z_q = C_0 x_q + C_1 x_{q-1} + \dots + C_k x_{q-k} \quad (2.7)$$

If Δt is the increment of the series, then the power spectra of z_q and x_q are related as shown in ref (13) by:

$$S_z(f) = |Y(f)|^2 S_x(f) \quad (2.8)$$

where

$$f = \frac{\omega}{2\pi} \quad (\text{Hz})$$

and $|Y(f)|^2$ is the transfer characteristic of the auto-regressive filter, given by:

$$|Y(\omega)|^2 = b_0 + 2b_1 \cos \omega\Delta t + 2b_2 \cos 2\omega\Delta t + \dots + 2b_k \cos k\omega\Delta t \quad (2.9)$$

where

$$\begin{aligned} b_0 &= C_0^2 + C_1^2 + \dots + C_k^2 \\ b_1 &= C_1 C_0 + C_2 C_1 + \dots + C_k C_{k-1} \\ b_2 &= C_2 C_0 + C_3 C_1 + \dots + C_k C_{k-2} \end{aligned} \quad (2.10)$$

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$$b_{k-1} = C_{k-1} C_0 + C_k C_1$$

$$b_k = C_k C_0$$

A record with a specific power spectrum is generated as follows. Initially a sequence of random numbers is generated with zero mean and unit variance as before, which have the power spectrum $S(\omega)$ given by equation (2.3). The transfer characteristic of the required autoregressive filter to produce the target spectrum $s_g(\omega)$ is given by:

$$|Y(\omega)|^2 = S_g(\omega)/S(\omega)$$

The coefficients $(b_0, b_1, \dots, b_k)$ in equation (2.9) are obtained by solution of equation (2.9) for $(k+1)$ values of ω evenly spaced on $(0, \omega_{\max})$. The coefficients $(C_0, C_1, \dots, C_k)$ of the autoregressive filter are given by the solution of the non-linear equations (2.10) which is achieved numerically by Newton-Raphson's method.

The target power spectrum chosen for the present study is again the Kanai-Tajimi function given by equation (2.6), with the values of ω_g and ζ_g given previously. The parameters in the generation procedure were given the values $\Delta t = 0.05 \text{ sec}$, $k=10$, $\omega_{\max} = 62.83 \text{ rad/sec}$ (10 Hz).

Interpolation points in each time interval $(t, t+\Delta t)$ can be obtained using the following approximate wave form as given by ref.(13):

$$x(t + \tau\Delta t) = \sum_{n=-3}^4 x(t + n\Delta t) \frac{\sin\pi(\tau-n)}{\pi(\tau-n)} \quad (2.11)$$

In this formula n is an integer and $\tau < 1$. Interpolation points in each time interval $(t, t+\Delta t)$ can easily be obtained by insertion of appropriate values of τ . This formula is such that it does not introduce any additional frequency components in the time history.

A typical earthquake produced by this procedure using the shaping function given in equation (2.4) and bedrock spectral intensity $S_0 = 50 \text{ cm}^2/\text{sec}^3$ is shown in fig. 2.5, together with its acceleration spectrum for zero damping. Integration of the record shows a progressive growth of velocity and displacement with time, which can be eliminated by a small uniform correction term. Figs. 2.6 and 2.7 show velocity and displacement before and after this correction.

Twenty records of duration 100 sec generated in this way with a constant (unshaped) variance intensity function gave approximately 10% higher rms value than the analytic solution for the given bed-rock spectrum and soil system transfer-function. This is believed to be primarily due to the upper frequency limit set at 10 Hz. Taking $k=10$, $f_{\max}=20$ Hz improved the rms but led to perceptible discrepancies in spectral shape. $k=20$, $f_{\max}=20$ Hz would be preferred if the improvement in accuracy was felt to justify the additional computation.

2.2.4 Power spectra of actual earthquakes

The Fourier spectrum of the longitudinal component of the Pacoima Dam acceleration record is shown in fig. 2.8. This is computed by a Fast Fourier Transform Routine using 2048 points and a 0.025 Hz frequency interval, as described in detail by Newland (14). The Pacoima Dam record indicates effective frequency and damping close to the values of $\omega_g = 5\pi$ rad/sec and $\zeta_g = 0.6$ used with the Kanai-Tajimi power spectrum in the preceding sections, and has a duration of 41 sec. The Fourier spectrum plotted in fig. 2.8 is the modulus of the computed complex Fourier Transform of the acceleration record.

For comparison in fig. 2.8 the Fourier spectrum of an artificial earthquake record generated using the autoregressive filter is also shown. The artificial earthquake has a duration of 30 sec and was computed to match the Kanai-Tajimi power spectrum with effective frequency and damping of $\omega_g = 5\pi$ rad/sec and $\zeta_g = 0.6$. The overall shape of both records is similar, but they also indicate local spikiness which is not consistent.

Real records show differing effective frequency and damping according to site and it is not clear whether local peaks (and troughs) depend upon the site. Real acceleration records also do not usually have significant low-frequency components as the Kanai-Tajimi power spectrum suggests.

The scatter appearing in the spectral estimates, however, in real and artificial earthquake records is believed to be predominantly an inevitable result of the finite length of sample constituting each record. For this reason, the response of a particular system to an earthquake input can vary significantly from one earthquake record to another. The generation of artificial earthquakes using a smooth target spectrum is therefore suggested as an acceptable procedure since, by using a large number of artificial earthquakes, it is possible to examine the statistical response of a particular system subjected to earthquakes with the same frequency content and intensity.

2.3 Random vibration

2.3.1 Equations of motion for a single-degree-of-freedom elastic system

Attempts to improve the design procedure for structural systems have been directed in recent years towards a probabilistic design which takes account of the major sources of variation in material and structural strength properties, including variability introduced by the design and production processes on the one hand and the variability in the load environment in which the structure operates on the other hand. Stochastic dynamic analysis of elastic or inelastic structures can play a central role in developing such methodologies for safety analysis and design of structures under natural or man-made hazards.

Seismic ground accelerations are generally modelled for random vibration analyses by non-stationary Gaussian processes. The Gaussian hypothesis is mathematically convenient and is physically acceptable since the characteristics of seismic accelerations depend on numerous contributions. In the present study the ground acceleration is described by a power spectrum $S(\omega)$ and a variance intensity function $\phi(t)$.

In this section the response of the structure with random properties subjected to stochastic dynamic excitation is analysed. Initially the response of a single-degree-of-freedom system is considered, and subsequently this is extended to the response of an inelastic system.

In these studies, the mean value and the correlation function are two quantities of vital importance in the statistical representation of the excitation and the response process, although they do not necessarily describe the random process completely. In particular, these two functions of the response process play an essential role in the safety analysis of structures subjected to random loading:

in estimating fatigue life, in evaluating the probability of catastrophic failure, etc. Furthermore, since the process is presumed to be Gaussian, these two functions determine the probability density of any order. Therefore the major concern of the present study is to evaluate the mean value and the correlation function of the response process of a general structure with the knowledge of the mean value and the covariance function of the excitation process or its equivalent.

The probability theory is described in many textbooks and papers, as for example in ref. (15). This theory is briefly summarised.

The equations of motion for a single-degree-of-freedom system with frequency ω_n and damping ζ , on an elastic soil foundation (cf section 2.2.2) are

$$\begin{aligned} \ddot{x} + 2\zeta \omega_n \dot{x} + \omega_n^2 x - 2\zeta_g \omega_g \dot{x}_f - \omega_g^2 x_f &= 0 \\ \ddot{x}_f + 2\zeta_g \omega_g \dot{x}_f + \omega_g^2 x_f &= -\ddot{x}_g \end{aligned} \quad (2.12)$$

In these equations x is the displacement of the single degree-of-freedom system relative to the ground and x_f is the response of the soil (which can be considered as a filter). ω_g and ζ_g are the effective ground frequency and damping (generally taken in this study as 5π and 0.6 respectively, as discussed in section 2.2.2).

Equations (2.12) are converted to the following system of first order differential equations:

$$\frac{d(Y)}{dt} = [P] [Y] + [f] \quad (2.13)$$

in which $[Y]^T = [x \ \dot{x} \ x_f \ \dot{x}_f]$, $[f]^T = [0 \ 0 \ 0 \ -\ddot{x}_g]$

and

$$|P| = \begin{vmatrix} 0 & 1 & 0 & 0 \\ -\omega_n^2 & -2\zeta\omega_n & \omega_g^2 & 2\zeta_g\omega_g \\ 0 & 0 & 0 & 1 \\ 0 & 0 & -\omega_g^2 & -2\zeta_g\omega_g \end{vmatrix} \quad (2.14)$$

Application of the standard methods of analysis of linear systems to equation (2.13) leads to the following equation for the zero time-lag covariance matrix $|S|$

$$\frac{d|S|}{dt} = |P| |S| + |S| |P|^T + |Q| \quad (2.15)$$

where matrices $|S|$ and $|Q|$ are given by:

$$S_{ij} = E(y_i y_j), \quad i, j=1, \dots, 4$$

$$Q_{ij} = E(f_i y_j), \quad i, j=1, \dots, 4$$

in which $E(\)$ stands for the expected value. As $|f|$ only contains one non-zero element, it is clear that the only non-zero element of $|Q|$ is Q_{44} , which is $Q_{44} = \phi(t)$.

Equation (2.15) results in a system of ten first order differential equations which are solved by the fourth order Runge-Kutta method.

The stationary solution requires the solution of the matrix equation

$$|P| |S| + |S| |P|^T + |Q| = 0$$

with $\phi(t) = \pi S_0$

A typical result of this analysis is presented in fig. 2.9 for the response of a system with frequency of 1 Hz and 3% critical damping having a constant variance intensity function $\phi(t)$. The system starts from rest at $t=0$, i.e. the problem is reduced to the transient at the start of the excitation, becoming asymptotically stationary as t increases.

One more example is shown in fig. 2.10 for the variance of the displacement of the 1Hz system with 1% critical damping using now the variance intensity function as given in equation 2.4. A value of $S_0 = 50 \text{ cm}^2/\text{sec}^3$ is used for both examples.

Stationary response always constitutes an upper bound to the nonstationary solution, but for this lightly damped system stationarity is never reached.

2.3.2 Non-linear systems

Following Wen (16) the restoring force, Q , in a hysteretic damper is described by the relation:

$$Q(x, \dot{x}) = c\dot{x} + \alpha \frac{F_y}{Y}x + (1 - \alpha)F_y Z \quad (2.16)$$

where c = a viscous damping coefficient and Z = a dimensionless hysteretic component satisfying the following non-linear first order differential equation:

$$Y\dot{Z} = -\gamma|\dot{x}|Z|Z|^{N-1} - \beta\dot{x}|Z|^N + A\dot{x} \quad (2.17)$$

Here Y and F_y are the yield displacement and force of the hysteretic damper, respectively; β , γ , and A are dimensionless parameters. The parameter N is an integer which controls the smoothness of the transition from elastic to plastic range (the higher the value of N , the more localised the transition), and α is the ratio of the asymptotic values of stiffness in the plastic range to the elastic stiffness. The $(\dot{})$ denotes the time derivative. All varieties of model, including the bi-linear, can be obtained by suitable choice of parameters.

Baber et al (17) showed that strength and stiffness deterioration may be obtained by allowing A and Y to be functions of the absorbed energy, although there is no model yet available which wholly satisfactorily describes the hysteretic behaviour of bracing elements.

Fig. 2.11 shows two rate-independent hysteresis loops generated by the Wen model under sinusoidal displacement of amplitude 14 cm and frequency 0.9 Hz. The value of $N=1$ is used here since this considerably simplifies the solution. This model has been used extensively by many investigators, for example by Constantinou and Tadjbakhsh (18,19 and 20) for optimum design of a base isolation system.

To apply equations (2.16) and (2.17), the conventional procedure is first to substitute the system by an equivalent linear system and then solve numerically the resulting system of non-linear equations. This involves solution of a large number of non-linear equations, approximately n^2 for an n -degree-of-freedom system. Another approach can now be presented which makes the solution of the multi-degree-of-freedom system relatively easy for the stationary case.

The equations of motion of a single-degree-of-freedom system without viscous damping and $N=1$ are:

$$M(\ddot{x}_g + \ddot{x}) + \alpha \frac{F_y}{Y} x + (1 - \alpha) F_y Z = 0 \quad (2.18)$$

$$g(\dot{x}, z) = Y\dot{z} + \gamma\dot{x}|Z| + \beta\dot{x}|Z| - A\dot{x} = 0 \quad (2.19)$$

Exact solution of these equations is very difficult. The conventional procedure is to linearise by replacement of (2.19) by:

$$Y\dot{z} + c_e \dot{x} + K_e Z = 0 \quad (2.20)$$

According to Atakil and Utku (21), the mean square error in replacing (2.19) by (2.20) is a minimum if c_e and K_e are given by:

$$c_e = E\left(\frac{\partial g(\dot{x}, Z)}{\partial \dot{x}}\right) \quad K_e = E\left(\frac{\partial g(x, Z)}{\partial Z}\right)$$

in which $E(\)$ denotes the expected value.

If the ground acceleration $\ddot{x}_g$ is a stationary Gaussian process of zero mean with spectral density $S_g(\omega)$, then Z , $\dot{x}$ are also jointly Gaussian zero-mean processes, and we can easily show for the case $N=1$:

$$c_e = \left(\frac{2}{\pi}\right)^{\frac{1}{2}} \left(\gamma \frac{E(\dot{x}Z)}{\sigma_{\dot{x}}} + \beta \sigma_z - A \right) \quad (2.21)$$

$$K_e = \left(\frac{2}{\pi}\right)^{\frac{1}{2}} \left(\gamma \sigma_{\dot{x}} + \beta \frac{E(\dot{x}Z)}{\sigma_z} \right)$$

The procedure now proposed for evaluation of the stationary response of multi-degree-of-freedom systems makes use of the complex frequency response functions for x and Z , $H_x(\omega)$ and $H_Z(\omega)$ respectively, corresponding to $\ddot{x}_g = e^{i\omega t}$. Then (2.18) and (2.20) give:

$$\left(\alpha \frac{F_Y}{Y} - \omega^2 M\right) H_x + (1 - \alpha) \frac{F_Y}{Y} H_Z = -M \quad (2.22)$$

$$(i\omega Y + K_e) H_Z + i\omega c_e H_x = 0$$

Equation (2.22) can easily be solved to give H_x and H_Z as functions of ω .

c_e and K_e are then obtained from (2.21) by the substitution:

$$E(\dot{x}Z) = \int_{-\infty}^{\infty} H_x^* H_Z S_g(\omega) d\omega$$

$$\sigma_x^2 = \int_{-\infty}^{\infty} |H_x^*|^2 S_g(\omega) d\omega$$

$$\sigma_Z^2 = \int_{-\infty}^{\infty} |H_Z|^2 S_g(\omega) d\omega$$

in which $H_x^* = i\omega H_x$, and H_x^* the complex conjugate of H_x .

This approach requires a double sided spectrum symmetric about $\omega=0$ for $E(\dot{x}Z)$ to become real. Since K_e and c_e are not known, an iterative procedure is proposed. Assuming initial values of c_e , K_e , new values of c_e , K_e are obtained by use of equation (2.21) and this is repeated until convergence is obtained. Rapid convergence was obtained when this was tested on a series of models. For an n -degree-of-freedom system this method requires the solution of $(2n)$ equations. For example, for a shear building, for each storey there are two unknowns, H_{x_i} and H_{z_i} , where x_i is the displacement of the given storey and z_i is the dimensionless hysteretic component for this storey.

It should be mentioned that the double sided spectrum used here has ordinates one-half of those of the single sided spectrum with which the engineers are more familiar.

2.3.3 Energy dissipation

The energy dissipation process can be formalised by reference to fig. 2.12; clearly energy dissipation is the summation of the shaded areas. It can be seen that

$$S = dx - \frac{dQ}{\text{elastic stiffness}} = dx - \frac{Q}{(F_y/Y)} = dx - \frac{Q'(x)dx}{(F_y/Y)}$$

where $Q'(x)$ is the derivative of the damper force Q with respect to the coordinate x

Then the cumulative energy dissipated

$$W = \int_0^t Q \left(1 - \frac{Q'(x)}{(F_y/Y)} \right) dx$$

and the rate of energy dissipation

$$R = \frac{dW}{dt}$$

is given by

$$R = Q \left(1 - \frac{Q'(x)}{(F_y/Y)} \right) \dot{x} \quad (2.23)$$

Substitution of

$$Q = \alpha \frac{F_y}{Y} x + (1 - \alpha) F_y Z$$

into equation (2.23) gives the required solution for the expected (average) energy dissipation rate, $E(R)$.

2.3.4 Example

Consider a single-degree-of-freedom system with the restoring force shown in fig. 2.11(a). Assume that a mass is attached to the system such that when the system is elastic it has a fundamental frequency of 1 Hz

$$(f = \frac{1}{2\pi} \sqrt{\frac{F_y}{YM}})$$

No viscous damping is assumed.

Table 2.1 summarises the response of this system for various earthquake intensities S_0 (as defined in section 2.2.2). The average number of crossings of the level zero with positive slope, which is a measure of the effective frequency of the system, i.e.

$$f = \frac{1}{2\pi} \frac{\sigma_{\dot{x}}}{\sigma_x}$$

is shown in the fourth column.

The value of the hysteretic loss factor η , such that the linear system with equivalent stiffness

$$\frac{F}{Y} (1 + i\eta)$$

would have the given displacement variance, is shown in the final column of table 2.1. It can be seen that for small excitation levels the value of η is approximately constant, but falls as the excitation is increased to large values. Numerical evaluation of the hysteretic element model at various amplitudes of cyclic motion shows that for small excitation levels the energy dissipation is approximately proportional to the square of the amplitude, but it increases less rapidly as the excitation level is increasing. This is clearly reflected in the values of η shown in table 2.1.

Typical results giving the complex frequency response function of the velocity are shown in fig. 2.13 for the case $S_0=50 \text{ cm}^2/\text{sec}^3$.

2.4 Applications of randomly-generated ground motions

2.4.1 Response of a degrading system

A numerical study of the response of a degrading system was carried out using 20 artificial earthquakes generated using the autoregressive filter as described in 2.2.3. The structure examined was an X-braced undamped single-degree-of-freedom frame, with all the lateral resistance derived from the braces, which were chosen to produce a structure with an elastic frequency of 1 Hz and maximum ductility not more than 3.0. Two different cases were examined, having braces with slenderness ratios $l/r = 100$ and $l/r = 300$ respectively. Greater degradation would be expected in the latter case (cf discussion in section 3.2.2). It was found that in 16 of the 20 earthquakes the structure with the bracing slenderness ratio of $l/r = 100$ suffered smaller maximum displacements than that having braces with $l/r = 300$, illustrating the proposition that degradation leads to increased deformation.

2.4.2 Optimum profile in a multi-storey building frame

The inelastic approach used in section 2.3.2 can be applied to find the optimum strength profile for a multi-storey frame. As an example consider a two-storey shear building. The restoring force in the first storey is assumed to be hysteretic, as shown in fig. 2.11(b). For the second storey, the same hysteretic response is assumed but with a reduction factor applied to both the yield force and the yield displacement. Two equal floor masses are assumed, such that the resulting elastic structure has frequencies of 1 Hz and 2.5 Hz. Table 2.2 summarises the reduction factor which must be applied to the hysteretic response of the second storey in order to produce balanced σ/Y ratios, i.e. the same value in each storey of the standard deviation of the storey displacement relative to the yield displacement of that storey. It was observed (see table 2.2) that for higher excitation levels weaker upper storeys are required.

2.4.3 Extreme-value response of a single-degree-of-freedom system

The extreme-value response of elastic, classical elasto-plastic, and a degrading elasto-plastic system was examined by numerical evaluation of the response to the first 20 sec of 50 artificial earthquakes generated using the autoregressive filter. The third case represents the hysteretic response of an X-braced system in which the braces have no buckling resistance. The elastic stiffness was selected to give a natural frequency of 1 Hz, and the yield displacement (for the elasto-plastic cases) was set at 17 mm, corresponding to a single-degree-of-freedom system designed according to the UBC design code using a design value of $C=1/15$ with unity for the other seismic loading parameters. The earthquake intensity was set at $S_0=50 \text{ cm}^2/\text{sec}^3$. Zero viscous damping was assumed.

The response of the three systems is presented in fig. 2.14 in the form of Gumbel plots. This presentation follows that used in ref (10) (p.621), i.e. for each case two lines are plotted, a broken line representing the actual ranked final relative displacements for the 50 trial runs, and a solid line representing the best fit type I extreme-value distribution. It is shown in these graphs that the degrading response is generally bigger than the elasto-plastic response.

The effect of the duration of the excitation is shown in fig.2.15.

The lines shown are best fit to plotted ordinates equal to the ensemble average values of the peak relative displacements of the whole run (20 seconds), divided by the ensemble average values within the first 10 seconds. It is shown here that the increase of the response of the degrading system after the first 10 seconds is bigger than the increase of the response of the elasto-plastic system.

2.4.4 Inelastic displacement spectra

Response spectra for an elasto-plastic system and for a bi-linear system with the bi-linear stiffness 10% of the associated elastic stiffness are shown in figs. 2.16(a) and 2.16(b) respectively for one artificial earthquake record generated using the autoregressive filter, with $S_0=50 \text{ cm}^2/\text{sec}^3$, and zero damping.

For frequencies more than 10 Hz the inelastic displacement is bigger than the elastic; thus in agreement with previously reported results for actual earthquakes (22). In this frequency range the bi-linear maximum displacement is smaller than the elasto-plastic but still bigger than the elastic. For this earthquake record in the intermediate frequency range (2 Hz - 10 Hz) the elastic displacement seems to be bigger than the inelastic. Finally for frequencies smaller than 2 Hz the displacements are close, probably increasing slightly for higher ductilities.

It should be mentioned that these curves correctly represent the maximum displacement; for the non-linear system the tripartite logarithmic plot cannot rigorously be interpreted as showing the maximum pseudo-velocity and pseudo-acceleration.

2.5 Errors expected in an analysis based upon the foregoing probabilistic approach.

2.5.1 General comments

The theory of probabilistic dynamics described in section 2.3 can lead to very simple expressions concerning the variance of the response or the energy dissipation.

This approach may, however, also result in significant errors. The errors may be due to the relatively small duration of the time histories and the various assumptions made, such as stationary response

or narrow-band response. For a non-linear system additional errors may be introduced because of the equivalent linearisation technique.

A parametric study was carried out to investigate the errors involved when the design is based entirely upon the probabilistic approach.

2.5.2 Elastic systems

For an elastic single-degree-of-freedom system an analytic solution is available for the average number of crossings of a particular level with positive slope. The analytical prediction is

$$N_Y^+ = N_0^+ e^{-\frac{Y^2}{2\sigma^2}}$$

This is based upon narrow-band stationary response, where

N_0^+ = the number of crossings with positive slope of level zero

N_Y^+ = the number of crossings with positive slope of level Y

σ^2 = the stationary variance of the response variable Y.

For a single-degree-of-freedom elastic system N_0^+ is equal to the natural frequency.

The time domain solution using artificially generated records is compared with this formula in figs. 2.17 to 2.19. Five earthquake records were generated using the autoregressive filter with the effective frequency and damping values reported in section 2.2.2, and duration of 100 sec, and applied to a single-degree-of-freedom elastic system with frequency of 1 Hz. Three different damping ratios were studied, i.e. 1%, 2% and 3% of critical respectively. The analytical prediction represents a reasonable average of the time domain solutions.

A similar comparison of the probability distribution of the envelope process (cf section 2.3.1) is presented in figs. 2.20 to 2.23. The results are all normalised with respect to the standard deviation of the stationary response predicted by the frequency domain solution. The analytic solution for this case is the Rayleigh distribution.

In fig. 2.20 a single degree-of-freedom elastic system with a frequency of 1Hz and for three different damping ratios is subjected to ten artificial earthquakes generated using the autoregressive filter with a duration of 100 sec.

In fig. 2.21 the same system with 1% critical damping is subjected to ten artificially generated earthquake records using the second order filter with a duration of 90 sec. As shown in figs. 2.20 and 2.21, even for these relatively long durations, the individual realisations of the envelope process substantially diverge from the Rayleigh distribution.

Fig. 2.22 examines the influence of slightly changing the frequency of the system. The damping was again at 1% critical. This figure shows response to two earthquake records, namely the fourth and the fifth used in fig. 2.21. As shown in fig. 2.22 even for a small change in the frequency, the envelope distribution has changed significantly. This result is not surprising since it is well known from the classical response spectra that even small changes in the frequencies can cause substantial differences in maximum response in lightly damped systems.

In the results shown so far, a constant variance intensity function was presumed in generation of the earthquake records (for the definition of the variance intensity function see section 2.2.2). The non-constant variance intensity function given by equation (2.4) was used in a final example. Using this variance intensity function ten artificial earthquake records were generated using the second order filter with a duration of 30 sec. These earthquakes were again used to excite the 1Hz system with 1% critical damping. As shown in fig. 2.23 the envelope distribution of the response now diverges even more from the Rayleigh distribution. This is probably due to the shorter effective duration of the earthquake.

2.5.3 Inelastic systems

The response of an inelastic system is now considered, subjected to artificially generated earthquake records. The artificial earthquake records were generated using as effective frequency and damping of the soil the values reported in section 2.2.2.

The greater part of this study was based on the structure that was used in section 2.3.4 to illustrate the theoretical probabilistic dynamics approach, i.e. that shown in fig. 2.11(a), with the mass chosen to give an elastic natural frequency of 1 Hz. The analytic results, shown in table 2.1, are now compared with time domain solutions using ten artificial earthquakes generated by the autoregressive filter to the basic spectral shape, each record being of 100 sec duration, and scaled as required to various excitation levels (S_0).

Typical hysteresis loops obtained for one earthquake record are shown in fig. 2.24(a) and typical energy dissipation in fig. 2.24(b). As shown in fig. 2.24(b) the energy dissipation is almost stationary. This is due to the fact that the hysteretic models used here introduce high damping and stationarity is very quickly achieved. The response of this structure for two different excitation levels of $S_0=200 \text{ cm}^2/\text{sec}^3$ and $S_0=800 \text{ cm}^2/\text{sec}^3$ is shown in figs. 2.25(a) and (b) respectively. The results shown in these figures are normalised by division by the corresponding results of the stationary solution found through probabilistic analysis; for example, the mean maximum values shown are divided by the standard deviation of the stationary solution. We observe that the variance and (most notably) the energy dissipation are well predicted by the probabilistic theory. However, the probabilistic theory predicts zero mean value, as a consequence of the equivalent linearisation technique, whereas the time domain solution always produces a non-zero mean value. This is not surprising, since it is well known that the inelastic response of a system may shift in one direction resulting in a non-zero mean. Similar results are also produced by the 1940 N-S El Centro earthquake.

The same system was then subjected to these ten earthquakes generated with $S_0=200 \text{ cm}^2/\text{sec}^3$, with the duration now reduced to 30 sec, together with the variance intensity function given by equation (2.4). The results of this analysis are presented in fig. 2.25(c); again normalised by the corresponding results of the stationary response. Typical energy dissipation is shown in fig. 2.26.

Finally, the same earthquakes with total duration of 100 sec, and intensity of $S_0=200 \text{ cm}^2/\text{sec}^3$ and constant intensity envelope were used to excite the model shown in fig. 2.11(b), the mass again being chosen to produce a system with an elastic frequency of 1 Hz. The results are presented in fig. 2.25(d). Similar conclusions regarding the response can also be drawn for this system.

2.6 Final remarks

A simple way of finding the response of inelastic systems based upon random vibration has been presented. This method is only applicable for stationary response, an approximation which is probably acceptable, since stationary response always corresponds to the maximum of the non-stationary solution. This method is especially applicable for structures incorporating hysteretic devices. A knowledge of the energy dissipation of these devices is particularly helpful to the designer.

Since, as shown earlier, this approach can sometimes lead to significant errors, it is suggested that such results be accompanied by studies using time domain solutions, using either actual or artificial earthquakes. As artificial earthquakes, earthquakes generated using the autoregressive filter are recommended, since random earthquakes with any power spectrum can be generated.

Artificial earthquakes are also very useful for experimental work on shaking tables, since the earthquake records can easily be adapted to suit the structural modelling.

CHAPTER 3

HYSTERETIC RESPONSE OF BRACES AND BRACED FRAMES

3.1 Introduction.

Analytical and experimental work done in the last twenty years on the hysteretic response of braces and braced steel frames is initially reviewed in this chapter. Analytical models are then presented describing the hysteretic response of braces which are used throughout this project. Finally, experimental and analytical work into the cyclic response of single and double angle braces and into the response of a k-braced frame is presented.

3.2 Literature survey and general remarks of previous investigators on the response of steel building frames

3.2.1. Response of moment resisting frames (MRF).

In unbraced building frames a considerable reduction is expected in the horizontal force-displacement relation due to column loads. This destabilising phenomenon, however, becomes more important in a higher structure, where the vertical loads are large. It is also significant when the horizontal displacement is large, even if vertical loads are small (23).

The ratio of column-to-girder strength has a pronounced effect on the dynamic response of the structure. Strengthening the girders reduces their ductility requirements and forces a corresponding increase in the column ductility, and so the columns rather than the girders control the lateral deflections. In this situation the columns will tend to absorb most of the ductile yield energy, leading to large deformations and danger of collapse in these vital load-carrying elements. The design problem of a multi-storey building is even more difficult because the ductile deformations tend to vary widely through the structure in a manner which depends not only on the structural properties but also on the intensity of the ground motion and undoubtedly on its character as well.

For columns in strong-axis bending, the column web at a joint or, as it is more often called, the panel zone, plays a particularly important role. This zone is large, being more than 10% of the column height in upper stories and 20% or more at the lower floors of tall buildings. Due to lateral loads, high shears are developed in the panel zone which may rise to high local distortions. The comparisons of experimental and analytical results show the importance of the panel zone distortion, as shown for example in refs. (24) and (25).

Refs. (26) and (27) conclude that welded connections produce a superior hysteretic response than bolted connections for cyclic loading on connections between beams and columns.

3.2.2 Hysteretic response of bracing elements.

Both experimental and analytical studies, see for example refs. (28) to (49) make it possible today to know the cyclic behaviour of individual bracing elements accurately enough. The general cyclic behaviour of a pin-ended bracing element follows the force-deformation relationship shown in Fig. 3.1. The first zone OA is the elastic zone generated by applying a compressive strain to the brace, until finally the brace buckles.

The buckling load for the first load cycle, as recommended by the American Iron and Steel Institute (AISI) is:

(a) For class A (annealed or hot-rolled) tubes

$$P_{cr} = \left(1 - \frac{\left(\frac{KL}{r}\right)^2}{2C_c^2}\right) \sigma_y A; \quad \text{for } \frac{KL}{r} < C_c$$

in which

$$C_c = \sqrt{\frac{2\pi^2 E}{\sigma_y}}$$

(b) For class B (unannealed) tubular sections

$$P_{cr} = \left(1 - \frac{\frac{KL}{r}}{1.5C_c}\right) \sigma_y A; \quad \text{for } \frac{KL}{r} < C_c$$

in which

$$C_c = \sqrt{\frac{3\pi^2 E}{\sigma_y}}$$

For $C_c < \frac{KL}{r} \leq 200$ and class A and B tubes

$$P_{cr} = \frac{\pi^2 EA}{\left(\frac{KL}{r}\right)^2}, \quad \text{the Euler buckling load.}$$

In the previous expressions KL/r and A are the effective slenderness ratio and the cross-area respectively and σ_y is the yield stress of the material. The second zone AB is dominated by the development of a plastic bending moment in the middle of the brace due to the moments induced by the compressive load P . The magnitude of the load P in this zone is decreasing since the $P\delta$ moment cannot exceed the member's plastic moment capacity. This zone is characterised by very large lateral deflections of the centre of the brace and by very large inelastic curvature in the centre region. If cyclic reversal takes place at point B then the inelastic strain portion of the brace will again begin to behave elastically. Zone BC corresponds to the elastic unloading of the member. The slope of this zone is much smaller than the virgin elastic curve due to the large permanent lateral deflection of the centre of the brace, which also results in a curved rather than a straight member. The lateral deflection decreases due to $P\delta$ bending moments induced by the tensile load. Point C is the start of yielding due these bending moments. Since these $P\delta$ bending moments are of opposite sign to those induced by compressive loads, this inelastic bending partially restraightens the brace as it lengthens. Since the lateral deflection δ is reduced

as the brace straightens, the tensile load required to sustain yielding must increase. Point D is the point at which the brace is fully straightened; any elongation beyond point D is purely plastic uniaxial elongation. In zone DE the tensile load P is nearly constant with increasing elongation Δ for elastic-perfectly-plastic material or it is increasing with increasing elongation for a strain hardening material. Point E is the load reversal point. Thus the final zone EF consists of elastic unloading, in which the elongation will decrease linearly with decreasing axial load, and the slope is essentially the same as the virgin elastic curve.

Subsequent cycles show the same general characteristics but with the origin shifted to a new location in the P - Δ space. This translation in the origin results in the so-called pinched hysteresis loops. For the braces to yield in tension in subsequent cycles, they must be stretched to the previous elongated length.

Another very important characteristic is the reduction of the buckling load of braces as a function of the number of cycles the braces have undergone. This reduction is bigger for braces having smaller slenderness ratios than for braces having larger slenderness ratios. The decrease of the buckling load is also greater in the initial cycles than in the later cycles. Typical results for the reduction of the buckling load of braces in relation to the number of cycles are shown in Fig.3.2 taken from ref. (47).

One reason for this reduction of the buckling resistance is that the brace develops increasing permanent deformation. This means that in subsequent cycles the brace must be treated as a strut subject to an eccentricity equal to the current value of this imperfection, characterised by a lateral deflection, e (say). Typical plots for the buckling load of a strut from mild steel subjected to a load with an eccentricity e are shown in Fig. 3.3. Here r is the radius of gyration and c is the distance of the extreme fibre from the centroidal axis. In these curves it was assumed that buckling occurs when the stress at

this extreme position equals the yield stress of the material (42). These graphs indicate that the buckling resistance is not much affected by the eccentricity of the applied load for braces with high slenderness ratios compared with those with small slenderness ratios. Another reason for the reduction of the buckling load by the number of cycles is the Bauschinger effect which lowers the apparent yield stress of the material.

It can thus be seen that the loading history is very important in the hysteretic response of braces. Braces which are initially stretched and then compressed show different hysteresis loops from those which were compressed directly. It is evident that a more confident prediction of the hysteretic response of braces would require a detailed finite element model taking into account progressive plastification, local buckling, end restraint conditions and so on, of considerable complexity and expense. Many simplified models have therefore been developed.

The first model put forward to describe the hysteretic response of braces is the so-called slip-model shown in Fig. 3.4. The braces are assumed to have elasto-plastic behaviour in tension and no compression capacity. Once the braces yield, the model translates in the $P-\Delta$ space, reflecting in this way the pinching effect as shown in Fig. 3.4. This model is clearly only asymptotically correct for braces having large slenderness ratios. Powell (41) developed a computer program in which he uses the two models shown in Fig. 3.5. Model (a) is an element yielding in both tension and compression and model (b) is an element yielding in tension and buckling in compression. It is common to classify axially loaded members into three regions according to member slenderness ratio, namely: short ($L/r < 60$), intermediate ($60 < L/r < 120$), and long ($L/r > 120$). It is believed that most bracing members would lie in the intermediate to long regions. So the first model shown in Fig. 3.5 is especially suitable for stubby members and the second for very slender members.

These simplified models have been greatly improved during the last decade. Based upon experimental work many analytical models have been reported from many countries, but especially from Japan and USA. Singh (37), Nonaka (35), Ikeda (49) are three typical publications from a large number. In these studies it is assumed that a plastic hinge forms in the middle of the bracing element when it is loaded in compression and the rest of the brace remains elastic. Simple yield criteria are used in these studies to take into account the influence of the axial load on the member's plastic moment capacity. For rectangular cross-sections and wide flange cross-sections subjected to bending about the weak axis, the fully plastic bending moment is modified as:

$$\frac{M_{pr}}{M_p} = 1 - \left(\frac{P}{P_y}\right)^2 \quad (3.1)$$

where:

- M_p = the plastic bending moment
- M_{pr} = the reduced plastic bending moment
- P = the axial load
- P_y = the yield force in tension.

This method of analysis is mainly applicable to slender beams which buckle elastically or nearly elastically. Members with low slenderness ratios are not controlled by elastic buckling and, as a result, the expansion of the yield zone during cyclic axial load must be considered. Finite element modelling, like that developed by Fujimoto (38), is very accurate for these circumstances.

In the foregoing description it was assumed that the brace was pin-ended. The effect of the end restraint is commonly regarded in terms of a change of the effective length, KL , of the member.

Experimental results show that the actual buckling load of braces differs from their theoretical buckling load. This is due to initial imperfections of the member, to the unavoidable eccentricity of the applied load, and to the estimation of the effective slenderness

ratio of the member. As shown in Fig. 3.2 the buckling load obtained at the first cycle is bigger than the expected value given by the AISI formula. This is due here to the estimation of the effective slenderness ratio of the member.

The normalised hysteresis curve of a brace (P/P_y versus Δ/Δ_y plot) is independent of the cross-sectional area and is mainly affected by the effective slenderness ratio KL/r . The cross-sectional area mainly affects the stiffness and strength of a brace.

The dynamic response of the bracing members has been found to be almost the same as the quasi-static response described so far, although high speed loading causes an increase in the effective yield stress of the material (48).

3.2.3 Hysteretic response of braced frames

Goel and Hanson (8) studied different kinds of diagonal bracing in multi-storey buildings and compared these results with the unbraced frame. The most important results of this study are summarised here. The $P-\Delta$ effects are very important in multi-storey buildings, and this makes the assumption of linear elastic column response of questionable accuracy. The addition of minimum cross-bracing (area 2.88 in^2 and $L/r=300$) results in limiting the drifts and the inelastic activity of the girder, although the accelerations are increased. Column axial forces are considerably increased in the braced frames. The columns on the unbraced frames generally remain elastic, but the braced frames show considerable inelastic activity in the columns. The addition of viscous damping reduces the diaplacements and stresses.

Wakabayashi (refs. 30, 44 and 45) reported that frames with braces having small slenderness ratios are more suitable for tall buildings since they are able to absorb a larger amount of energy.

A rather different bracing system, the multi-phase system, has been analysed and tested by Shepherd (36). He assumes that in the case of a braced tower with essentially pinned joints the bracing may itself be of a two-phase nature, possibly provided by the use of pairs of parallel braces, one of which is designed to remain elastic under all anticipated loading conditions and the second being selected with the intention that it should yield and finally break in a major earthquake.

In a report from Berkeley (46) three different types of bracing systems are examined experimentally, and the results compared with unbraced frames. The results of this study indicate that diagonal bracing systems such as tubular or double-angle braces are very effective in reducing lateral displacements of buildings for moderate earthquakes and that their energy dissipation will be significant, if their compressive capacity is not less than 50% of their tension capacity.

Designs using the tension-only concept have resulted in a rather large amount of damage to adjoining elements. This is especially true for K-braced frames, where brace buckling can cause significant damage to the beam, rather than to X-braced frames. As the result of such evidence, SEAOC (1985) recommends that braces in regions of high seismicity be designed for compressive stresses at code design loads whether acting in tension or compression. The braces should be so arranged that the compression and tension capacities of the braces are within 10% of each other for either lateral force direction.

Introduction of pre-tension can have beneficial results in an X-braced frame. Consider for example the frame shown in Fig. 3.6. Assume that the braces have no compressive strength. The strength of a frame with slender braces can be regarded as the sum of the unbraced frame strength and the strength of the bracing system. For very slender bracing members, the relationship between the restoring force of the bracing and the lateral deflection of the total frame is shown in Fig. 3.6.

If the bracing is initially straight and no pre-tension is introduced, the relation follows the line OAB. If pre-tension is introduced in the bracing, the compressed bracing also contributes to the rigidity until it eventually relaxes at C, following the line OCDB. When the magnitude of introduced pre-tension is equal to one half of the yield force P_y of the brace, the relation is as shown by the line OEB. This suggests that the required cross-sectional areas could be reduced by introducing pre-tension to the braces.

3.3 Hysteretic models used to describe the response of bracing elements

The computer programs developed for this study have made use of two simplified models describing the hysteretic response of bracing elements. These are described in the following paragraphs.

A model describing the hysteretic response of a single brace as shown in Fig. 3.1 is given by Singh (37). This model assumes that after buckling all the plastic deformation is concentrated in the middle of the brace whereas the rest of the brace remains elastic. Each one of the curves shown in Fig. 3.1 is found by writing the virtual work equation in incremental form. The solution is developed by applying an incremental longitudinal displacement $d\Delta$ in the brace and then finding the force in the brace by an iterative procedure (37). Simple expressions are used for the relation of the plastic moment capacity of the brace to the axial load; for example, equation (3.1). This model has been used only for the quasi-static response of braces and braced frames. For the dynamic time domain solution computer programs, the model was developed as shown in Fig. 3.7(a). This model is a piecewise linear approximation to the previous one, thus avoiding the need for iteration within each time step in the time domain solution. The out-of-balance forces have to be corrected only at the transition points. This model was again developed by Singh (37).

The buckling resistance of the braces in this model is approximated by a constant value; this was generally taken as $P_{cr} = -25P_y / (KL/r)$. According to Singh, this is an average buckling load of braces corresponding to the 3rd or 4th cycle (37). Another characteristic of this model is the distance $OA = OB * 55 / (KL/r)$ as shown in Fig. 3.7(b), which is a result of previous experimental work. After the brace yields in tension the origin of the model translates in the $P-\Delta$ space, reflecting in this way the pinching effect. For high values of the KL/r ratio the model reduces to the slip model. It will be noted that these models ignore the progressive reduction of buckling load with successive cycles of loading.

3.4 Current experimental verification

3.4.1 Experiments on the hysteretic response of bracing elements

Single and double angle braces which were extensively used in the experimental work on braced frames reported later were tested to clarify their hysteretic response.

The braces tested are shown in Fig. 3.8 together with a photograph of the arrangement in Fig. 3.9. The load was applied by a 120 kN screw jack and was measured using a load cell. The longitudinal displacement was recorded using an AC/DC variable-resistance displacement transducer. The applied force versus displacement was plotted using an X-Y plotter. All the braces tested were pin-ended, with lengths from pin to pin of 1m.

Initially some flat very slender braces were testing having an effective slenderness ratio of $KL/r=550$, mainly for calibration purposes. As expected, it was found that the buckling resistance of these braces was hardly affected by the number of cycles. A typical normalised hysteresis loop corresponding to those very slender braces given by the analytical model is shown in Fig. 3.10.

The properties of the single and double angle braces tested are shown on Table 3.1. The double angle braces were connected at two intermediate points with battens, which ensured that the two angles formed an effective compound member and that buckling occurred only in the plane of symmetry. The single angle braces buckled about their weak axis. The angles were of bright mild steel with a yield stress of 330 N/mm^2 .

Two sets of double angles were tested and the hysteresis loops obtained are shown in Figs. 3.11 and 3.12. The first set was tested as follows. Denoting the yield displacement in tension as Δ_y , the angles were first compressed to a displacement of Δ_y . They were then tensioned to a load equal in magnitude to the critical buckling load, compressed to a displacement of $2\Delta_y$, tensioned to current compression capacity, compressed sufficiently to establish the renewed maximum load, tensioned again to basic buckling load, and compressed again to $3\Delta_y$. A few more hysteresis loops were obtained by compressing the angles at higher displacements, as shown in Fig. 3.11. Upon the last cycle, the load was increased to yield the angles in tension, and then removed. The second set of double angles was first compressed to a displacement of $2\Delta_y$ and then tensioned to a load equal to approximately 80% of the yield load in tension. After two compression cycles at $2\Delta_y$ the angles were compressed to higher displacements. Finally the load was increased to the yield load in tension and then removed.

The buckling resistance of the first set of double angles is almost the same as the theoretical buckling load, whereas in the second set it is smaller, probably due to a small eccentricity of the applied load. A noticeable result is the significant reduction of the buckling load of the first set of double angles in successive cycles. The reduction is smaller in the second set of double angles, which are stretched more strongly after each buckling.

A similar loading history was followed for the two single angle braces, as shown in Figs. 3.13 and 3.14. Here both braces show a first buckling load slightly higher than the theoretical, and this is probably due to the small degree of restraint provided by the two ends. Due to their higher slenderness ratio, these braces show a smaller reduction of buckling load in successive cycles. The pinching of the hysteresis loops can clearly be seen in Fig. 3.14 for the single angle brace which is stretched significantly in order to yield in tension.

3.4.2 Response of a small-scale K-braced frame

3.4.2.1 Selection of member sizes.

Cyclic static loading tests were carried out on a frame designed to approximate the first storey of a multi-storey K-braced frame at $\frac{1}{4}$ linear scale. The leading dimensions are shown in Fig. 3.15.

Since the braces in multi-storey frames usually dominate the lateral stiffness and strength, the same decision was taken for the test model, in which the braces provide 95% of the total lateral stiffness. The braces were single angles, the same as used for the member tests described above, oriented to buckle in-plane about their weak axis. The angles were welded at both ends to the RHS beam and column members through stiff gusset plates which provide (as will be shown) full restraint at both ends. The buckling resistance of those angles is about 40 kN, which is about 80% of their yield resistance in tension. This is in agreement with recommendations from SEAOC to have buckling resistance in the braces in K-braced frames close to their tensile yield resistance.

The critical parameter in modelling a K-braced frame for the present purpose is the relative strength between bending behaviour and the braces. As we shall see shortly, brace buckling introduces a downwards vertical force in the middle of the beam which is the vertical component of the resultant of the two brace forces. This causes a

downwards deflection in the middle of the beam and probably the development of a plastic bending moment at that point. The importance of this effect depends upon the relative strength of the braces and the beam. For the dimensions of the K-braced frame chosen here, the downwards deflection in the middle of the beam is important, because the braces provide a much higher strength than the beam.

All the connections in this frame were welded and conservatively designed. The beam-column intersections were reinforced with plates which minimised the relative joint rotation. Out-of-plane deflections both in the frame and in the angles were very small.

3.4.2.2. Response of the frame

The load was applied quasi-statically with a hydraulic jack, pin connected to the frame. A load cell was incorporated to measure the load. The displacement-time-history applied to the frame is shown in Fig. 3.16 and the resulting force-displacement hysteresis loops are shown in Fig. 3.17. The displacements measured in the test were the horizontal deflection at the beam and the vertical deflection at the point of intersection of the two braces at the middle of the beam.

Strain gauges were attached in pairs on the minor centroidal axis, symmetric about the major axis at three points in the length of the angles. The readings of these gauges in the linear elastic range, in conjunction with the differential equation describing the buckling of a strut with equal end restraints, suggest an equivalent effective length for the angles of approximately $0.53L$, where L is their actual length. This indicates that essentially full restraint is provided at both ends of the braces.

As a result of the end fixity, plastic hinges developed in the angles in the post-buckling regime not only in the middle of their length but also at the ends. The strain gauges at the quarter length of the angles (which roughly correspond to the points of contraflexure) were used to find the axial force in the angles. The readings of the results at two strain gauges at the quarter length point were not in general exactly equal. This is due to a small out-of-plane deflection which is possibly caused by a small asymmetry of the total arrangement. The maximum difference between strain gauges which were expected to give the same results was about 7%.

Vertical displacements and also strain-gauge readings were measured for the first three cycles at frequent intervals. The vertical displacements in the middle of the beam and the hysteresis loops of the braces are shown in Figs. 3.18 and 3.19 respectively.

An analytical prediction for the first two load cycles, based on the first element model described in § 3.3, is also shown in Fig. 3.17 for comparison with the experimental results. The element model was suitably modified to take account of the development of additional plastic hinges in the two fixed ends. For the frame itself elastic-perfectly-plastic behaviour was assumed once the bending moment at a point reached the plastic bending moment capacity of the cross-section.

The actual linear elastic stiffness of the frame is smaller than the value predicted analytically. This discrepancy is probably due to the flexibility introduced to the frame by the end connections. Analytical results for the frame beyond the elastic range under monotonically applied load are shown in Fig. 3.20, and the predicted sequence of plastic hinge formation is shown in Fig. 3.21.

The experimental results show that hysteresis loops of the frame are pinched, but the pinching is not of the same degree as that of individual braces. This is clearly due to the integration and partici-

pation of all the structural components to form the frame. The vertical displacement in the middle of the beam, which is effectively zero before brace buckling, is very high after buckling, increasing notably in each successive buckling. This vertical displacement can clearly be seen in the photograph in Fig. 3.22. Collapse occurs in the combined mechanism, while one brace buckles and the other is in tension, as is typical of K-braced frames in practice.

3.4.3 Response of the moment resisting frame

A second frame was tested without the braces to examine the hysteretic response of the frame itself. The force-displacement loops obtained are shown in Fig. 3.23. A kinematic hardening model is sufficient to describe the hysteretic response of this frame.

3.5 Dynamic response of the K-frame

Analytical predictions are now given for the dynamic response of the K-braced frame considered in §3.4.2.2. It is assumed that a mass is attached to the beam so that the fundamental frequency of the frame is 8 Hz. The structure is assumed to have zero damping. Results are presented for this structure subjected to the first 7 seconds of the 1940 NS El Centro earthquake. Because of the modelling, the earthquake causes only moderate displacements in the frame. In particular the columns remain elastic while brace buckling occurs and plastic deformation is restricted to the beam. The horizontal displacement on the beam is shown in Fig. 3.24 and the vertical displacement in the middle of the beam in Fig. 3.25. Brace buckling and the development of a plastic bending moment in the middle of the beam causes a continuous downwards deflection in the middle of the beam.

3.6 Final remarks

The introduction of braces in multi-storey building frames is generally considered to be advantageous since the braces reduce the inter-storey deflections and minimise the damage to non-structural components. On the other hand, severe earthquakes can cause significant damage to the braces and costly repairs are subsequently needed. Mechanisms which could absorb the earthquake energy, thus protecting the braces, are sought in the next chapter.

CHAPTER 4

ENERGY DISSIPATION FOR BRACED FRAMES

4.1 Introduction

It was shown in the previous chapter that braced frames offer considerable strength and stiffness, but their inelastic behaviour, especially energy dissipation capability can be poor. This is due to the poor hysteretic response of the bracing elements, as has been discussed in detail.

The frames considered so far were concentrically braced frames, i.e. for an X-braced frame the centre lines of the beam, the column and the braces meet at a single point, and in the case of a K-braced frame the centre lines of the two braces intersect at the neutral axis of the beam. In the first part of this chapter an alternative configuration, the eccentric bracing system, is discussed. In this system, which has principally been developed in California, an eccentricity is introduced at the connection of diagonal bracings to the beams which form horizontal members of the bracing truss. The proportions are so chosen that the beam yields in shear before significant damage is done to the bracings or their immediate connections. Because the stress gradients in the web of the beam are small, plastic deformation can take place over a sufficiently large region to accommodate large overall deformations and ensure substantial energy dissipation.

In the second part of this chapter alternative configurations are sought, and in particular friction devices which, suitably placed, can be used as the main source of energy dissipation. These friction devices have the advantage over the eccentric bracing system that damage to the primary members is completely prevented.

4.2 Eccentrically braced steel frames

4.2.1 General

Fig. 4.1 shows alternative bracing arrangements for eccentrically braced frames. The characteristic feature of this bracing system is that the axial forces induced in the braces are transmitted either to a column or another brace largely through shear and bending in a segment of a floor beam called a link. Yielding of the beam imposes a limit on the load that can be transmitted to the bracings, giving protection against the progressive deterioration of strength which occurs in the event of cyclic deformation that causes buckling of bracing members. The braces must be designed so that they do not buckle under extreme loading conditions. This basic requirement is assured by an accurate estimation of the ultimate shear capacity of the link section of the beam; the bracing is then proportioned to ensure that the major inelastic activity takes place in the link. This system has been based upon studies conducted mainly in USA, with some contribution from Japan; refs. (50) to (57) are typical examples from a large number of publications in this field. Design recommendations for links, beam-column connections and beam-brace connections for eccentrically braced frames are given in these publications.

4.2.2 Stiffness and strength of short beam elements yielding in bending or shear

The links can be classified as either shear links or bending links, depending on whether they yield predominantly in shear or bending. As noted earlier, shear links are more effective as energy dissipators, and the following discussion focuses on this type, with I-section beam members. In presentation of examples, the beam dimensions will be given in the format $b/d/t_w/t_f$; b = the flange width, d = the beam depth, t_w = the web thickness and t_f = the flange thickness.

For the link to yield predominantly in shear, ref (56) concludes that the shear link strength e should satisfy the following condition:

$$e \leq 1.6 \frac{M_p}{V_p} \quad (4.1)$$

in which M_p is the plastic bending moment of the cross-section, and $V_p = 0.55 \sigma_y d t_w$ is the shear yielding load (assuming that all the shear load is carried out by the web) for steel of yield stress σ_y .

Ref (56) also concludes that the ultimate shear force V_{ult} of a shear link which satisfies equation (4.1) should be taken as at least

$$V_{ult} = 1.5 V_p$$

This is largely due to the significant additional shear carried by the flanges in I-sections after the web has yielded in shear.

Three different stages can be distinguished for the stiffness matrix of a shear link. The first stage is when the shear link remains elastic. Consider an element of a shear link as shown in Fig. 4.2, subjected to transverse loads and moments at the element ends. Then the stiffness matrix of this element, as given for example, by ref(6) is:

$$|K| = \begin{bmatrix} K_1 & K_2 & -K_1 & K_2 \\ & K_3 & -K_2 & K_4 \\ \text{symmetric} & & K_1 & -K_2 \\ & & & K_3 \end{bmatrix} \quad (4.2)$$

where

$$K_1 = \frac{12EI}{L^3} (1 + \phi_s)$$

$$K_2 = \frac{6EI}{L^2} (1 + \phi_s)$$

$$K_3 = \frac{EI}{L} \frac{4 + \phi_s}{1 + \phi_s}$$

$$K_4 = \frac{EI}{L} \frac{2 - \phi_s}{1 + \phi_s}$$

in which

$$\begin{aligned}
 E &= \text{Young's modulus} \\
 G &= \text{shear modulus} \\
 L &= \text{length of the member} \\
 I &= \text{moment of inertia} \\
 S_w &= \text{cross-section of web} \\
 \phi_s &= \frac{12EI}{L^2 G S_w}
 \end{aligned}$$

This matrix connects the forces in directions 1, 2, 3, 4 respectively, with the corresponding displacements.

The second stage occurs after the web yields in shear while the flanges still remain elastic. In this case, as shown in ref. (50) sandwich beam theory can be used to describe the stiffness matrix of the element. This theory utilises the fact that two different materials, a weak one, which in our case is the web which yields in shear, and a strong one, which in our case are the two flanges which remain elastic, are connected together.

This theory also includes the effects of warping restraint provided by the two ends of the element, using 'warping' in this context to refer to out-of-plane distortion of an initially plane cross-section of the beam. This vertical-plane shear deformation of the flange is clearly likely to be small. This effect (which is not to be confused with the warping which is usually associated with out-of-plane deflections due to torsion) is thus likely to be important when the web has yielded in shear.

Since the shear links in eccentrically braced frames are usually fully welded to the surrounding beams and columns, warping restraint at both ends of the shear links is usually almost complete. A very convenient formulation describing the stiffness matrix of a shear link after the web has yielded in shear is shown in Appendix A, based on the sandwich beam theory. As shown in Appendix A, the stiffness matrix of a shear link can again be described by equation (4.2) by changing only the ϕ_s term.

The final stage of the hysteretic response of a short I-section beam yielding in shear is when the flanges yield as well and in this case the stiffness of the beam is reduced significantly.

4.2.3 Inelastic deformations of eccentrically braced frames

Typical admissible fields of deformations for two eccentrically braced frames are shown in Fig. 4.3. The storey drift of this frame is designated by the angle θ . While the moment resisting frame has to sustain the plastic hinge rotation of θ , the short links experience significant larger deformations, the centre line change of shape being approximately $(\frac{L}{e})\theta$. The link length must therefore be chosen with care. If the link length e is very short, the imposed inelastic rotation, $\gamma = (\frac{L}{e})\theta$, may become excessively large, and possibly cause premature failure of the link. Design recommendations for shear links, based principally on the experimental program conducted at Berkeley are briefly given in the next section.

4.2.4 Design recommendations for shear links

Extensive experimentation at Berkeley has shown that shear links with well-stiffened webs can sustain without failure a total cyclic angular rotation γ up to ± 0.10 rad. However to minimise damage it is recommended by SEAOC (1985) that γ be limited to ± 0.06 rad.

The generation of stable hysteresis loops without any degradation requires reinforcement of the web of the shear links with transverse stiffeners. The stiffeners must have sufficient axial strength to permit the web to develop tension field action and must also be rigid enough to prevent buckling of the whole link web as a single panel. Refs. (51) to (54) give design recommendations concerning stiffener dimensions and stiffener spacing as a function of the ductility or energy absorption requirements in the shear link. Fig. 4.4 shows typical hysteresis loops of an eccentrically braced frame tested at Berkeley with links designed according to these recommendations (57).

4.2.5 Response of a single-storey Y-braced frame

4.2.5.1 Stiffness of the single-storey Y-braced frame

A single-storey Y-braced frame is shown in Fig. 4.5 . The braces are not connected directly to the beam but to a stub which suitably designed can yield either in shear or in bending, protecting the braces from buckling.

Fig. 4.5 compares the lateral stiffness provided by this Y-braced frame with the stiffness provided by the corresponding K-braced frame without a stub. In this figure I_b , I_c , and I_{st} stand for the second moment of area of the beam, column and stub respectively, and A for the cross-sectional area of the braces. In these results shear deformation is ignored and the braces are assumed to be axially loaded members only.

This figure shows, as expected, that the introduction of the stub generally reduces the stiffness of the Y-frame relative to the corresponding K-frame, due primarily to bending of the beam. Since shear deformation in the stub is in fact significant, especially for the small e/h ratios, a Y-braced frame with realistic dimensions will provide somewhat smaller stiffness than the value shown in Fig. 4.5 and less than the concentric K-frame in all cases.

4.2.5.2 Monotonic response of a single-storey Y-braced frame

The monotonic response of a single-storey Y-braced frame is examined here using a finite element package FINAS developed at Imperial College. The frame is shown in Fig.4.6 . Beam, columns, stub, and braces are I-sections and their dimensions are described using the notation for beams described in section 4.2.2. All cross-sections are oriented to bend about their strong axis. The braces were conservatively designed to avoid buckling at any stage. The dimensions of this single-storey frame are based upon the experimental study of a Y-braced frame reported in ref. (55).

Fig. 4.7 shows the monotonic response of this frame for different stubs and different eccentricities e . In the figures the most significant inelastic activity occurring in the frame is highlighted by using three different symbols.

In these finite element runs a strain-hardening ratio of $H=0$ was taken for simplicity. For the stub proportions shown, the bending resistance of the individual flanges of the stub is significant, so the stub has a significant stiffness even for the non-strain-hardening model ($H=0$, see Appendix A) when the stub web is yielding. For the small link length of $e=20$ cm, for which mainly shear yielding occurs, this leads to the effectively bi-linear restoring force relationship shown in Fig. 4.7.

For the higher link length of $e=40$ cm, the stub yields from combined shear and bending stresses. The restoring force relationship in this case is more complicated as a result of the more progressive plastification of the stub.

As shown in this figure, the Y-frame can be designed suitably to have a desired yielding load and reach a desired level of ductility.

4.2.5.3 Final remarks

The Y-braced frame, although not very popular in the USA, might have some advantages in comparison with other eccentrically braced frames. In particular, if the shear link is located in the beam in real buildings the influence of the dead weight might cause problems by acting against reverse plasticity. This problem does not exist in the Y-frame.

Experimental work on the hysteretic response of a shear link and on a Y-frame is presented in the following section.

4.2.6 Experimentation

4.2.6.1 Cyclic response of a small shear link

To assist in the design of a suitable test-model Y-braced frame, a small shear link was tested in order to clarify the hysteretic response of this type of link.

The shear link was a fabricated all-welded I-section beam, shown in Fig. 4.8. It was designed to yield predominantly in shear while the bending stresses are small. The yield stress of the material used was $\sigma_y = 500 \text{ N/mm}^2$.

The design recommendations given by ref. (56) for the dimensions and spacing of stiffeners relate to large I-section beams, and clearly are not applicable here. It was decided simply to reinforce the web with transverse double-sided stiffeners at the point of application of the load and at the two reactions. Warping restraint was provided by extending the beam on both sides of the supports.

The hysteresis loops obtained from this test are shown in Fig. 4.9. The loops proved very stable, although the strains are higher than those recommended for large I-sections on the basis of the Berkeley experiments. There is a general increase in the load sustained by the shear link, especially in the first cycles. This is a typical characteristic of shear yielding, as was pointed out by Roeder (50), and is due to the significant strain-hardening associated with shear yielding. At the later cycles the web buckles and develops the diagonal tension field.

4.2.6.2 Cyclic response of a Y-braced frame

The hysteretic response of a Y-braced frame is considered here. This is a first attempt to improve the hysteretic response of the K-braced frame described in the previous chapter, by introduction of a shear link at the intersection of the two braces. The dimensions of the Y-braced frame so produced are shown in Fig. 4.10. Photographs of this frame and of the shear link at the end of the test, are shown as Figs. 4.11 and 4.12.

The stub shear link is similar to each of the panels of the shear link tested earlier. The design capacity of the shear link was based upon the buckling resistance of the braces. Warping restraint in the shear link is provided by welding both ends to very stiff gusset plates.

The hysteresis loops of the Y-frame are shown in Fig. 4.13. For the first five cycles the hysteresis loops are very stable, but subsequently degrade significantly. This is especially due to the rupture of the web of the stub as shown in the photograph (Fig. 4.12). An additional reason which probably causes damage to the stub was the fact that this stub was a fabricated cross-section with a web machined down to the desired thickness. It is possible that unequal thickness of the web may concentrate damage at some points. Buckling of one of the two braces, which was experienced at the end of the fifth cycle, is a further source of degradation.

After the first three cycles where mainly shear yielding occurred, the stub web buckled and the diagonal tension field then developed. This was measured at positions (1), (2), and (3) in Fig. 4.13 and plotted in Fig. 4.14 using a standard computer software.

The vertical displacement in the middle of the beam is shown in Fig. 4.15. This displacement is zero for the first three cycles where mainly shear yielding occurs, and only small vertical displacements appeared after the buckling of the stub web; significant vertical displacements developed after brace buckling.

4.2.6.3 Hysteretic response of a device suitable for an X-braced frame

The hysteretic response of a device based upon bending deformation is considered here. This device, which is shown in Fig. 4.16, is suitable for insertion in an X-braced frame.

Under test, as shown in Fig. 4.16, the device is initially elastic, yield then commencing in bending, near the corners. The corner connections were reinforced, as shown in Fig. 4.16, so that plastic bending moments developed at two points in each corner, left and right from the reinforcing position. This device is similar to the bending device tested by Tyler (58).

The hysteresis loops of this device are shown in Fig. 4.17. Detail of the monotonic response is shown in Fig. 4.18. This device was also used in the experimental work carried out at the shake table, described in detail in section 6.5.4.

4.3 Response of braced frames incorporating friction devices

4.3.1 General

It has been shown that devices which are based upon shear or bending deformation can considerably increase the ability of a braced frame to absorb the imparted earthquake energy. However, a device closely approximating ideal elasto-plastic deformation tends to encourage concentration of permanent deformation at one panel, the "weak link" in the structure, wherever that may be.

It is therefore proposed to seek modifications that will retain the advantages described previously, but in combination with a hardening characteristic. Significant progress was made in this direction some fifteen years ago (59), by the application of high strength friction grip (HSFG) bolts using an arrangement of slotted holes to permit large movements. This approach has been extended in this study to be suitable also for dynamic applications.

4.3.2 Description of the friction devices

Clark (59) was one of the first investigators to report on the use of a linear slipping device. This device consisted of three plates preloaded together with HSFG bolts as shown in Fig. 4.19(a). The

middle plate having slotted holes was able to slip relative to the two outer plates when the applied load exceeded the frictional resistance of the joint. Clark (59) applied this device in the foundations of a building in order to minimise the effect of differential settlements on load distribution within the building, and therefore only tested the device in one direction of relative movement, i.e. a monotonically applied load, until finally the clearance in the slotted hole in the middle plate was eliminated. Such a device has been tested more recently in alternating loading (refs. 60 and 61) for application in earthquake engineering. The authors of ref. (60) inserted pads of an asbestos-based friction material between the plates to improve the hysteretic response of the joint.

A device of this type was also tested on this project. The results of the extensive friction tests carried out are shown in the next chapter. It is shown that by introducing suitable material between the slipping surfaces, the device is able to deform the number of cycles expected in an earthquake while keeping the same characteristics.

In addition to the linear slipping devices, a novel approach was made by extending the study to devices based upon rotational friction, as shown, for example, in Fig. 4.20. This is referred to as the rotating device in the sections that follow. Annulae, or "washers" of a selected material are inserted in the corners of the rectangular plate assembly, preloaded by HSFG bolts as before. The resisting bending moment at the corners thus depends upon the dimensions of the washers and on the pressure caused by the bolt. The friction faces remain locked for a certain load, permitting only a small elastic deformation, and start rotating when the load transmitted by the braces produces bending moments at the corners of the device which are equal to the resisting slipping bending moment.

As shown in the friction tests in the next chapter, such an arrangement, also, is able to deform for the expected number of cycles in an earthquake, producing broad, stable hysteresis loops (i.e. approaching rectangular).

4.3.3 Slip coefficient for a bolted joint

The slip coefficient of a bolted joint is examined in many publications; refs. (62), (63) and (64) are typical examples. The parameter mainly investigated in these publications is the coefficient of friction and the conditions affecting it, such as the influence of different steels, different surface conditions, exposure of the specimen in the open air, and additional faying surfaces between the slipping surfaces. In these publications generally the initial slip coefficient is investigated and not any possible variation in repeated cycles of deformation; the latter is of paramount importance in the present investigation.

4.3.4 Response of linear friction-type bolted joint

The general response of a friction type bolted joint, for example as shown in Fig. 4.19(a), is shown in Fig. 4.19(b). If the joint is subjected to an in-plane load passing through the centroid of the group of fasteners, four characteristic loading stages exist, as illustrated. In the first stage, static friction prevents slip; in the second stage, the load exceeds the frictional resistance and the joint slips into bearing when the slack in the slotted hole in the middle plate closes; in the third stage the fasteners and the plates deform elastically and consequently the load-deformation relationship remains linear; in the fourth stage, yielding of plates, fasteners, or both occurs and results in plate fracture or shearing of the fasteners. This clearly presumes that at the third stage there is a whole-surface contact between the bolts and the plates, so that both bolts and plates can remain elastic.

An analytical model describing the response of a friction type bolted joint is given by Fisher (65) and this model is also used here. The model is designed to find the force-displacement hysteretic response of the friction joint at the third and fourth stages as shown with the distinct cycle points in Fig. 4.19(b). For this, simple expressions were used relating the forces on the bolts and on the plates as a function of the existing strain. It is assumed that the slope of the force-displacement curve of the third stage when the bolts and the plates come into bearing is the same as for the first stage when static friction prevents slip.

4.3.5 Response of a single-degree-of-freedom structural model incorporating friction devices

4.3.5.1 Description of the models

The most basic form of the frictional device produces a load-deformation relationship which is essentially the same as the ideal elasto-plastic element. However, it is easily possible to visualise frictional devices of limited travel as described in §4.3.4 above. These cases can be regarded as defining two limiting models; for clarity of description these will be referred to as "elasto-plastic" and "hardening" models of frames incorporating the devices in question.

A typical hardening model is shown in Fig. 4.21. The additional force $F - F_y$ on the hardening model as shown in Fig. 4.22 is, in the case of a linear slipping device, a force exerted on the bolts in bearing. This force, or the ratio F/F_y , is a very useful indication to the designer wishing to guard against failure of bolts, or excessive excursion into the non-linear range of bolts or braces.

4.3.5.2 Excitation of the hysteretic models by an impulsive load

Consider first for simplicity the response of the two hysteretic models to an impulsive load which gives an initial velocity to the system. Assume also for simplicity that the system has no damping. The response is described in Fig. 4.23(a) and Fig. 4.23(b) respectively.

Assuming a yield displacement of unity, we can denote the maximum displacement of the elasto-plastic model as a ductility requirement, μ . This ductility depends upon the magnitude of the impulsive load, and can be regarded in terms of energy, since the kinetic energy imparted by the impulsive load minus the energy dissipated by the model is equal to the strain energy stored in the model. Without any damping in the system, the elasto-plastic model vibrates about the final yield position indefinitely.

Assume that the hardening model becomes elastic at a displacement a , $a < \mu$, reaching a maximum force F as shown in Fig. 4.22. For conservation of energy, the total energy dissipated and the total energy stored by the two models must be equal. By equating the shaded areas in Fig. 4.22 it is easy to show in this case that

$$\frac{F}{F_y} = \sqrt{2\mu - 2a + 1} \quad (4.3)$$

As shown on Fig. 4.23(b) for the hardening model, after reaching the maximum deflection x it returns and yields in the other direction. The hardening model dissipates the same amount of energy as the elasto-plastic model and finally vibrates about the final position indefinitely. It was found by analytical solution of the differential equations that the introduction of damping makes very small difference in equation (4.3). However, as might be expected, as the amount of damping is increased, the energy dissipated by the hardening model is reduced, compared to the energy dissipated by the elasto-plastic model.

4.3.5.3 Response to earthquake excitation

The foregoing observations suggest that the hardening model may have some advantages compared with the elasto-plastic model, in particular by reducing the maximum displacements of the structure. On the contrary, the additional forces F may, in the case of a linear slipping device, cause damage to the bolts or to the braces. A parametric study is carried out here to compare the performance of these two hysteretic models under earthquake excitation.

The earthquake records considered were generally artificially generated records using the second order filter described in § 2.2.2, with effective damping and frequency of $\zeta_g = 0.6$, $\omega_g = 5\pi$ rad/s. The two components (NS and EW) of the 1940 El Centro earthquake were also used.

The first study was based on a single degree of freedom system structure having a frequency of 1.2 Hz and 1% critical damping; the responses were normalised by considering the yield displacement as unity. Ten earthquake records (El Centro and eight randomly-determined artificial records, curtailed as mentioned below) were suitably scaled in order to produce a selected value of ductility on the elasto-plastic model, in particular $\mu=2$ and $\mu=6$. In order to achieve this the response analysis was repeated iteratively for each earthquake record. Since only a small number of iterations was performed, the final value of ductility obtained was close to the desired, but not exactly equal, varying slightly from one earthquake record to the other. Only ten seconds of the earthquakes and the response were used on this example.

The ten earthquakes thus determined were also used to excite the hardening model. For the hardening model it was assumed that it became elastic again at a displacement $a=(\mu+1)/2$, where μ is the ductility reached by the elasto-plastic system. The normalised response of the two systems is presented in Fig. 4.24 for the two target ductilities $\mu=2$ and $\mu=6$. In this figure the response values of the hardening model

are divided by the response values of the elasto-plastic model. As can be seen in this figure the final energy dissipation by the two models is approximately the same. On the contrary the maximum displacements of the hardening model are generally smaller than the displacements of the elasto-plastic model especially for the high ductility $\mu=6$.

The number of crossings of a particular inelastic displacement level (i.e. plastic displacement or the movement of the friction device, as appropriate) has been studied in Figs. 4.25 and 4.26 for the two structures and for the two basic ductilities. As shown in these figures, the elasto-plastic model tends to produce higher displacements by shifting the response in one direction, whereas in the hardening model all the inelastic activity happens in the range $(-a, a)$. In this range the number of contacts between the two points is generally higher than in the elasto-plastic model and also more uniform.

The same system is investigated on Table 4.1 for twenty artificial earthquakes generated using the second order filter. The system was again defined to give frequency of 1.2 Hz, 1% artificial damping and a yield displacement of unity. For this study the yield force of the system is set at one-quarter of the maximum earthquake forces produced by the earthquakes on a rigid structure of the same mass, i.e. the product of the mass of the system and the maximum ground acceleration. This procedure was adopted as a simple method to produce response ductilities of around 4. The total duration of the earthquake records was used in this example. For the hardening model it was again assumed that $a=(\mu+1)/2$ where μ is the ductility reached by the elasto-plastic model.

As shown on this table the energy dissipation by the two models is approximately the same and the maximum displacements of the hardening model are generally smaller than the maximum displacements of the elasto-plastic model. As is also shown on this table, the maximum force F/F_y is generally lower than the prediction given by equation (4.3). This table is especially interesting in emphasising the large scatter of the maximum response values between randomly-generated earthquakes with the same frequency content and the same intensity.

4.4 Final remarks

Different arrangements to enhance the energy absorption of steel braced frame structures have been considered in this chapter. From these possibilities, devices which are based upon friction are the most promising. These devices can completely protect the structural elements from damage, while they can easily be replaced if necessary after a severe earthquake event.

The development of the friction devices is described in detail in the next chapter.

CHAPTER 5

DEVELOPMENT OF FRICTION DEVICES

5.1 Introduction

This chapter first describes extensive experiments on friction in cyclic linear and rotational motion. Experimental work is then described on a small-scale K-braced frame incorporating a friction device. Finally a possible configuration is presented of a full-size rotating device suitable for an X-braced frame.

5.2 Friction tests

5.2.1 General

The experimental work reported here relates to two different friction devices. The first is a slipping device where friction is associated with linear relative motion between the plates, while in the second, rotations of bars forming a small mechanism are subjected to controlled friction couples.

The first tests reported were carried out at the beginning of this project in order to get a broad grasp of the problems intrinsic in the hysteretic response of a friction joint. These were centred on the linear slipping device. The second group of tests was much more laborious, examining many different materials under different applied loading conditions, in order to find the materials most suitable for the present application. These were centred on the rotating device.

5.2.2 Experiments on friction in cyclic linear motion

5.2.2.1 Experimental work on a small-scale slipping device

The device tested is shown in the photograph in fig. 5.1 and the instruments used in the photograph in fig. 5.2. The dimensions of the device are shown in fig. 5.3(a). The load is applied with an Amsler multi-range testing machine for tension and compression, capable of inducing a maximum force of 10t. The device is tested in both directions using the arrangement shown in fig. 5.3(b).

The device consists of two outer plates and a middle plate. All plates are made from bright mild steel. The middle plate, having slotted holes, is able to slip relative to the two outer plates when the load applied by the testing machine reaches the limiting slipping load. The plates are connected with high strength 8.8 grade bolts of 8mm diameter. The diameter of the bolts was reduced to 4.5mm in their part passing through the middle plate and to 6mm in the rest. This decision was taken in order for the device to have a slipping load of about 3t, but had the further advantage of producing a high-grade reduced shank bolt of consistent performance in retightening. The nuts were tightened by turning two flats of the nut from the hand-tightened position; this corresponds to 10 Nm torque applied with the torque spanner (the bolts having a breaking torque of 15 Nm). A rebate in each outer plate keeps the contact area between faying surfaces constant as movement progresses and helps to prevent an increase in the amount of load sustained as the result of wedge action.

The load is first applied in one direction, applying displacement control by the machine. As successive slips take place, the individual maximum and minimum loads form the upper and lower bounds of an envelope, the width of which is a function of the faying surfaces, the machine characteristics, and the loading cycle. After

closing the gap in the slotted hole in one direction, the device is turned over and a suitable packing piece inserted so that the centre plate can be driven back (as shown in fig. 5.3(b), finally completing one cycle. This device was tested only quasi-statically with a maximum loading rate of 100 kg/sec.

Typical load-deformation curves for this device are shown in fig. 5.4. With simple steel-to-steel contact, repeating the testing procedure for successive cycles rapidly damages the mild steel surfaces. As a result of this the upper envelope curve is continuously increasing, exceeding finally the capacity of the machine after the fourth cycle.

An effort was made to find additional materials which can be used as faying surfaces in order to avoid this unsatisfactory performance. Initially, brass plates with thicknesses of 0.5, 1 and 1.5mm were inserted between the mild steel plates. The brass plates have only 6mm holes in order to pass the bolts. Typical results obtained are shown in fig. 5.5. This unsatisfactory hysteretic response is mainly due to the fact that the brass plates suffer damage in bearing on the hole face. Even by introducing slotted holes into the brass plates, the hysteretic response was still not wholly satisfactory, as shown in fig. 5.6.

Introduction of mild steel washers between the slipping surfaces significantly improved the results. The washers move relative to both plates, reducing in this way the relative movement between the washers and the plates and reducing the damage on the plates. Typical results are shown in fig. 5.7 for the case where the internal diameter of the washers is sufficiently large. The washers in this case were W/15/36/2/ (notation explained below) corresponding to contact pressure of 8.5 N/mm^2 .

Since the idea of washers was extensively used in this project either as faying surfaces in a slipping joint or as rotating frictional washers in a rotating joint, a simple notation is used to describe their dimension, i.e. W/I/O/T/, where W stands for a washer, I and O the inner and outer diameter of the washer in millimeters and T its thickness in millimeters.

Changing to mild steel washers with the small internal diameter of 7mm produced the unsatisfactory results shown in fig. 5.8. These are due to the fact that the washers come into bearing with the bolts, restricting in this way the relative motion only in one interface.

The best results were obtained using high-strength washers W/15/36/2/ as faying surfaces. Typical results are shown in fig. 5.9. These washers have all the advantages of the washers mentioned previously and in addition, being from a high strength material, have a very high wear resistance. A small increase in the load happens at the first cycles after which the load stabilises with no further increase up to the 12th cycle tested. The mild steel plates were almost completely undamaged by the end of the test.

It is essential to leave the washers able to move relative to both plates. By fixing the washers on the middle plate, restricting in this way all the movement relative to the two outer plates, the unsatisfactory results shown in fig. 5.10 were obtained.

5.2.2.2 Tests on a large slipping joint

The dimensions of a large slipping joint tested are shown in fig. 5.11. The plates were connected by an M16 HSFG bolt. The washers inserted between the plates as faying material were W/35/80/6/ made from ground flat stock after heat treatment for maximum hardness. More details about this material are given later. Both ground flat stock and bright mild steel plates were sand blasted. The device was again

tested on a very slowly applied load for a displacement of 2 cm in each direction. The variation of the maximum slipping load obtained at every half cycle is shown in fig. 5.12. A coefficient of friction corresponding to this particular slipping load is also shown. The variation of the maximum slipping load by the number of cycles, as shown in this figure, is less consistent than it was in the small slipping device. This is attributed to the substantially higher pressure of 22 N/mm^2 now existing between the plates and the washers. Significant damage was observed after the test on the mild steel plates which is the result of scuffing between the ground flat stock washers and the mild steel plates during the test.

One more device was tested for comparison, without insertion of additional material between the plates. New sand blasted bright mild steel plates were used. The variation of the slipping load by the number of cycles is shown in fig. 5.13. After the first five cycles there is a continuous increase of the slipping load, finally exceeding the capacity of the machine after the tenth cycle.

The conclusion reached from the tests so far is that the hysteretic response of a friction joint can be significantly improved by the insertion of additional materials between the slipping surfaces. High strength steel washers which are left loose between the plates are a possible solution giving very consistent hysteretic response so long as the pressure between the surfaces is kept relatively low.

5.2.3 Experiments on friction in cyclic rotational motion

5.2.3.1 Description of the test rig

The tests reported in §5.2.2 above were carried out at the beginning of this research in order to get some idea of the response of a slipping joint in alternating loading. These tests were carried out quasi-statically and in the early stages no special provision was made for the surface finish.

The tests reported now were carried out on a rotating joint, i.e. an assembly representing one corner of the rotating device introduced in the previous chapter. The assembly comprised the main plates, with intervening washers of selected materials including proprietary friction-lining materials. Many materials were tried for these washers, in order to find the materials giving the most consistent hysteresis loops. Special care was taken for the surface finish in both the plates and the washers. Different velocities and pressures were also used in order to cover a wide range of results for the application of this device in earthquake engineering.

A photograph of the test rig is shown in fig. 5.14, and a photograph of some of the materials tested in fig. 5.15. A sketch of the overall arrangement is also shown in fig. 5.16. The load is applied through a 70 kN hydraulic jack which is connected with a pin joint from one end to a rig frame and from the other end to the specimen. The specimen is a cantilever arm with the frictional washers placed on the 'fixed' end. Both the middle plate and the outer plates were surface ground. After the end of each test the plate surfaces were re-ground or replaced if the damage which occurred during the test was significant.

External non-frictional washers were also used, as shown in fig. 5.16. These washers were 25mm thick and were rebated in order to transfer the pressure from the bolts to the radius overlying the friction material. Without these external washers the bending deformation of the plates under the high externally-applied pressure from the bolt may reduce the effective radius of contact between the plates and the washers and so reduce the slipping load. For the particular plate thickness used (20mm) the use of these washers made only a small difference in the few comparisons made. On the scaled-down device shown, later used on the shake table, the use of external washers became absolutely necessary, however, since this device has very thin

plates which bend easily. Generally the use of the external washers is necessary for reliable results.

The thickness of 25mm of the external washers is entirely sufficient to cause uniform pressure distribution between the frictional washers and the plates in the present tests. This was verified by repeating some of the friction tests reported later with new external washers having 100mm thickness. The results produced by those washers were identical with the results given by the 25mm thickness washers.

In the descriptions that follow the frictional washers are described with the same notation as before, i.e. W/I/O/T/. For many of the tests reported later, the internal diameter of the washer was chosen to equal the diameter of the bolt. When the internal diameter of the frictional washer was bigger than the diameter of the bolt then special provision was made to keep the frictional washer central relative to the bolt, by inserting an appropriate close fitting but thinner washer between the frictional washer and the bolt.

The plates were of bright mild steel.

5.2.3.2 Test procedure and format of results

The jack force was measured by a piezo-electric pressure transducer connected adjacent to the jack, feeding a charge-amplifier. The output was regularly calibrated against a large load cell inserted in place of the friction device. The jack force was thus displayed on an X-Y plotter against the stroke of the jack.

The basic parameters for the various friction tests are summarised on Table 5.1. The first column of this table gives a test number, and the second column shows a reference number identifying the graph which is typical for results of this test, in the form of hysteresis loops of force versus the stroke in the jack. The next column of Table 5.1 shows

the material from which the washers were made, assumed to be tested against mild steel plates unless it is otherwise specified on this column. The washer material and the normal applied pressure is also shown on each graph.

The next column shows the effective coefficient of friction which is inferred as follows:

$$M = P \cdot \ell = \frac{2}{3} \pi \mu m (r_1^3 - r_2^3) p_r$$

where P = the applied load on the jack

ℓ = 20 cm, the cantilever length

M = the couple acting on the friction joint

$r_1 = D_o/2$

$r_2 = D_i/2$

$m = 2$, the number of faying surfaces

p_r = the nominal pressure between the plates and the washers

μ = the coefficient of friction

This formula assumes that the applied couple M is taken by friction whereas the force P is taken by the bolt in bearing with the plates. In test cases where the hysteresis loops were not stable over the number of cycles, a range of μ values is given, corresponding to the minimum and maximum values of the slipping load.

The next column shows the total angle of rotation of the outer plates relative to the middle plate, ϕ . Next is shown the frequency of the test or range of frequencies tested in the given combination. New specimens were always used for each frequency. The applied cycle has the time history shown in fig. 5.17, so the frequency shown on Table 5.1 is $f=1/T$ and the angular velocity $2\phi f$. Quasi-static response refers to a very low rate of oil delivery ($f < 0.25$ Hz).

The next column of Table 5.1 indicates the nominal applied pressure between the friction materials and the plates for the particular test. The next column shows the bolts used, as detailed on Table 5.2, together with the torques applied. The selected torque was generally such as to develop in the bolt 70% of the tension yield load. A reduced torque was also used for the M24 10.9 grade bolt for convenience in some tests. The torques were applied and measured with a torque spanner; the calibration was checked.

The next column shows the number of cycles for which the test was carried out and the next column the surface finish for the washers. The parent plates were mild steel with ground surface finish. The final column of Table 5.1 shows the dimensions of the washers for the particular test.

A brief summary of the chemical composition of the materials tested is shown on Table 5.3 and the hardness of each material on Table 5.4. The hardness was measured with the Hardness Tester DHV 10 with an estimated maximum possible error in the results of 5%.

A short description of each of the materials tested is given in the general discussion of the specific test results, §5.2.3.4.

5.2.3.3 Selection of the load and motion parameters.

The selection of the excitation parameters is based upon the performance of the 10-storey braced shear frame described in §1.3.3.2. When this frame is subjected to 3 artificial earthquakes generated using the autoregressive filter described in § 2.2.3 with intensity of $S_0 = 50 \text{ cm}^2/\text{sec}^3$ (which is the same as the intensity of the 1940 NS El Centro earthquake), the maximum recorded interstorey velocity in undamped response is 15 cm/sec. This value appears at one particular storey for one particular earthquake, while the other values are smaller. The values of maximum interstorey velocities are also reduced if damping is introduced.

The corresponding maximum rotation at each node of the device envisaged is $\phi=15^{\circ}$, and this was set as the target value for testing. Difficulty was experienced in setting the jack stroke, however, and for this reason some values of ϕ departed from the target, as shown in table 5.1

The maximum relative velocity between faces of the rotating device was similarly related to the inter-storey velocities discussed above. For most tests, however, a slower rate was used, for practical convenience.

In general, the tests were taken to 20 cycles, which is regarded as a conservative representation of the cycle count expected in response to a real earthquake, bearing in mind that the real response will comprise cycles of motion less than the maximum. It would appear on first consideration of the problem that rate of heating would be a most important parameter. This in turn would focus attention on the slipping velocity, although, as indicated later, the performance in practice does not appear unduly sensitive to this factor.

Analytical predictions were made for the expected energy dissipation by the hysteretic devices using either the probabilistic theory as described in section 2.3 or by time domain solutions as shown in Appendix B. These predictions suggest rate of temperature rise in the friction materials during earthquakes with the same intensity as the El Centro of 5° C/sec. This value is found by using the specific heat coefficient for steel and assuming that half of the total volume of the washers are heated, contributing to the energy dissipation. This value occurs only in the stronger part of the record, being smaller when the earthquake loses its intensity. This small rate of temperature increase in the friction material may explain why the velocity does not significantly affect the results.

5.2.3.4 Cyclic moment/rotation results

The main objective of the programme described above was to identify materials having good hysteretic response over the defined range of excitation parameters. Good hysteretic response is considered to be the response where the slipping load is predictable and remains unchanged over the required number of cycles.

One problem in the friction tests is wear, i.e. the loss of material from the surface by transfer to another surface, or the creation of wear debris, and a review of this problem is given by Colangelo (66). According to the present tests, the wear seems to be affected mainly by the following parameters:

- a) The most important parameter seems to be the hardness of the materials, the greater the hardness, the less the wear and the better the consistency in the hysteresis loops.
- b) The normal pressure, the wear is higher for higher pressures.
- c) The slip travel, the bigger this distance, the bigger the wear.

Point (c) was also clearly verified in the tests by observing greater wear damage in the washers at the points farther away from the centre of the washer. The damage was also oriented in a circumferential manner.

From the experience drawn from these tests it was concluded that for small "rotations" or "slips" (say 5° rotations in a rotating joint or 5mm slip in a slipping joint) there is very good consistency. This was due to the polish of the contact surfaces by the number of cycles.

The wear is also correlated with the total cumulative slip (or alternatively the "energy dissipation") but to a lesser degree than to the range of the individual slip cycles. As noted earlier, for the present value of velocities, slip velocity seems to be a relatively unimportant parameter.

The response of each material is affected by all these parameters. Materials suitable for a certain range of those parameters may become unsatisfactory for another range of values. Exact identification of the "interaction surface" above which wear starts, clearly requires a large amount of experimental work, which is outside the scope of the present study. Each of the materials discussed below was tested on a moderate range of parameters, in an attempt to establish in an economical way the materials and the conditions most suitable for the present application.

Ground flat stock

The ground flat stock (alternatively referred to as "gauge plate") was the first material tested, proving finally to be the best from the metallic materials tested. For hardening it was heated to 780°C and quenched in oil, which was conveniently placed to avoid loss of heat. For tempering it was heated for one hour at 150°C .

The first three tests, namely G1, G2, G3, were carried out with sand-blasted plates and washers. The sand-blasted surface finish was generally found to give higher coefficients of friction than the basic ground surface finish. However, this finish was generally found to be unsatisfactory, because the damage commonly concentrated at limited areas, while many parts between the plates and the washers were not in contact. The coefficient of friction of the sand-blasted materials seems also to be easily affected by touching the prepared faces. Tests carried out a few months after the original tests, using sand-blasted materials, show a different coefficient of friction, although the original results can approximately be restored by sand-blasting the materials again. For these reasons, it was concluded that sand-blasting the materials is not sufficient for reliable coefficients of friction.

Test G1 was the only one of the tests reported here which was carried out without the use of the external washers; this seems to reduce slightly the coefficient of friction as compared with test G2. The unsatisfactory hysteresis loops of test G3 were due to the high contact pressure between the plates and the washers.

Contrary to the sand-blasted finish, the ground surface was found to give very reliable results; the comparable tests are G4, G5 and G6. These results show that a contact pressure of 20 N/mm^2 is the limiting pressure for the ground flat stock after which significant wear occurs. For smaller pressures the ground flat stock gives very satisfactory results even for high strokes. For higher pressures the graphs are generally inconsistent, being more inconsistent for the higher strokes, as shown by comparing tests G4 and G5.

In all the results reported thus far, the ground flat stock washers were left free between the plates to rotate in their own preferred direction. In the tests G7, G8 and G9, ground flat stock washers were welded on the outside plates, imposing in this way all the movement on the other interface. The initial thought in doing this is that restricting the relative motion to one interface effectively doubles the relative movement at this face. The results obtained by doing this were unfavourable to a degree that cannot be explained on the basis of doubling the relative motion. Typical results for quasi-static applied loading are shown in tests G7, and G8. A further development was to weld washers of ground flat stock to all the meeting faces, so that movement occurred between two faces of the harder washer material. The results, shown as test G9, are still unsatisfactory.

A simple explanation of this effect can be put forward. The relative rotations between the two washers of test G5 and one of their attached plate were shown in fig. 5.19. Initially, for a few cycles, the relative movement concentrates on one interface, until this face damages. The relative motion is then concentrated at the other inter-

face until this interface damages as well. At this stage the second face siezes at one extreme of the relative motion and an additional rotation is enforced on the first face, so that a new relative contact position exists between plates and washers and the foregoing process is repeated. This additional rotation of the washer is very significant for materials such as the ground flat stock which are susceptible to damage, and leaving the washer free to rotate is very important in proving the hysteretic response of the joint. In all subsequent tests the washers were left free to rotate unless otherwise specified.

Although for high pressures the ground flat stock siezes up and the couple increases successively, the rate of increase decreases and becomes much smaller after a substantial number of cycles than in the first few cycles. It would appear that there is an upper bound to the couple in the case of this material.

Ground flat stock is clearly an acceptable material for the present application, although it does not give the excellent hysteresis loops which can be obtained by using proprietary purpose-formulated friction materials as described later.

Cast Iron

The grade 14 cast iron, which had a yield stress of 220 N/mm^2 , showed a very bad hysteretic response. Typical results for the cast iron washers against mild steel plates are shown in test C1 and against ground flat stock in test C2. In the later case the cast iron washers were inserted between ground flat stock washers welded on the mild steel plates.

The response of this particular cast iron was in some subsequent tests found to be more favourable when used in conjunction with stainless steel and with some of the friction materials reported later. Details of these tests are not shown since the final arrangements are complicated and are not regarded as practically applicable. Higher grade cast irons or alloy cast irons might prove more suitable, but this line of development was not pursued.

Nitroy 40B

Nitroy 40B alloy steel was the next material tested, surface-hardened using the nitriding process. The hysteretic response of this material, tested as it emerged from the nitriding process without regrinding the surface, is shown in tests N1 and N2 versus mild steel plates and ground flat stock respectively.

The very bad hysteretic response may be attributed to the nitriding process; in this process the nitrogen dissolves in iron only up to a certain concentration. When the nitrogen exceeds this value a surface layer called the "white layer" is formed. Dr. T.J.Baker, of the Department of Metallurgy at Imperial College, has suggested that this layer might be responsible for the bad response, and test N3 shows the behaviour after surface grinding of the Nitroy 40B which is presumed to have removed the "white layer". It can be seen that the hysteretic behaviour is consistent. Nevertheless, this material must be considered unsuitable for the present application, taking into account the cost of the material and of the nitriding process and the final uncertainty concerning its hysteretic response.

500K-Monel

The response of 500K-Monel stainless steel is shown in tests M1 and M2. As shown in test M1, the hysteretic response of 500K-Monel is very promising for relatively high pressure and high stroke at low speeds, but is less consistent for higher speeds, as shown in test M2.

This stainless steel was not hardened. The hysteretic response of this material would probably be improved by hardening, but this possibility was not examined.

3501F Ferodo friction material

The five materials reported in the rest of this section are proprietary fibre-based friction materials selected from the range manufactured by the Ferodo company (67) in collaboration with the

company. All the Ferodo materials tested were asbestos free.

The first of these materials to be studied was Ferodo friction material type 3501F; because of the small quantity of the material available, only one washer was prepared while in the other interface a washer from ground flat stock with the same dimensions was used. Results are presented as test F1.

Despite the relative consistency of the graph for this particular pressure and stroke, this material showed unsatisfactory wear resistance and was severely damaged after the test.

Type FF Ferodo friction material

The hysteretic response of this material is shown in tests F2 and F3. In test F2 the couple falls in successive cycles. This is due to the fact that this friction material is very soft, resulting in high compressibility.

The hysteretic behaviour of this material was significantly improved in test F3. Here four ground flat stock washers W/50/115/6/ were welded on the plates and between them were inserted pieces of the friction material 150mm square, 9mm thick. The larger area of friction material used in this test with significantly smaller normal stress improves the hysteretic response.

Nevertheless, this friction material is unsatisfactory for the present application, given the problems noted.

An obsolete-stock Ferodo friction material

The best hysteresis loops were obtained using the friction material shown in the photograph in fig. 5.15. This was obtained from the stock held by a structural engineer and could not be equated with any current-catalogue material. The fabric in this material is formed in a circumferential manner. Typical results are shown in test F4. This material was tested faying with various other surfaces,

showing very stable hysteresis loops; even when the washers were bonded to the outside plates and tested against mild steel, the procedure which produced quite unsatisfactory results with metal-to-metal faces, the hysteresis loops were still very stable.

Exact identification of this material with the present available Ferodo friction materials was impossible. Two materials which are believed to be very close are next considered.

3701F Ferodo friction material

This was the best of the materials tested from the current catalogue of Ferodo friction materials. Typical hysteretic response is shown in tests F5 and F6. The friction material as well as the plates were completely undamaged by as many as 50 cycles of motion. A summary of the significant physical properties of the 3701F material is shown on table 5.5.

Test F6 was repeated by machining the friction material in order to reduce its thickness. It was found that as long as the thickness exceeded 6mm, the hysteresis loops were identical without any damage to the materials, whereas for a thickness of 5mm the friction material crushes under this particular stress.

Friction tests using the 3701F Ferodo friction material against mild steel plates with ground surface finish show very good repeatability. The friction tests F6 were repeated a few months after the original tests, showing identical results.

3904F Ferodo friction material

This material was less satisfactory than the 3701F. In test F7 the load fell in successive cycles, due to the crushing of the 3904F friction material; for the same thickness the crushing of the 3701F was not so pronounced. It lacks in comparison with the 3701F because it is more flexible and has lower ultimate strength properties.

5.2.3.5 Creep tests

Creep tests were carried out on the 3701F Ferodo friction material using the arrangement shown in fig. 5.20. Three washers W/30/85/10/ having total thickness of 30mm were inserted between two rigid steel blocks and compressed using an M24 grade 10.9 bolt with total grip length of 160mm. A torque of 1200 Nm was applied to the bolt, introducing a tension of 290 kN, which is about 70% of the tensile yield load on the bolt. Two dial gauges were used to measure the creep on the friction material, as shown in fig. 5.20. Two strain gauges of 120 ohms resistance each were connected on opposite sides of the bolt to compensate for bending, producing a total active resistance of 240 ohms. Two extra dummy strain gauges were used on a second unstrained bolt for thermal compensation, connected to form a quarter bridge circuit with two fixed arms of 120 ohms each, provided by an electronic test instrument. The experiment was carried out in a temperature-controlled room.

The strain gauge readings for the first test are shown in fig.5.21, and the dial gauge readings in fig.5.22, on a linear-logarithmic scale. The strain gauge reading is converted to a load on the bolt, assuming a Young's modulus for the bolt of 210 kN/mm^2 . As shown in these figures, the decay of the force on the bolt as well as the dial gauge readings are approximately logarithmic with time. The creep was very significant on this first test and approximately 1/3 of the load on the bolt was lost after two weeks.

The test was extended by re-tightening the bolt at the end of the first two weeks. The decay of the readings as shown, denoted "test two", is now much smaller. After further re-tightening at the end of the second two weeks, the decay in the readings is almost negligible as shown, denoted "test three".

It was found experimentally that for the present dimensions the stiffness of the friction material is approximately 40% more than the stiffness of the bolt. This means that these tests were carried out neither under constant stress nor under constant strain conditions, and in consequence the present results cannot directly be used to predict for other conditions. The insignificant creep after the second re-tightening is, however, strong evidence that for any practical dimensions, creep in service can be eliminated by repeating this procedure.

5.2.4 Development of a rotating device for braced frames

5.2.4.1 Tests on a large-scale model rotating device

A large rotating device with the dimensions shown in fig. 5.23 was tested using the arrangement shown in this figure. The device was pin-connected with the testing machine, with 25mm pins on both ends. Having tested the device by compression across one diagonal, constituting half a cycle, it is turned 90° and tested in the other direction to complete the cycle.

Tests were carried out quasi-statically with the same conditions as the GI tests reported earlier for the single rotating joint, i.e. both plates and ground flat stock washers were sand-blasted. Typical results for the first and the fifth cycle are shown in fig. 5.24. The slipping load in the early cycles correlates reasonably well with the loads from the tests carried out at the single rotating joint. In later cycles the load increases more than in the tests on the single rotating joint. This may be due to the fact that the present tests were carried out at higher stroke.

It should be noted that when this device activates, the transition from the elastic range is not so sharp as that with the single rotating joint reported before, because not all the four friction joints on this device activate at the same time. This happens since without grinding the parent plates and the washers, the contact area between the washers and plates varies from one friction joint to the other.

5.2.4.2 Tests on a small-scale model rotating device

A model device which was prepared for use on the shake table is shown in the photograph in fig. 5.25 and its dimensions in fig. 5.26. External washers, not shown in this photograph, were also fitted. For completeness, table 5.6 summarises friction tests associated with this small device.

Although plates and ground flat stock washers were only sand-blasted, the hysteresis loops obtained in these tests are very consistent, similar to those given by the 3701F friction material in the single joint tests. This consistency is probably assisted by the fact that for the very small washers used in this device the slipping movement is small even for high rotations.

The 3904F Ferodo friction material which gave unsatisfactory response in tests F7 gives also very consistent hysteretic response on the small slipping device. This is again due to the fact that the relative movement between washers and plates is now smaller than the pressure in the F7 tests.

A wide range of slipping loads can be obtained using this device by varying the amount of torque applied to the bolts or by slightly varying the washer dimensions.

Other configurations, except the basic rectangular configuration used so far, are also possible. Consider for example the brace damper shown in fig. 5.27. This mechanism was developed by Scholl(68) and the device incorporates a visco-elastic material between the plates. It is possible using a similar configuration to produce a friction device with very large frictional washers.

5.2.5 Further examination of proprietary friction-lining materials

5.2.5.1 Detailed tests on the 3701F Ferodo friction-lining material

A slipping test was carried out on a single slipping joint using the 3701F Ferodo friction material. The dimensions of the plates and the Ferodo friction material which is used as faying surface are shown in fig. 5.28. The tests were carried out quasi-statically for a slip of 2cm in each direction. The plates were bright mild steel, surface ground, connected with an M16 10.9 grade bolt. External washers were

also introduced for more uniform pressure distribution.

The variation of the maximum slipping load obtained at each half cycle is shown in fig. 5.29, together with the equivalent coefficient of friction. Typical results for the first three cycles are shown in fig.

5.30.

The direction of grinding does not affect the coefficient of friction. Tests repeated with the two outside plates turned so that slip now occurs perpendicular to the direction of grinding, give unchanged results.

The slipping load was generally consistent over the number of cycles tested, although not so consistent as the corresponding rotating tests, despite the fact that the pressure between the plates and the friction material is now very small (only 5 N/mm^2 , assuming uniform pressure distribution throughout the area of the friction material). Plates and friction material were almost completely undamaged by the end of the test.

It was generally observed that the rotating tests were more consistent than the linear slipping tests for the same friction material. This is probably mainly due to the fact that the relative movement on the washers is small, even for relatively high rotations.

5.2.5.2 General comments concerning the coefficient of friction

Individual rotating or slipping friction tests using the 3701F Ferodo friction material show very good consistency. However, the apparent coefficient of friction varies according to application, as shown by comparing the linear slipping tests described in the foregoing section with the rotating tests F5 and F6.

This is attributed mainly to the variation of the coefficient of friction according to the normal applied pressure. An indication that the coefficient of friction may vary as a function of the normal pressure is given by the Ferodo manual, but only for small applied pressures.

The present tests suggest that for the very high pressures desired for the rotating devices there is a significant reduction of the coefficient. Specifically, it was found from the linear slipping tests, where the contact pressure was small, that the coefficient of friction is around 0.25; whereas in the rotating tests, which have much higher contact pressure, the coefficient of friction can be as small as 0.14.

In order to verify this point, a few additional tests were carried out for both linear and rotational motion. First some linear motion tests were carried out using the arrangement described in the previous section, but with faying surfaces of 3701F Ferodo friction washers with the same dimensions as in the F6 tests. Plates and washers were screwed together using an M24 10.9 grade bolt, producing a linear slipping joint with the same normal pressure as in the F6 rotating tests. The coefficient of friction found from these tests is very close to the value of 0.14 found from the rotating tests F6.

Finally a few more rotating tests were carried out. The tests were similar to the rotating tests F6, but the M24 10.9 grade bolt was replaced by an M16 10.9 grade bolt. The joint now had a normal pressure of 25 N/mm^2 . The coefficient of friction found from this test was $\mu=0.18$ compared with $\mu=0.14$ at the higher stress given by the M24 10.9 grade bolt. These results clearly verify that for this friction material the coefficient of friction reduces with higher normal pressure.

5.3 Tests on a small-scale K-braced frame with a slipping device

A further test was carried out on the K-braced frame, with a slipping device at the intersection of the two angles. The frame is shown in fig. 5.31 and also in the photograph in fig. 5.32. The slipping device used in this frame is the same as the small-scale slipping device described in §5.2.2.1.

The two braces are welded to the middle plate of the device (which has the slotted hole), whereas the two outer plates are welded to the beam. The welding of the two outer plates was done with great care to minimise distortions. The two braces were aligned to intersect at the middle of the device; eccentricity could cause a reduction of the slipping load of the device, as shown in fig. 5.33. Applying the equilibrium equations to the fasteners shown in fig. 5.33, we can find the instantaneous centre of rotation and then the reduction of the slipping load as a function of the eccentricity e . The resulting reduction for this device for its basic slipping load of $3t$ is shown in fig. 5.34.

The hysteresis loops for the first 7-cycles in the frame with the device are shown in fig. 5.35. The slipping load at the device was set at $3t$. The vertical displacement in the middle of the frame remains zero throughout the response. Five more cycles, but now with a smaller slipping load are shown in fig. 5.36, the slipping load being reduced by reducing the amount of torque applied to the bolts using the torque spanner. Extensive testing of this frame for different slipping loads extending up to 30 cycles of deformation gave very consistent results.

5.4 Design of a full size rotating device

5.4.1 General

A possible configuration of a rotating device for use in a realistic X-braced frame is now developed. The device incorporates the idea of frictional washers and the design is based upon the tests reported in the earlier sections of this chapter.

5.4.2 Desirable slipping loads and expected deformations in a device

The design loads of a multi-storey frame depend upon many factors, such as the importance of the building, the seismicity of the area, the soil-structure amplification factor, etc. To establish the desirable values of slipping loads for a device, a multi-storey frame is designed

using realistic values of the various design parameters. The building analysed has ten storeys, with a storey height of 3m and plan dimensions 24mx12m. Bracing members are introduced only in the end frames of the building. The earthquake is assumed to act in the direction of the 12m dimension. The bracing configuration used is shown in fig. 5.37.

Since an identical frame exists at the other end of the building, each frame is assumed to carry dead load and live load from a length of 12m, being half of the total length of the building. The structure was designed for the following loads:

Dead load of roof	3.2 kN/m ²
Dead load of floors	5 kN/m ²
Live load of roof	1 kN/m ²
Live load of floor	2.4 kN/m ²
Wind loading	1.5 kN/m ²

The mass subjected to the earthquake ground motion was presumed to correspond to the specified live load. The seismic design was based on the formulation discussed in §1.1.2. The soil interaction factor (S) was taken as 1.5 which corresponds to a period of soil of 1 second. The importance factor of this building (I) and the seismicity of the area factor (Z) were taken as unity. The system ductility factor was taken as $K=0.67$. These values are used to evaluate the base shear which is then distributed in the various storeys. Part of the shear in each storey is taken by the columns and part by the braces.

The bracing elements were designed as tensile members with negligible compressive strength. The fundamental period of the structure was estimated at slightly more than 1 second. Details from the selection of cross-sections for beam and columns are not shown. According to this design, the device at the bottom braces requires to have a slipping load of 600 kN, assuming that it is activated only by the tensile braces. For this slipping load, the

device remains elastic for wind loading and for the design seismic forces, while it is expected to activate for a moderate and a strong earthquake. For the higher storeys a minimum area for braces (2.88 in^2) was presumed and on these storeys the slipping load on the devices was set at 400kN.

From the design of this building as well as from the design of other building frames reported by other authors, it was concluded that any developed mechanism must be able to cover slipping loads in the range from 300kN to 700kN.

The design philosophy of multi-storey braced steel frames requires a moderate earthquake to be limited to non-structural components. For a major earthquake the necessity is to avoid collapse while some structural damage is allowed. The mechanism now developed imposes some limitations since it only permits a certain amount of rotation, corresponding to the limiting case when the edges of the rotating plates come together. The mechanism must be able to deform for the expected deflections in a strong earthquake before it reaches this limiting rotational position. As maximum inter-storey deflection, a value of 80mm (slightly over 3 inches) was chosen, which is considered to be representative of practical building response to very strong earthquakes, for example 1.5 times El Centro. This choice was based on dynamic analyses carried out by other authors, as for example by Goel and Hanson (8), who reported the response to specific ground motions of braced frame multi-storey buildings designed in a closely similar way. Similar dynamic analyses are also reported in this thesis in appendix B.

5.4.3 A typical full size device

5.4.3.1 Dimensions of the device

A possible mechanism for use in an X-braced frame is shown in fig.5.38. This device is basically a combination of a number of the basic rectangular rotational devices described previously. From the 16 intersections, 12 are friction joints, whereas the 4 joints in the

corners are pin joints for attachment to the braces. The advantage of this device over the simple rectangular device is that it makes the connections between the device and the braces relatively simple.

Each friction joint has six frictional washers W/30/85/6/ (for the notation describing the washers see section 5.2.2) from 3701F Ferodo friction material. The plates and the washers are connected using an M24 10.9 grade bolt. External washers of 25mm thickness are also used to ensure uniform pressure distribution between frictional washers and plates. The pressure between washers and plates is thus the same as in the friction tests F6 reported in the previous chapter, so the respective results regarding the coefficient of friction can be used. Concerning creep, the total thickness of the friction material in each friction joint is 36mm, which is very close to the 30mm thickness of the friction material in the creep tests reported in section 5.2.3.5, and the procedure suggested to eliminate creep can confidently be used in this application.

The virtual work equation and the results of the friction tests F6 reported in the previous chapter predict the slipping load of this device to be 400kN when loaded as shown in fig.5.39(a).

The design of the plates, the bolts and the pins follows the same lines as the design of any structural component in a building frame. The bending moment diagram for the plates is shown in fig.5.39(b) making the conservative assumption that the storey shear acts solely through the tension bracing, as shown in fig.5.39(a). In this diagram M stands for the total slipping bending moment developed at each joint as a result of friction in all six interfaces, which in this case is $M=7000 \text{ Nm}$ using the results from the F6 friction tests.

As shown in this figure, the largest bending moments on the device occur close to the points of application of the load by the tensile braces; the moments are zero close to the connection points of the device with the compressive braces. When the device is activated by both the tensile and the compressive braces, the bending moment diagram becomes anti-symmetric with a maximum bending moment

of M (instead of the value $2M$ shown in fig.5.39(b)). Even in the case when the device activates primarily by the tension bracing, the compressive bracing will always have a small buckling resistance, so the actual bending moment diagram on the device will always be smaller than the one shown in fig.5.39(b).

Each of the plies of this multi-layer device is subjected to an appropriate fraction of the bending moment shown in fig.5.39(b). In particular the portion taken by all the inner plies is twice as much as that taken by the outer plies.

Assuming that this device is accommodated in a square X-braced panel, table 5.7 shows two typical changes of the geometry of the device. The first case in table 5.7 shows the limiting rotation that the device can accept which corresponds to the case when the rotating plates meet each other. The second case in table 5.7 shows the deformation of the device for the design safe-limit deflection of 80mm. It can be seen on this table that the maximum deformation that this device can accept is much bigger than the safe-limit deflection of 80mm.

The device as drawn, has a slipping load of 400 kN. A slipping load of 600 kN, such as might be required for the bottom storey of the multi-storey frame discussed above, can be obtained by increasing the number of plies.

5.4.3.2 Remarks concerning hardening of the device

In the design of these devices, two different approaches can be followed. In the first approach the requirement is to avoid brace buckling. In this case the design of the device is based upon the buckling resistance of the braces and both the compressive and the tensile brace activate the device. The second approach can be followed when the buckling resistance of the braces is very small and in this case the design of the device is based upon the tensile strength of the braces, as shown in fig.5.39(a). In particular, the device is

expected to activate at a load which is smaller than the yield strength of the braces.

In the foregoing discussions it has generally been assumed that once the device activates, the forces on the device and on the braces are constant. There are two forms of non-linearities which can cause an increase in the forces in the braces and correspondingly increase the forces transmitted to the device.

The deformation of a single storey braced frame incorporating the rotating device is shown in fig. 5.40. The first non-linearity arises from the kinematics of the device, which causes the reduction in the length of the one diagonal of the device to be greater than the increase in the length of the other diagonal. This effect is important when both braces activate the device, i.e. when the design is based upon the buckling resistance of the braces. This effect causes a positive increase in the force in both braces. The net change in brace force on the compressive diagonal is thus reduced.

The second non-linearity is an increase of the slipping load of the device because of the change of the geometry of the device. The new slipping load can be found by writing the virtual work equation on the device in its deformed position. For the maximum expected storey deflection of 80mm, of which (say) 10mm is elastic storey deflection before the device activates, while the remaining 70mm corresponds to slipping deformation of the device, the slipping load of the device shown in fig. 5.38 increases by 25%. The net result is a significant hardening characteristic, even prior to closure of the gaps between the bars.

Allowance was made for these non-linearities in the design of the device. In particular for the maximum expected storey deflection of 80mm the brace yielding force is still greater than the new slipping

load of the attached device. The device at each storey is also designed to remain elastic for the code design lateral forces.

This example was, however, only created to show typical dimensions for a full-size device, the expected deformations, and the possible hardening of the device resulting from the change of its geometry. As shown later, the slipping load of the device at the various storeys should be suitably chosen to cause reasonably uniform ductility requirements at the various storeys.

CHAPTER 6

SHAKE TABLE TESTING

6.1 Introduction

This chapter describes the shake table testing of a single-degree-of-freedom X-braced frame. The major part of the experimental work concerns the introduction of the rotating device at the intersection of the two cross-braces. The hysteretic response of the same frame with a plastic-ending device is also described, as well as the comparable behaviour without any hysteretic device. The behaviour of an unbraced frame was also briefly examined.

6.2 The Imperial College shake table

The shake table at Imperial College is a biaxial table with one axis of horizontal and one of vertical excitation capable of simultaneous independent movement. The table size is 2mx2m and it is capable of supporting a 4 tonne payload while imparting peak accelerations of 1g horizontally and 0.47g vertically at frequencies between 1 and 40 Hz. These accelerations may be increased or decreased according to the total mass in motion. Only the horizontal actuator was used in the present investigation. Details of this shake table are given in ref (69).

The line of horizontal excitation of the table is defined as the longitudinal axis, and roll is the rotation about this axis. Pitch is the rotation about the horizontal axis perpendicular to the direction of excitation. Typically the natural frequencies of the predominantly pitch and roll modes are approximately 16 Hz and 8 Hz respectively for the unloaded table.

The table and the electro-hydraulic actuators are carried on a 140t reaction mass in the form of a concrete tank. This is located on 24 coil springs which react against the foundations of the building. The springs minimise the dynamic forces transmitted to the building and provide an acceptable degree of seismic isolation. The frequencies associated with the movement of the tank lie between 1.1-1.7 Hz.

When the platform of the shake table is shaken only by the horizontal actuator, small pitch and roll rotations as well as small lateral and vertical accelerations are also experienced. These interact, in particular, with the concrete tank movement which has frequencies within the seismic bandwidth. It is sometimes necessary to filter acquired data to eliminate these effects.

A further source of distortion of the pre-determined excitation is the oil-column resonance of the electro-hydraulic actuator, which mainly affects the frequencies around 20 Hz.

The interactions discussed above commonly have the effect on first trial operation that the acceleration appearing on the platform varies substantially from the target acceleration introduced on the computer. This effect may be minimised by using an iterative procedure, i.e. continuously filtering the acceleration on the computer until it finally matches the acceleration appearing on the platform, a procedure called "time history matching".

6.3 Description of the test model

The dimensions of the test model are shown in fig. 6.1. It is also shown by photograph in fig. 6.3. This frame represents approximately a 1/3 scale model of the first storey of a multi-storey building frame. A value of 850kg was chosen as the mass on the top of the beam. A larger mass would be possible but could introduce significant supporting problems and greater error due to table pitch.

The frame was mounted at the longitudinal centre line of the table to restrict the motion ideally to this direction. With the present dimensions the frame spans almost from one end of the shake table to the other.

The photograph, fig. 6.3, also shows two braced towers, which are provided solely as safety restraints. The main mass is linked to these frames by light, flexible slings so that it would be safely supported in the event of accidental failure of the test frame.

6mm solid round bars were chosen for the bracings in the test frame, these being probably the minimum diameter for which the simple threaded end connection is suitable. This material was cold-rolled mild steel with the monotonic force-displacement curve shown in fig. 6.2. Four bars were used in each frame, as shown in fig.6.1. They were threaded at both ends; at one end they were connected to the energy-dissipation device through pin connections. At the other end they passed through holes in corner-braces of the frame, located by nuts to facilitate adjustment, including possible pre-loading. This detail provides some rotational restraint.

The forces on the bars were measured with strain-gauges fitted at the quarter point of the length of the two bars below the device; at this position the bar remains elastic. Two strain gauges were attached opposite each other in order to compensate for bending. For the bars above the device, it was assumed that the force is in each case the same as the force in the lower bar on the same diagonal.

For the frame beam and columns, RHS sections 80x40x4 were chosen, oriented with their weak bending direction in the plane of shaking. The choice of this cross-section was made in order to have a reasonable ratio between stiffness and strength provided by the frame and by the braces. The two columns are rigidly connected to the top mass, approximating in this way the model of a shear building frame. The beam is also rigidly connected with the top mass, as shown in fig. 6.1; there was thus no plastic deformation of the beam throughout the experiments.

Two displacement transducers were used, one measuring the displacement of the mass relative to the platform of the shake table and the second on the device. The transducer on the device, as shown in fig. 6.1, gives the relative movements, and in particular the transducer reading is equal to half of the change of length of the one diagonal of the device. The strain gauge readings, together with the readings of this transducer, permit full identification of the slip movements of the device in the dynamic response.

Eight accelerometers were also used measuring various horizontal out-of-plane and vertical accelerations. The accelerometer outputs were filtered using either electrical or digital filters to remove any components above about 30 Hz.

6.4 Generation of synthetic time-history input functions

The measured in-plane natural frequency of this braced frame was 7.6 Hz. It was decided to excite the frame with artificially generated earthquakes, designed to approximate the response of the frame to the first mode response of a multi-storey braced frame.

The artificial earthquakes on the shake table were generated using the second order filter procedure. A duration of 10 sec was used for each earthquake record and an effective frequency of $\omega_g = 15\pi$ rad/sec, i.e. duration and frequency were set three times lower and three times higher respectively than the values reported in section 2.2.2 for real earthquakes. It will be noted that this leads to spectral peak which is very close to the natural frequency of the frame ($2\pi f_1 = 2\pi \times 7.6 = 47.75$ rad/sec). The effective damping of the filter was again set at $\zeta_g = 0.6$. The selected normalised variance intensity function, $\phi(t)$ was:

$$\begin{aligned}\phi(t) &= 0.5t && \text{for } t < 2 \text{ sec} \\ \phi(t) &= 1 && \text{for } 2 < t < 3.83 \text{ sec} \\ \phi(t) &= e^{-0.456(t-3.83)} && \text{for } t > 3.83 \text{ sec}\end{aligned}$$

This is consistent with the full-scale intensity function discussed in section 2.2.2, given the timescale of 3:1, but modified by an initial ramp function to meet the practical requirements of the shake table control and actuation. A time interval of $\Delta t = 0.01$ sec was used.

The power spectrum of the artificial earthquake thus generated is shown in fig. 6.4. A problem was experienced at this stage since the displacement time histories included major components of very low

frequency. This problem was circumvented by passing the acceleration records through one more filter to eliminate the low frequency components producing the filtered acceleration power spectrum shown in fig. 6.4. This approach has the further advantage of eliminating possible interference with the concrete tank resonances.

Ten basic artificial earthquake records were generated with an intensity of $S_0 = 900 \text{ cm}^2/\text{sec}^3$ (intensity S_0 is explained in fig. 6.4). These records, having a peak acceleration of about 5g, corresponded approximately to the maximum possible performance of the unloaded shake table. A reduction factor was always applied in the subsequent tests. A simple notation was used to describe the produced artificial earthquake, i.e. E/a/b/, where the letter E stands for earthquake, the a varies from 1 to 10 according to the earthquake number and the letter b introduces the reduction factor as a percentage of the basic ground motion.

Clearly the artificial earthquakes thus produced are subject to a small modification because of the dynamic characteristics of the shake table. The differences between the target earthquake and the earthquake appearing on the platform proved to be very small in the frequency range of structural significance, as discussed in the following paragraph, and accordingly for the present investigation no attempt was made to modify the input to match precisely the target acceleration record.

A typical acceleration record and a displacement record appearing on the platform are shown in figs. 6.5 and 6.6 respectively. Figs. 6.7, 6.8, 6.9, 6.10 and 6.11, show the SDF acceleration spectrum for 1% structural damping for one of the target earthquake motions, together with the corresponding spectra based on the accelerations appearing on the platform for different load conditions on the platform. As shown in these figures, although there is a small difference between the target acceleration on the computer and the acceleration appearing on the

platform, the platform motion is very consistent, being unaffected by the mass or by the frequency of the specimen on the platform. Significant spectral differences appear only at very high frequencies not affecting the response of the specimen. Normalised time histories of displacements and accelerations of the earthquake on the platform are also identical for different excitation levels and for different non-linearities of the specimen on the platform. Similar results apply to the other earthquake records.

The relative consistency of the acceleration appearing on the platform may be attributed to the fact that the acceleration record has no spectral components at 1-2 Hz, so it is not affected by the concrete tank resonances, so it is not severely affected by the oil column resonance of the hydraulic actuator. The consistency of the acceleration time history appearing on the platform was also aided by the relatively small value of the mass used on the test frame.

6.5 Response of the test model to earthquake excitation

6.5.1 Response of the frame without braces.

Since the frame lies in the direction of the longitudinal actuator and is placed symmetrically with respect to it, motion should be restricted to this plane. Because of the small asymmetry of the shake table which produces small out-of-plane accelerations, as well as to a small possible asymmetry of the frame itself, out-of-plane accelerations as well as vertical accelerations on the top mass of the frame were observed during the test. No provision was made to restrict out-of-plane and vertical accelerations, since these additional motions seem to be independent of the in-plane motion and also of small magnitude. With the unbraced frame, out-of-plane and vertical accelerations of the top mass are less than 15% of the corresponding values of accelerations in the horizontal direction. It was estimated that the frame would remain elastic for a lateral deflection of the frame relative to the platform of around 4cm.

Initially the ten earthquake records were used at the small excitation level of 6% to excite the frame within the elastic range. Using the frequency analyser the frequency of the frame was found to be 3 Hz. The damping of the frame was found to be around 1% critical from observation of free vibrations.

Table 6.1 summarises the maximum displacements reached by the top mass relative to the platform, positive and negative, from the mean position for the ten earthquake records (positive being defined as the forward direction of the actuator). A typical such displacement time history is shown in fig. 6.12 and a typical acceleration time history in fig. 6.13. These show the typical characteristics of the narrow band response of a lightly damped single-degree-of-freedom system. These results were found to be well reproducible, i.e. approximately the same response was obtained by repeating the same earthquake record. On the other hand, on changing the excitation level to 3%, the response was not exactly half that caused by the 6% excitation level. This is due to the fact that at such a very small excitation the shake table is highly non-linear due to frictional effects.

Comparison between experimental and analytical results is relatively easy. Fig. 6.14 shows a typical comparison between experimental and analytical results of the displacement time history of the top mass. In this figure are shown two plots, the experimental one, and the analytical one based upon the response of a single-degree-of-freedom system using as input the acceleration appearing on the platform. The analytical result is based on a natural frequency of 2.96 Hz, marginal adjustment of this value showing that this gave the best fit between experimental and analytical predictions. An equivalent viscous damping of $\zeta=0.7\%$ critical was again determined by trial and error in order to have the same peaks in the cycles between experimental and analytical results. A similar procedure to determine damping had been developed by the author in previous experimental work (70).

6.5.2 Response of the braced frame incorporating the rotating device

6.5.2.1 Test parameters

The main part of the present experimental work was concentrated on the dynamic response of the braced frame incorporating the rotating device as shown in the photograph in fig. 6.3. The range of values of the principal test parameters was fixed by the considerations discussed below.

Since the 6mm bars were threaded, the elastic limit was estimated on the basis of the reduced diameter of the bars to be about 1000 Kg. In order to avoid any non-linearity in the bars, the slipping load on the device was set at 700 Kg. Small discrepancies above and below this value were experienced during the tests, due probably to a small variation of the torque applied to the bolts using the torque spanner.

The 6mm bars used for braces were very slender with an estimated effective slenderness ratio of around 300. They thus have a very small buckling resistance, so the device mainly activates from the tensile brace forces.

It was found on trial runs that the device and the braces developed large out-of-plane movements, which introduced a lot of noise to the accelerometers. To prevent this, an arrangement was prepared to restrict the out-of-plane movements. Each of the four corners of the device was connected by horizontal tensioned wires to the two safety frames; being perpendicular to the plane of the frame, these wires restrict the out-of-plane movements without affecting the in-plane motion.

Before each test the braced frame was excited with the very small excitation level of 1% of the earthquake to verify that the elastic frequency of the frame remained the same. At the end of each test the same procedure was repeated to find the new frequency of the frame.

The 10% excitation level is approximately the limiting excitation level for which the device remains elastic, although at this level significant buckling occurs in the braces. The two excitation levels of 40% and 65% were used in the dynamic response of the braced frame with the rotating device. The latter is approximately the maximum excitation level that the shake table can accept with the particular mass of 1000Kg at height 2m (this includes the safety frames as well as the test model).

6.5.2.2 Conduct of the test

Each test was conducted in the following way. The device was first readjusted to its rectangular configuration in case it was deformed significantly in the previous test. A small pretension of 500 N was then introduced in the braces, using the readings of the strain gauges, by adjusting the end nuts by equal amounts. Out of the total number of tests reported, half were performed using the original four braces and the other half using four new braces.

Using the 1% excitation level it was found that the braced frame has an elastic frequency of 7.6 Hz. This is the frequency of the frame where both braces remain elastic and participate in the lateral stiffness, due to the pretension introduced in the bars. During the dynamic response the compressive braces easily buckle and the effective frequency of the response reduces to around 6.5 Hz. There are approximately 60-70 cycles of vibration for each of the earthquake records used, varying slightly as a result of the random generation process. However, the elastic frequency of the frame after the test remains at 7.6 Hz. This is due to the fact that almost all the deformation was taken by the device, so that by the end of the earthquake the braces return effectively to their original condition. In a very small number of cases it was found that the elastic frequency of the frame had reduced after the test to around 7.2 Hz. This was due to the fact that at the end of the earthquake

the device had remained deformed significantly from its initial rectangular shape, so that two of the braces forming one diagonal were buckled and thus made little contribution to the stiffness.

6.5.2.3 Results

Table 6.2 summarises the maximum positive and negative displacements reached by the frame for the ten earthquake records and for the two excitation levels. The scatter of the values of maximum displacements reached by the frame when it is subjected to earthquakes having the same frequency content and the same intensity is seen to be considerable.

Since the braced frame without the device requires horizontal deflection of approximately 9 mm to yield the bars, the horizontal deflections shown on table 6.2 could cause very severe yielding in the bars and produce pinched hysteresis loops.

Comparison between tables 6.1 and 6.2 shows the very substantial reduction of the maximum deflections resulting from the addition of braces to the simple beam-column frame, even though the resulting natural frequencies are more unfavourable in relation to the ground-motion frequencies in the case of the braced frame.

The hysteresis loops of the friction joint are shown in fig. 6.15. The ordinates show the difference between the forces in the two braces, inferred from the strain gauge readings using a Young's modulus of $E=210 \text{ kN/mm}^2$. The abscissae show the displacement measured by the transducer on the device which is equal to half of the change of the length of the one diagonal. These figures show that the friction device produces almost perfect elasto-plastic hysteresis loops irrespective of the amount of lateral displacement reached by the frame.

Global hysteresis loops of the frame are shown for the majority of the earthquake records in fig. 6.16. In these figures the displacement of the mass of the frame relative to the platform is plotted against the total acceleration of this mass in the horizontal direction. The introduction of the device converts the hysteresis loops of the frame to bilinear, the second linear stiffness being the stiffness of the moment resisting frame, which remains elastic throughout the response. The number shown next to the extreme positive and negative displacements (fig. 6.16) indicates the cycles when these extreme displacements occur; it can be recalled that the response consists of 60-70 cycles of vibration and the device activates at almost every cycle.

Figs. 6.17 to 6.21 show typical response time histories of the frame. Fig. 6.17 shows a typical displacement time history of the mass of the frame relative to the platform and fig. 6.18 shows an acceleration time history of the mass. The spectrum of the response acceleration is shown in fig. 6.19. Clearly the activation of the device reduces the effective frequency of the response; frequencies from 4 Hz to 7 Hz being now predominant. Fig. 6.20 shows a typical slip time history given by the transducer on the device. Finally fig. 6.21 shows typical strain gauge readings for the two braces. The two strain gauges used for each bar were connected to compensate for bending so the readings shown in fig. 6.21 represent the mean value, and each directly indicates the axial force in the corresponding bar.

These results proved to be very closely reproducible, i.e. the same response time histories were obtained by repeating a test. An example is given as fig. 6.22, which shows a displacement time history of the transducer on the device by repeating an earthquake record. These results are almost identical with the slip time history shown in fig. 6.20. The response time histories given by the other instruments are also almost identical. This consistency is mainly due to

the fact that by using the device, the braces are only allowed to buckle but not to yield in tension. For each new test the braces are easily retensioned to their initial elastic values. The slipping load of the device is also very consistent, due to the friction material used in this device.

Out of plane and vertical acceleration of the top mass of the frame were at this time more important than in the case of the frame alone without braces. These accelerations were now approximately 25% of the corresponding in-plane accelerations.

6.5.3 Introduction of pretension in the braces

The response of the braced frame with the rotating device is considered here but now modified by the introduction of significant pretension in the braces. Using the readings from the strain gauges, all braces are pretensioned to a value of 3000 N. The design of the device can now be related to the compressive resistance and thus, if desired, it is possible completely to avoid buckling. In particular, the slipping of the device is now caused by the difference of the forces in the two braces. It was, however, found during the tests that 3000 N pretension was not entirely sufficient to prevent buckling in the braces.

In this situation, as discussed in section 5.4.3.2, there are secondary forces appearing on the device and on the braces, resulting from the change of the geometry of the device. These additional forces can be found by writing the compatibility equations for the device and for the braces. This is done in fig. 6.23 for the geometry of this particular frame used on the shake table. Fig. 6.23 shows the increase in tension (V) in comparison with the compression (decrease in pretension) (U) as a function of the displacement (S), which is the lateral displacement of the frame measured from the point at which the device activates. It will be seen that these secondary forces are very small for the displacement levels encountered in these tests.

Figs. 6.24 and 6.25 show the hysteresis loops of the friction joint and of the frame respectively for two earthquake records. Again, these hysteresis loops are almost perfect elasto-plastic or bilinear. The secondary effects mentioned above have no significant effect here, due to the limited displacement magnitudes as noted above, and that the pretension was sufficient to limit buckling to insignificant occurrences. The elastic stiffness of the braced frame before the activation of the device, shown in fig. 6.25, is bigger than that of the frame shown in fig. 6.16, since now both braces remain elastic and participate in the lateral stiffness.

Fig. 6.26 shows typical strain gauge readings for one earthquake record and fig. 6.27 a typical slip time history given by the transducer on the device. In the strain gauge readings, as shown in fig. 6.26, the compressive strain reached by the bars is slightly smaller than the tensile strain, due to the fact that slight buckling still occurs. As also shown in this figure, the device activates at almost every cycle of vibration.

In fig. 6.28 a comparison is attempted between experimental and analytical results for the displacement of the mass of the frame relative to the platform. In this figure are shown two plots. One is the experimental displacement for one earthquake record. The other is the analytical prediction which is based upon the response of a single-degree-of-freedom system braced frame incorporating the device and using as input the earthquake appearing on the platform. Viscous damping of $\zeta=0.7\%$ is used in the prediction, which is the value found previously in order to have best fit between experimental and analytical results of the frame alone. Analytical results giving the distribution of input energy are also shown in fig. 6.29; the notation used in this figure to describe the various energy expressions is described in detail in section 1.3.2.6. As shown in this figure, more than 95% of the total input energy is dissipated by the device.

There is very good fit between experimental and analytical results in fig. 6.28, right through to the end of the record. This is attributed to the very high value of equivalent damping of the braced frame caused by the activation of the device. The very high equivalent damping of the system smooths the response spectra, and the system now has a very limited 'memory'.

6.5.4 Response of the braced frame incorporating the bending device

The bending device discussed in greater detail in section 4.2.6.3 was introduced at the intersection of the four braces. The dimensions of the bending device were such that the braces used in the rotating device could also be used with the bending device. The only difference is that now the four braces were fully fixed to the device, which slightly increases the buckling resistance of the braces.

The braces were pretensioned at the beginning of each test to 500 N, following the same procedure as with the rotating device. The elastic frequency of this frame is now 4 Hz. This significant reduction is due to the bending elastic flexibility of this particular device.

Two runs were performed using the bending device. The hysteresis loops of the frame are shown in figs 6.30 and 6.31 and a record of the reading of a typical strain gauge on the braces in fig. 6.32.

A careful examination of the hysteresis loops shown in figs. 6.30 and 6.31 shows that they are non-degrading, due to the introduction of the bending device. However, they do not have the perfect bi-linear shape given using the rotating friction device, because the solid cross-section used for the bending device results in a progressive rather than a sudden plastification. Other cross-sections for bending devices having a sharper transition to the inelastic range are possible.

6.5.5 Response of the unprotected braced frame

An attempt was made to test the same braced frame without providing any hysteretic device. A single round bar was now used for each diagonal brace. The braced frame without any device again has a frequency of 7.6 Hz. The arrangement for connecting the braces to the frame is shown in fig. 6.33. Each bar was inserted at each end into a drilled bolt and silver soldered to it. The bar is then connected to the frame by these bolts; the objective of this arrangement was to permit uniform elongation throughout the length of the bar in the event of brace yielding, which is not possible with the direct threaded connections used hitherto. However, it was found that the part of the bar next to the bolt was annealed and consequently this part has reduced yield properties. On further examination in a conventional static machine it was found that the bars broke at this position at a load of 16 kN before any significant non-linearity in the remaining length of the bar. The same problem was also experienced on the shake table. Although the braced frame was excited by only 40% of the first earthquake record, the two braces broke at these weak positions. No further attempt was made to test the braces further without any device.

Analytical predictions are shown in fig. 6.34 for the resulting hysteresis loops of the two braces and of the frame without any device. In these results it is assumed that the braces have a tensile strength of 17 kN and yield uniformly throughout their length. The abscissae show the horizontal displacement of the mass and on the alternates show the difference between the two forces in the braces. The resulting hysteresis loops are clearly pinched, as shown in this figure, and the maximum displacement reached by the frame for this particular earthquake is much higher than the displacement reached introducing the device.

CHAPTER 7

BEHAVIOUR AND DESIGN OF MULTI-STOREY FRAMES INCORPORATING THE FRICTION DEVICE

7.1 Introduction

The hysteretic behaviour of the single-storey X-braced frame was shown by shake-table testing (Chapter 6) to be drastically modified by incorporation of the friction device. The dynamic response of a multi-storey building is further complicated as a result of the energy exchange between different modes resulting from non-linearity exemplified by the slip devices.

Problems associated with the design and the behaviour of multi-storey buildings incorporating the device are addressed in this chapter.

7.2 Distribution of energy between the various storeys of a multi-storey building.

One of the most difficult decisions in the design of multi-storey buildings to resist earthquakes is the relative stiffness and strength of the various storeys throughout the height of the building. A good design requires beams, columns and braces to be proportioned in such a way that the ductility demands are appropriately related to the actual limiting ductility of the various structural elements.

Appendix B summarises different optimisation techniques which may be used to derive such a defined optimum distribution of stiffness in multi-storey steel building frames; for example the distribution of stiffness which causes the same ductility demands for all the structural elements. As shown in this Appendix, despite the uncertainty of future earthquakes, there are certain decisions concerning the stiffness distribution that a designer can take in order to improve the performance of steel structures.

Similar decisions can also be taken for braced frames when they incorporate friction devices. The slipping load of the devices at the various storeys should usually be chosen to cause fairly uniform distribution of the slip movements. In some cases hysteretic devices may only be needed at the storeys where analytical predictions show a concentration of earthquake energy.

7.3 Dynamic response of a multi-storey braced frame incorporating the rotating device

The response of a multi-storey braced frame incorporating the rotating device is examined here. The frame considered is the 10-storey shear braced frame discussed in detail in section 1.3.3.2. It is now assumed that the rotating device is introduced at the intersection of all the cross braces. The braces here have very small buckling resistance so the design of the device is based upon the tensile strength of the braces while the compressive braces are allowed to buckle. In particular, it is assumed that the slipping load of a device in a storey is equal to 80% of the tensile yield of the braces for that storey.

This frame is subjected to an artificial earthquake generated using the second order filter. The effective frequency and damping of the filter are given average values for real earthquakes as set out in section 2.2.2. The generated earthquake has an intensity of $S_0=100\text{cm}^2/\text{sec}^3$ (for the definition of this intensity parameter see section 2.2.2). This is approximately twice that of the 1940 NS El Centro earthquake and will therefore produce significant inelasticity in the frame. The maximum storey displacements at the end of the first 20 sec of the earthquake are shown in fig. 7.1(b) and the maximum absolute displacements in fig. 7.1(a). The results from the response of the braced frame without the device are also shown.

In both cases greater inelastic activity and higher ductility requirements are concentrated at the lower storeys.

As shown in this figure, the introduction of the device significantly reduces the absolute displacements and the storey displacements of the frame. A particularly beneficial result of the introduction of the device is that the braces are not allowed to yield in tension, whereas without the device, significant restoration would be needed after an earthquake as a result of the braces yielding in tension.

7.4 Shake table testing of a two-storey braced frame incorporating the device

7.4.1 General

The dynamic response on the shake table of a two-storey braced frame with friction devices is considered here. This work is a continuation of the study of the single-degree-of-freedom frame reported in Chapter 6. The two-storey frame constitutes a more realistic model of a multi-storey braced frame. A full report of the extension test programme carried out on this frame is given in detail in a separate report (71). A brief account only is given here.

7.4.2 Dimensions of the two-storey frame

The elevation on the plane of shaking is shown in fig.7.2 and a photograph in fig.7.3.

This frame represents a (1/6) scale model of the first two storeys of the multi-storey building designed in section 5.4.2. The two devices introduced at the intersection of the braces are also (1/6) scale models of the full-size device discussed in section 5.4.3. The equal scaling of the geometry of the frame and of the two devices was selected so that the resulting hysteresis loops for the model frame would be similar to the hysteresis loops of the full-size frame.

Flat bars were chosen as braces for both storeys. These had negligible buckling resistance. The braces again dominate the lateral stiffness and strength. A larger mass on the top (second) floor was chosen to approximate the influence of the upper floors in the full-size structure. The higher value of the mass on the second floor therefore produces a more uniform distribution of the input energy to the two storeys of the model frame.

Out-of-plane deflections are avoided in this frame by connecting the frame to the two safety supports with extremely slender flat-strip braces.

7.4.3 Selection of earthquake excitation

This frame was subjected to the artificial earthquake records generated using the second order filter. The effective frequency and damping of the filter was again scaled down and the duration of the generated earthquakes was set at 15 seconds. Since the fundamental period of the two-storey frame is 3 Hz, this results in approximately 40-45 cycles of vibration in the dynamic response.

7.4.4 Response

The hysteretic response of the two friction joints in this frame for small activation of the device is almost ideally elasto-plastic, as reported in Chapter 6 for the single-degree-of-freedom system. A typical result is shown in fig. 7.4. Here again the horizontal axis shows the displacement of the device whilst the vertical axis shows the difference of the forces of the two attached braces. Fig. 7.5 shows a typical example of the hysteretic response of the top storey of this frame. The horizontal axis shows the top storey displacement and the vertical axis shows the storey shear force. The latter is found as the product of the mass on the top floor and the corresponding acceleration. This figure again shows that the hysteresis loops are almost bi-linear, with the upper-range stiffness being the stiffness of the moment resisting frame, which again remains elastic throughout the response.

These results are, however, obtained only for small rotations of the friction devices. In fig. 7.6 hysteresis loops are shown for the friction joint in one of the two devices with higher rotation. The hardening of the hysteresis loops resulting from the change of the geometry of the device is clearly shown in this figure.

The material used for the braces in this frame was hot-rolled mild steel and these braces were able to yield significantly in tension before breaking (in contrast with the braces used for the single-degree-of-freedom system reported in Chapter 6). In this way comparison with the response of the braced frame without the device was possible, the latter case being simulated by locking the device so that it remained elastic throughout the response. Fig. 7.7 shows typical hysteresis loops for the top storey. As shown in this figure, the resulting hysteresis loops are now pinched. By the end of the earthquake a very significant slack was observed in the braces as a result of yielding in tension.

The displacements achieved by the frame when the device is locked are much higher than the displacements with the device activating; typically 3-4 times higher. This is mainly attributed to the development of slack; it may be noted that this example comprises a relatively large number, 40-45, cycles of response. For the first part of the earthquake response, before the braces yield significantly, the frame does not reach very high deflections, but in the latter part, although the earthquake intensity has reduced, the deflections increase significantly since the braces now make little contribution to the lateral stiffness.

7.4.5 Concluding remarks from the response of the two-storey frame

From the dynamic response of the two-storey frame on the shake table the advantages of introducing the rotating device were obvious. With the device activating, the braces are now allowed to yield and by the end of the earthquake they are in the same state as they were before the test. With the devices locked, the braces yield in tension, developing a slack by the end of the earthquake and produce very high deflections in the frame.

Problems encountered by non-linearities exemplified by the devices can easily be solved by analytically predicting the maximum expected rotation of the device and adjusting the slipping load of the device accordingly.

7.5 Dynamic testing of full-size devices

Tests on full-size devices are currently being carried out at Imperial College. These include devices incorporating the idea of frictional washers similar to the device discussed in section 5.4.3 and a few other devices with different configurations.

7.6 Design philosophy of multi-storey buildings with the device

There are distinct differences between the responses of multi-storey buildings to wind and to earthquake loading. For wind loading the building is safe for loads smaller than the design loads, whereas for higher loads the probability of failure as well as the damage to the structural and non-structural components increases very steeply, because the long duration of a wind storm and the presence of a sustained time-average loading ensures severe cumulative damage. For earthquake loading the damage to the building and the probability of failure is a progressively increasing function of the load.

A design requirement for braced frames incorporating hysteretic devices is for no slip movement to occur under the design wind forces. This requirement ensures that there will be no damage to the main structural elements and no deformation of the device for wind loading which is a frequent event with short return period.

For earthquake loading the introduction of the device positively increases the survival probability of the structure. The device ensures that the elasticity of the braces is sustained. Only when the device reaches its limiting rotation position can the braces yield and finally break at high displacements.

The introduction of the device is also generally expected to reduce the amount of remedial work following an earthquake. It is well known that significant damage can occur in a building, with very expensive repairs in the non-structural components which sometimes match the cost of the building itself. These are clearly a result of excessive lateral displacements of the frame. The device is generally expected to reduce the damage since it reduces the lateral deflections. Damage to the structural components is also expected to be reduced. Even at the design stage the cost of the building with the device can be reduced (albeit probably only slightly) because it can be designed with a reduced design earthquake loading as a result of allowing a reduction in the design K coefficient in recognition of the energy dissipation capability of the system.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

8.1 Conclusions

The main part of this thesis was concentrated in the development of friction devices suitable for braced frames. Many different such devices tested in this project were found to deform a large number of cycles, always keeping the same basic characteristics. Both in the static and in the dynamic tests, these devices completely change the hysteretic response of braced frames. The hysteresis loops of braced frames incorporating these devices are very stable for a large number of cycles. Should these devices be damaged by the end of the earthquake, they can easily be replaced.

Comparisons were also attempted in this thesis with existing ideas and in particular with devices which are based upon deformation of the primary structural members in bending or shear. Such ideas tend to concentrate a lot of deformation at one panel in the structure, requiring expensive restoration afterwards.

For these reasons it is thought that devices based upon friction are more advantageous for braced frames. In particular, these devices are cheap, easily replaceable, very effective in dissipating more energy than that imparted by one earthquake and, finally, protect all the structural elements from damage.

8.2 Recommendations for future work.

The results of the friction tests are reported in Chapter 5. Some uncertainty remains in predicting the coefficient of friction for a given material under high (and probably non-uniform) normal stress. In particular, two nominally identical slipping devices used in the two-storey frame reported in section 7.4 show slightly differing slipping loads, although this difference may have been accentuated by the small scale of these devices. The accurate prediction of the slipping load for the full-size device is clearly very important and this requires to be thoroughly investigated by examining different full-size examples.

Tests on full-size devices with different geometric configurations are also recommended. Devices similar to the one reported in section 5.4.3 require a substantial amount of friction material. If the washer dimensions are changed to produce devices with larger capacity, the amount of friction material required is correspondingly increased, and the cost may become significant. Greater economy can be obtained by modified configurations, but at the expense of reduced ease of assembly in service.

Analytically the selection of the slipping loads for the devices must be thoroughly investigated as well as the benefits to be gained from systematic variation of the slipping loads throughout the height of the building (for example, by extension of the procedures presented in Appendix B).

It has been shown that severe hardening of the hysteresis loops of the device can be avoided by suitably choosing the geometry of the device. However, as shown in section 4.3.5, hardening of the hysteresis loops may be advantageous in some cases. On the other hand, since such hardening may result in overstressing of the braces, such options need to be thoroughly investigated.

APPENDIX A
STIFFNESS MATRIX OF I-SECTION BEAMS
WITH THE WEB YIELDED IN SHEAR

A1. Derivation of the stiffness matrix

Consider the I-section beam shown in fig. A1.

When the web of an I-section member yields in shear, it is common to assume a strain hardening relationship that is compatible with a presumed uniaxial strain hardening ratio, H say. The von-Mises yield criterion and the Prandtl-Reuss equation, assuming that the volume of the material is constant during plastic deformation, lead to an equivalent plastic shear modulus G_p (say), given by $G_p = HE/3$. As has been pointed out by Roeder (50), this simple theory predicts excessive warping since it assumes that the incremental shear stress and strain are uniformly distributed over the web and are zero in the flanges. For short elements the resulting prediction of overall deformation is significantly in error. This can be overcome by application of sandwich beam theory.

This theory is described in detail in reference(72). An element of a sandwich beam comprises two 'faces' and a 'core'. The faces correspond to the flanges of an I-section beam, and the core corresponds to the web. The longitudinal (x-direction) stiffness of the core is assumed to be negligible by comparison with that of the faces. The core acts to transmit stresses in shear between the faces, thus developing the conventional St. Venant bending resistance of the sandwich overall, but itself carries no longitudinal stresses. There is generally significant shear deformation on the core, since it has a low transverse modulus of rigidity G_p . Under a loading consisting of a bending moment M and a shear force Q an infinitesimal element dx will deform as shown in fig. A2(a). This deflection w is composed of two partial deflections w_b and w_s .

The former is the deflection which occurs if the shear modulus, G_p , of the core is infinite. The deflection w_s arises from shear strain in the core, as shown in fig. A2(c). It is postulated that the total deflection, w , is given by:

$$w = w_b + w_s \quad (A1)$$

As shown in fig. A2, the global bending component w_b can be evaluated using the ordinary theory of beam bending

$$w_b'' = \frac{M}{EI} \quad (A2)$$

where M is the bending moment of the beam,

E is the Young's modulus of the face,

I is the moment of inertia of the two faces about the centroidal axes, and

w_b'' is the second derivative of w_b with respect to the longitudinal coordinate x .

The component w_s is accompanied by distortion of the cross-section, or 'warping', as shown in fig. A2(c). If this warping is restricted, significant bending moments M_f are developed in the individual faces, with corresponding shear forces Q_f . These bending moments are basically a result of the shear deflection w_s . Denoting the face bending and shear deflections as w_{bf} and w_{sf} , since the faces are forced to undergo the same deflections w_s as the core, we have:

$$w_s = w_{bf} + w_{sf} \quad (A3)$$

The total shear force Q is carried partially by the core and partially by the face, hence

$$Q = Q_c + 2Q_f \quad (A4)$$

where Q_c is the shear carried out by the core and

Q_f is the shear carried out by a single face.

Conventional shear deflection theory gives:

$$Q_f = G S_f w'_{sf} \quad (A5)$$

$$Q_c = G_c S_c w'_s \quad (A6)$$

where S_c, S_f : effective reference areas for shear in core, face
 G_c, G : shear modulus for core, face respectively.

The face bending deflection term, w_{bf} , is determined by:

$$M'_f = -Q_f \rightarrow E I_f w'''_{bf} = -G S_f w'_{sf} \quad (A7)$$

where $I_f = \frac{1}{12} b t_f^3$

in which t_f = face thickness.

Warping is fully prevented at an end when $w'_{bf} = 0$ at that point, and is free when $w''_{bf} = 0$ at that point. Assuming now that the sandwich beam is not subjected to any axial force, we have:

$$M = E I w''_b \quad (A8)$$

$$Q_c = -M' = -E I w'''_b \quad (A9)$$

where M is the moment carried according to the beam theory.

Ignoring now the axial degrees of freedom and using only two degrees of freedom for each end of a beam with length L , the boundary conditions become:

$$\text{at } x = 0, \quad w = w_1, \quad \theta = \theta_1$$

$$\text{at } x = L, \quad w = w_2, \quad \theta = \theta_2$$

Instead of using the previous conditions, we can for simplicity use

$$\text{at } x = 0, \quad w = 0, \quad \theta = \theta_1$$

$$\text{at } x = L, \quad w = w_2 - w_1, \quad \theta = \theta_2$$

Equations (A1) - (A9) can now be combined to give:

$$w_{bf}''' - \alpha^2 w_{bf}' = - \frac{\alpha^2 Q}{S_c G_c} \quad (A10)$$

$$\text{where } \alpha^2 = \frac{G_c S_c G S_f}{E I_f (G_c S_c + 2G S_f)}$$

Equation (A10) can be solved with the appropriate two boundary conditions concerning warping, together with the condition that $w_{bf}=0$ at $x=0$. Having found w_{bf} , w_{sf} can be found from (A7) with the boundary condition that $w_{sf}=0$ at $x=0$, and w_b from equation (A9) with the boundary conditions at $x=0$, $w_b=0$, $w_b'=\theta_1$ and at $x=L$, $w_b'=\theta_2$.

We can also write the boundary conditions concerning warping in a more general form as:

$$\text{at } x = 0, \alpha w_{bf}' + R_2 w_{bf}'' = 0$$

$$\text{at } x = L, \alpha w_{bf}' + R_1 w_{bf}'' = 0$$

where R_1, R_2 are coefficients depending upon the degree of warping restraint provided by the two ends.

For example, for $R_1=R_2=0$, we have the case where both ends are fixed against warping, and for $R_1=R_2=\infty$, the case where both ends are free.

The previous boundary conditions lead to the same stiffness matrix as the one given by formula 4.2 in section 4.2.2, but with the new ϕ_s given in table A1. Together are also given the results for a special case when $H=0$ and ϕ_s is found by applying the l'Hopital's rule:

In these results ϕ_s is always positive since the negative terms are always smaller than the positive. Using also simple expressions, such as $\tanh(\alpha L) < 2 \tanh(\frac{\alpha L}{2})$, we can observe the obvious result by comparing the various ϕ_s , that warping restraint increases the stiffness of the beam.

A2. Example

Consider the Universal beam section shown in fig. A3 which is assumed to have both ends fixed against warping. In this beam for lengths smaller than 3m mainly shear yielding is expected.

Under increasing load the web will eventually yield in shear and the plastic shear modulus G_p can then be used to find the inelastic stiffness matrix. Fig. A4 shows the contribution also of the sandwich action for this Universal beam section for different lengths L , assuming a uniaxial strain hardening ratio $H=0.75\%$. This value was said by Roeder (50) to give best fit between experimental and analytical results. For higher values of the length L the contribution of the sandwich beam theory is almost constant, since $\tanh(\frac{\alpha L}{2}) \rightarrow 1$.

The ϕ_s term plotted in fig. A4 corresponds to the two members left and right from the point of application of the load.

It is evident from this example that since the effects caused by complete or partial prevention of the cross-sectional warping quickly damp out as we move away from the clamped end, the global stiffness matrix of a sandwich beam is only affected if the length of the beam is very small.

APPENDIX B

DISTRIBUTION OF STIFFNESS IN MULTI-STOREY STEEL BUILDING FRAMES SUBJECTED TO EARTHQUAKE EXCITATION

B1. Introduction

This Appendix considers the desirable distribution of stiffness in multi-storey steel building frames subjected to earthquake loading. Due to the random nature of earthquakes, it is difficult to determine an optimum stiffness distribution in a multi-storey building frame suitable for any earthquake. Despite this, certain conclusions can be drawn from past experience and certain guidelines can be given in order to improve the performance of steel structures. Particular attention is given here to the distribution of the input energy between the storeys in braced steel frames and also in braced frames incorporating hysteretic devices.

Seismic design of structures must satisfy two basic criteria. During minor earthquakes the structure must have sufficient strength to avoid any structural damage, and during a major earthquake must possess sufficient ductility to prevent collapse. The best stiffness distribution is usually considered to be that which causes the same ductility requirements at all the floors in an earthquake. Clearly other definitions could also be considered, such as to equalise the relative storey drifts, or criteria concerning the distribution of energy.

B2. Linear systems.

Consider first the simplified model of a shear building where all the mass is assumed to be concentrated at the floors, which are themselves rigid. For this structure, a stiffness distribution leading to the same ductility requirements at all the floors can be found as follows. Assume a fundamental natural period, and a smooth

acceleration spectrum for the expected earthquake, for example the response spectra given by Housner (73) for American earthquakes. Assume also an initial stiffness distribution, for example a linear variation with storey number. Normalised values of the significant structural properties are thus entirely specified, and we can find the storey shear in each mode of vibration. Combining the modes we can estimate the total shear induced in each storey. For a shear building with well-separated frequencies the modes can be combined by taking the square root of the sum of the squares of each mode.

Assuming now that plastic deformation would occur at a limiting storey shear force that is proportional to the storey stiffness (any other relationship can also be considered), we can use the foregoing shear distribution as the basis for modifying the stiffness distribution. At this stage a small adjustment is made in order to keep the fundamental period constant; this is conveniently done by applying a scaling factor to the values of the masses. The procedure is then repeated. This method converges very quickly to a specific stiffness distribution, which depends upon the chosen acceleration spectrum (and so upon the assumed value of the damping), and upon the fundamental period of the building, with which the limiting storey shear would just be reached in all storeys; i.e. $\mu=1$ for all storeys.

Typical results for an 8-storey shear building are shown on table B1; the mass at each floor is assumed to be the same, and three different values of the fundamental period have been considered. An undamped acceleration spectrum typical of the American earthquake has been used for this illustration. For $T=0.4$ sec the stiffness distribution is mainly influenced by the first mode of vibration, and is approximately parabolic, corresponding to a straight-line first mode of vibration. The different results for the higher period structures show the influence of the higher modes.

This method only applies to elastic behaviour, i.e. ductility $\mu=1$. The value of this procedure is examined on an example. Four different stiffness distributions, namely, the parabolic, that found by the previous approach, and two linear with min/max values of the stiffness of 0.4/1 and 0.5/1 are compared in an 8-storey building frame. The building is assumed to have 2% critical damping in all modes of vibration. The building is subjected to an artificial earthquake having a specific acceleration spectrum using the method described in section 2.2.1 which is an average acceleration spectrum of American earthquakes. Linear elastic response only is considered. For two fundamental periods considered, namely $T=1$ sec and $T=2$ sec, the structure with the stiffness distribution found by the above approach, or the closest to it, give the most uniform ductility factors.

B3. Non-linear systems

In a structure where a high degree of non-linearity is expected, a stiffness distribution which causes the same maximum ductilities at all floors can still be found by an iterative procedure. Using a specific earthquake and an initial estimate of the stiffness distribution, we can find the maximum ductilities at all the floors through non-linear dynamic analysis. According to these ductilities we modify the structure slightly and repeat the same procedure until the ductilities are the same at all floors. The structure is modified at each step by increasing the stiffness at the storeys with high ductilities, keeping the value for the bottom storey as unity. The masses can be kept constant in this procedure without significantly affecting the fundamental period of the structure.

The stiffnesses at the floors are modified at each step by a very simple procedure. Assume for example that the restoring force at each floor is elasto-plastic as shown in fig. B1, where $F(x)$ is the restoring force, a is the yield displacement, b is the maximum

displacement reached at the previous step and K is the elastic stiffness of the floor. Then an equivalent elasto-plastic stiffness is given by

$$K_{eq} = \frac{1}{b} \int_0^b \frac{F(x)}{x} dx = \frac{K}{\mu} (1 + \ln \mu)$$

where $\mu = b/a$, the ductility.

This formula can easily be extended to include bilinear or any other hysteretic restoring force.

This method converges to a specific stiffness distribution. This again depends upon the earthquake chosen, the final ductility level obtained, the assumptions made about damping, upon the way that stiffness and ultimate shear in each floor are related, and upon the formulation describing the inelastic behaviour of the structural elements.

A "weak" storey (i.e. a storey with high ductility requirements) may not only be a storey with low elastic stiffness, but also a storey with low ultimate shear and high initial stiffness, and interaction between those characteristics must be considered. Typical results for three and four-storey shear building frames are shown in table B2, indicating the optimum stiffness distribution and the final uniform ductility obtained, using the first 10 sec of the N-S component of the 1940 El Centro earthquake. The earthquake is progressively multiplied by a factor to give higher ductilities and this factor is also shown in table B2. Rayleigh damping has been assumed based on linear dashpots of values $C=aM+bK$, where K is the initial elastic stiffness matrix and M the mass matrix and the scalar coefficients a and b are selected to give 2% critical damping to the first two modes.

For the linear response ($\mu=1$) the stiffness distribution found is very close to that given by the method first described above by application of the El Centro acceleration spectrum, but for higher

ductilities it requires weaker upper floors. For a small degree of non-linearity (up to $\mu=3$, say) the stiffness distribution found from the acceleration spectrum can be considered to be good enough, as the effect on the postulated stiffness distribution is less than the variation between estimates based on different realisations of the earthquake.

B4. Response of three multi-storey frames to earthquake excitation

The theoretical optimisation discussed in the previous section is probably of limited practical value. The optimum stiffness distribution found by optimisation with respect to a linear acceleration spectrum is mainly influenced by the fundamental mode as shown in table B1. Only for structures with a long fundamental period will the other modes (principally the second) become important. Such precise application of the stiffness distribution cannot be applied in practice since the actual stiffness is influenced by the non-structural components. The stiffness distribution found from non-linear dynamic analysis is even more uncertain since it depends upon the earthquake under consideration. A more useful approach is to compare directly different stiffness distributions and examine the variation in the response when the stiffness has changed by a certain amount. The dynamic response of three multi-storey shear building frames is considered here.

The frames considered are 8-storey shear building frames. It is assumed that the values of the masses at the floors are all equal to 34t. The three frames were assumed to have three different stiffness distributions, one parabolic, and two linear, of which one has relative min/max values of stiffness 0.3/1 and the other 0.5/1. The yield displacements for all the storeys were assumed to be equal to 17mm. The ultimate shear on the bottom storey was set for all three structures at twice the UBC design shear, using a value of $C=1/15$ and unity for all the other design factors. The capacities of the remaining floors were proportioned relative to the first according to the

specific stiffness distribution of each structure, assuming that stiffness and ultimate shear strength of each storey were proportional. The first three fundamental periods of the three structures are shown on table B3.

The restoring force at each floor is assumed to be either elasto-plastic or bi-linear with the bi-linear stiffness 10% of the corresponding elastic stiffness.

The three structures were subjected to the first 20 sec of the 1940 NS El Centro earthquake and also to the same earthquake with a multiplication factor of 2. Zero damping was assumed.

The ductilities obtained at the floors at the end of the record for the three structures are shown in fig. B2. A simple notation is used here to describe the response of the structure, i.e. the letter "E" which stands for "Earthquake response" and then:

Ep = Elasto-plastic storey characteristics

B1 = Bi-linear storey characteristics

Par = Structure with parabolic distribution of stiffness

Lin.1 = Structure with linear distribution of stiffness with min/max values of 0.3/1.

Lin.2 = Structure with linear distribution of stiffness with min/max values of 0.5/1.

1 or 2 = Earthquake multiplication factors.

As shown in fig. B2 the parabolic distribution of stiffness produces high deformations at the upper three floors, and thus a significantly non-uniform ductility distribution. Comparing the two structures with linear distribution of stiffness, the Lin.2 produces the more uniform result. Higher earthquake intensities result in the concentration of higher ductilities at places where there are already high ductility requirements in all three structures. The bi-linear model makes the ductility distribution at the floors more uniform especially by reducing the ductilities at the places where the elasto-plastic model produces particularly high ductilities.

Fig. B3 gives the cumulative plastic deformation per floor for the three structures. This is defined as the total inelastic displacement of the storey and is equal to the energy dissipation divided by the ultimate shear of this storey. The cumulative plastic displacement is less uniform than the ductility for the three structures, especially for the structure having the parabolic stiffness distribution. Again this is more uniform for the Lin.2 structure.

Finally, the time history of the energy dissipation per floor for the three structures is shown in fig. B4. The major energy dissipation occurs in the first 10 sec of the earthquake, which represents the stronger part of the record.

B5. Stiffness distribution including beam flexibility

The foregoing studies have treated the multi-storey frame as a shear building, ignoring the influence of beam flexibility. An approximate allowance for beam flexibility can be made by using values of lateral stiffness and strength found from an approximate static analysis taking beam flexibility into account.

An example of damage in a multi-storey frame which was attributed to a bad stiffness distribution is given in ref (74). This is a four-storey unbraced building in Japan, designed according to the strong beam - weak column approach (something which is not generally accepted in USA), and thus behaved approximately as a shear building. Damage was concentrated at the third floor. The authors found through non-linear dynamic analysis that the third floor actually concentrated high ductility requirements and cumulative earthquake energy dissipation, although it satisfied the minimum specified design forces. Provision of bracings is one of the solutions proposed in order to improve its dynamic response.

B6. Stiffness distribution in braced frames

It is customary in the design of multi-storey braced steel building frames to use the same brace for many storeys, or even all storeys. The braces usually dominate the lateral stiffness and strength so this results in an approximately constant distribution of stiffness over the height and as a result high ductility requirements are usually concentrated at the lower floors.

It was observed earlier that higher earthquake intensities concentrate higher earthquake energy most notably at the floors where there are already high ductility requirements. The same result was also noted by ref (75). This point is even more important for braced structures which may have degrading performance characteristics (cf. the typical results for the response of braces shown in section 2.4.3). For higher earthquake intensities the response of braces generally increases more than the response of a system having an elasto-plastic restoring force.

Similar ideas also apply in braced frames incorporating hysteretic devices, such as the "rotating" device. Consider for example the 10-storey braced frame used in section 1.3.3.2 having similar braces at all the floors, together with "rotating" devices of equal slip value. When this frame is subjected to the first 20 sec of the 1940 El Centro earthquake than at the bottom floor the device activates at every cycle with an average lateral slip of $\pm 3\text{cm}$ per cycle. On the contrary, virtually no slip occurs on the top two floors. It is evident that this is the result of an unfavourable distribution of stiffness and strength in the structure. However, an unfavourable distribution of stiffness in the structure can readily be compensated by varying the slipping loads of the devices in the various storeys. In particular the slipping loads of the devices in upper storeys must be set to smaller values than the bottom storeys. An appropriate set of values can be obtained by using the two iterative procedures described earlier for linear and non-linear systems.

To vary the slipping load the devices at every storey following precisely the values found from the optimisation techniques is probably not a practical proposition. A more practical proposition is to use two or three different slipping loads for the devices according to their location. Having done this it is clearly necessary to examine the variability of the response during non-linear dynamic analysis using earthquake records, following the general lines of the dynamic response of the three shear building frames described earlier in this appendix.

It is also evident that there are cases where it is not necessary to put hysteretic devices at all storeys, but only at the lower storeys where they are most effective. An extensive investigation of these problems is continuing and results will be presented in a separate report.

B7. General discussion

Three kinds of stiffness distributions in an unbraced multi-storey shear frame, namely uniform, linear and parabolic variations have been compared in ref (75). Uniform stiffness was found to give poor performance and the linear variation was in general better than the parabolic, but the balance depended upon the min/max values used. ref. (75) also reported that the increased ductility demand associated with the increase in duration of the excitation is concentrated where there are already marked plastic deformations.

In ref. (8) one unbraced and three braced ten-storey frames with different bracing arrangements have been examined, subjected to El Centro x 1.5 earthquake. Typical results are shown in fig. B5. The frame marked F1 is a fully braced frame, with brace slenderness ratio $l/r=300$; the braces have maximum area 3.28 in^2 (ground floor), and minimum 2.88 in^2 . The braces dominate the total stiffness and strength and the decrease in stiffness in height is thus much less optimum (the present results suggest a ratio around 0.3 compared to $2.88/3.28=0.88$). It is therefore not surprising that relatively high

ductility requirements are concentrated in the lower floors, although the absolute values are only modest. In the same frame only two sections are used for the beams, W18X50 for the upper five floors and W18X60 for the lower, resulting in high ductility demands at the beams in the lower floors.

Abrupt changes in lateral resistance have often caused undesirable performance, and sometimes collapse behaviour, as they are frequently overlooked when designing and analysing structures in accordance with the prescribed minimum forces. The structure marked as A1 in fig. B5 has braced elements at alternate storeys, causing extreme ductility requirements at the floors without braces. Similar conclusions can be drawn from ref. (76).

The practice of providing a "soft" lower level to absorb all the energy of the earthquake (77) has frequently shown poor performance. Large openings in shear walls, interruption of columns or beams, can strongly influence the lateral resistance and thus require consideration in the dynamic response.

Conventional design specifications are commonly in the form of quasi-static loads which are based on an idealisation of the earthquake spectrum. The UBC Code, referred to above, is one such which has been extensively used and critically examined. The way in which the structure is actually modelled to these loads can have a strong influence on its performance in any actual earthquake. Three different design philosophies, minimum weight design, allowable stress design and strong column design were compared in ref. (78). It was found that the structures designed with the strong column design philosophy performed better than the others. The strong column design philosophy, where the columns are designed to remain elastic and all the plastic deformation is concentrated on the beams, has generally been accepted in California.

The strength of the bracing elements can also strongly influence the dynamic response of the structure. Ref. (79) concludes that strong bracing elements (slenderness ratio 60 or less) must be avoided since they would result in excessive inelastic activity in the girder or columns.

B8. Experimental studies

Experimentation was carried out on a small model five-storey frame on the shake table. The frame, suitably modified, provided different stiffness distributions.

A sketch of the frame is shown in fig. B6. The floors were a thick slab of plastic material and were thus relatively very stiff. The columns were rectangular steel strips, oriented with their weak direction in the direction of shaking. Perpendicular to the direction of shaking the frame is much stiffer, since the columns are now oriented in their strong axis, but a small out-of-plane deflection as well as a small torsion of the frame was nevertheless experienced during the test. This was probably due to a small asymmetry in the frame and to the fact that the shake table produces small out-of-plane deflections.

The stiffness and strength of any specified storey was readily doubled by adding new columns next to the original ones. The new columns were separated from the original columns by a small distance to ensure that the total stiffness of the storey was just twice the former.

Maximum inter-storey displacements in the direction of shaking were measured by strings, as shown in fig. B6. Two strings were used per storey, as shown in this figure, to measure the maximum displacement left and right from the mean position. The strings are held in position by small frictional forces. Once the frame moves the strings develop a slack and measurement of this slack after the end of the earthquake gives the maximum inter-storey

displacement. This simple technique of measuring the inter-storey deflections showed very good repeatability.

Stiffness and strength at each floor are clearly proportional; the limiting elastic displacement (as defined in fig. B7) at all floors was 5cm. Experimentally the fundamental period of this frame was 0.66 sec (with a small variation according to each stiffness distribution) while the analytical prediction assuming rigid floors was 0.88 sec. The difference was probably mainly due to the fact that the floors were not entirely rigid.

The frame was subjected to the NS 1940 El Centro earthquake with a small excitation level so that the frame remained elastic with unimportant P- Δ effect.

The three different stiffness distributions used are shown in fig. B8(a) and the maximum relative displacements obtained by the end of the record are shown in fig. B8(b).

Uniform inter-storey displacements in the first mode would require a parabolic distribution of stiffness with values (1, 0.92, 0.78, 0.61, 0.4). The second stiffness distribution which is the closest to this one produces the more uniform relative displacements.

Again the constant distribution of stiffness produces higher relative displacements on the bottom floors.

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S_o (cm ² /sec ³)	σ_x (m)	Energy dissipation per second (Nm/sec)	Frequency (Hz)	η
50	0.018	5790	0.74	0.20
100	0.028	11360	0.64	0.18
150	0.035	16820	0.58	0.17
200	0.043	22180	0.54	0.15
800	0.115	82040	0.38	0.09

Table 2.1 Stationary response of the inelastic system to stationary ground excitation

S_o (cm ² /sec ³)	Reduction factor	σ/Y
10	0.66	0.46
20	0.66	0.60
40	0.65	0.80
100	0.65	1.21
200	0.64	1.71
300	0.64	2.15
800	0.63	4.04

Table 2.2 Optimum profile for a two-storey inelastic building frame.

	1/r	Area (cm ²)	P_{crit} (kN)
Single angle 25x25x3	204	1.52	7.6
Double angle 25x25x3	133	3.04	35.7

Table 3.1 Properties of the angle braces tested

Earthquake number

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Maximum elastic displacement	4.94	4.26	3.97	4.32	3.44	5.80	4.83	3.61	4.27	3.59	4.32	4.81	5.83	3.95	6.10	3.58	4.41	3.71	4.34	4.40
Maximum elasto-plastic displacement	2.21	5.11	5.54	3.06	2.77	9.70	4.68	3.58	5.02	3.81	2.39	2.62	4.49	4.64	5.33	5.50	3.94	3.95	4.73	7.72
Maximum displacement of the hardening model	2.08	3.31	3.35	2.65	2.45	3.76	2.79	2.48	3.11	2.48	2.34	2.72	2.61	3.18	3.62	3.09	2.93	2.98	2.99	2.91
F/F_y	1.09	2.32	2.36	1.65	1.45	2.77	1.80	1.49	2.12	1.49	1.34	1.73	1.61	2.18	2.62	2.10	1.94	1.99	1.99	1.91
predicted F/F_y	1.19	2.69	2.84	1.77	1.59	4.17	2.52	2.04	2.65	2.15	1.33	1.50	2.45	2.51	2.77	2.83	2.21	2.21	2.54	3.53
Energy dissipation of the hardening model divided by the energy dissipation of the elasto-plastic model	1.00	1.02	0.95	0.99	0.98	0.97	1.02	1.03	1.16	0.87	1.05	1.05	0.97	0.91	0.93	1.04	1.09	1.02	0.94	0.84

Table 4.1 Response of two hysteretic models to earthquake loading

Test	Graph	Material	μ	φ	Frequency	Pressure (N/mm ²)	Bolt	Number of cycles	Surface finish	Washer
G1	5.18(a)	Ground flat stock	0.46	20°	Quasi-static, 1Hz	22	1	20	Sand blasted	W/35/80/6/
G2	5.18(b)	Ground flat stock	0.56-0.62	18°	1Hz	31	2	20	Sand blasted	W/35/80/6/
G3	5.18(c)	Ground flat stock	0.25-0.47	14°	Quasi-static	48	4	20	Sand blasted	W/35/80/6/
G4	5.18(d)	Ground flat stock	0.18-0.24	5°	Quasi-static	48	4	10	Ground	W/35/80/6/
G5	5.18(e)	Ground flat stock	0.17-0.27	9°	Quasi-static	48	4	20	Ground	W/35/80/6/
G6	5.18(f)	Ground flat stock	0.15-0.19	9°	Quasi-static, 0.5Hz, 1Hz	23	6	10	Ground	W/50/115/6/
G7	5.18(g)	Ground flat stock	0.19-0.39	5°	Quasi-static	48	4	10	Ground	W/35/80/6/
G8	5.18(h)	Ground flat stock	0.19-0.34	15°	Quasi-static	48	4	20	Ground	W/35/80/6/
G9	5.18(i)	Ground flat stock versus ~~Ground flat stock~~	0.19-0.27	17°	Quasi-static	48	4	10	Ground	W/35/80/6/
C1	5.18(j)	Cast Iron	0.13-	20°	Quasi-static	44	4	6	Ground	W/38/85/8/
C2	5.18(k)	Cast Iron versus Ground flat stock	0.16-0.32	14°	Quasi-static	44	4	20	Ground	W/38/85/8/
N1	5.18(l)	Nitroy 40B	0.17-0.45	8°	Quasi-static	44	4	20	Unground	W/38/85/9/
N2	5.18(m)	Nitroy 40B versus Ground flat stock	0.23-0.39	8°	Quasi-static	42	6	10	Unground	W/38/85/9/
N3	5.18(n)	Nitroy 40B	0.18-0.23	8°	Quasi-static, 0.25Hz, 1Hz	42	6	10	Ground	W/38/85/9/

Table 5.1. Experiments on the single rotating joint

Test	Graph	Material	μ	ϕ	Frequency	Pressure (N/mm ²)	Bolt	Number of cycles	Surface finish	Washer
M1	5.18(o)	500K-Monel	0.14	9°	Quasi-static	23	4	10	Ground	W/50/115/9/
M2	5.18(p)	500K-Monel	0.15-0.22	9°	1Hz	23	4	14	Ground	W/50/115/9/
F1	5.18(q)	3501F	0.47	18°	1Hz	31	2	20	Unground	W/35/80/6/
F2	5.18(r)	FF	0.18-0.10	8°	Quasi-static	48	5	14	Unground	W/24/90/9/
F3	5.18(s)	FF versus Ground flat stock	0.24	7°	Quasi-static, 1Hz, 2Hz	23	6	10	Unground	W/50/115/9/
F4	5.18(t)	Proposed friction material	0.10	8°	Quasi-static, 0.5Hz, 1Hz	104	5	10	Unground	W/27/65/5/
F5	5.18(u)	3701F	0.16	8°	Quasi-static, 0.5Hz, 1Hz	39	6	10	Unground	W/30/85/10/
F6	5.18(v)	3701F	0.14	8°	Quasi-static, 0.5Hz, 1Hz	58	5	10	Unground	W/30/85/10/
F7	5.18(w)	3904F	0.15-0.11	8°	Quasi-static, 1Hz	58	6	10	Unground	W/30/85/4/

(continued)

Number	Bolt	Tightening torque(Nm)
1	M16 HSFG	260
2	M16 grade 10.9	360
3	M20 HSFG	490
4	M20 grade 10.9	690
5	M24 grade 10.9	1200
6	M24 grade 10.9	810

Table 5.2. Different bolts used in the experiment

Material	Chemical composition
Ground flat stock	Carbon 1%, Silicon 0.4%, Manganese 1.1% Chromium 0.4%, Tungsten 0.4%, Vanadium 0.25%
MM Cast Iron, Grade 14	Carbon 3.25%, Silicon 2%, Manganese 0.5%, Phosphorus 1%
Nitroy 40B	Chromium 3%, Molybdenum 3%
500k Monel	Chromium 15%, Nickel 3%, Manganese 1%, Carbon 0.2%, Silicon 1%
Ferodo Friction Materials	
3501F	Random base of synthetic fibre
FF	Flexible, solid woven, based on cotton fibre
3701F	Random base of synthentic fibre. A rigid moulded friction material which is mottled slate grey in appearance; it is a non-metallic asbestos free material having a random base of synthetic fibre and uses a specially developed resin which contributes to both the strength and performance of the material.
3904F	Having a basis of short filaments of steel and nonmetallic substances in a random dispersion.

Table 5.3 Brief chemical composition of the different materials tested

Material	Brinell hardness
Ground flat stock (before hardening)	260
Ground flat stock (after hardening)	830
Black mild steel	220
Bright mild steel	230
Gr.14 Cast Iron	500
Nitroloy 40B (after hardening)	950
500K-Monel	405

Table 5.4. Hardness of the different materials tested

Specific gravity	1.62
Ultimate tensile strength	15.6 MN/m ²
Ultimate compressive strength	118 MN/m ²
Ultimate shear strength	15.4 MN/m ²
Gear tooth strength	40.5 MN/m ²
Thermal conductivity	0.529 W/m ^o C

Table 5.5. Physical properties of the 3701F Ferodo Friction material

Material	μ	φ	Frequency	Pressure (N/mm ²)	Number of cycles	Surface finish	Washer
Ground flat stock	0.3	10°	Quasi-static, 1Hz, 2Hz, 4Hz	35	20	Sand blasted	W/20/44/3/
3904F	0.15	10°	Quasi-static	19	5	Unground	W/10/54/4/

Table 5.6 Experiments on a small rotating device

Maximum possible rotation of the device

D_i = Initial diagonal length = 42.42cm

D_e = Length of elongated diagonal = 52.41cm

D_s = Length of shorter diagonal = 29.19cm

d_e = Increase in length of the long diagonal = $D_e - D_i = 9.99$ cm

d_s = Reduction in length of the short diagonal = $D_i - D_s = 13.23$ cm

$d = d_s - d_e = 3.24$ cm

φ = Relative rotation of plates = 30°

Interstorey deflection = $9.99\sqrt{2} = 14.13$ cm

Rotation of the device for an interstorey deflection of 80mm

$d_e = 8/\sqrt{2} = 5.66$ cm

$D_e = d_e + D_i = 48.08$ cm

$D_s = 35.89$ cm

$d_s = D_i - D_s = 6.53$ cm

$d = d_s - d_e = 0.87$ cm

$\varphi = 16^\circ$

Table 5.7 Two typical deformations of the device

Earthquake	Maximum reached displacement (mm)
1	9.02/-8.50
2	7.64/-7.58
3	7.87/-7.64
4	9.99/-9.82
5	5.06/-5.28
6	4.71/-4.77
7	4.48/-4.99
8	6.81/-6.98
9	6.81/-6.69
10	3.61/-4.13

Table 6.1 Maximum displacements reached by the unbraced frame for the 6% excitation level

Earthquake	Maximum reached displacement (mm) (a)	Maximum reached displacement (mm) (b)
1	9.16/-9.79	15.2/-17.55
2	9.65/-9.01	16.2/-17.66
3	15.48/-11.23	25.91/-17.75
4	8.56/-10.11	13.59/-16.29
5	18.07/-12.36	21.08/-23.44
6	7.12/-10.69	14.39/-17.49
7	9.51/-8.87	12.87/-13.84
8	8.87/-10.37	21.03/-20.91
9	10.43/-9.39	15.86/-18.04
10	11.06/-10.48	16.63/-16.40

Table 6.2 Maximum displacements reached by the braced frame incorporating the rotating device for
(a) 40% excitation level
(b) 65% excitation level

General case	$\frac{12EI}{L^3} \left(\frac{L}{S_c G_c} - \frac{N}{D} \right), \text{ where:}$ $N = \left(1 - \frac{EI_f a^2}{GS_f} \right) (2 \cosh(aL) - 2R_2 \sinh(aL) + R_1 \sinh(aL))$ $D = a S_c G_c (\sinh(aL) (1 - R_1 R_2) + \cosh(aL) (R_1 - R_2))$
Both ends fixed against warping	$\frac{12EI}{L^3} \left(\frac{L}{S_c G_c} - \frac{4EI_f \operatorname{atanh}(aL/2)}{S_c^2 G_c^2} \right)$
One end fixed against warping and the other free	$\frac{12EI}{L^3} \left(\frac{L}{S_c G_c} - \frac{2EI_f \operatorname{atanh}(aL)}{S_c^2 G_c^2} \right)$
Both ends free against warping	$\frac{12EI}{L^2 S_c G_c}$
Special case $G_c = 0$	$\frac{12EI}{L^3} \left(\frac{L^3}{8EI_f} + \frac{L}{4GS_f} \right)$

Table A1. ϕ_s according to the boundary conditions

Floor level	Stiffness distribution		
8	0.23	0.27	0.31
7	0.42	0.45	0.47
6	0.58	0.59	0.60
5	0.72	0.71	0.70
4	0.83	0.81	0.79
3	0.91	0.90	0.87
2	0.97	0.96	0.95
1	1.00	1.00	1.00
	T=0.4 sec	T=1.00 sec	T=2.00 sec

Table B1. Optimum distribution of stiffness derived from a smooth acceleration spectrum

3 storey building frame, T= 1 sec, 2% critical damping				
Floor level	Stiffness distribution			
3	0.53	0.48	0.44	
2	0.82	0.79	0.75	
1	1.00	1.00	1.00	
	1.00	1.76	3.37	ductility
	1.00	2.00	3.00	Earthquake factor

4 storey building frame, T= 1 sec, 2% critical damping				
Floor level	Stiffness distribution			
4	0.45	0.41	0.38	
3	0.70	0.64	0.63	
2	0.89	0.85	0.82	
1	1.00	1.00	1.00	
	1.00	1.67	2.10	ductility
	1.00	2.00	3.00	Earthquake factor

Table B2. Optimum distribution of stiffness derived through nonlinear dynamic analysis

Structure			
	Par	Lin. 1	Lin. 2
T ₁	1.01 sec	1.06 sec	1.00 sec
T ₂	0.41 sec	0.41 sec	0.36 sec
T ₃	0.26 sec	0.26 sec	0.23 sec

Table B3. The first three periods of the structures

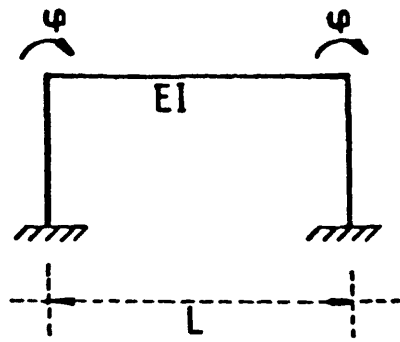


Fig. 1.1 A single-storey frame

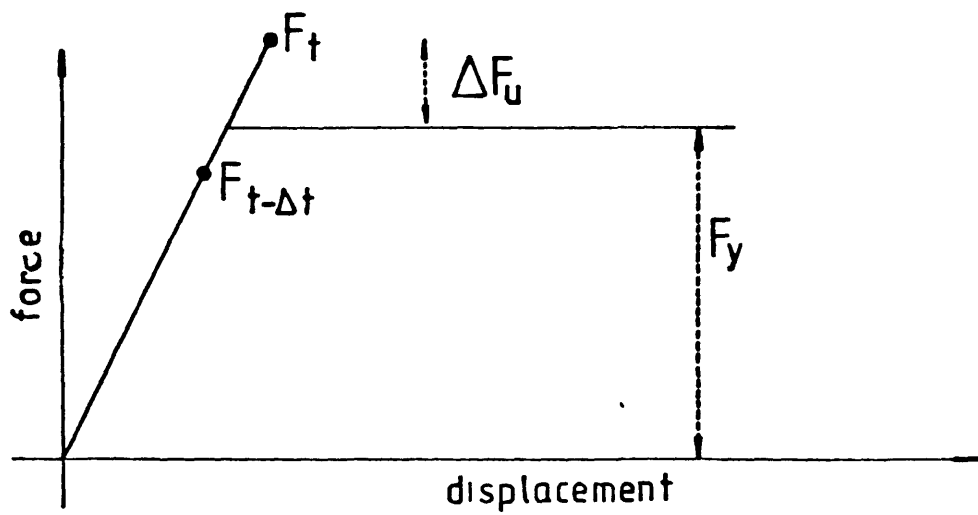


Fig. 1.2 Out-of-balance forces in the dynamic response

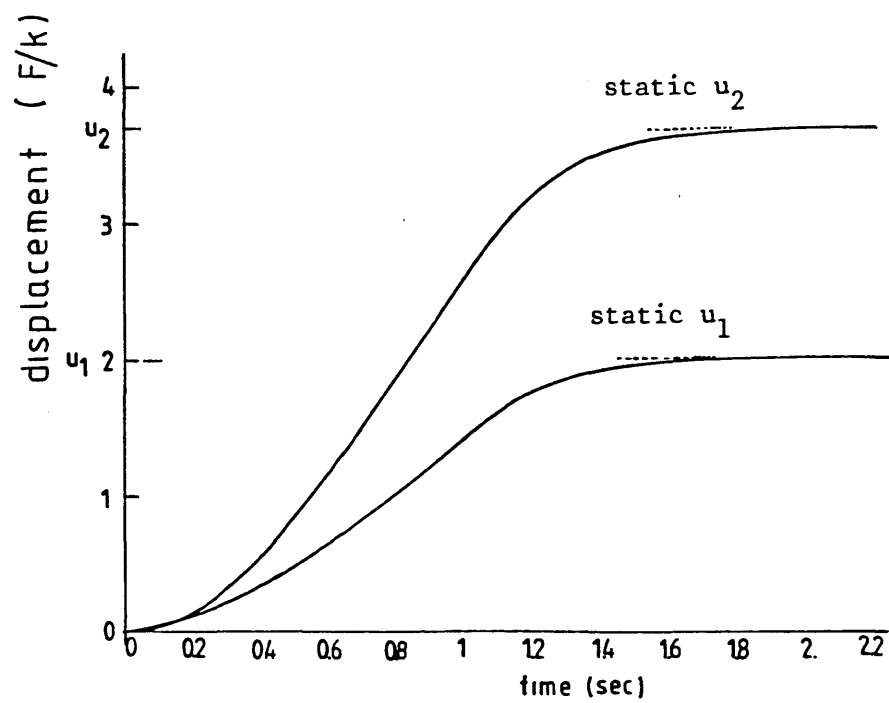


Fig. 1.3 Static solution through dynamic analysis

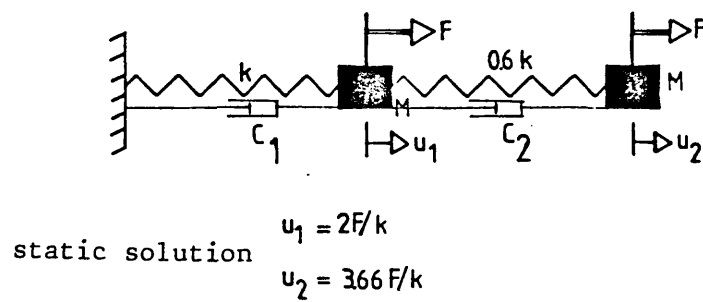


Fig. 1.4 The two-degrees-of-freedom system examined

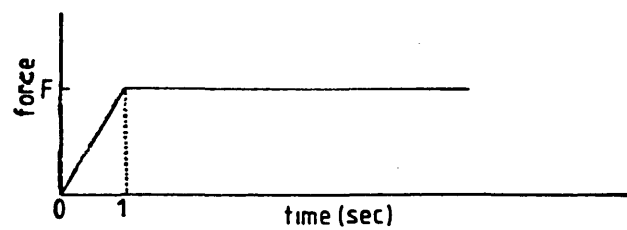


Fig. 1.5 Time history of the applied force

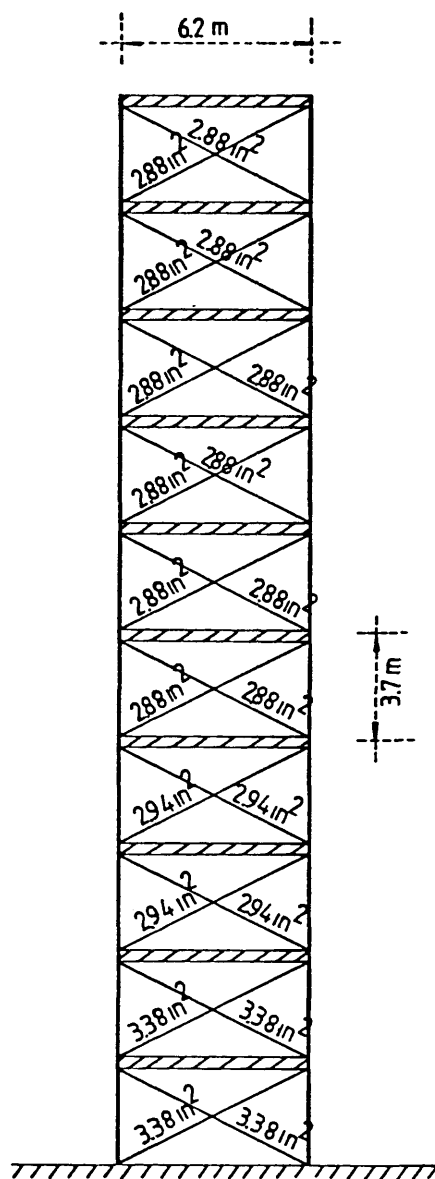


Fig. 1.6 The multi-storey braced frame examined

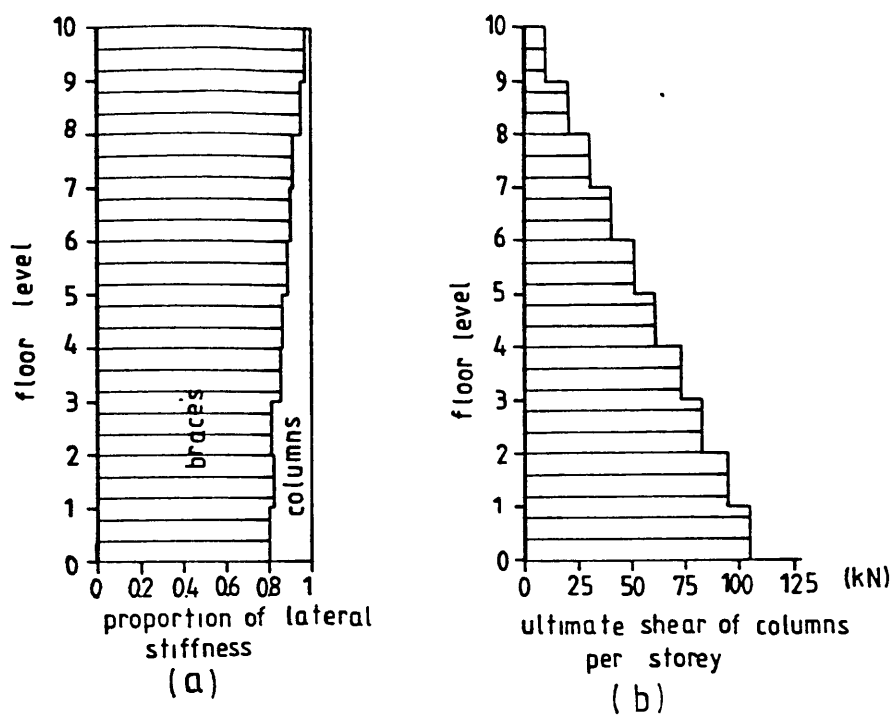


Fig. 1.7 Structural properties of the frame

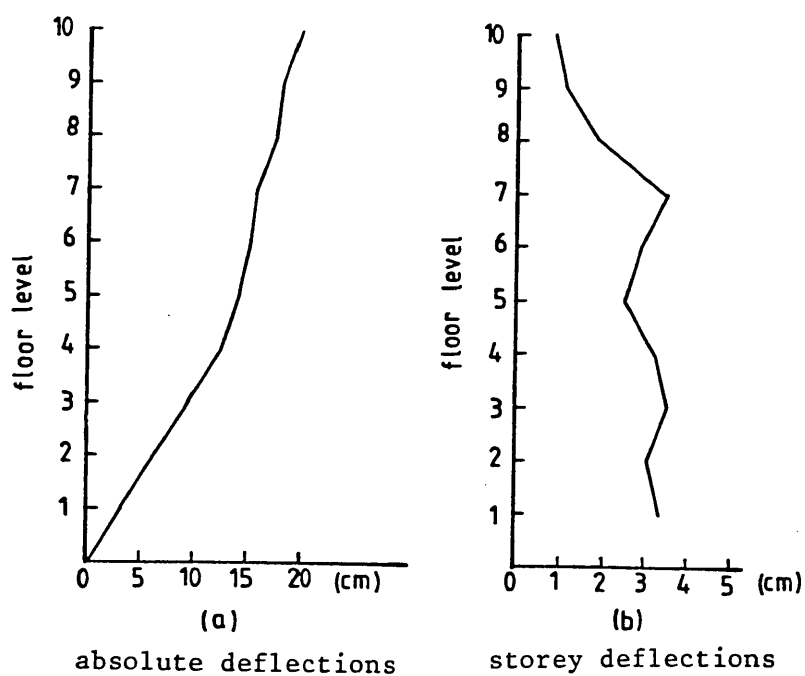


Fig. 1.8 Response of the frame

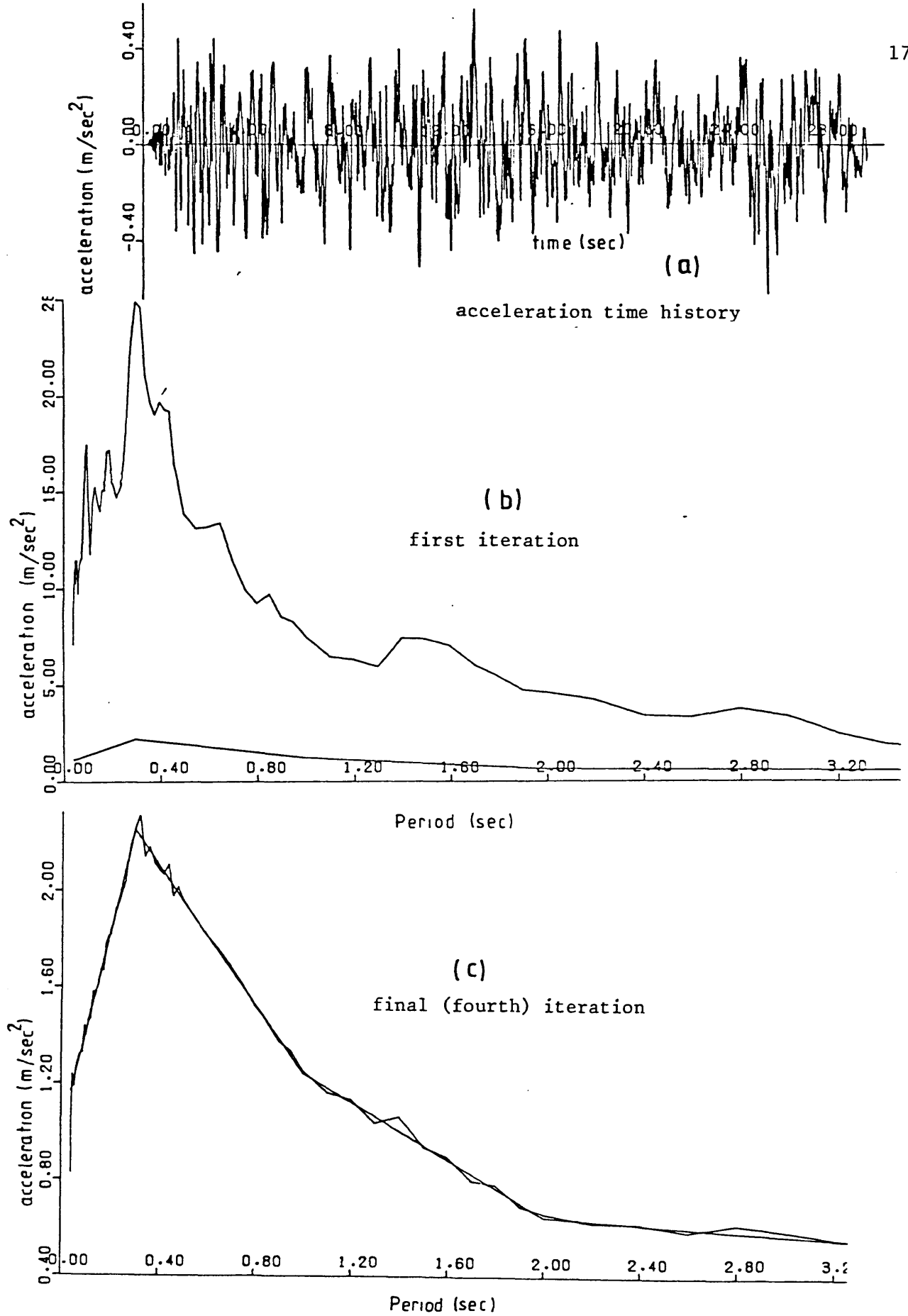


Fig. 2.1 An artificial earthquake having a specific acceleration spectrum

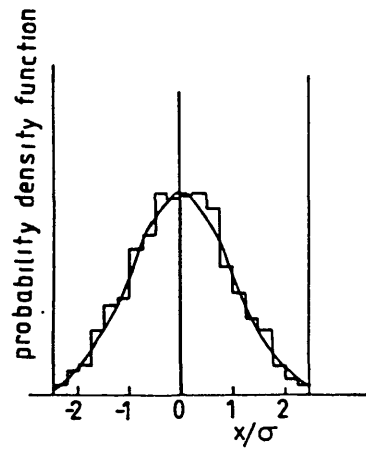


Fig. 2.2 Distribution of 2000 independent random numbers

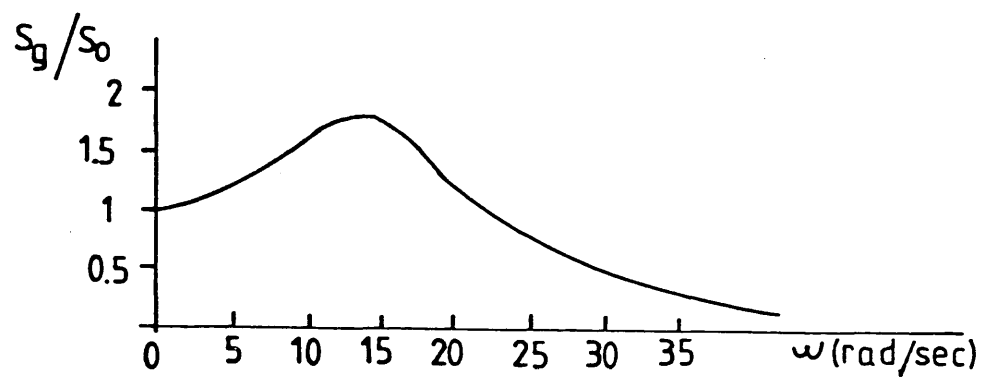


Fig. 2.3 The Kanai-Tajimi power spectral density function

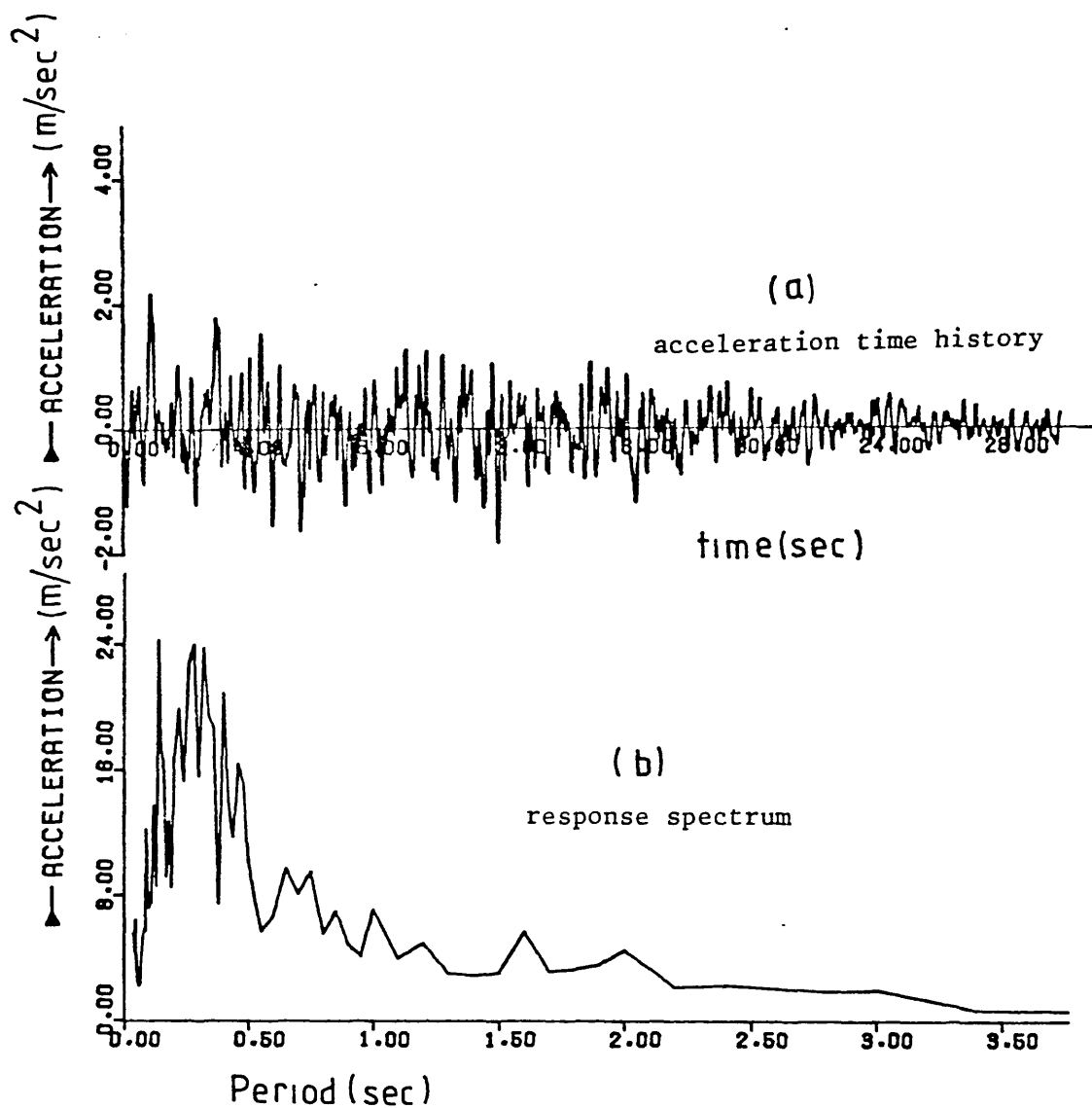


Fig. 2.4 An artificial earthquake generated using the second-order filter

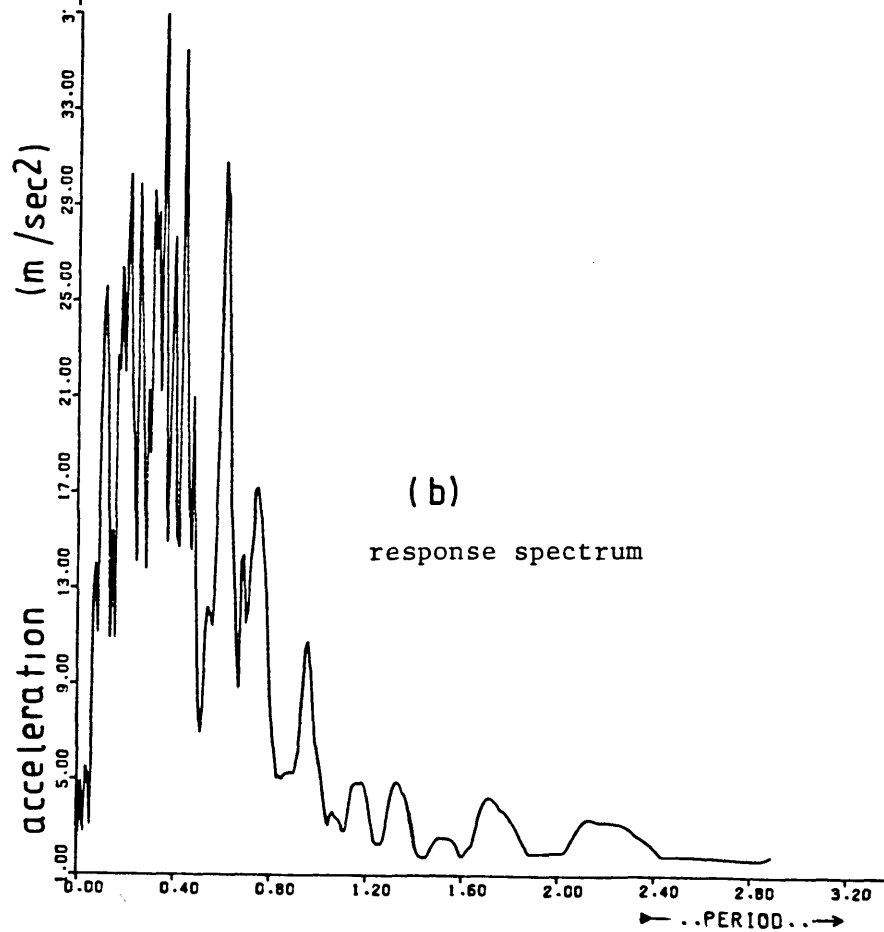
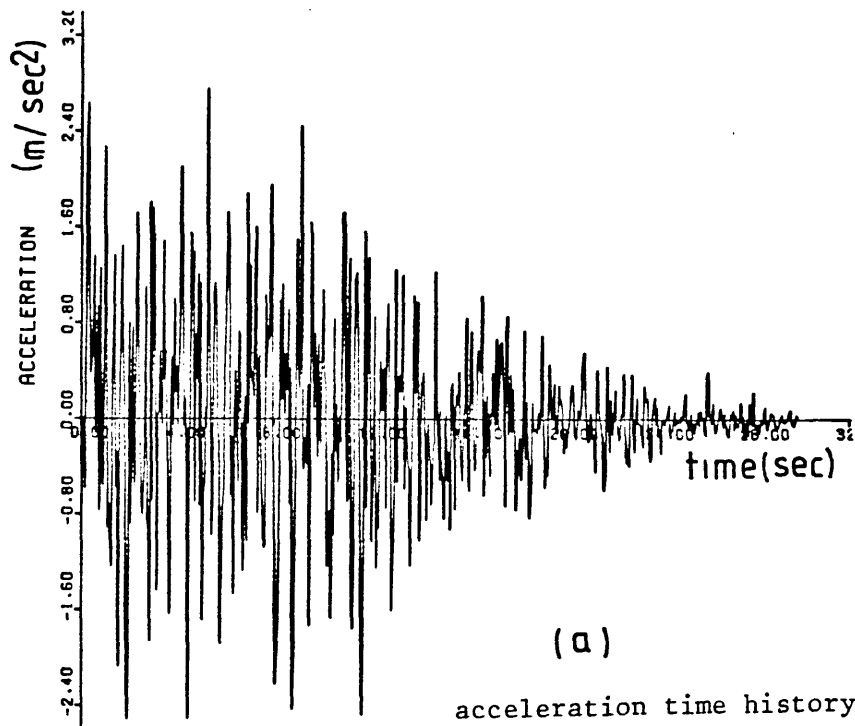


Fig. 2.5 An artificial earthquake generated using the autoregressive filter

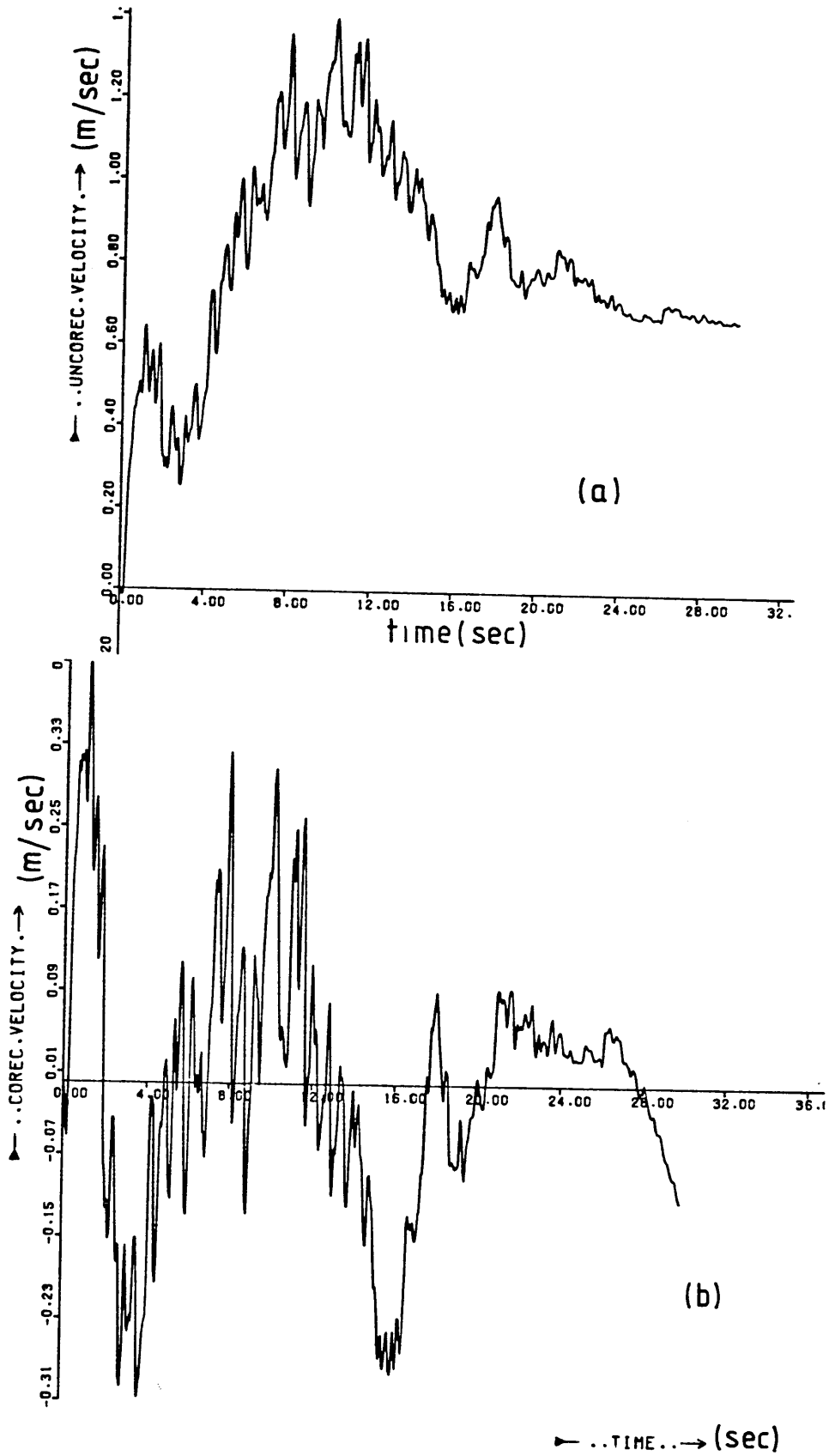


Fig. 2.6 Uncorrected and corrected velocity time history

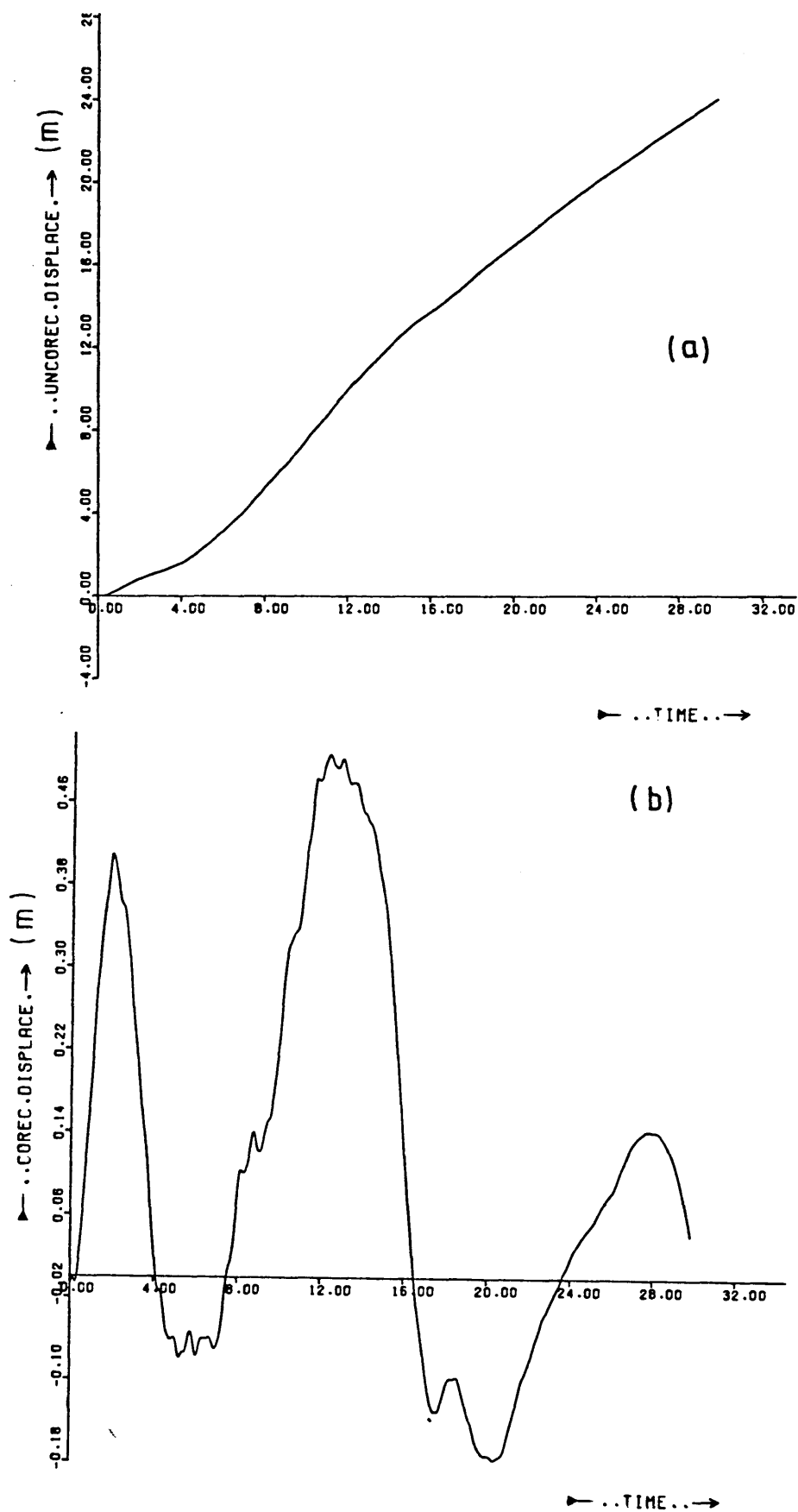


Fig. 2.7 Uncorrected and corrected displacement time history.

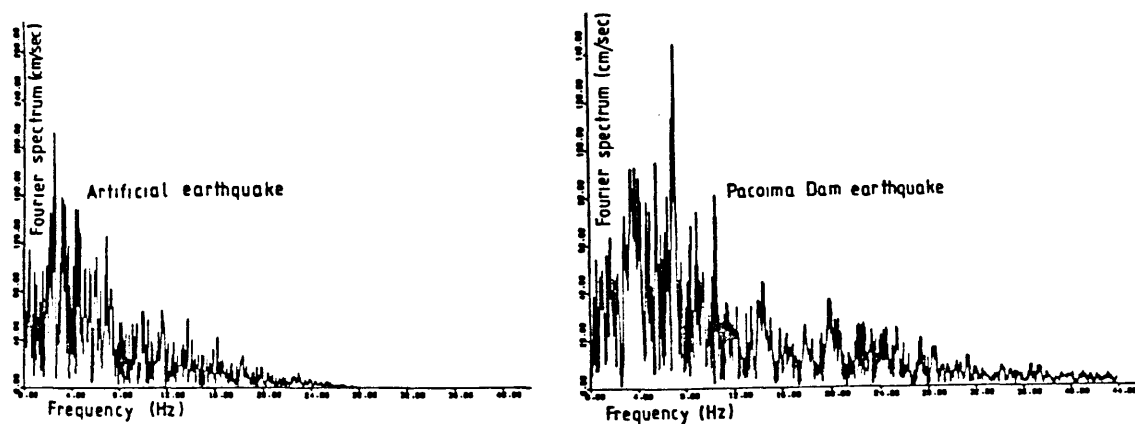


Fig. 2.8 Fourier amplitude spectrum of two earthquake records

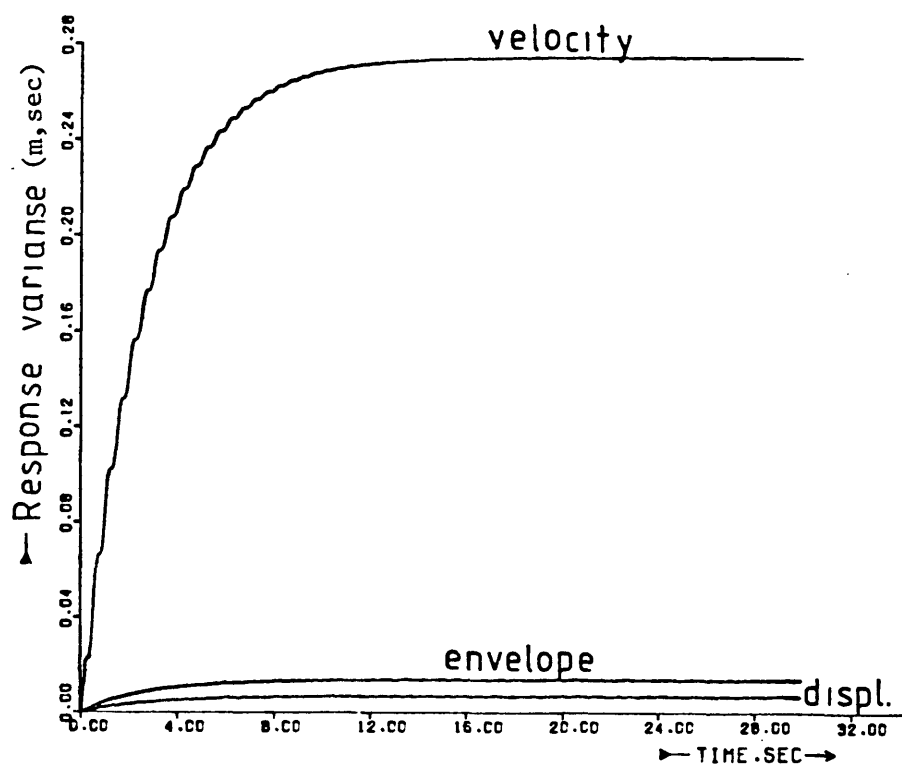


Fig. 2.9 Non-stationary response of a single-degree-of-freedom system

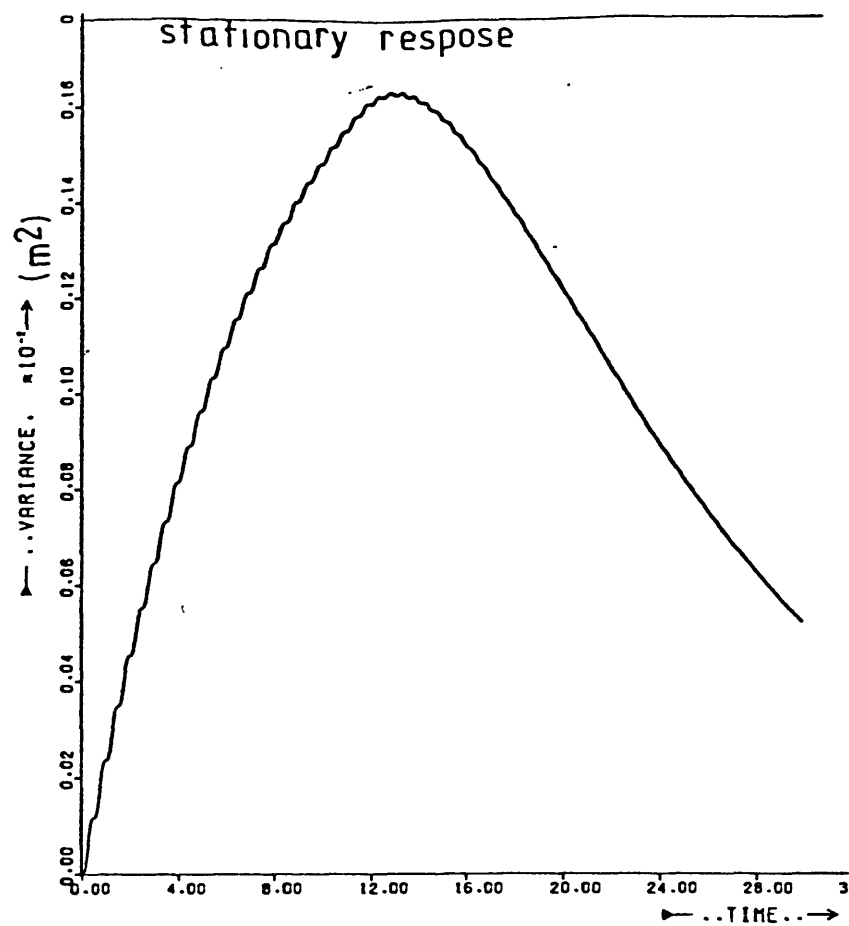


Fig. 2.10 Non-stationary response of a single-degree-of-freedom system using an intensity function

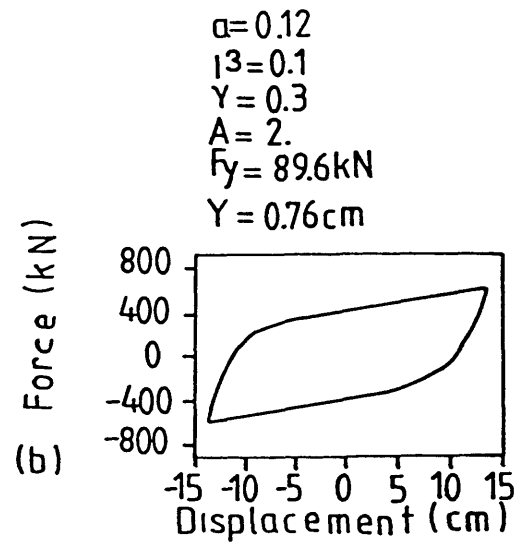
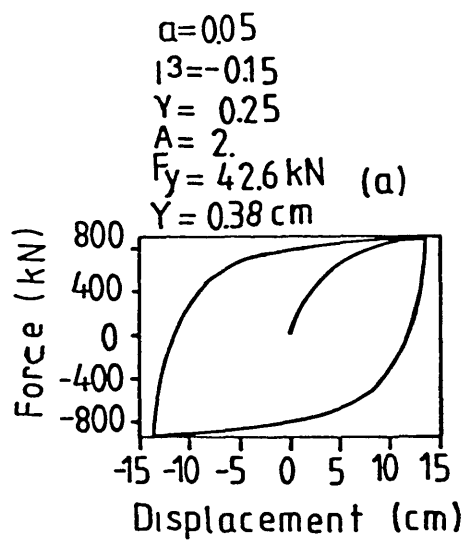


Fig. 2.11 Two rate-independent generated hysteretic models

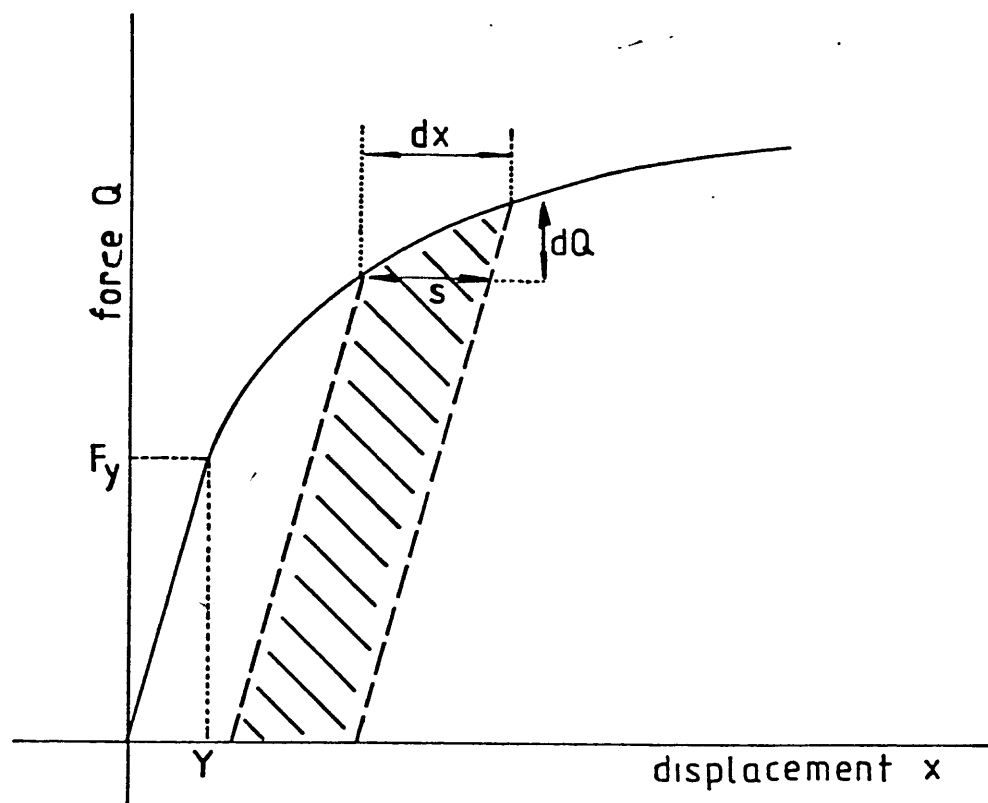


Fig. 2.12 Energy dissipation of an inelastic system

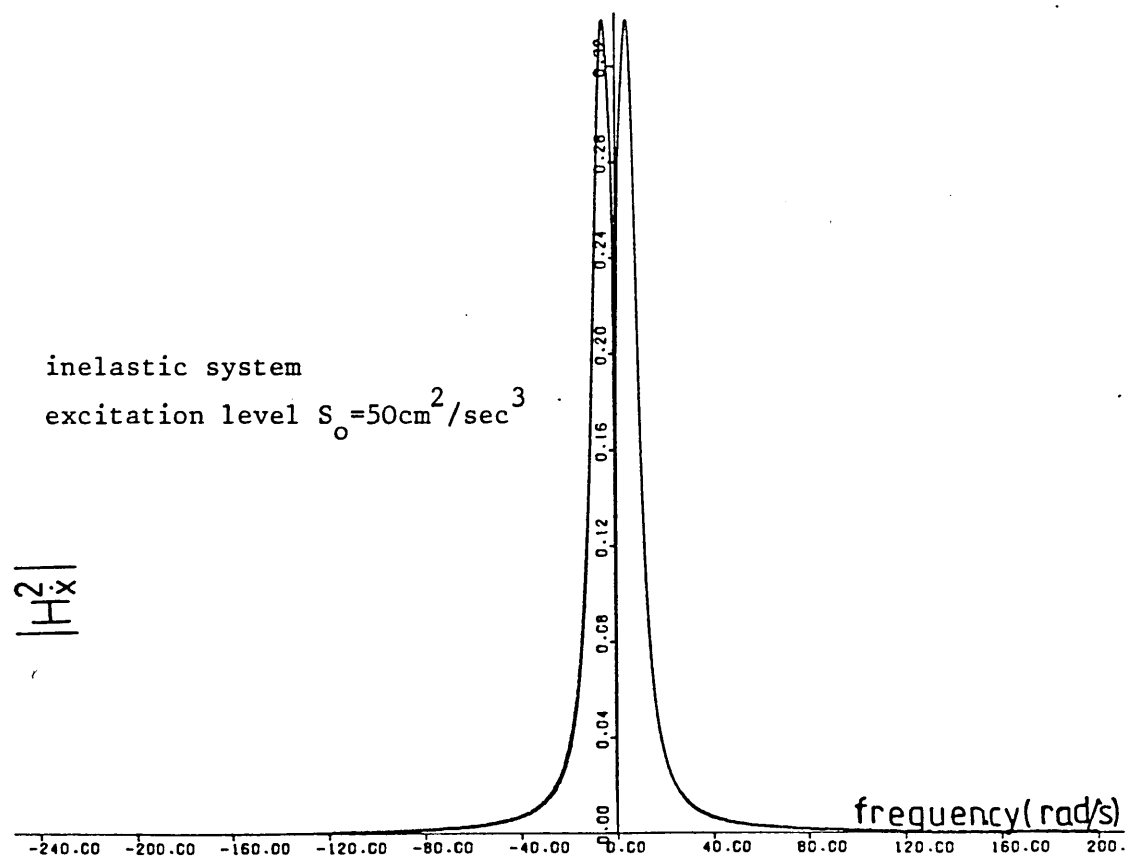


Fig. 2.13 Typical frequency response function of the hysteretic model

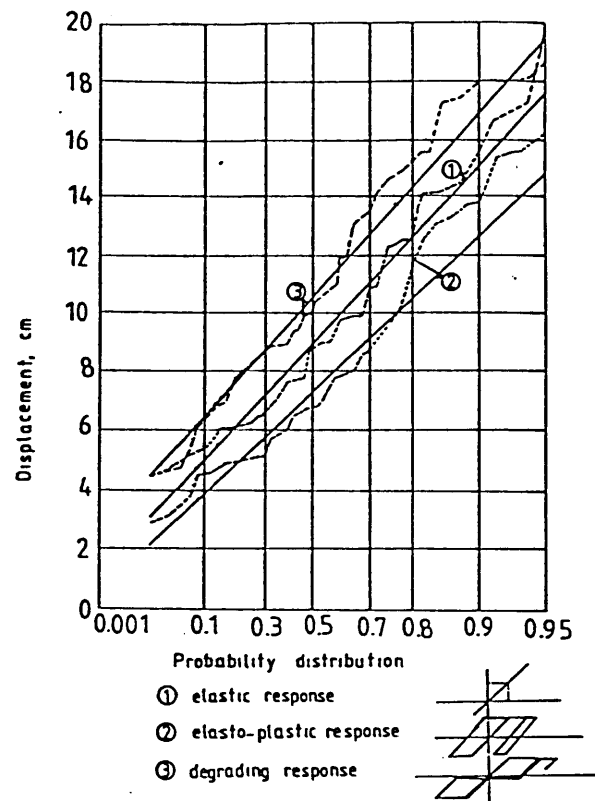


Fig. 2.14 Probability distribution for extreme values of relative displacement

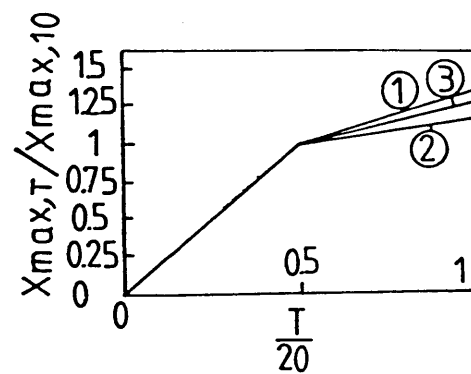


Fig. 2.15 Duration effect on mean peak response of the structures

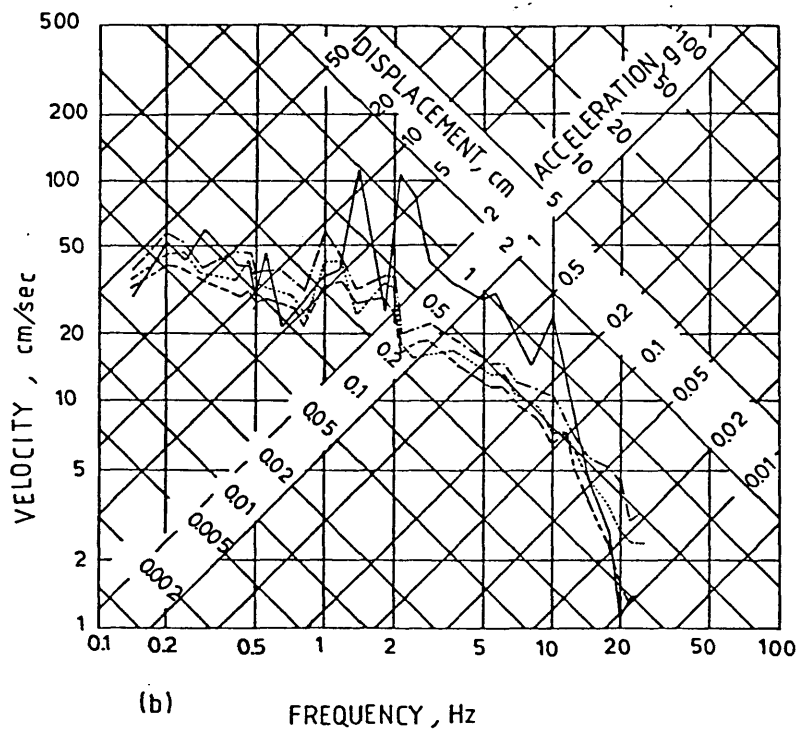
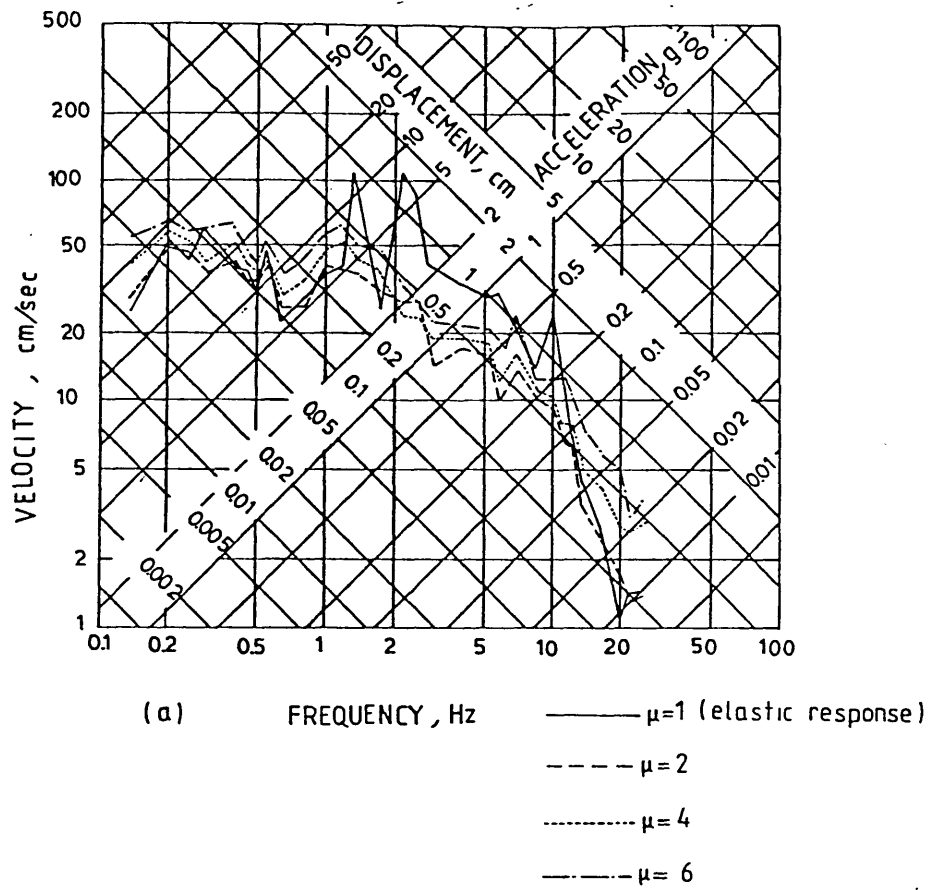


Fig. 2.16 Relative displacement response spectra

(a) Elasto-plastic system

(b) Bi-linear system

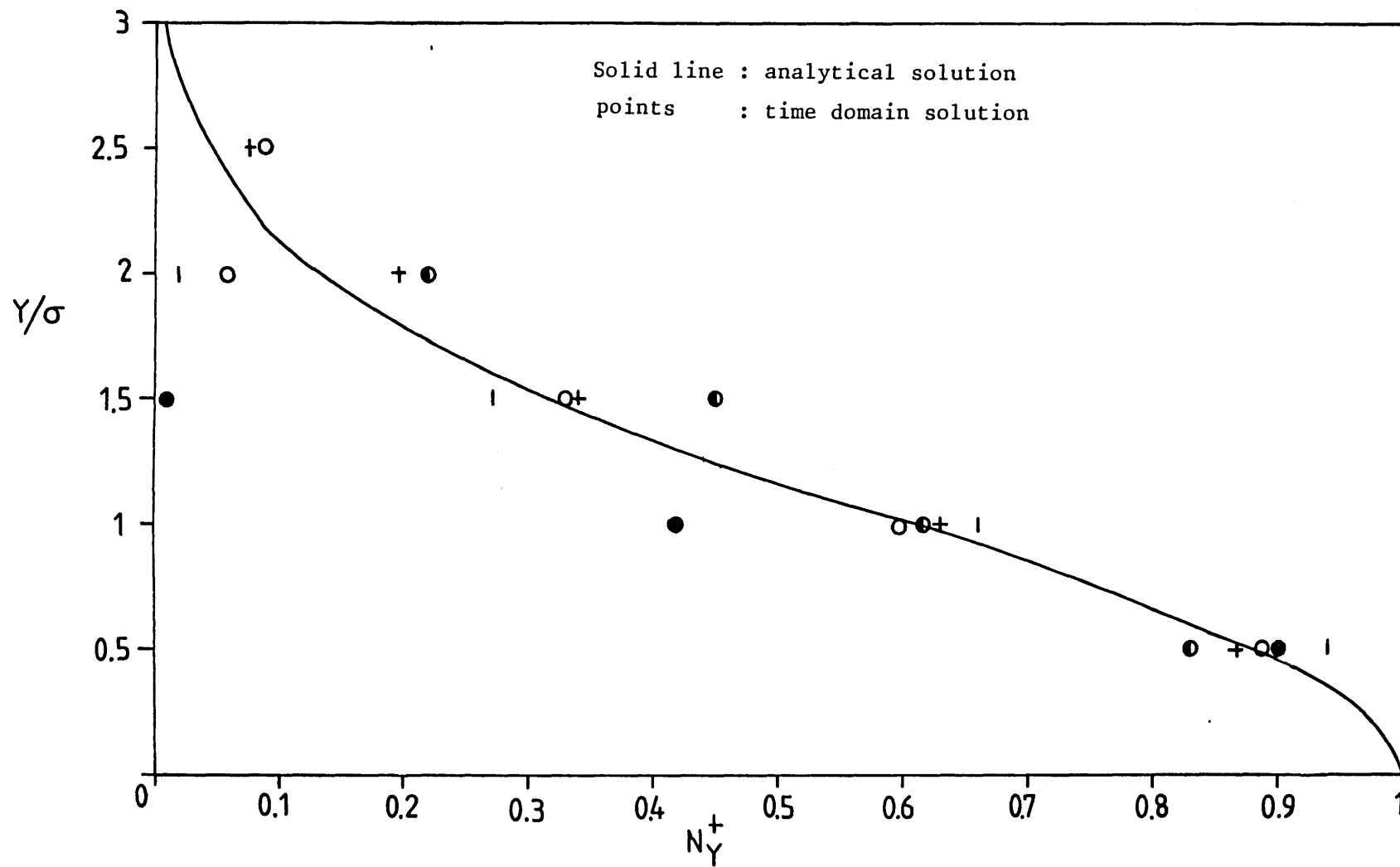


Fig. 2.17 Number of positive crossings per second for the 1 Hz system with 1% critical damping

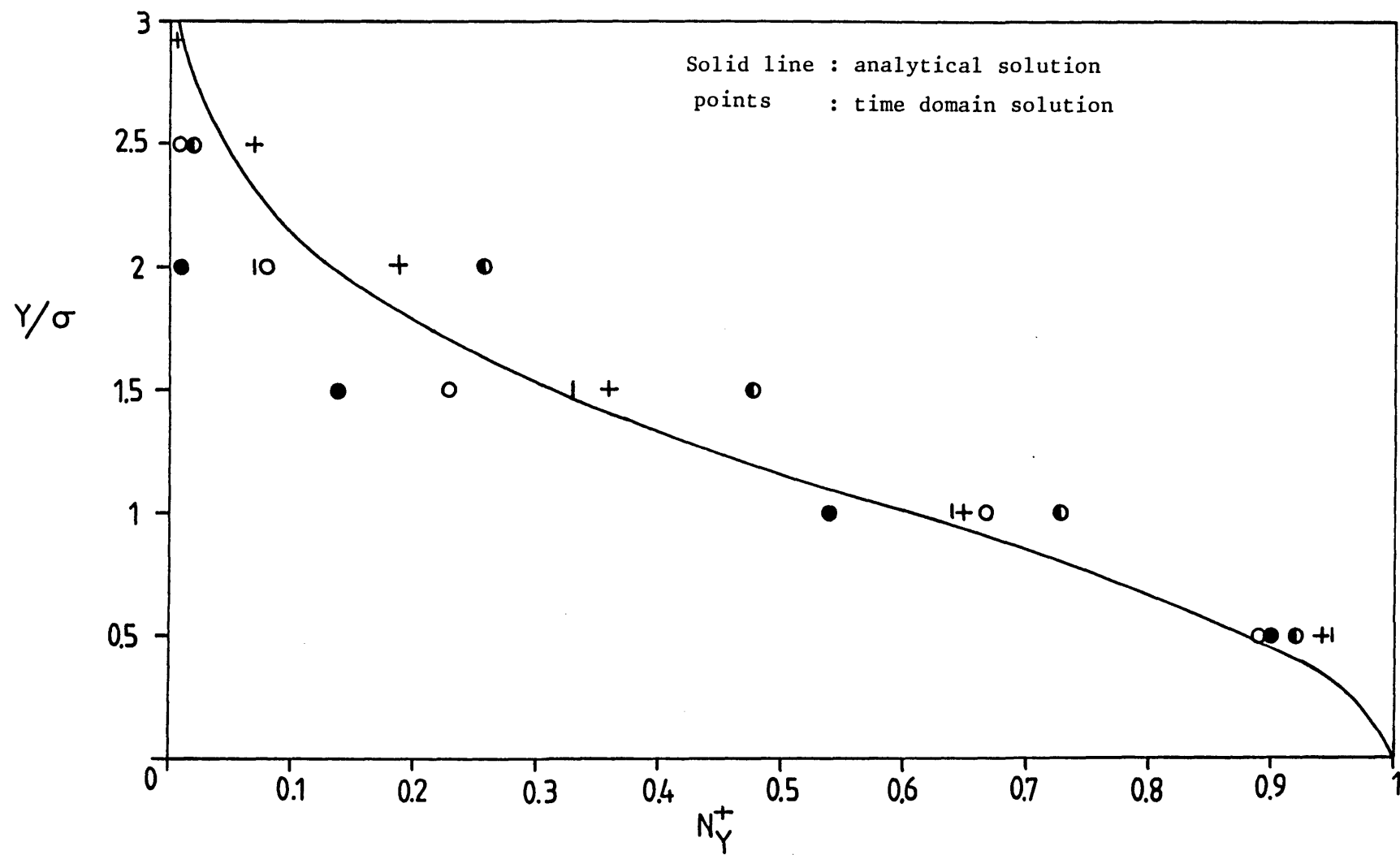


Fig. 2.18 Number of positive crossings per second for the 1 Hz system with 2% critical damping

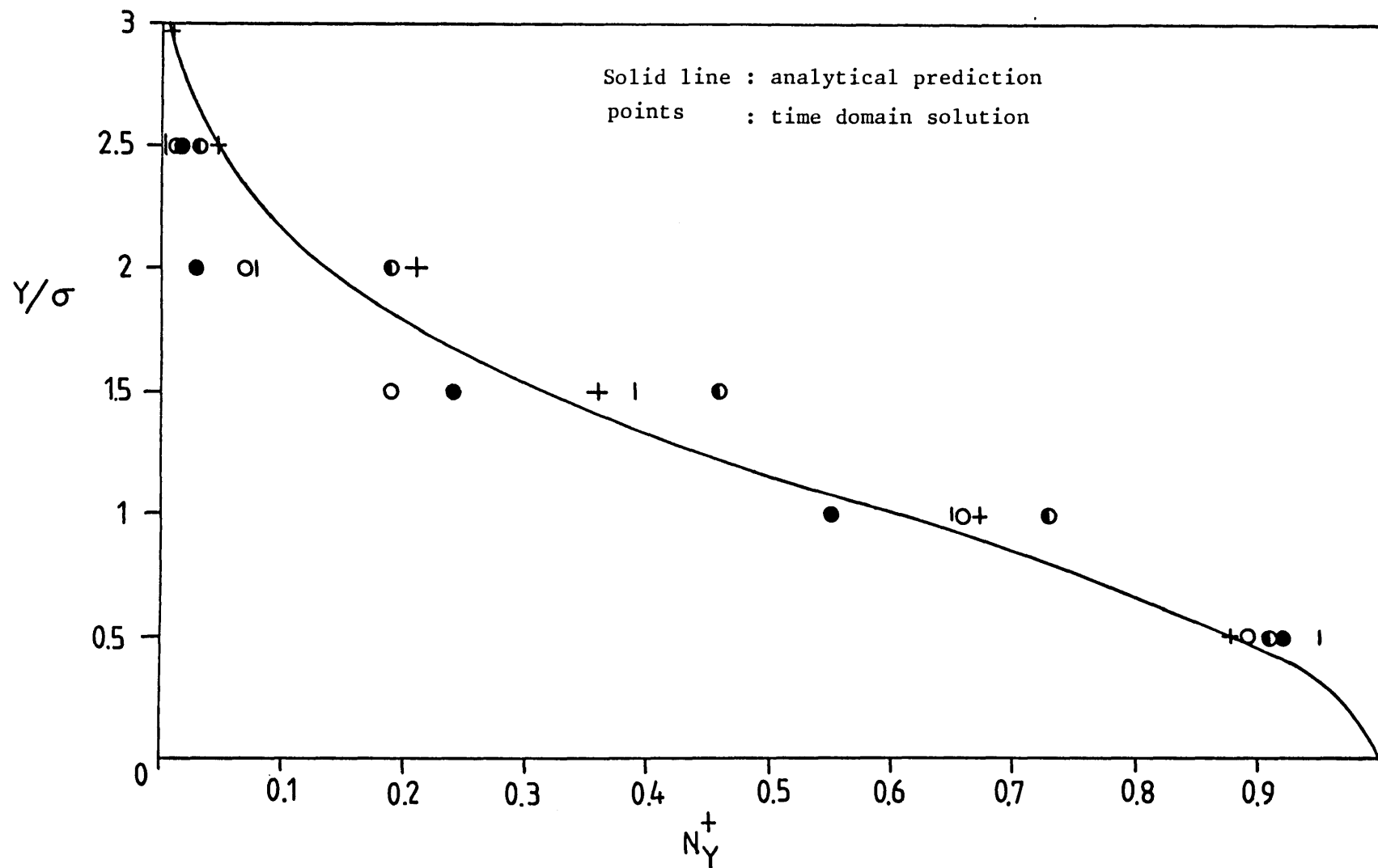


Fig. 2.19 Number of positive crossings per second for the 1 Hz system with 3% critical damping

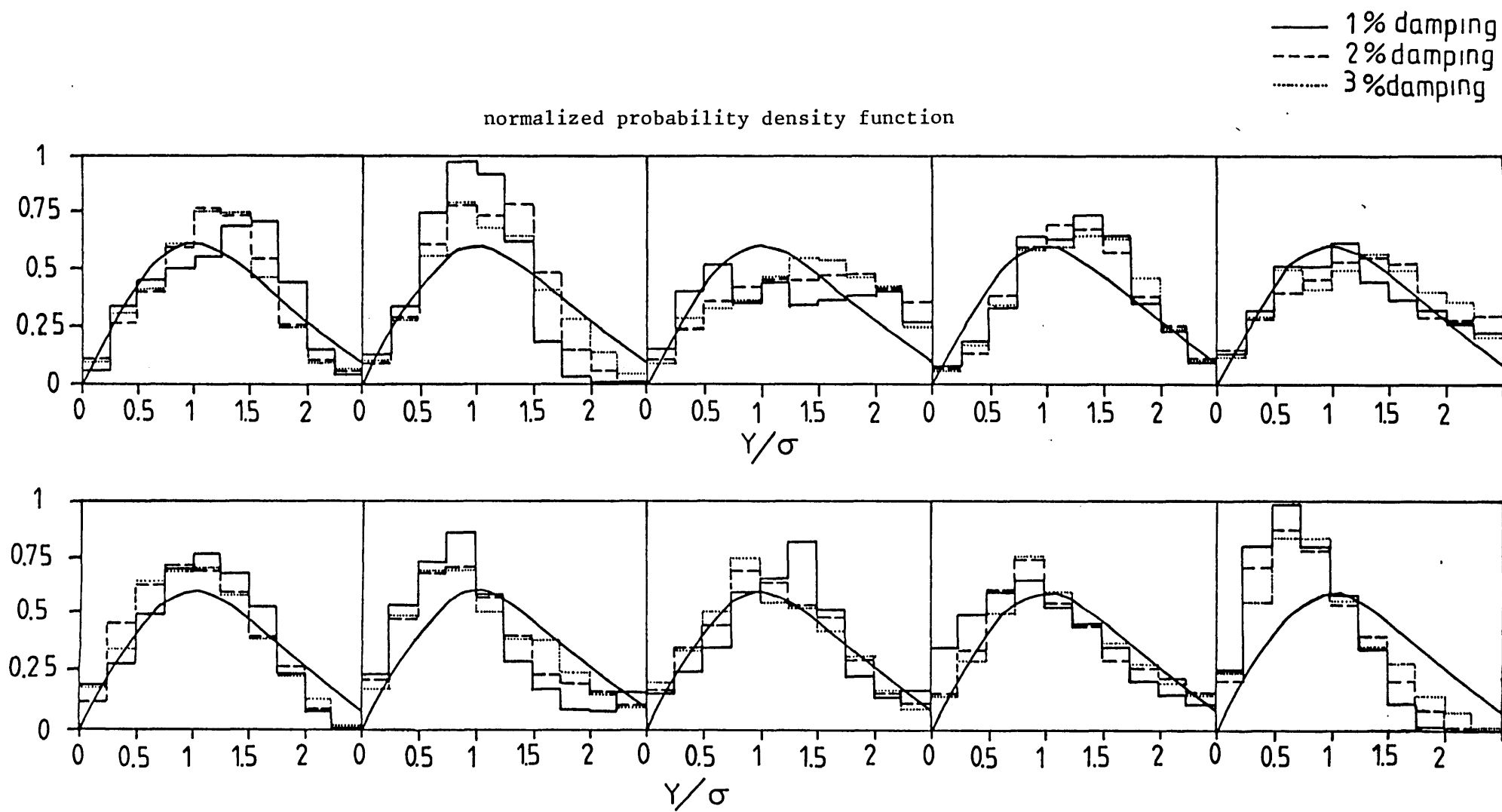


Fig. 2.20 Envelope distribution of the response for ten artificial earthquakes generated using the autoregressive filter

normalized probability density function

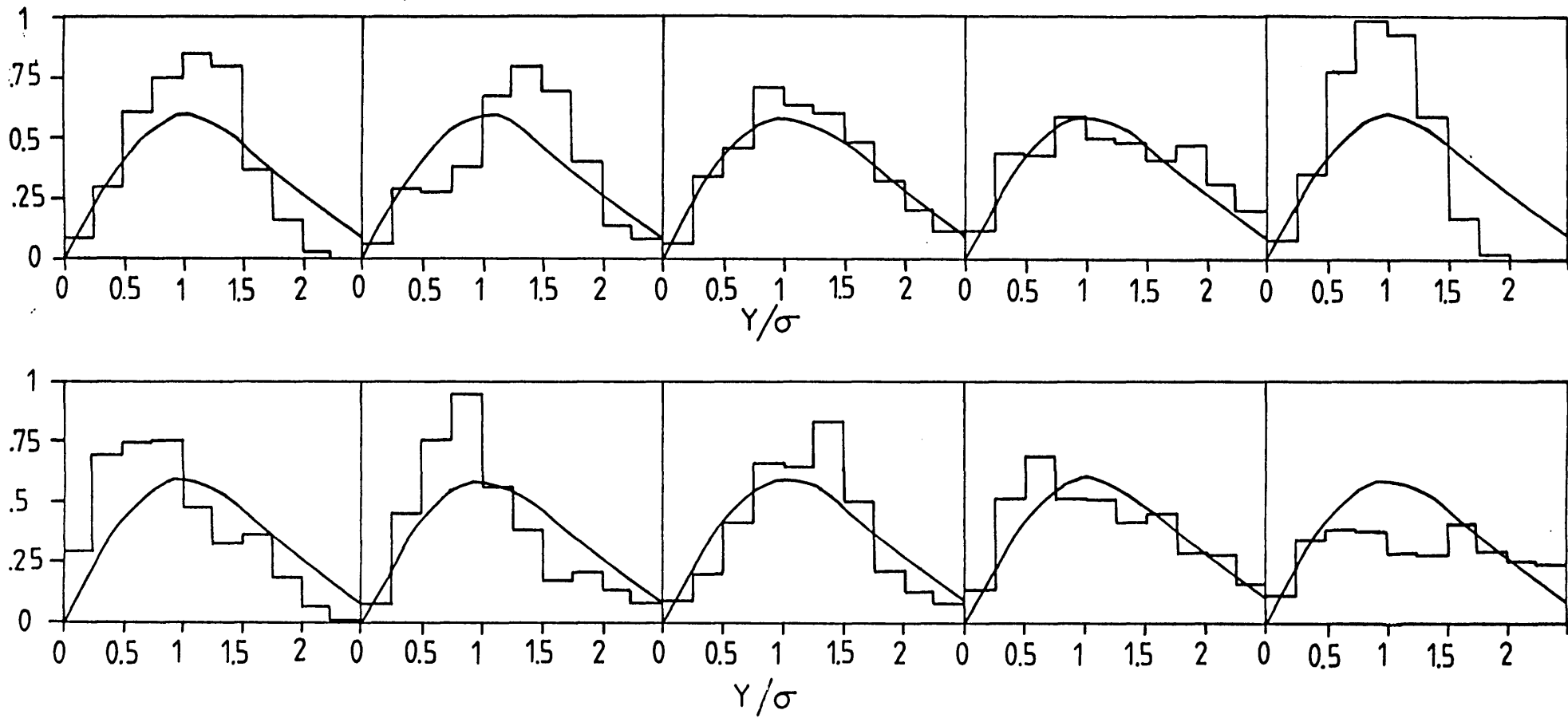


Fig. 2.21 Envelope distribution of the response for ten artificial earthquakes generated using the second order filter

normalized probability density function

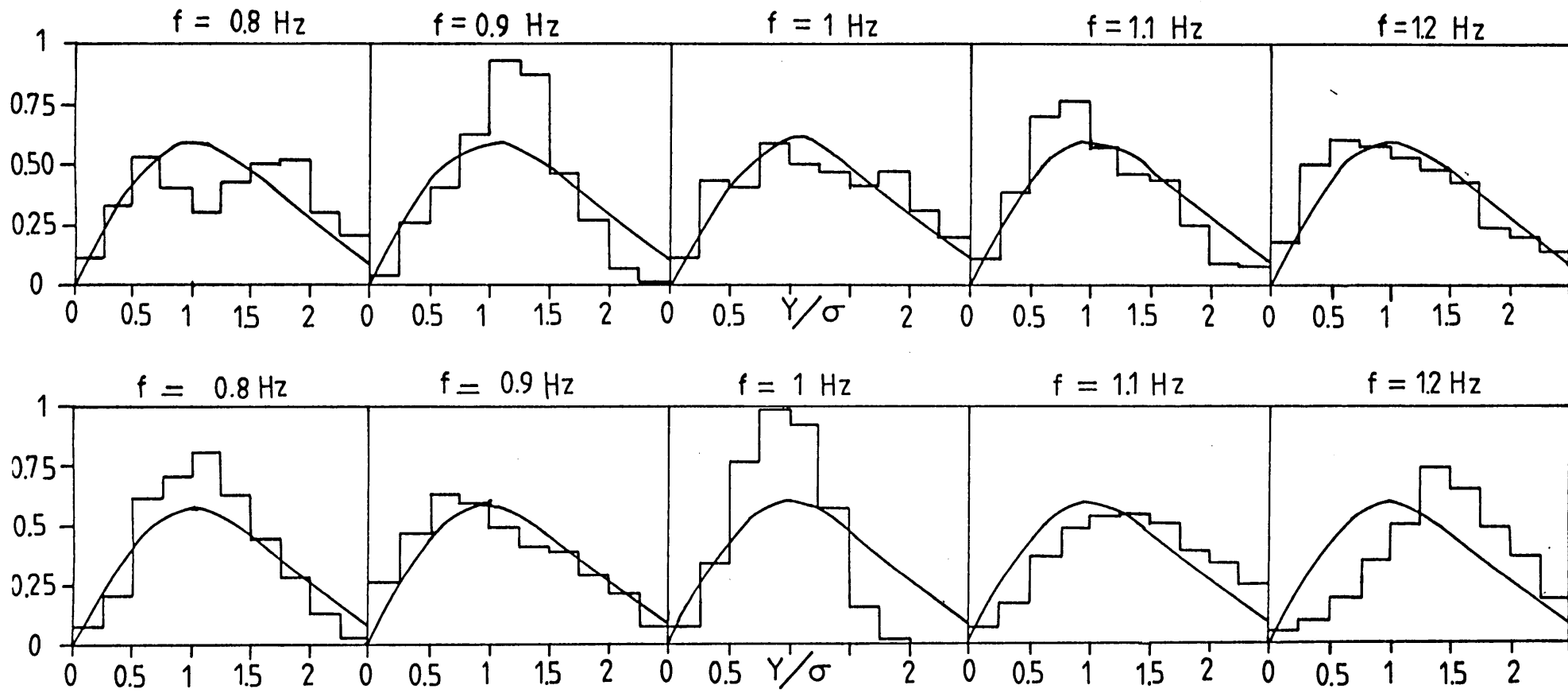


Fig. 2.22 Envelope distribution of the response for a system having a slightly different frequency

normalized probability density function

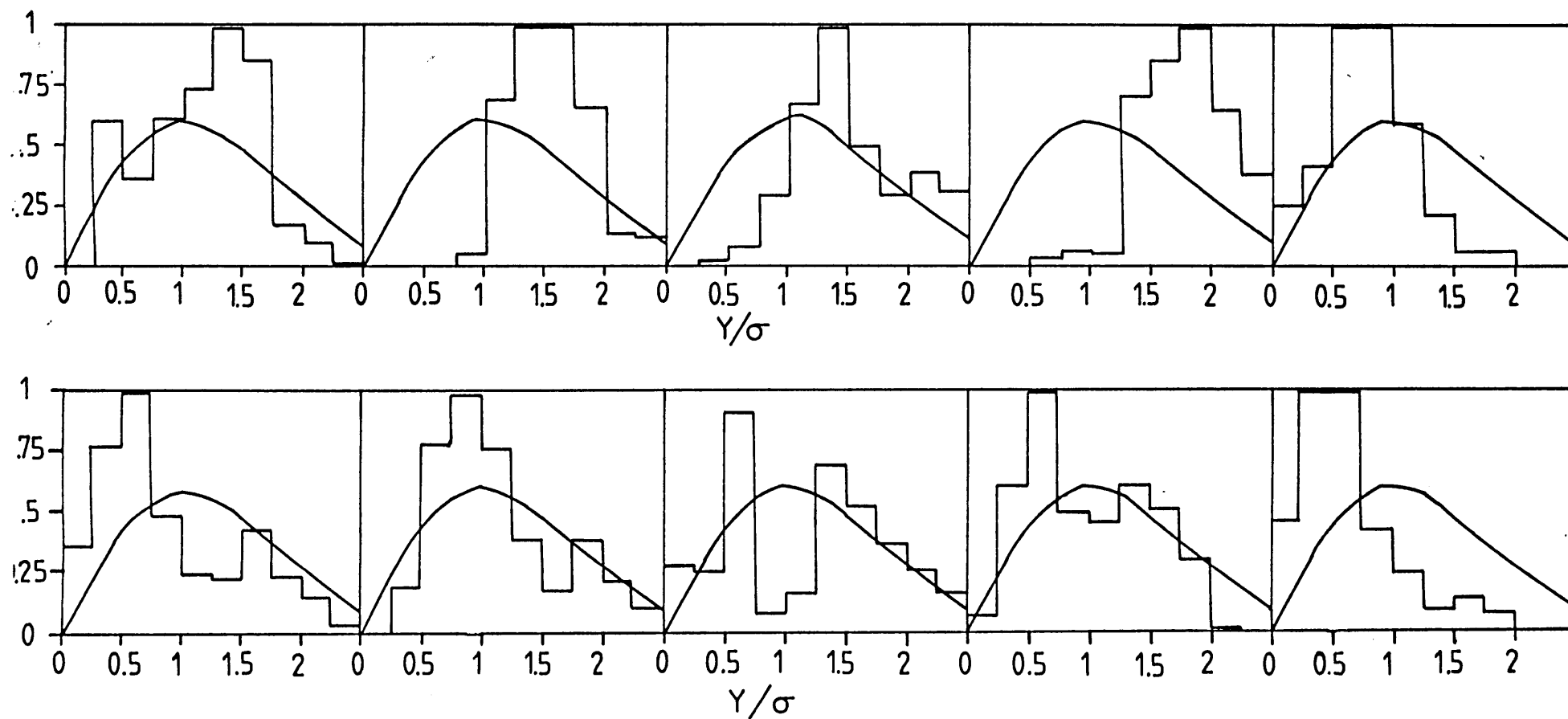


Fig. 2.23 Envelope distribution of the response using earthquakes having a non-constant variance intensity function

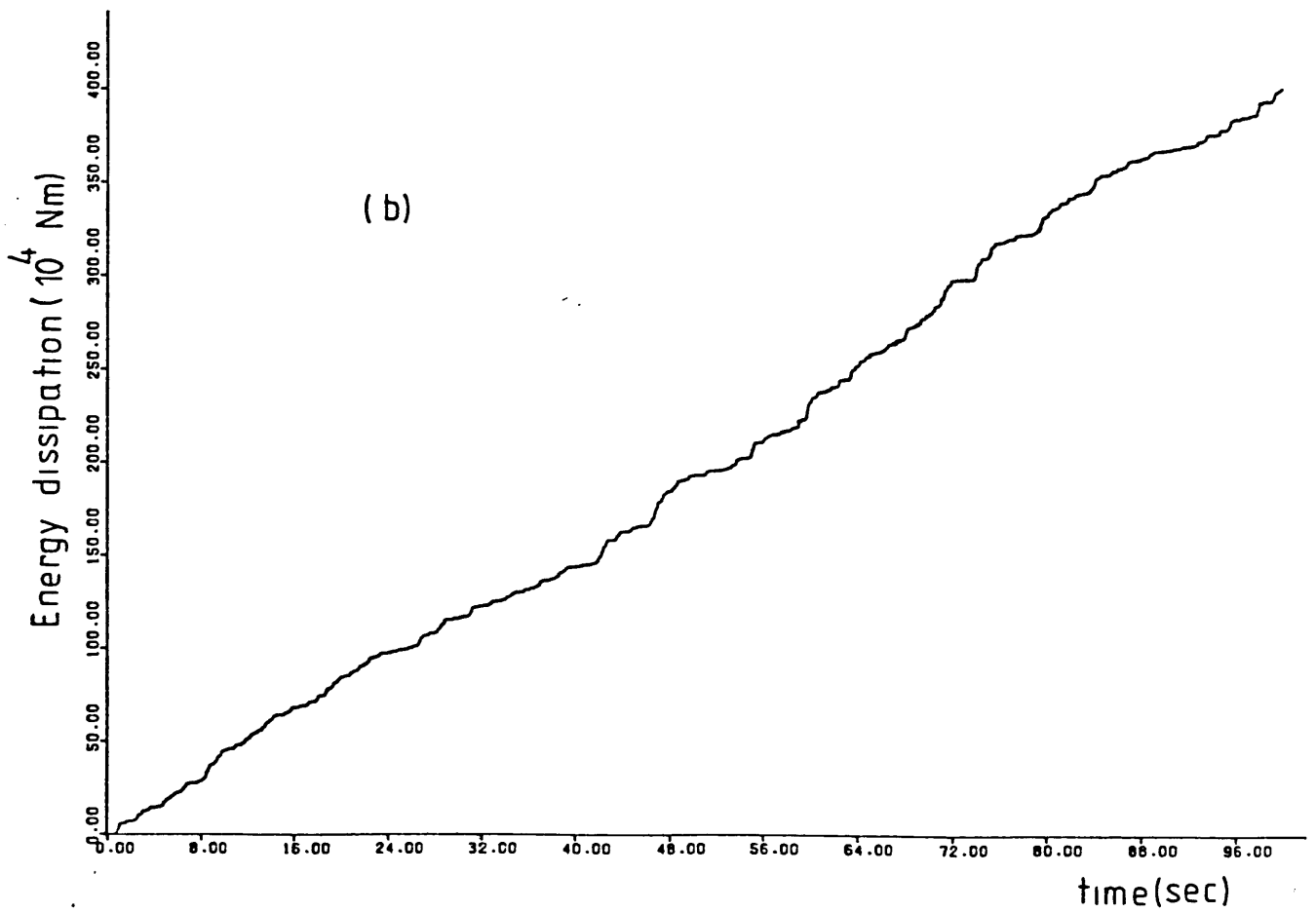
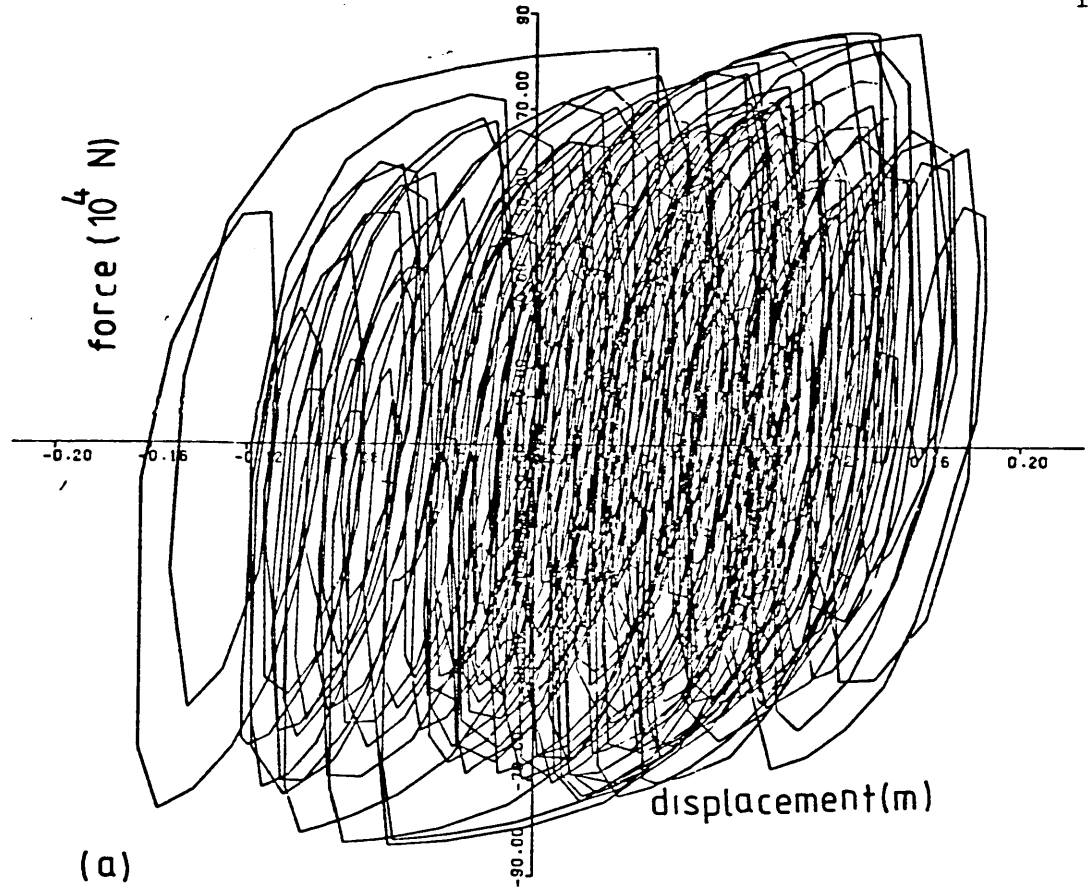
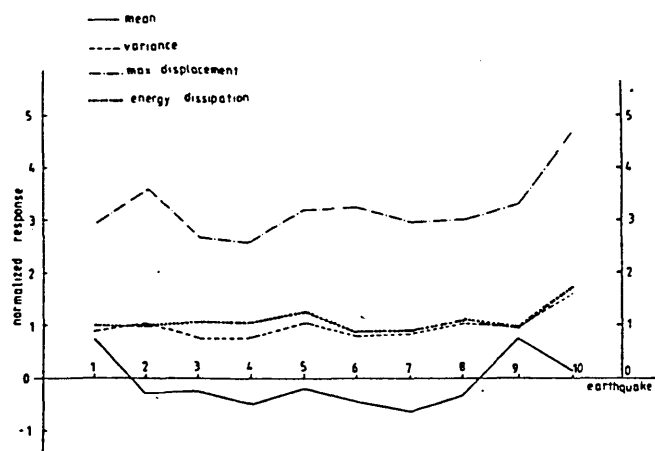


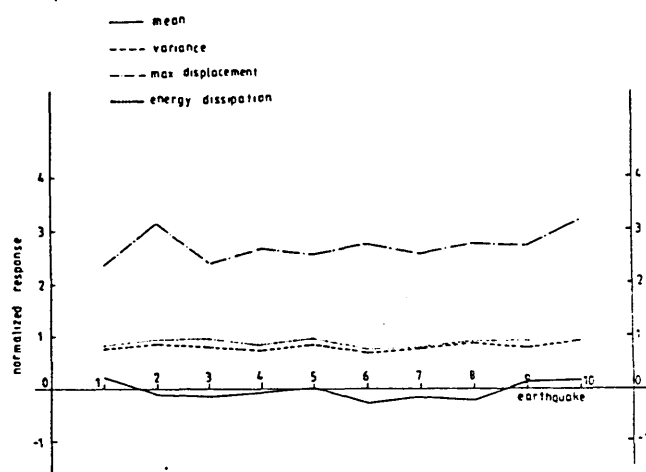
Fig. 2.24 Response of an hysteretic system to earthquake loading



(a)

$$S_0 = 200 \text{ cm}^2 / \text{sec}^3$$

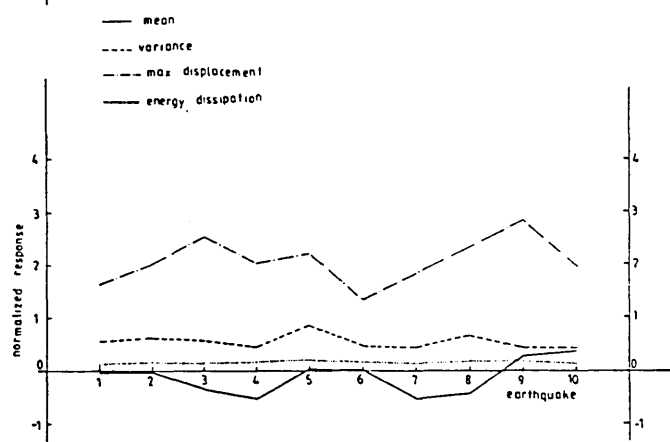
Model shown in fig. 2.11(a)



(b)

$$S_0 = 800 \text{ cm}^2 / \text{sec}^3$$

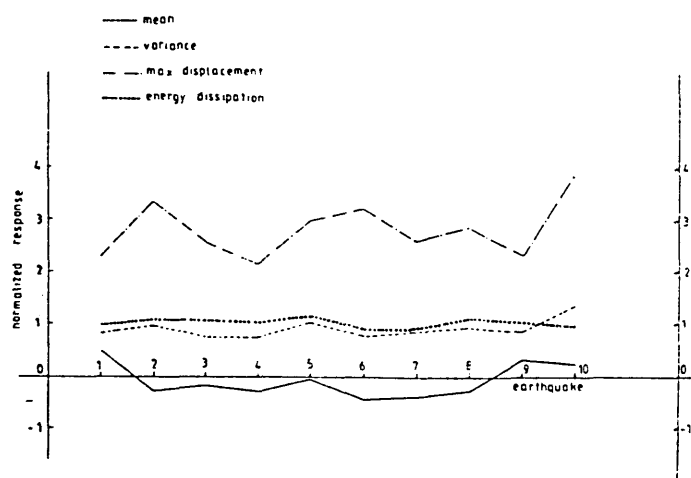
Model shown in fig. 2.11(a)



(c)

$$S_0 = 200 \text{ cm}^2 / \text{sec}^3$$

Model shown in fig. 2.11(a)



(d)

$$S_0 = 200 \text{ cm}^2 / \text{sec}^3$$

Model shown in fig. 2.11(b)

Fig. 2.25 Normalized response of hysteretic systems to ten realisations of generated ground motion

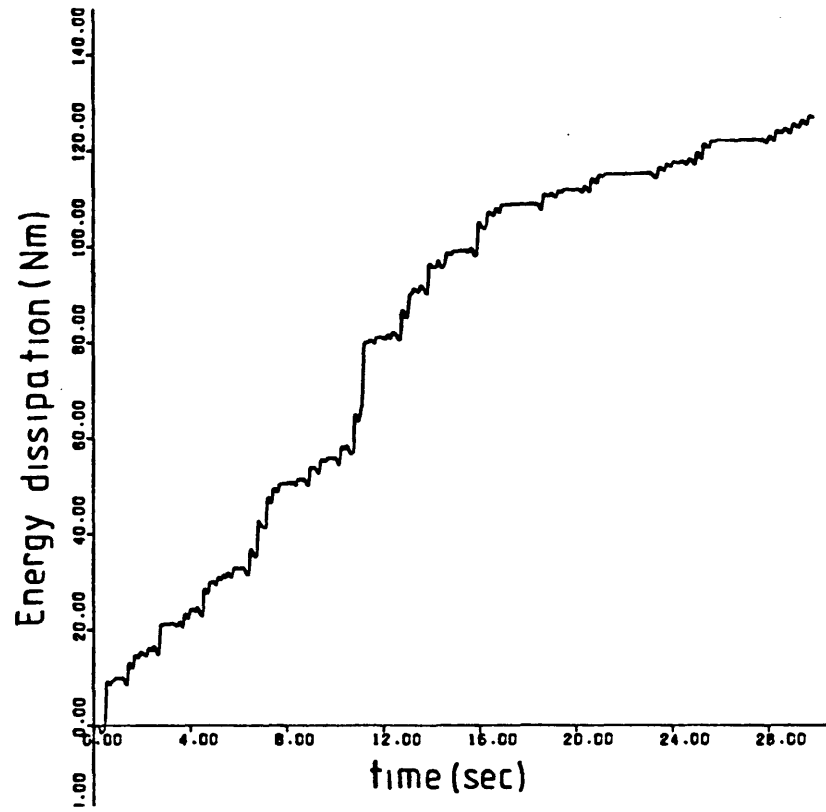


Fig. 2.26 Energy dissipation of the hysteretic model

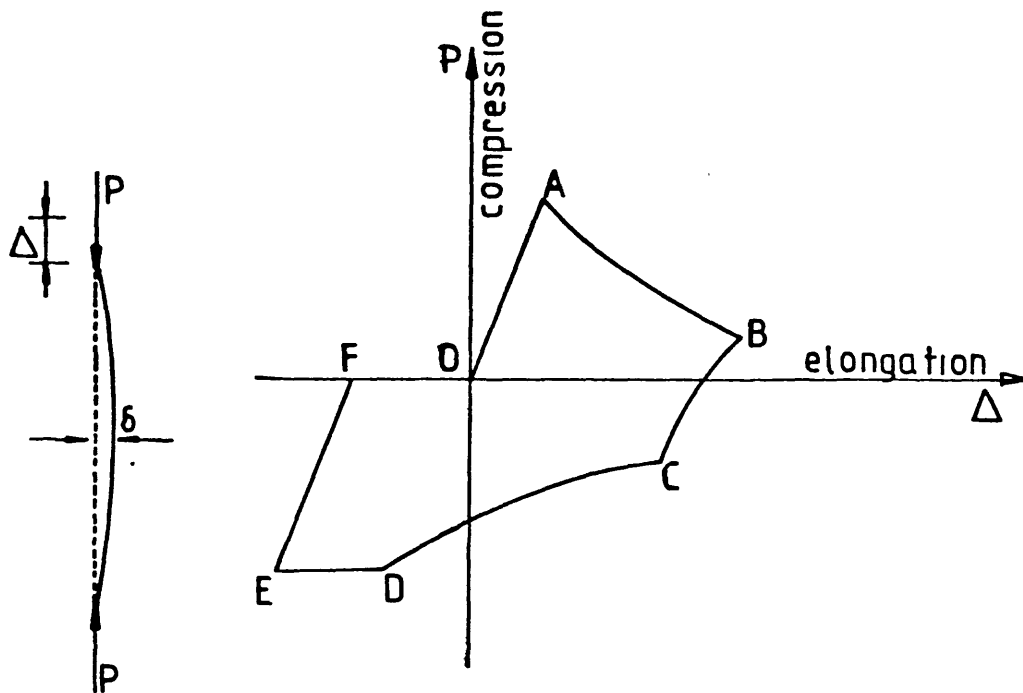


Fig. 3.1 Typical load deformation relationship of a slender bar

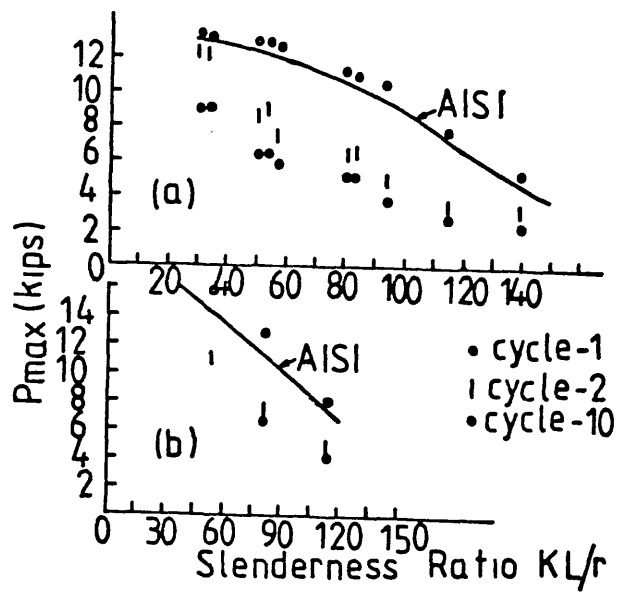


Fig. 3.2 Maximum compressive loads for:

(a) Annealed Specimens

(b) Unannealed Specimens (1 kip=4.482 kN)

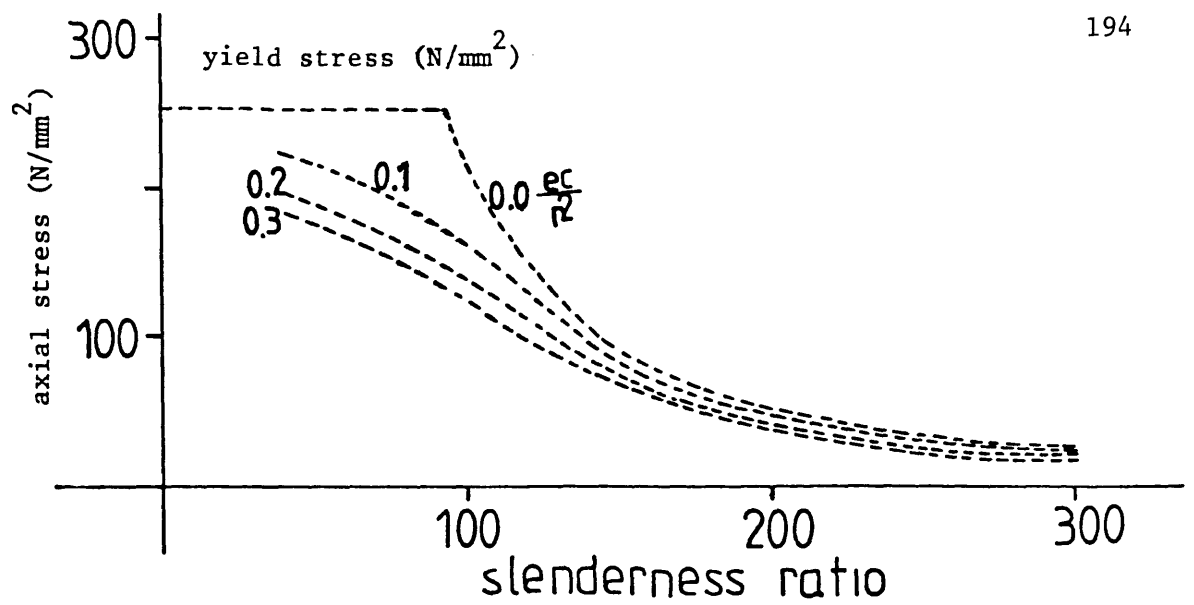


Fig. 3.3 Buckling stress of a strut subjected to a load with an eccentricity e

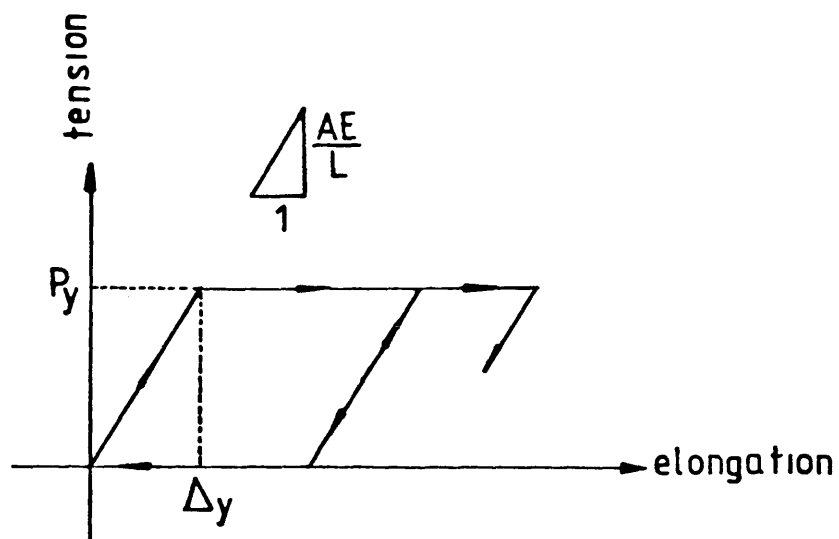


Fig. 3.4 Assumed force-displacement relationship of a slender bar

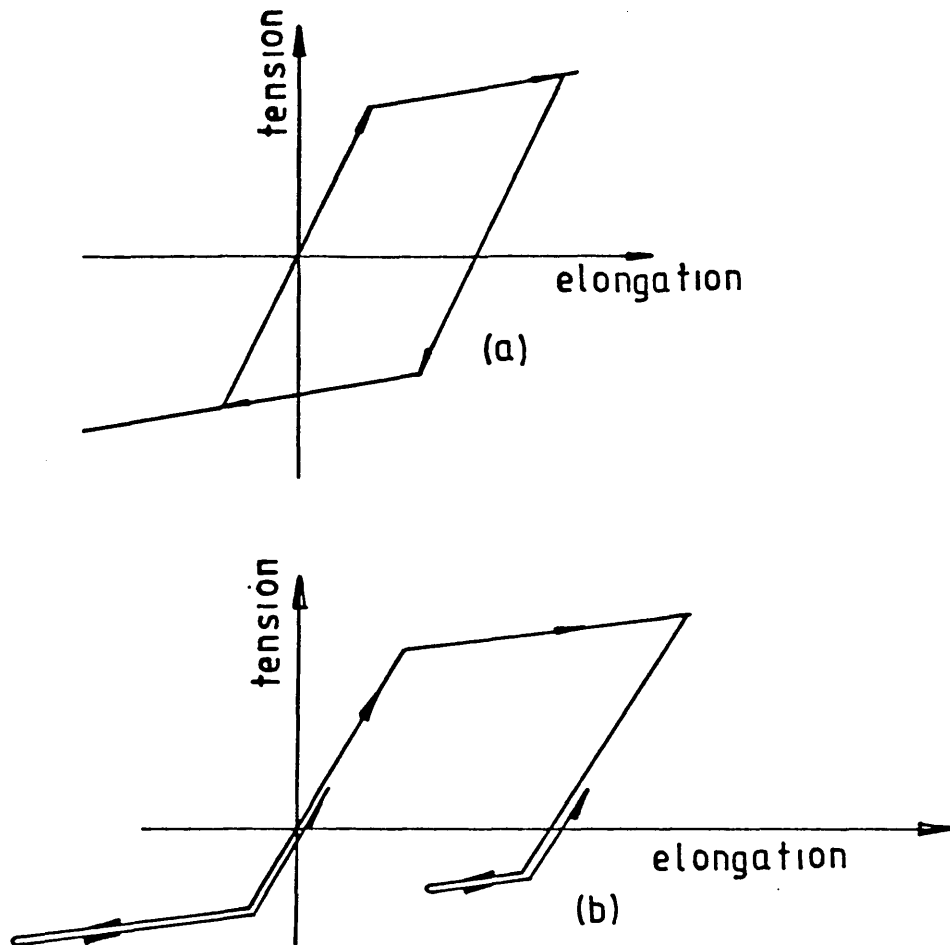


Fig. 3.5 Truss elements in the Drain-2D computer program

- (a) Yield in tension and compression
- (b) Yield in tension; buckling in compression

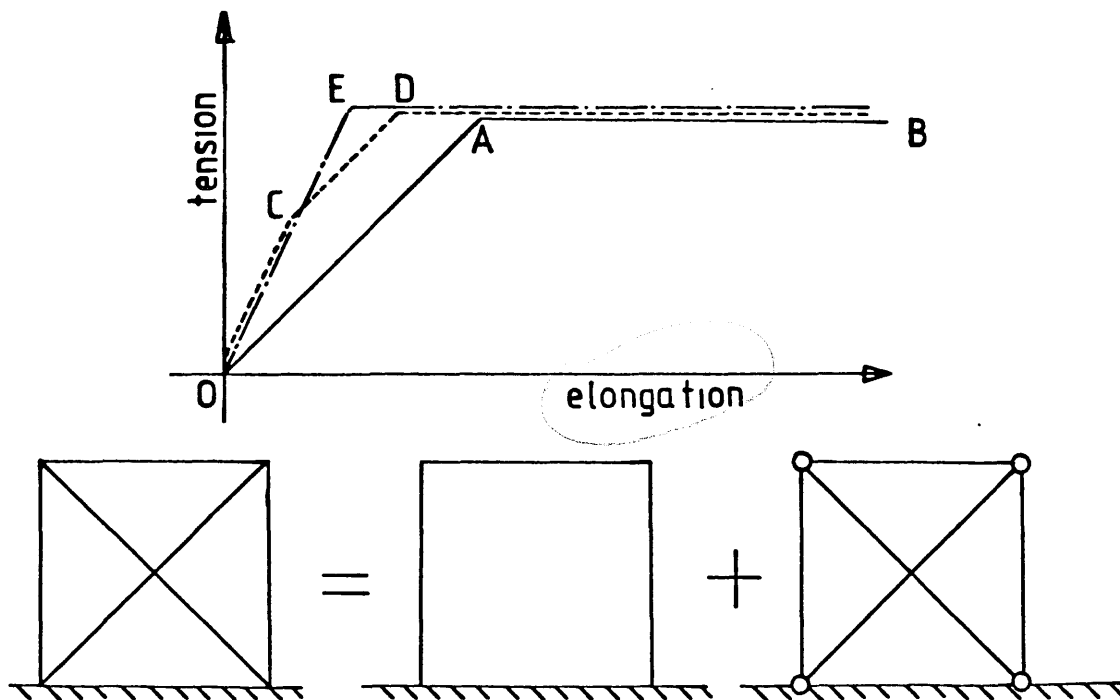


Fig. 3.6 Introduction of pretension in a braced frame

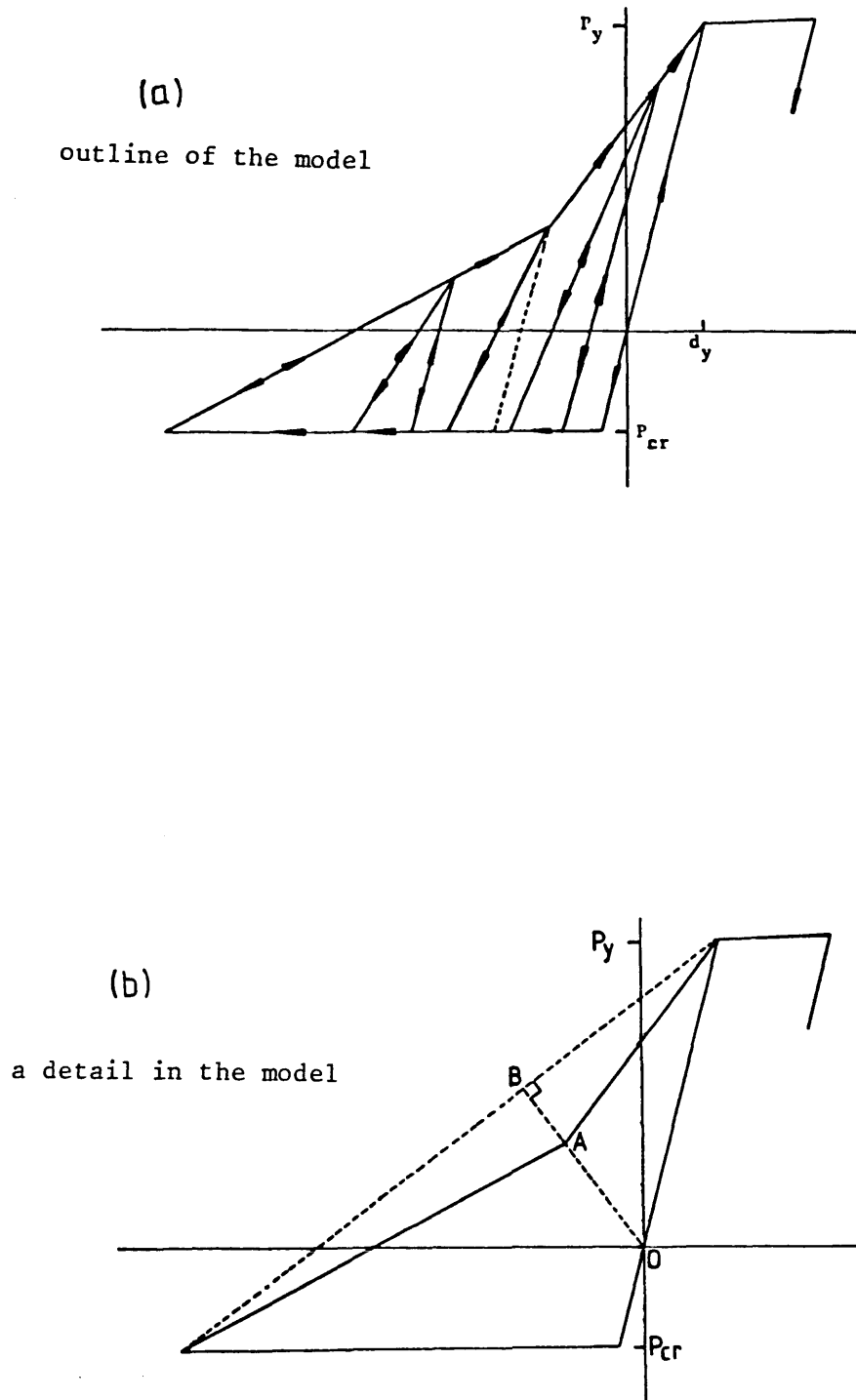


Fig. 3.7 Model describing the hysteretic response of a bracing member

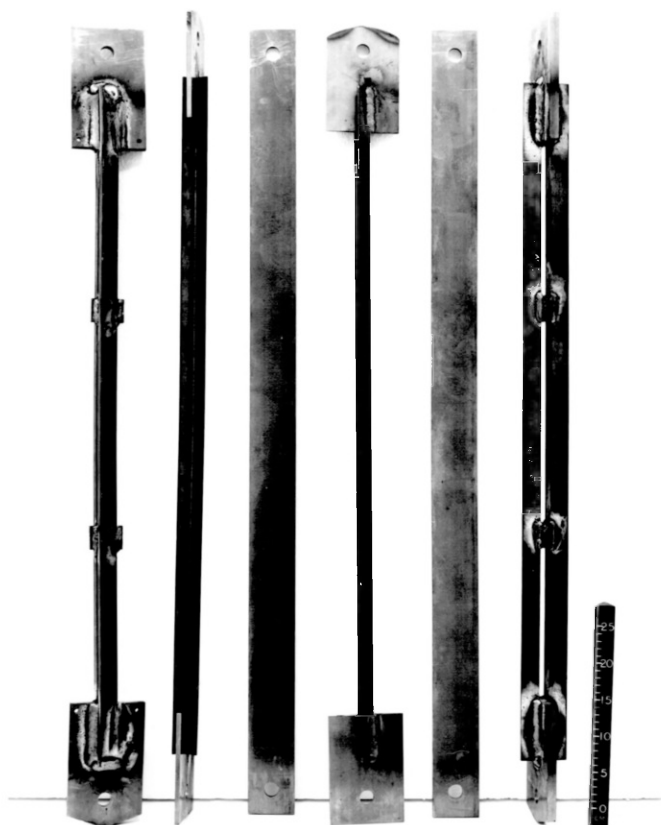


Fig.3.8. Different braces experimentally tested.

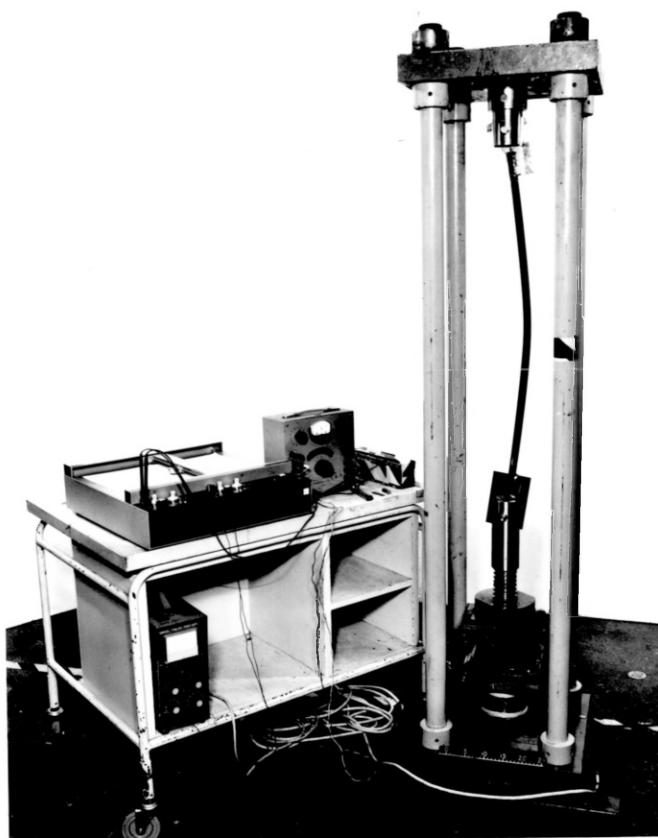


Fig.3.9. Experimentation on brace buckling.

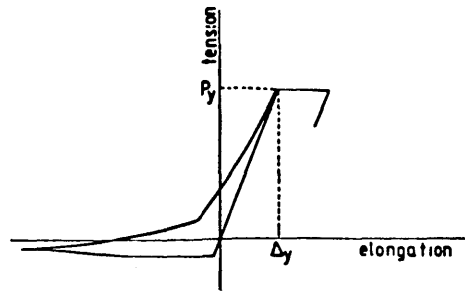


Fig.3.10 Normalised hysteresis loops for a very slender brace

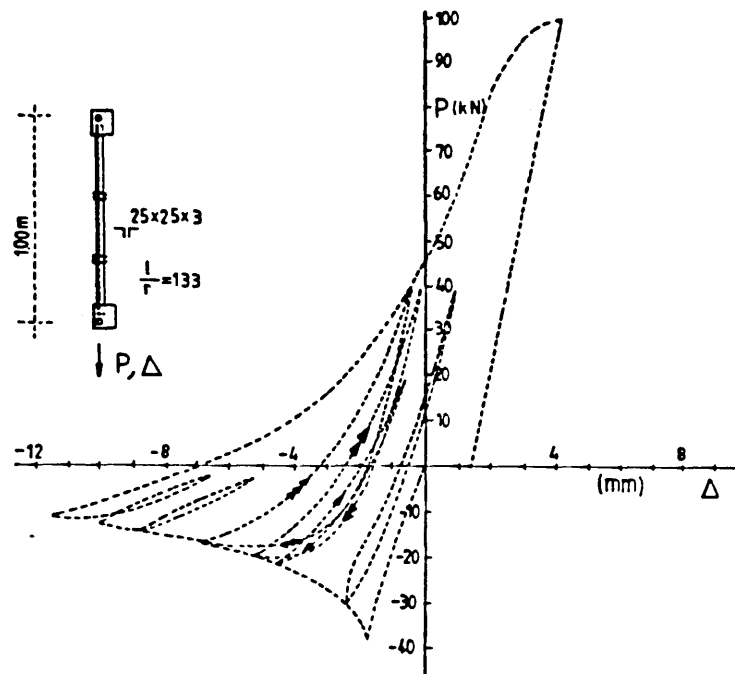


Fig. 3.11 Hysteresis loops of first set of double angles

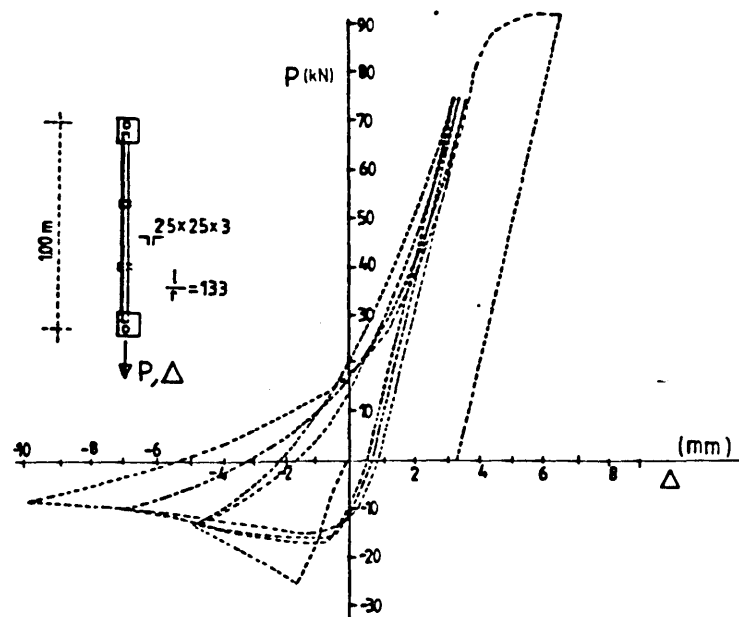


Fig. 3.12 Hysteresis loops of second set of double angles

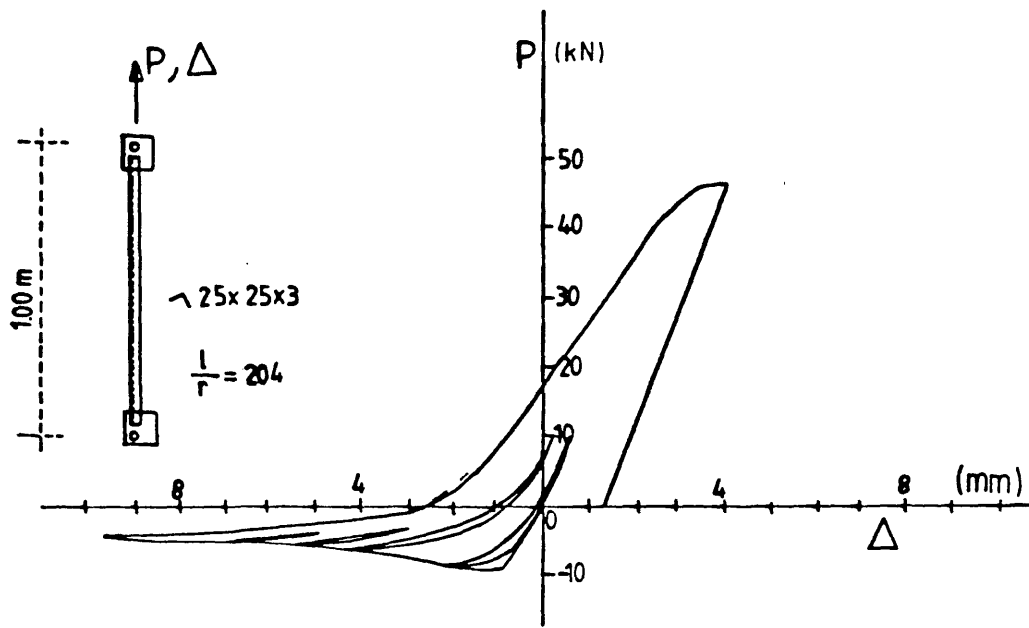


Fig. 3.13 Hysteresis loops of first single angle

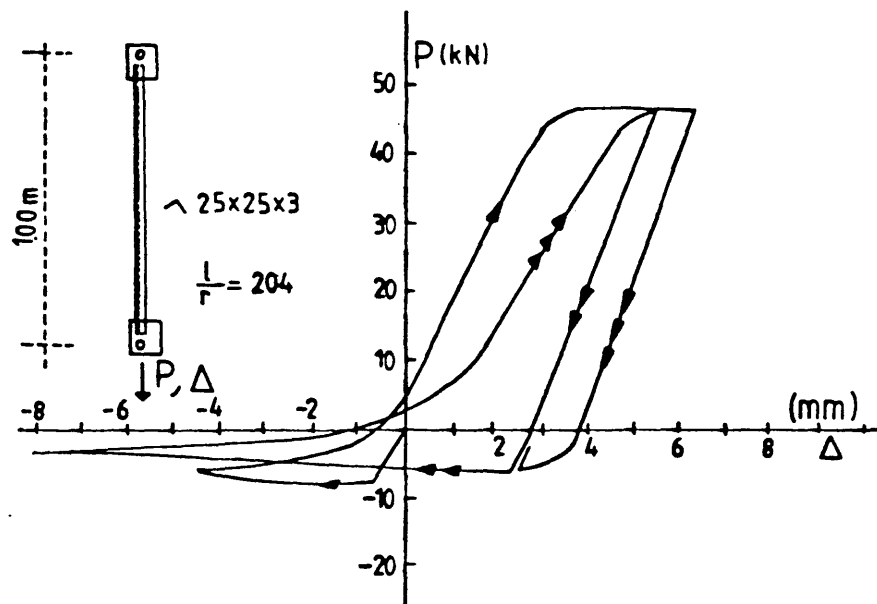


Fig. 3.14 Hysteresis loops of second single angle

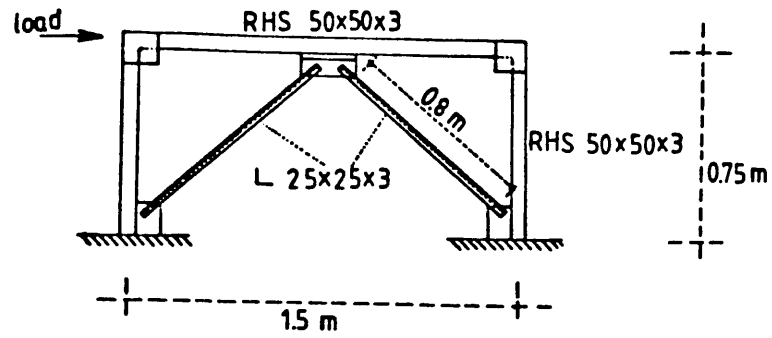


Fig. 3.15 The K-braced frame experimentally tested

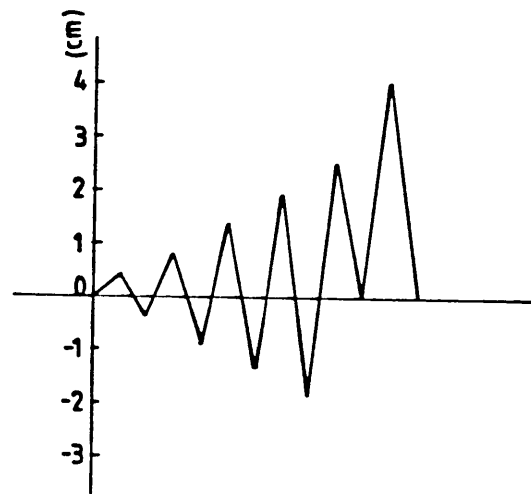


Fig. 3.16 Time history of the applied displacement

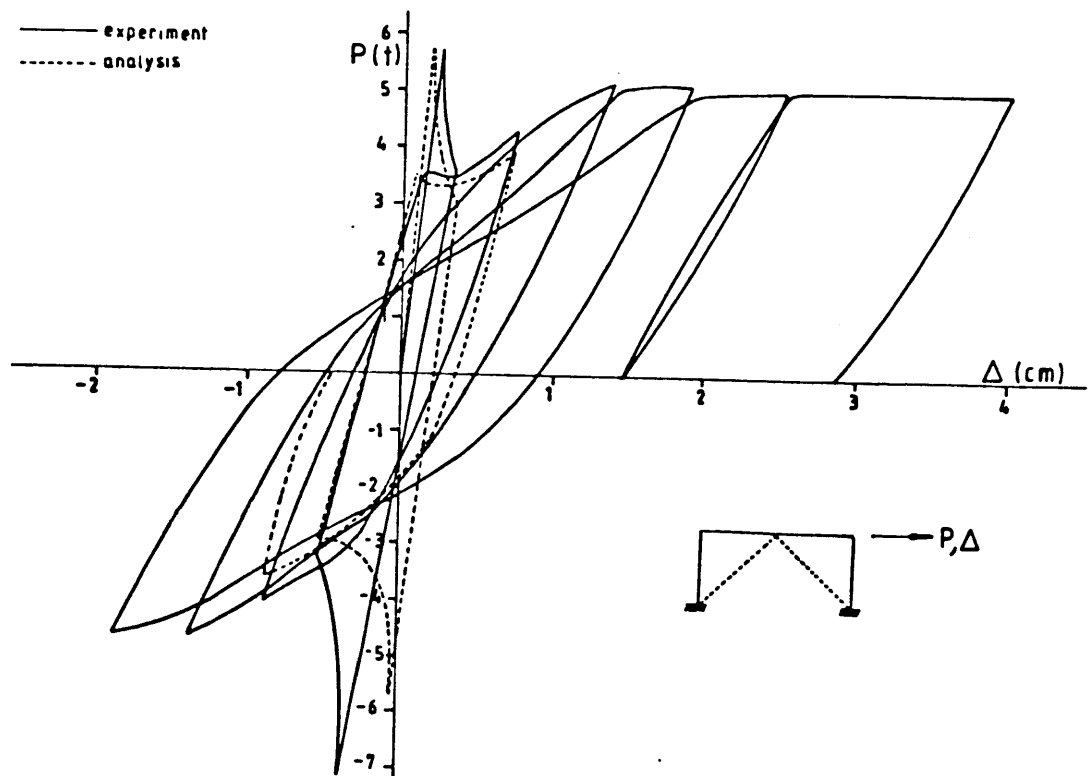


Fig. 3.17 Hysteretic response of the K-braced frame

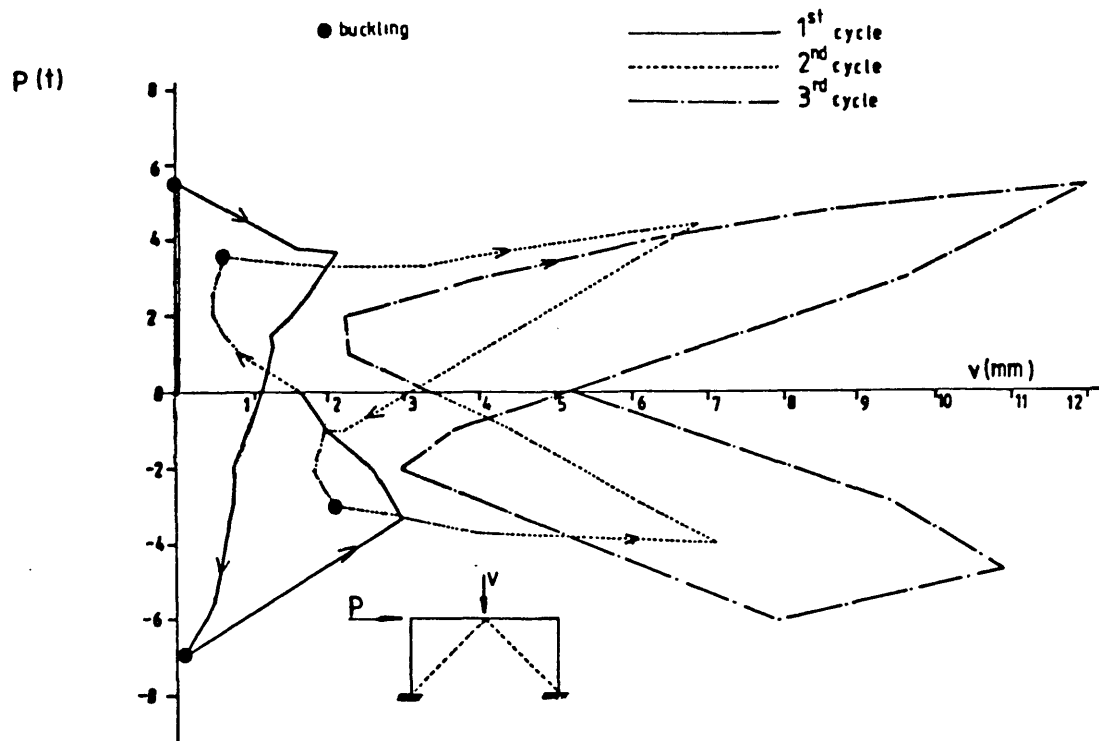


Fig. 3.18 Vertical displacement in the middle of the beam

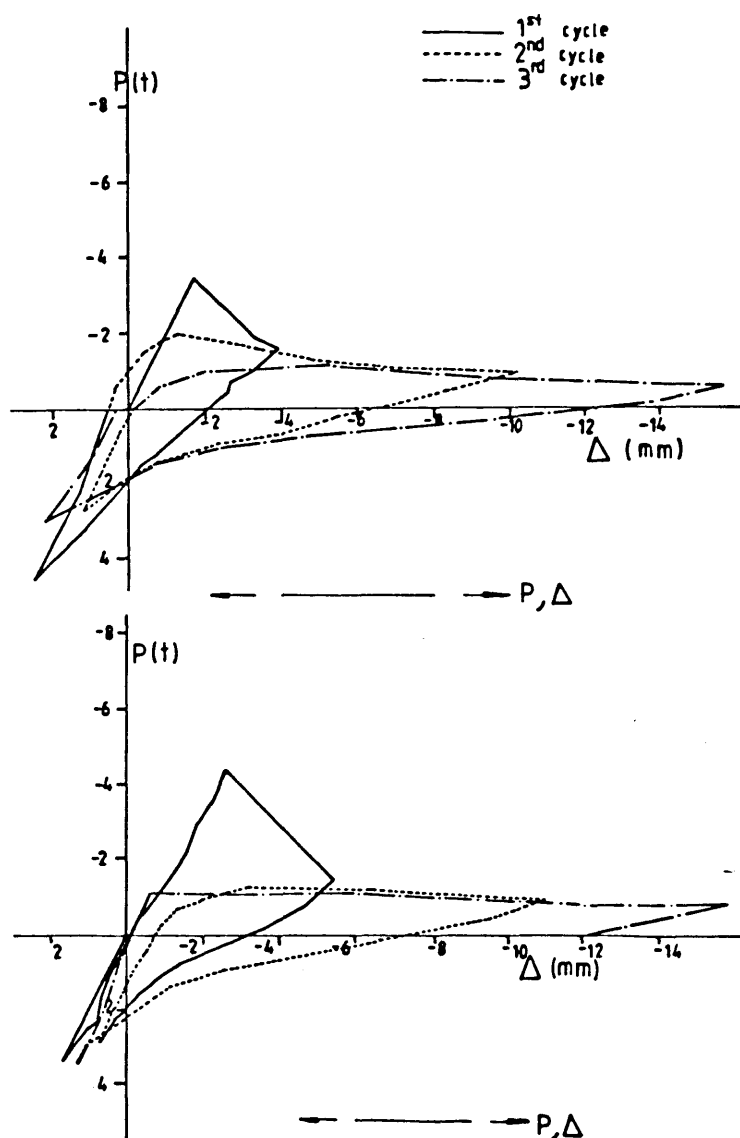


Fig. 3.19 Hysteresis loops of the two angles in the K-braced frame

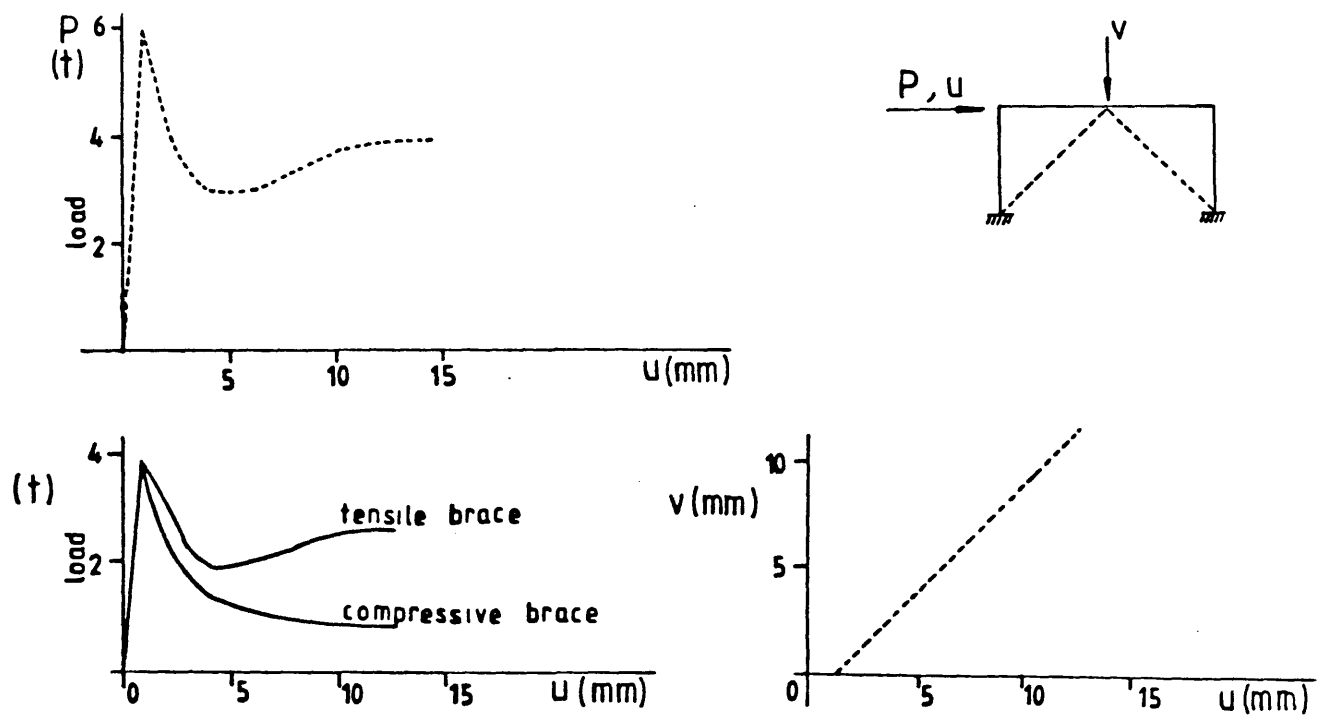


Fig. 3.20 Analytical predictions for monotonically applied load to the frame

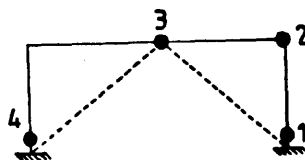


Fig. 3.21 Sequence of plastic hinge formation in the frame

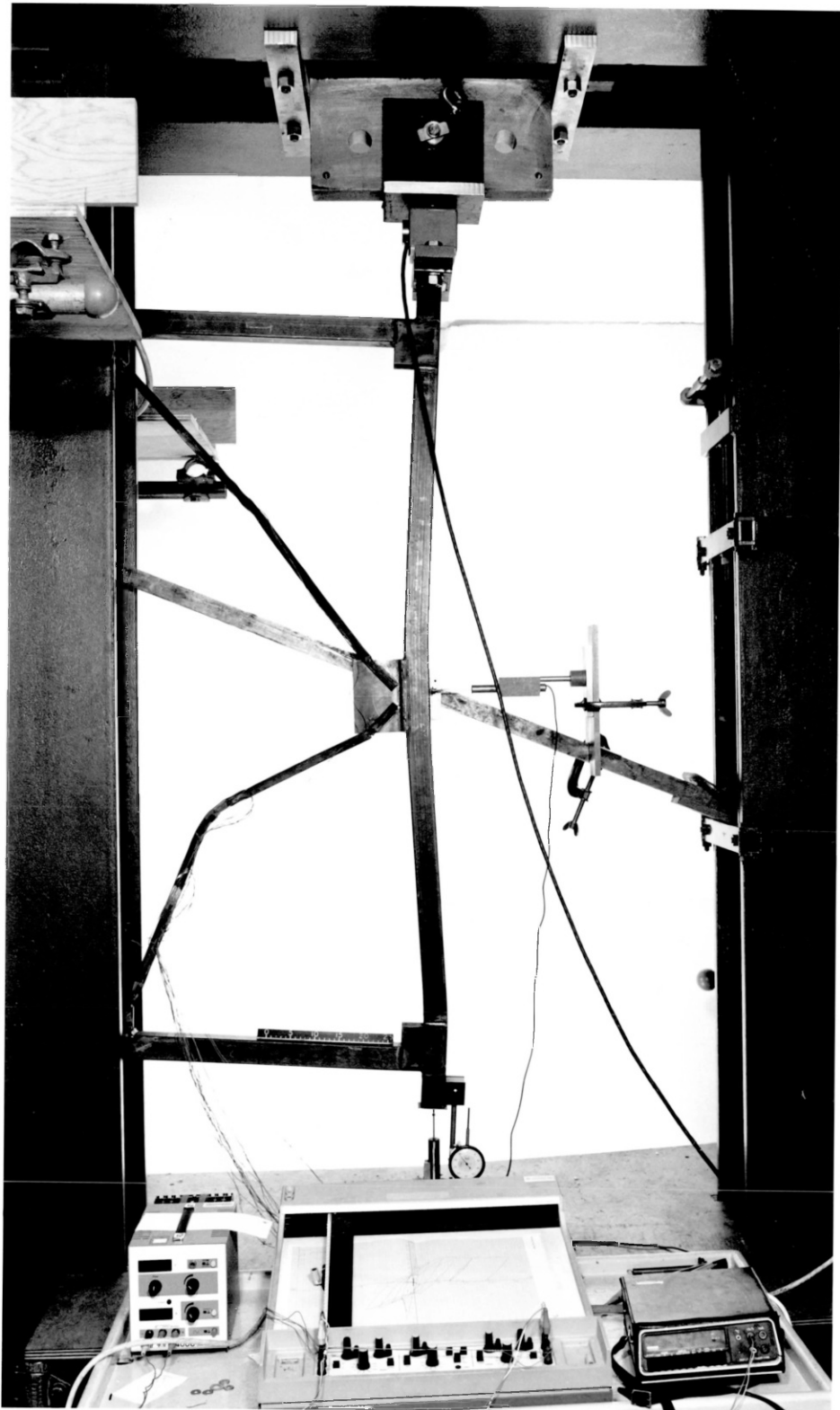


Fig.3.22. The K-frame experimentally tested.

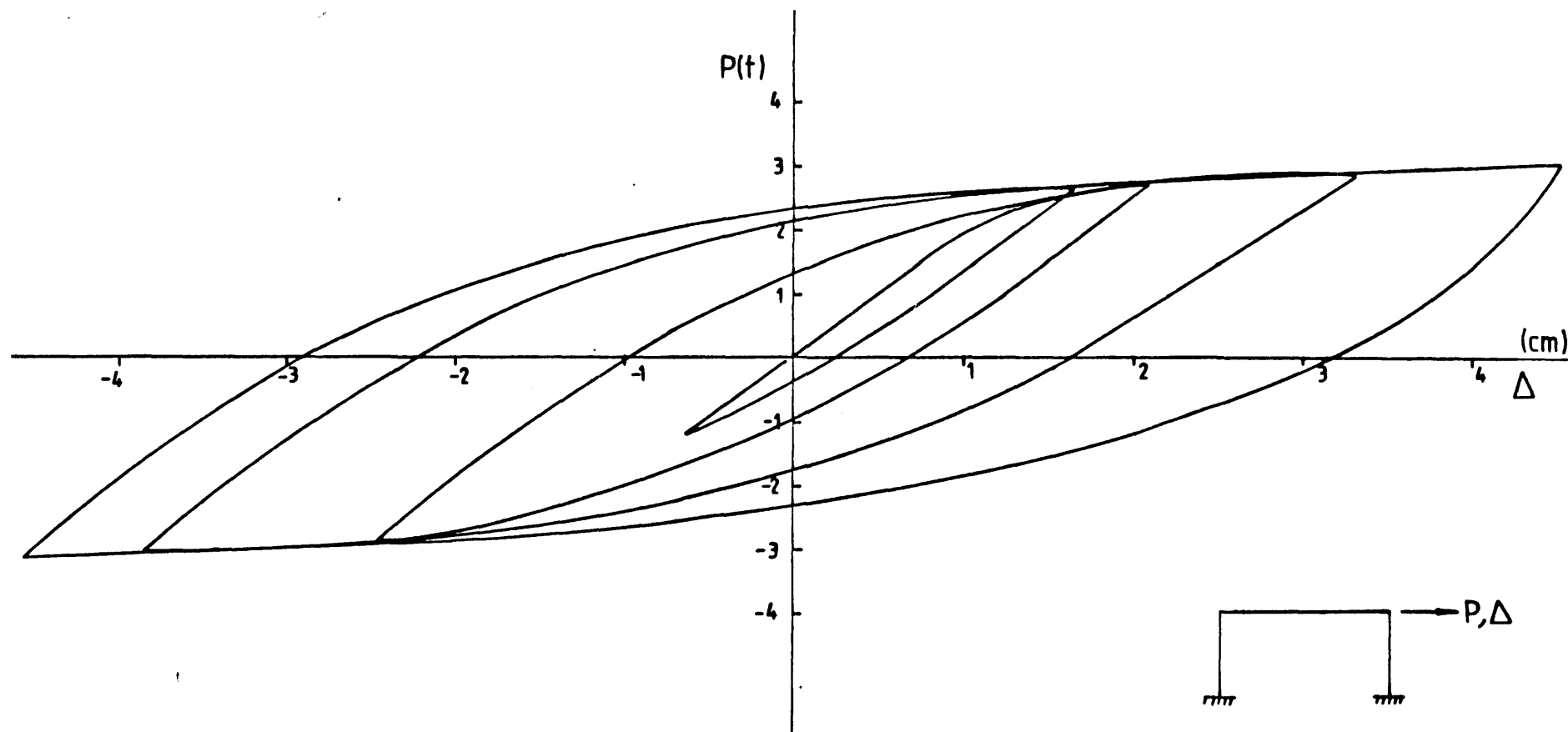


Fig. 3.23 Hysteretic response of the frame without the braces

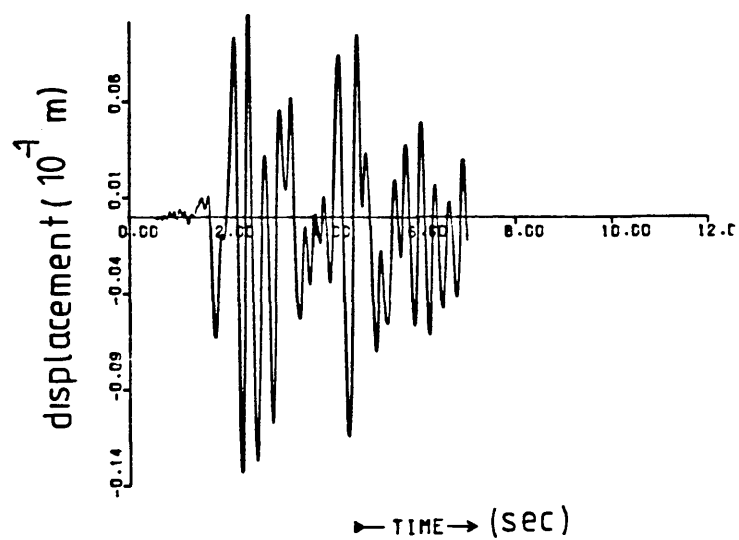


Fig. 3.24 Horizontal displacement on the beam in the time domain solution

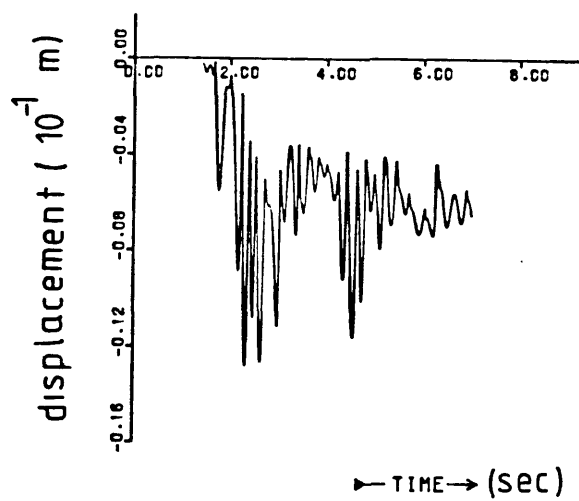


Fig. 3.25 Vertical displacement in the middle of the beam

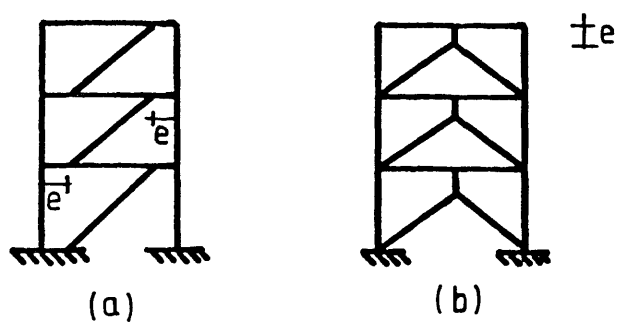


Fig. 4.1 Two typical eccentrically braced frames

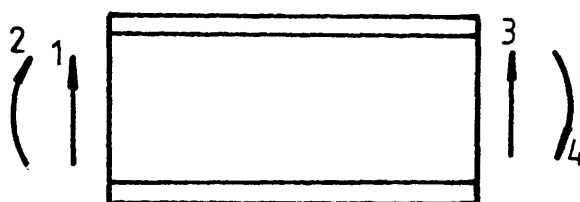


Fig. 4.2 An element of a shear link

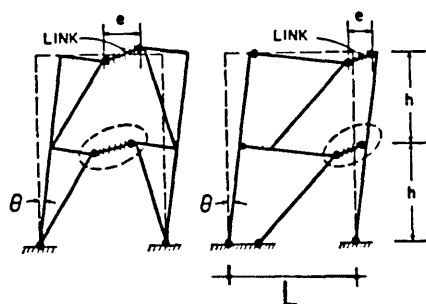


Fig. 4.3 Admissible fields of deformation for two eccentrically braced frames
(α Alex Popov (56))

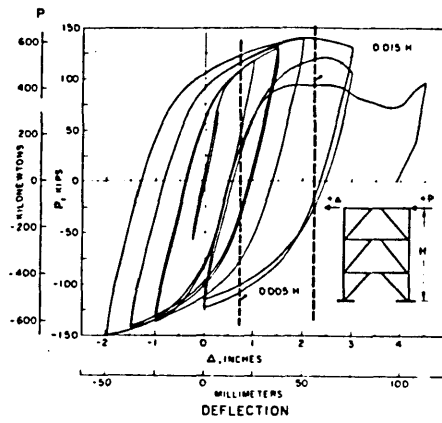


Fig. 4.4 Typical hysteresis loops of an eccentrically braced frame
(after Manheim (51))

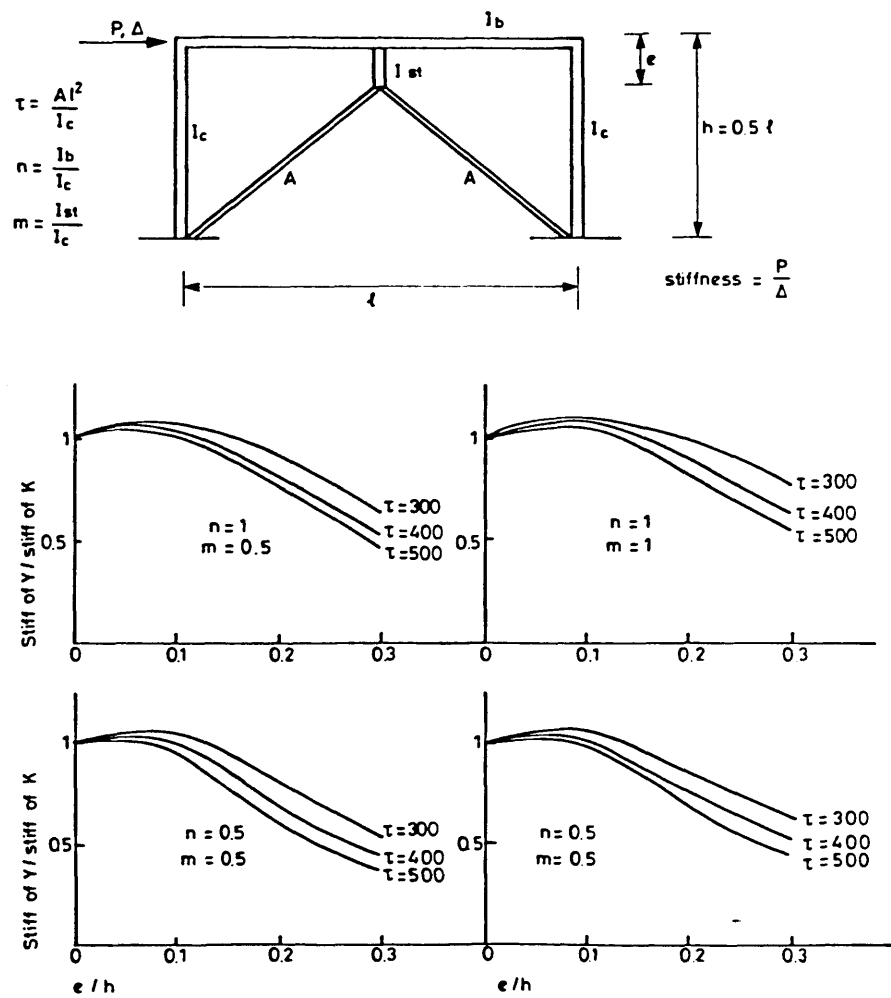


Fig. 4.5 A single-storey Y-braced frame

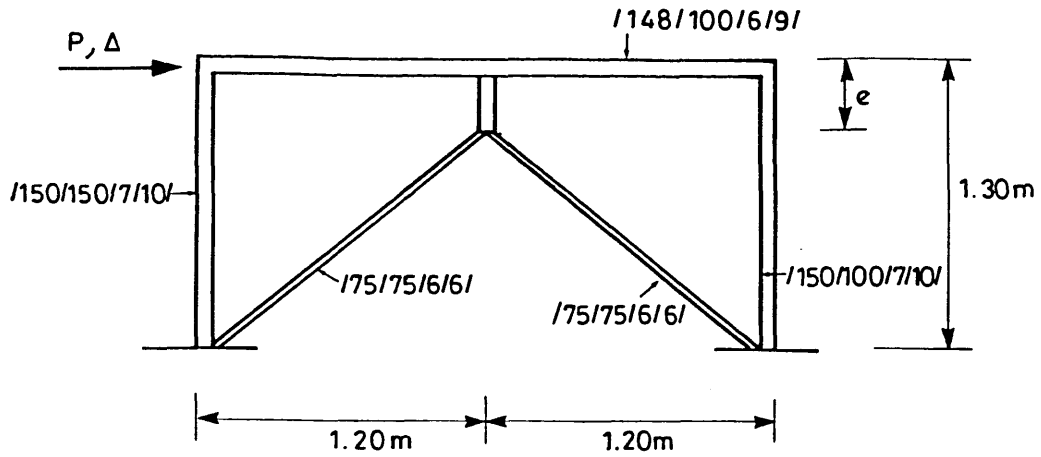


Fig. 4.6 The examined single-storey Y-braced frame

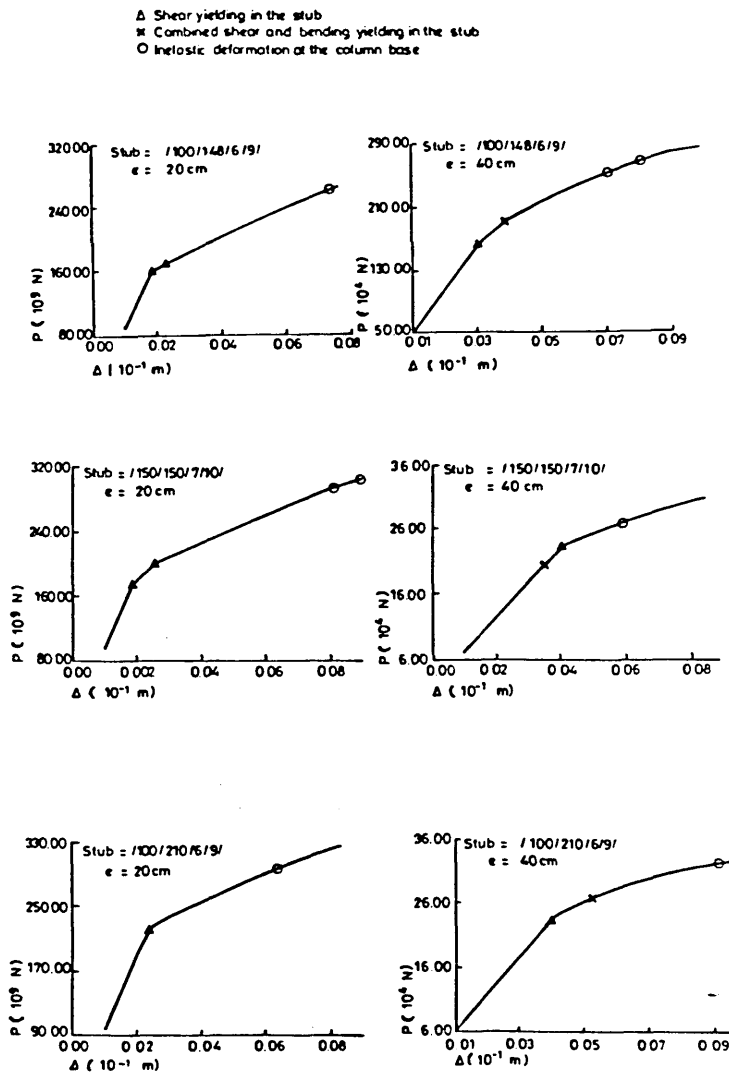


Fig. 4.7 Monotonic response of the Y-braced frame

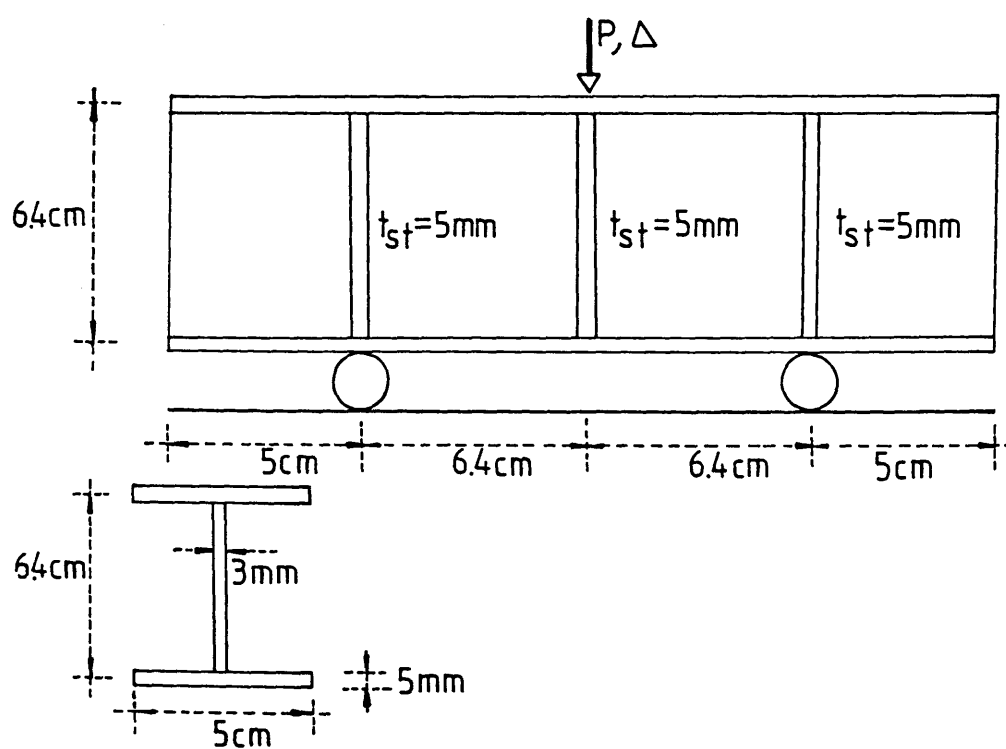


Fig. 4.8 The experimentally tested shear link

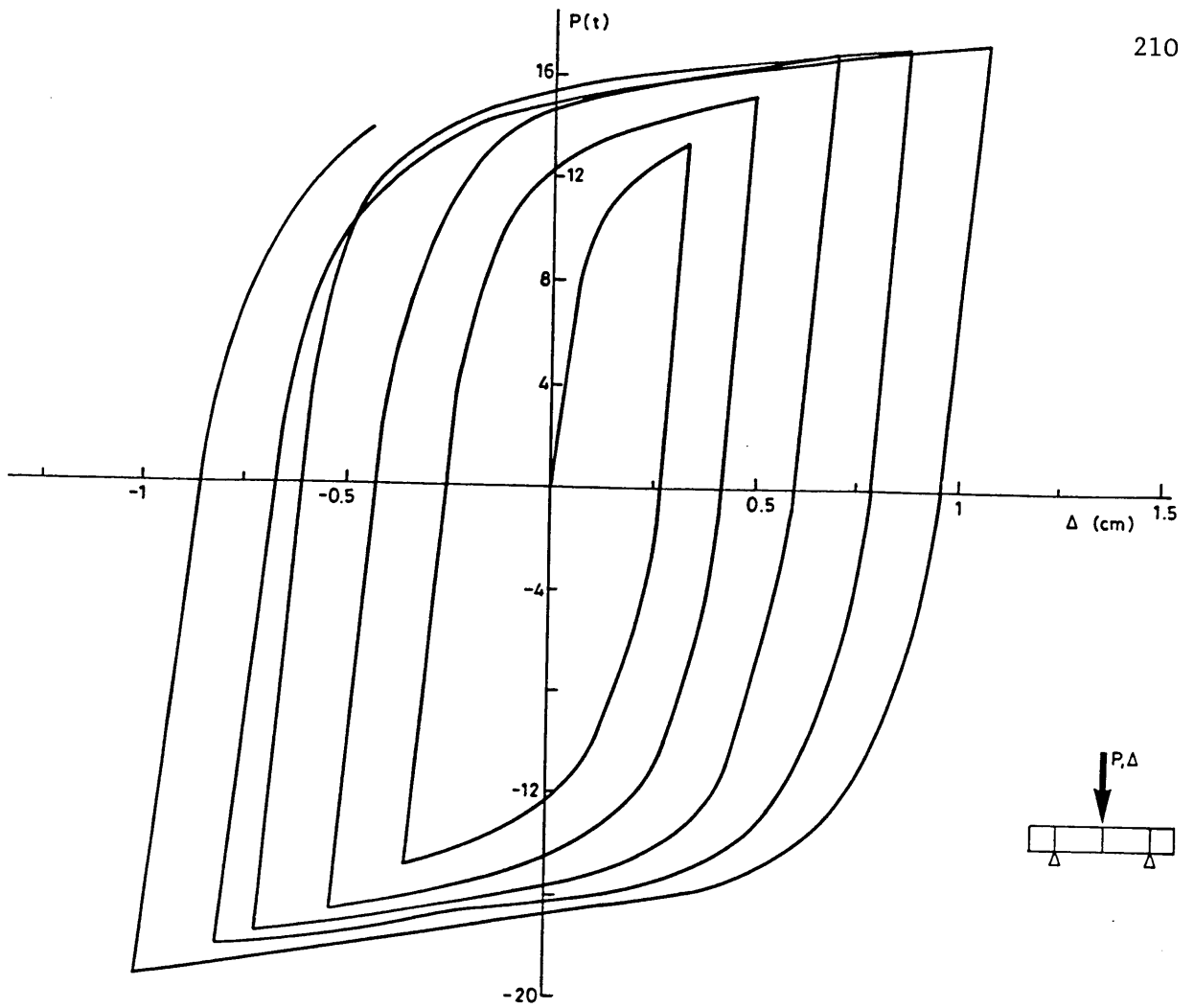


Fig. 4.9 Hysteresis loops of the shear link

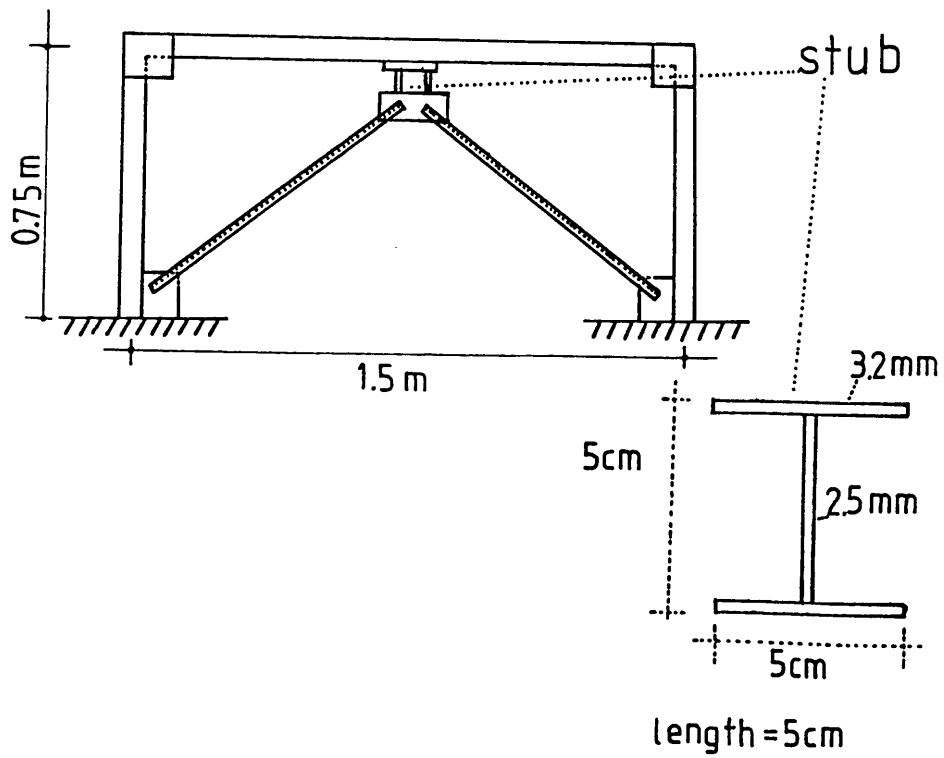


Fig. 4.10 Dimensions of the Y-frame experimentally tested

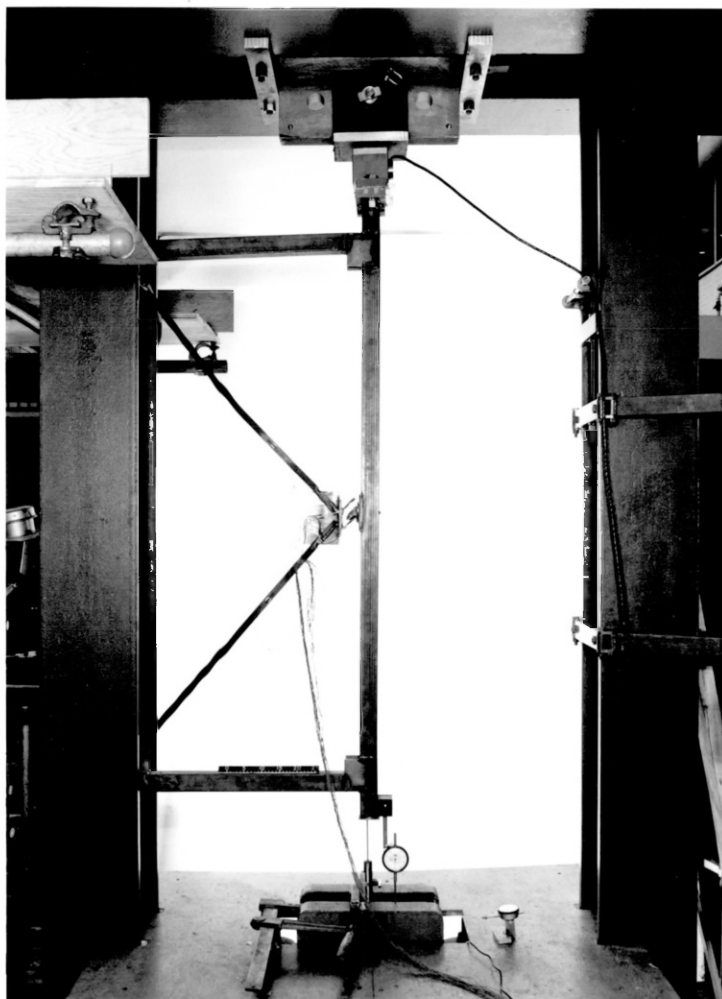


Fig.4.II. The Y-braced frame experimentally tested.

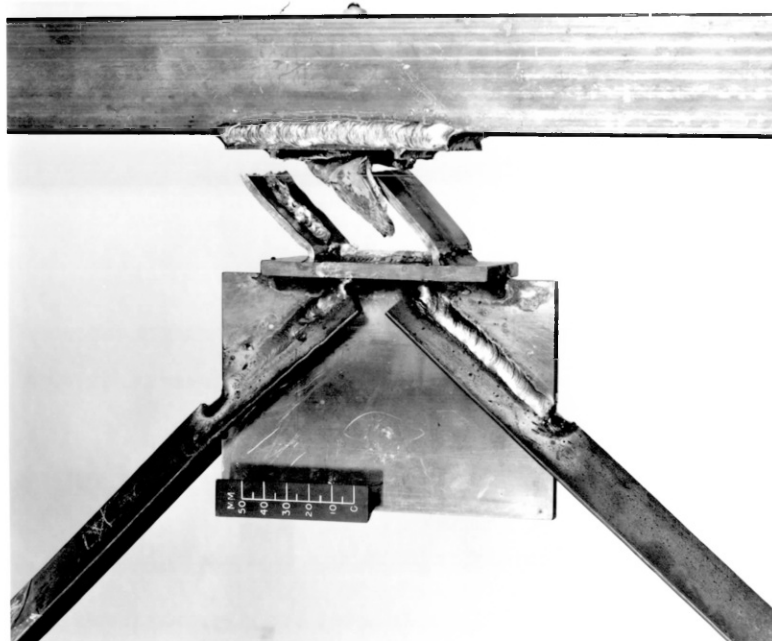


Fig.4.I2. A photograph of the shear link on the Y-frame after the test.

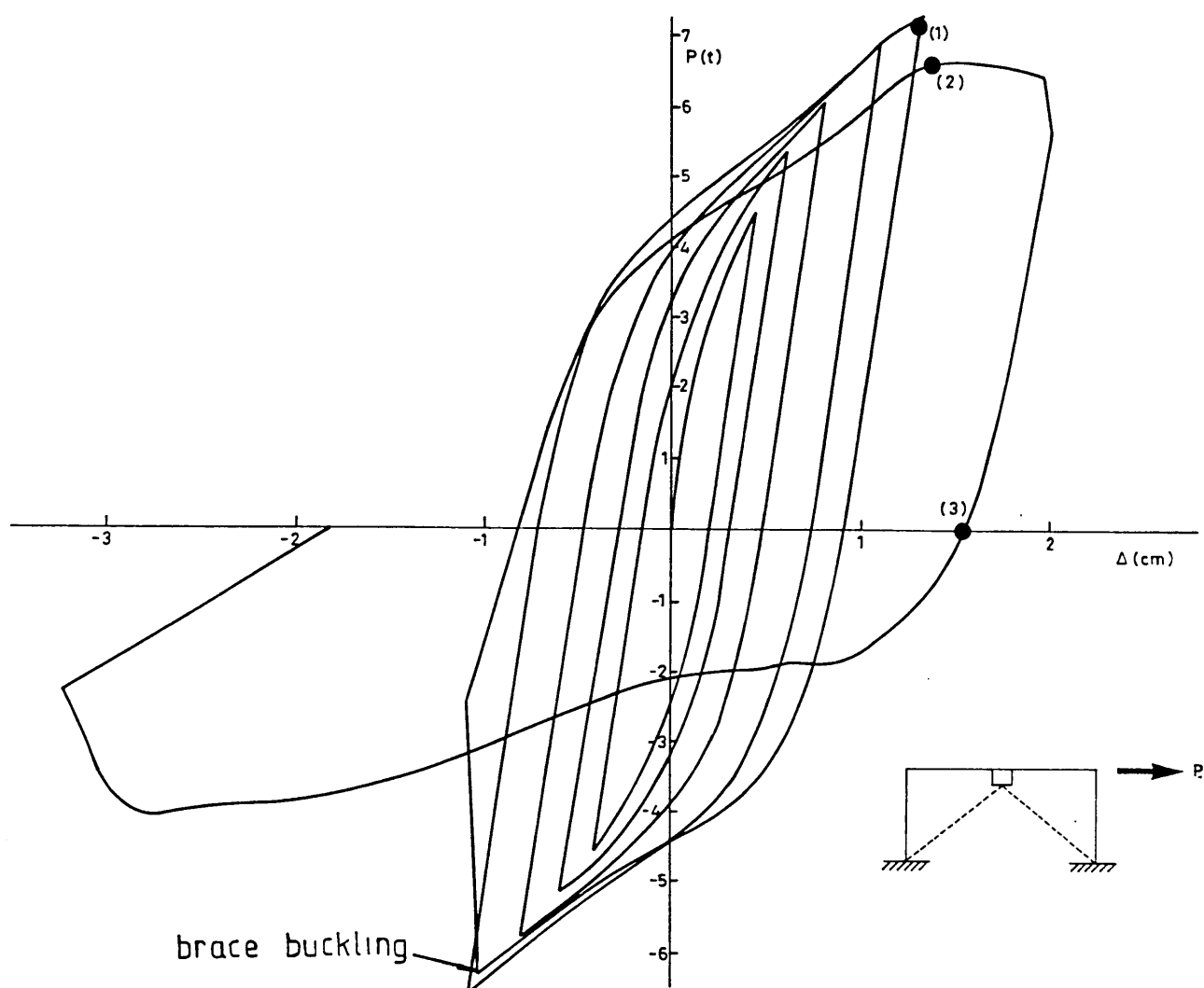


Fig. 4.13 Hysteresis loops of the Y-braced frame

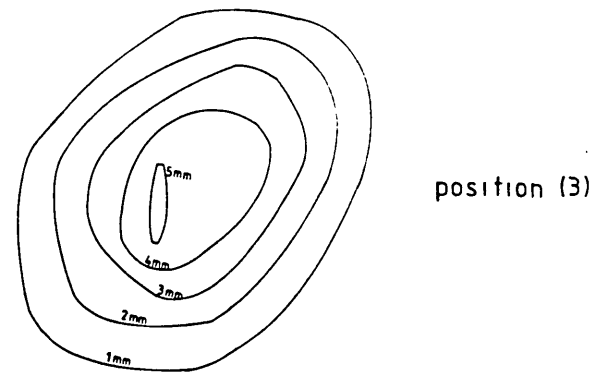
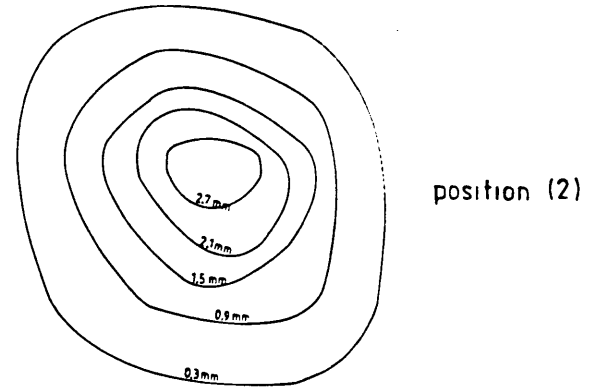
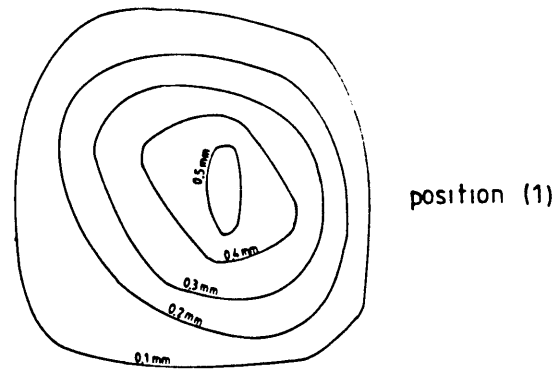


Fig. 4.14 The diagonal tension field at three positions on the Y-braced frame

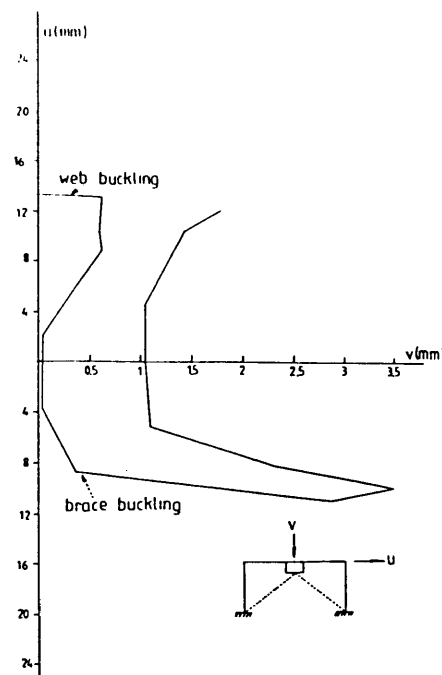


Fig. 4.15 Displacement in the middle of the Y-braced frame

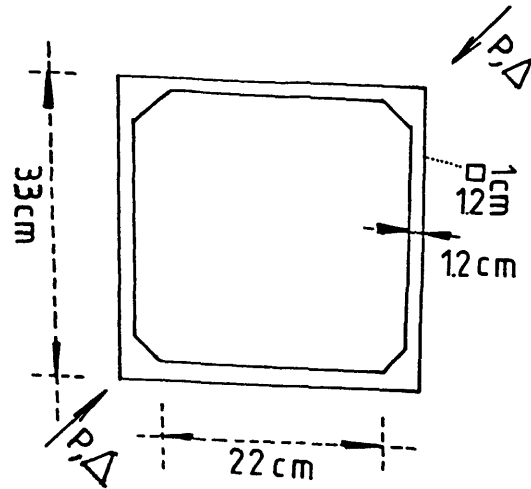


Fig. 4.16 Dimensions of the bending device tested

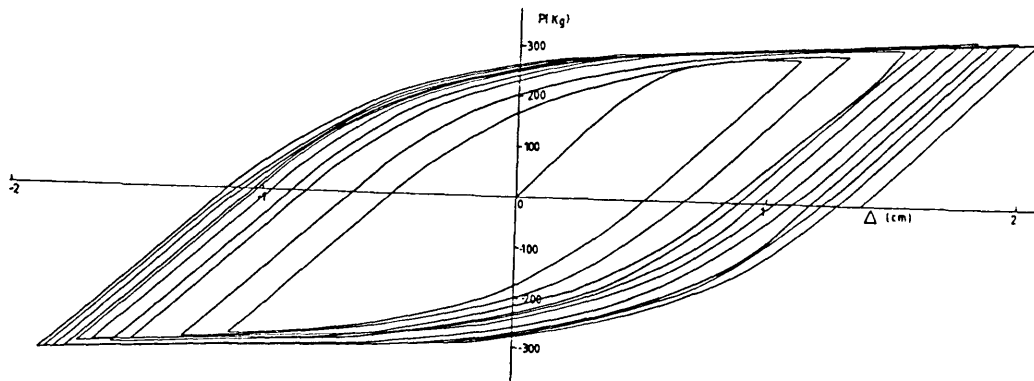


Fig. 4.17 Hysteresis loops of the bending device

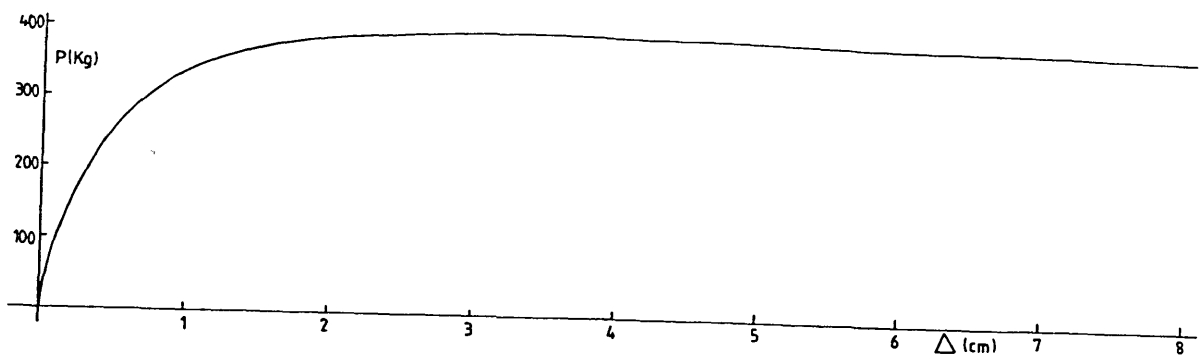


Fig. 4.18 Monotonic response of the bending device

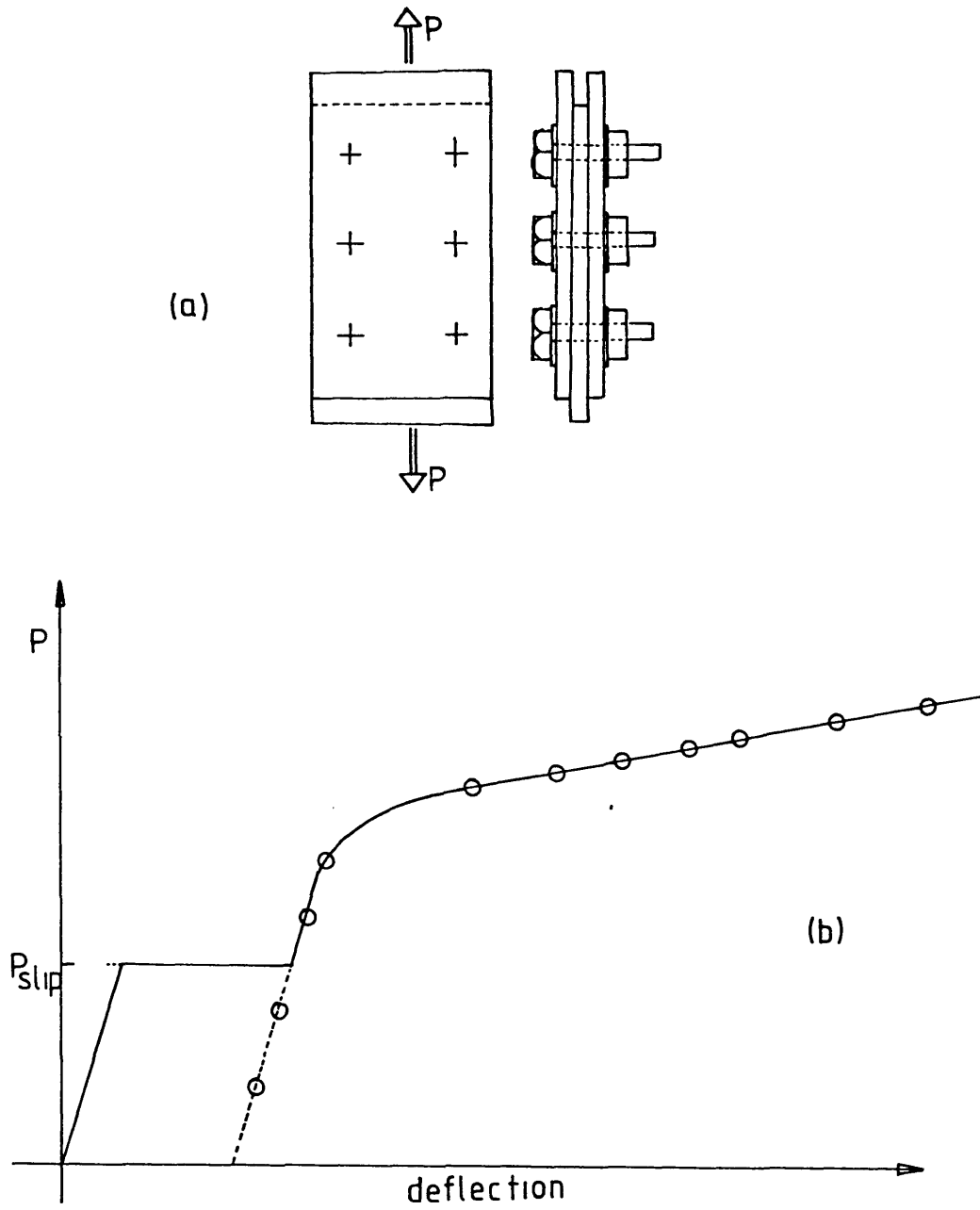


Fig. 4.19 A typical linear slipping device

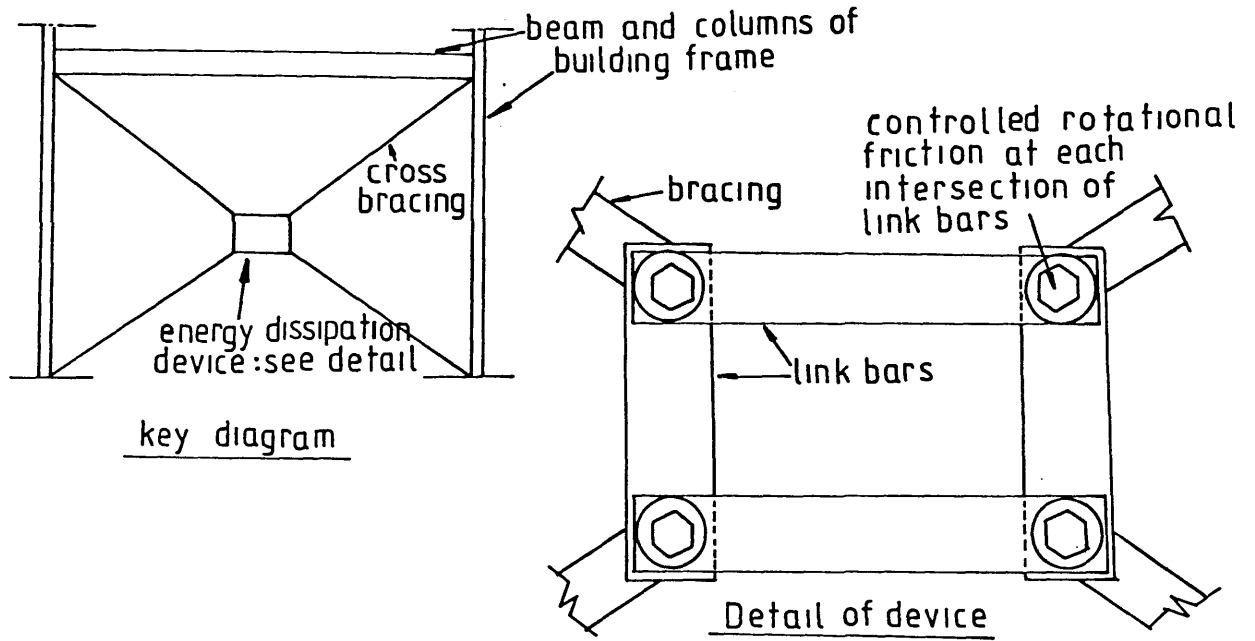


Fig. 4.20 A typical rotating device

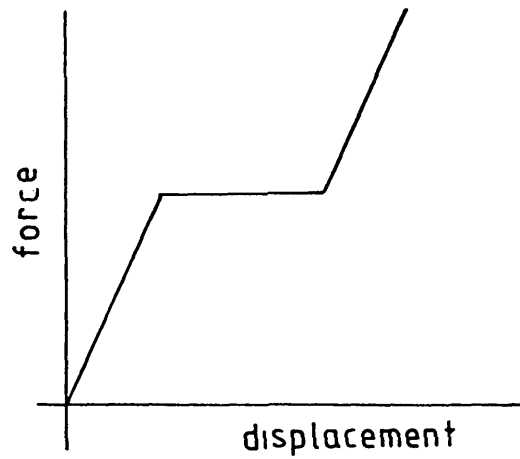


Fig. 4.21 The hardening model

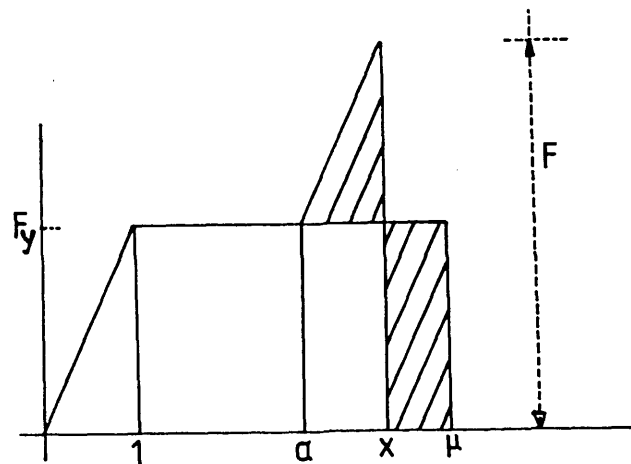


Fig. 4.22 Maximum force reached by the hardening model

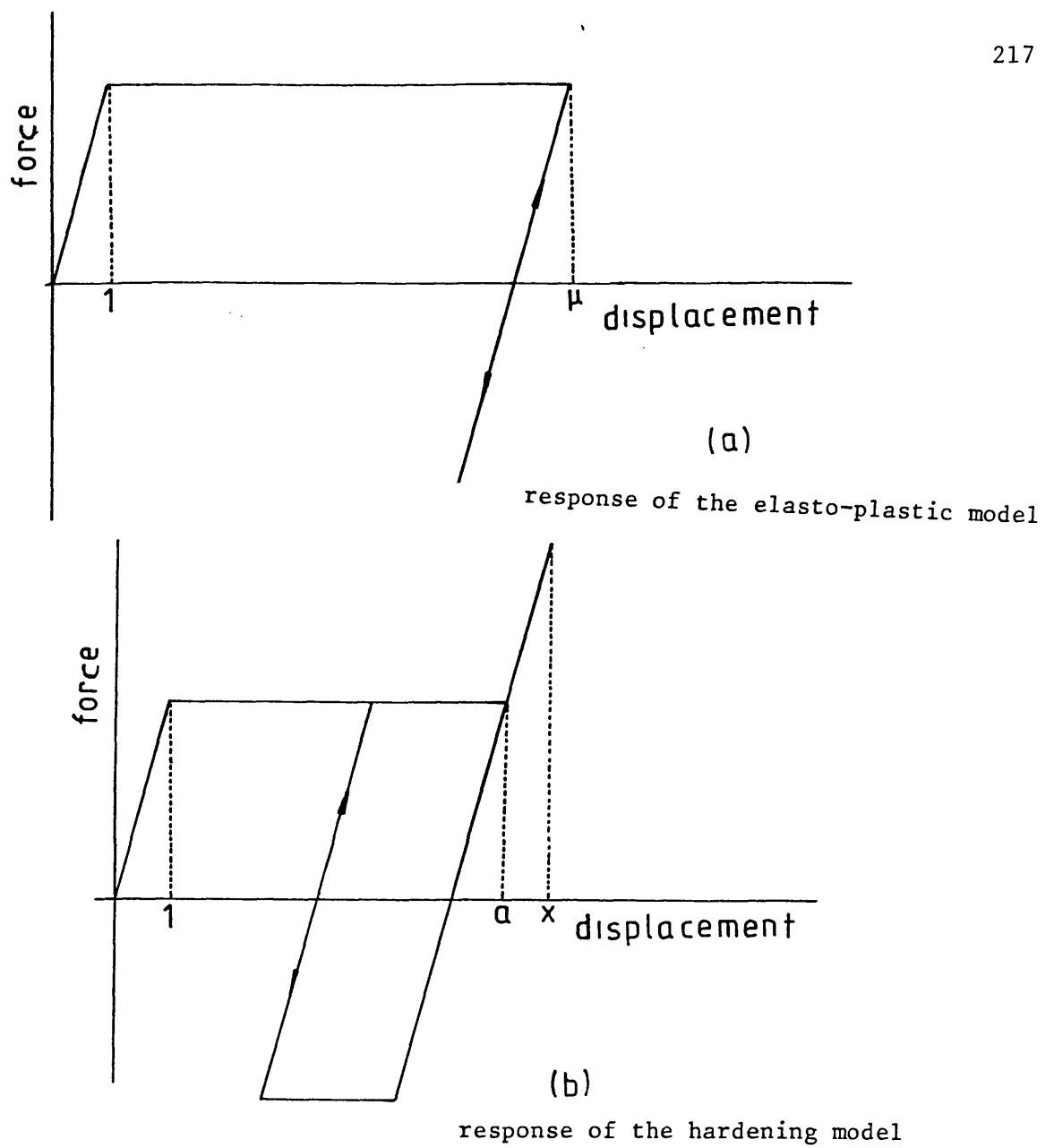


Fig. 4.23 Response of the two models to an impulsive load

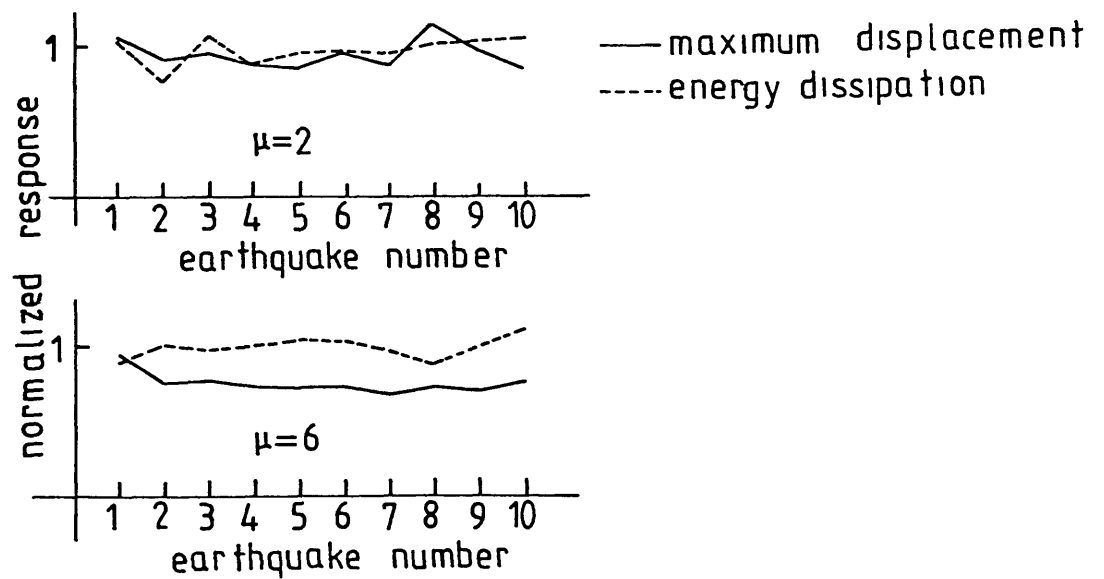


Fig. 4.24 Normalised response of the two models to earthquake loading

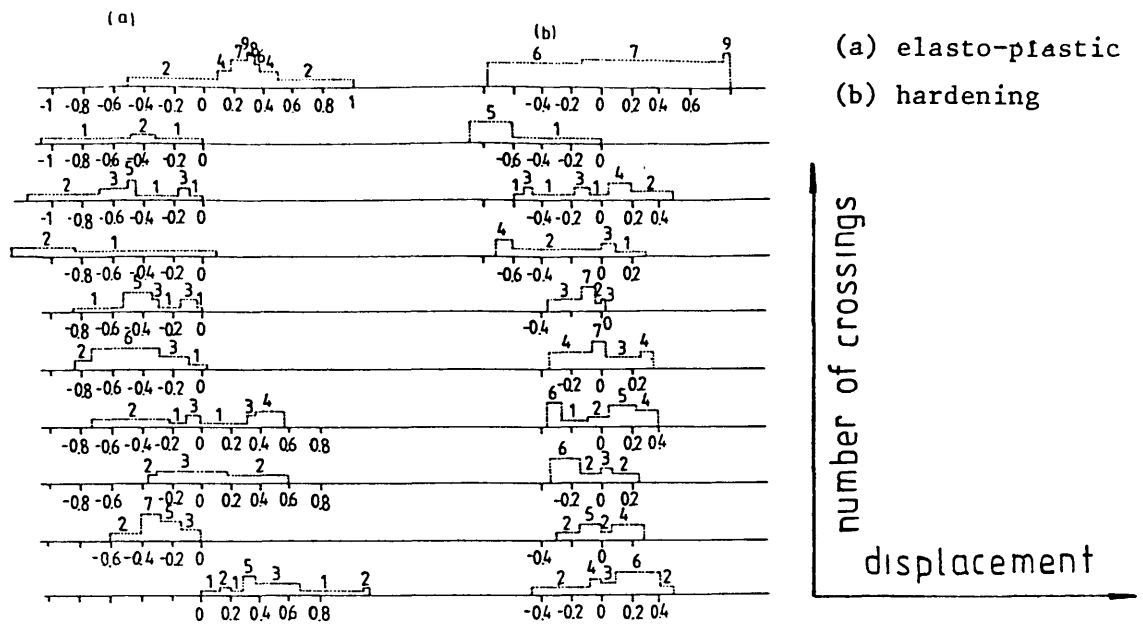


Fig. 4.25 Number of crossings of a particular inelastic displacement for $\mu=2$

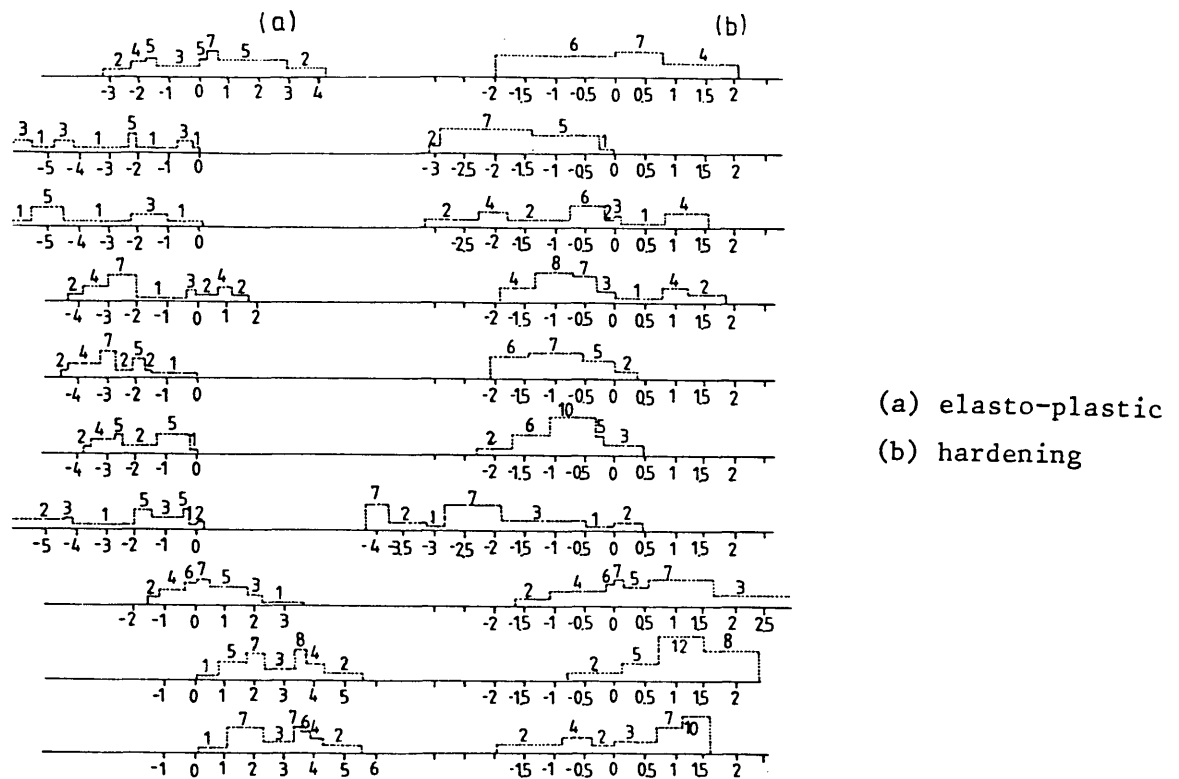


Fig. 4.26 Number of crossings of a particular inelastic displacement for $\mu=6$

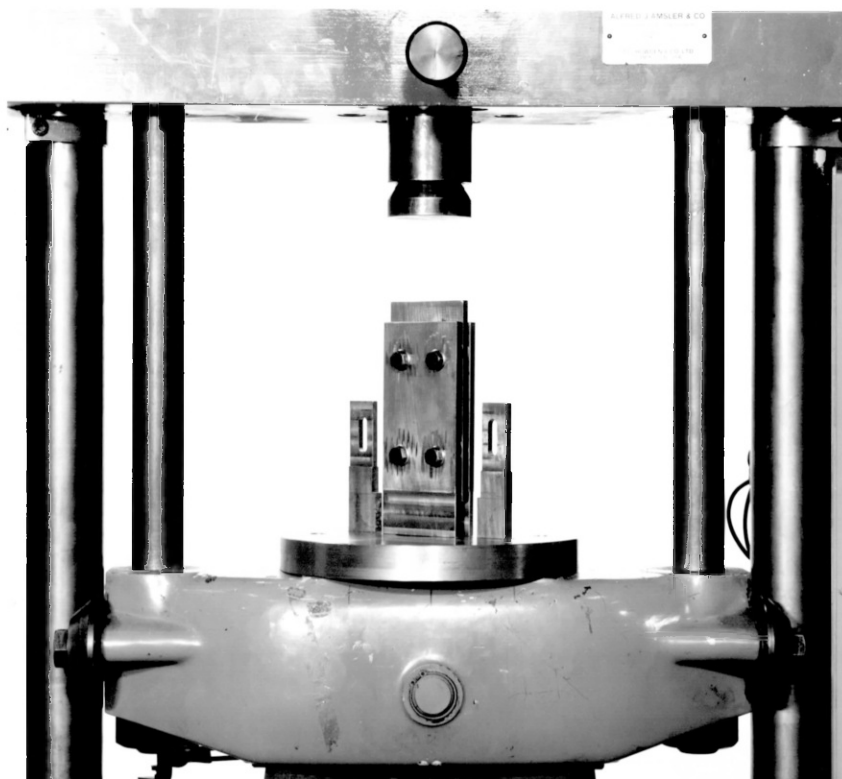


Fig.5.1. The small scale slipping device tested.

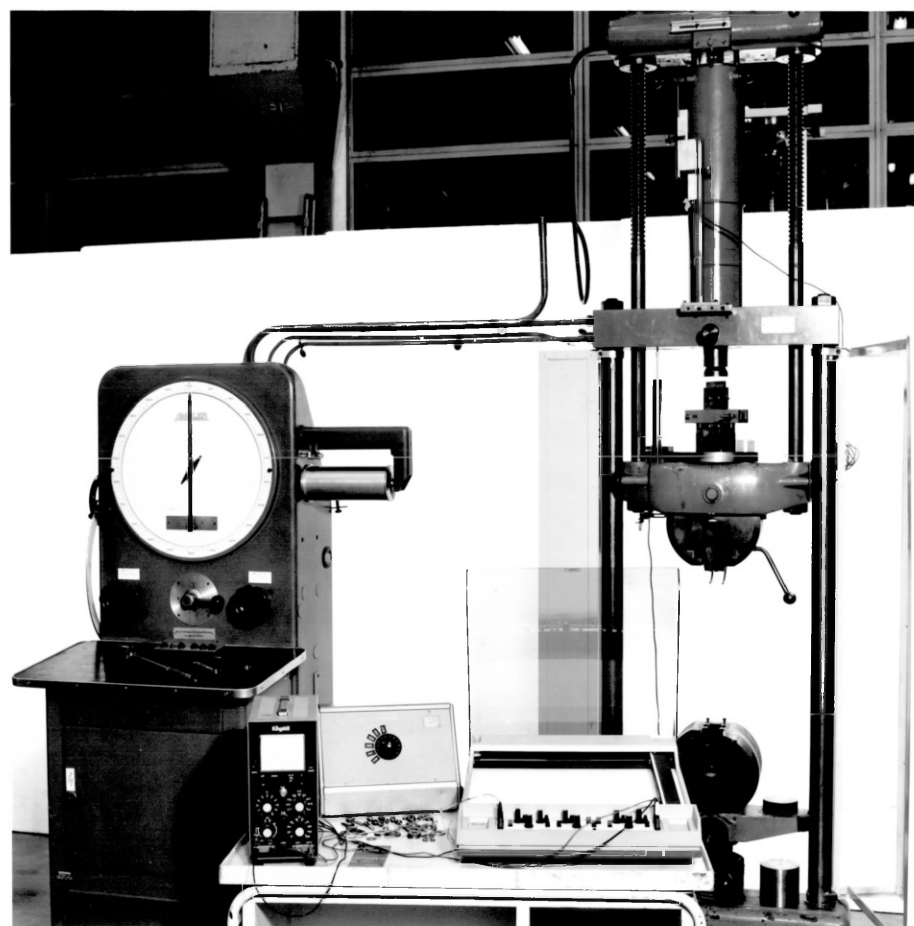


Fig.5.2. Instruments used in the slipping tests.

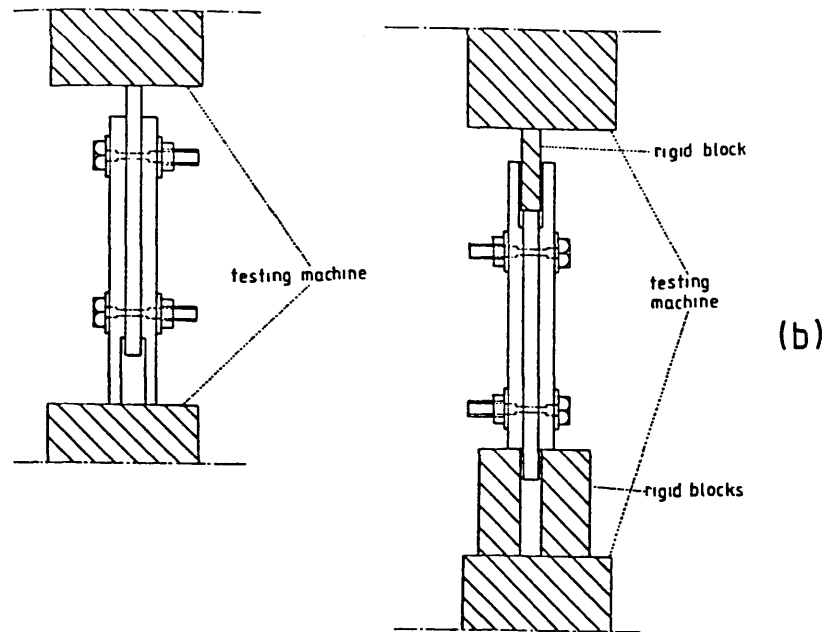
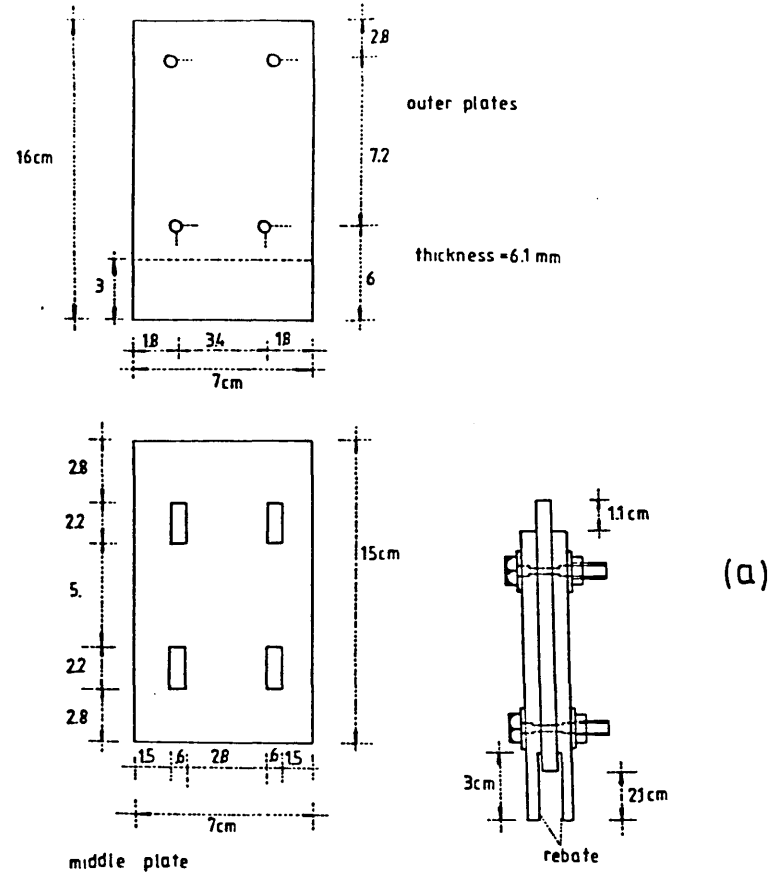


Fig. 5.3 Dimensions of the device tested and testing arrangement

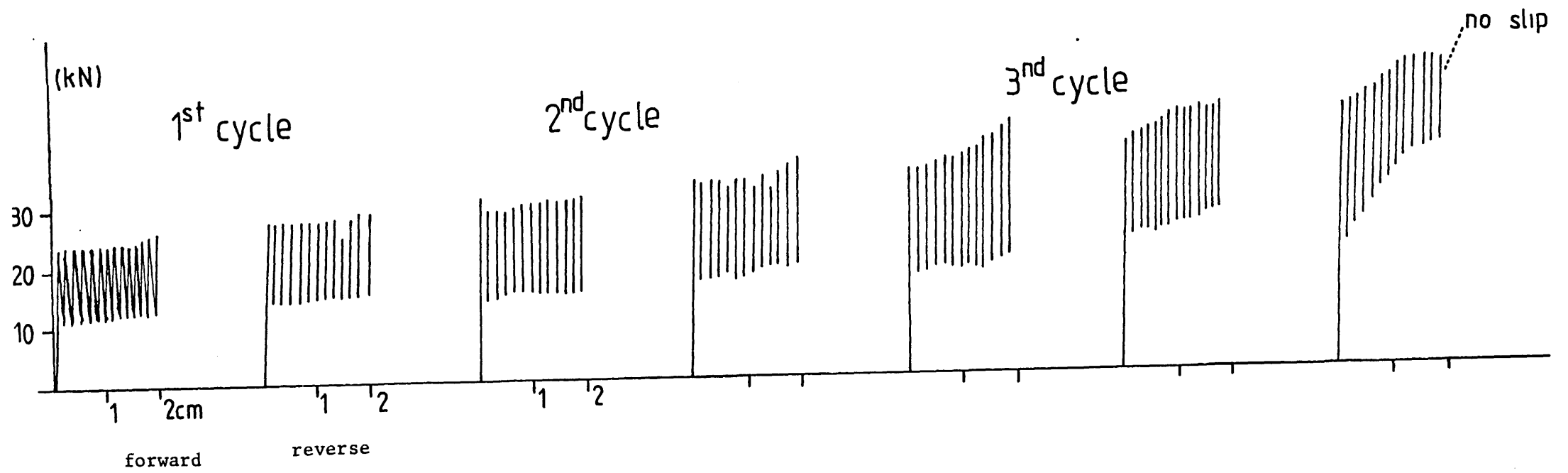


Fig. 5.4 Load-deflection curves for the small slipping device

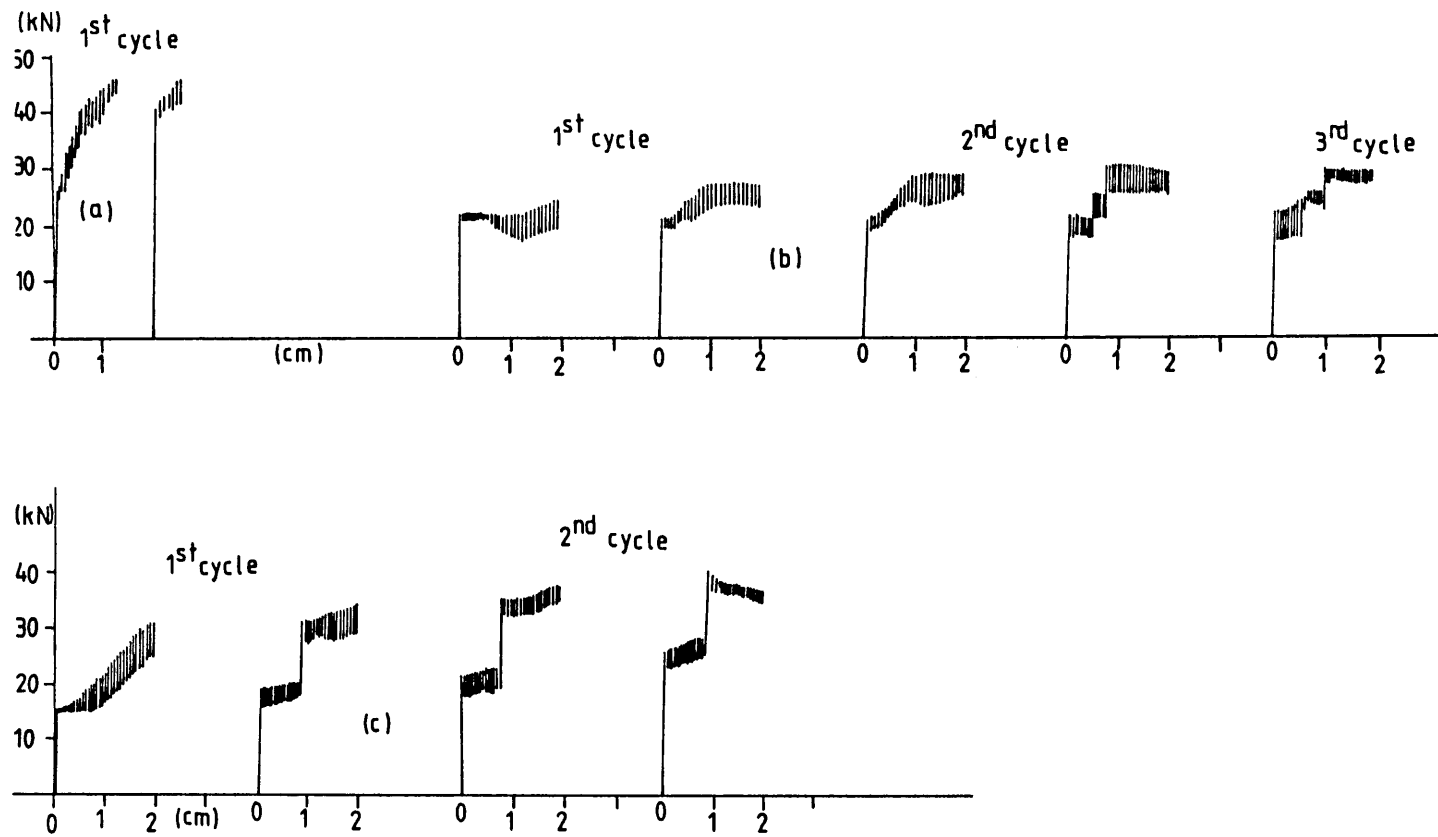


Fig. 5.5 Load-deflection curves for the small slipping device having brass plates as faying surfaces

Brass plate thicknesses

(a) 0.5mm (b) 1mm (c) 1.5mm

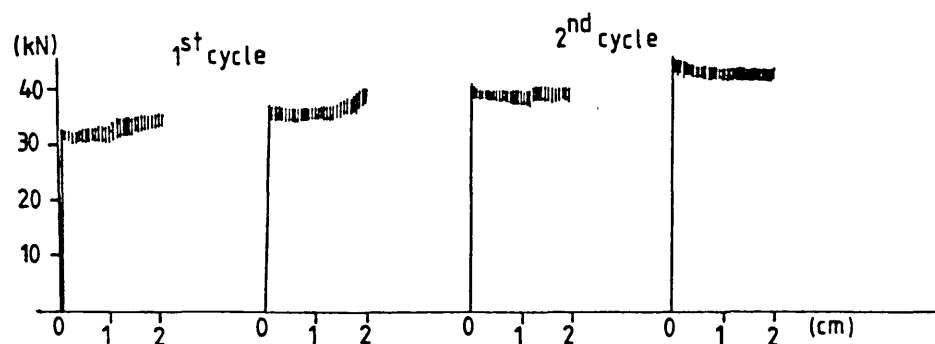


Fig. 5.6 Hysteretic response of the slipping device using the 1.5mm thickness brass plates with slotted holes

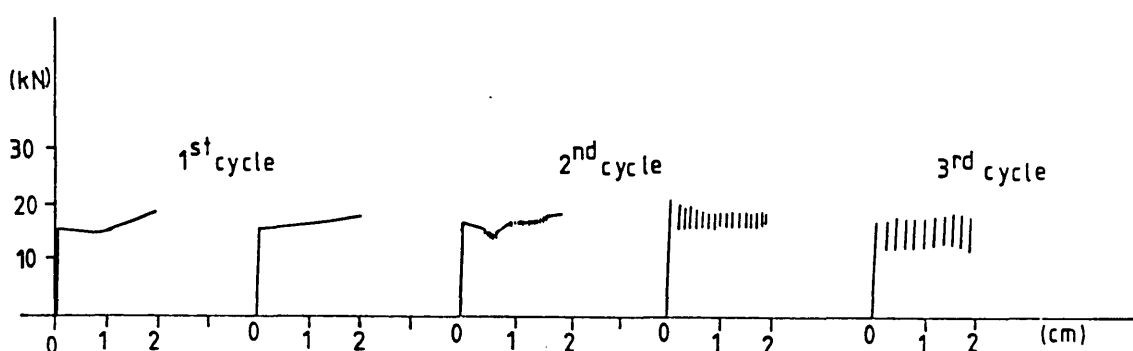


Fig. 5.7 Hysteretic response of the slipping device having mild steel washers as faying surfaces

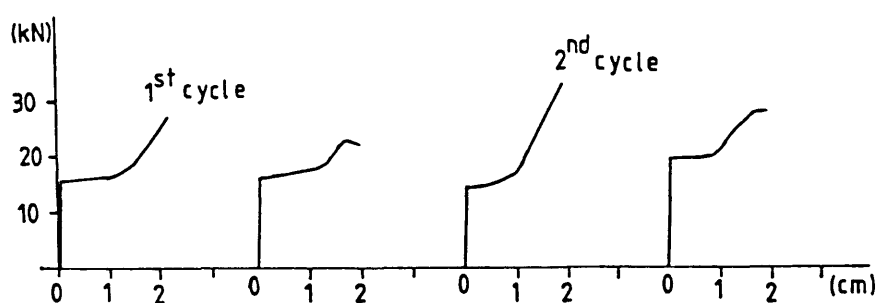


Fig. 5.8 Force-displacement curves using as faying surfaces mild steel washers having internal diameter of 7mm.

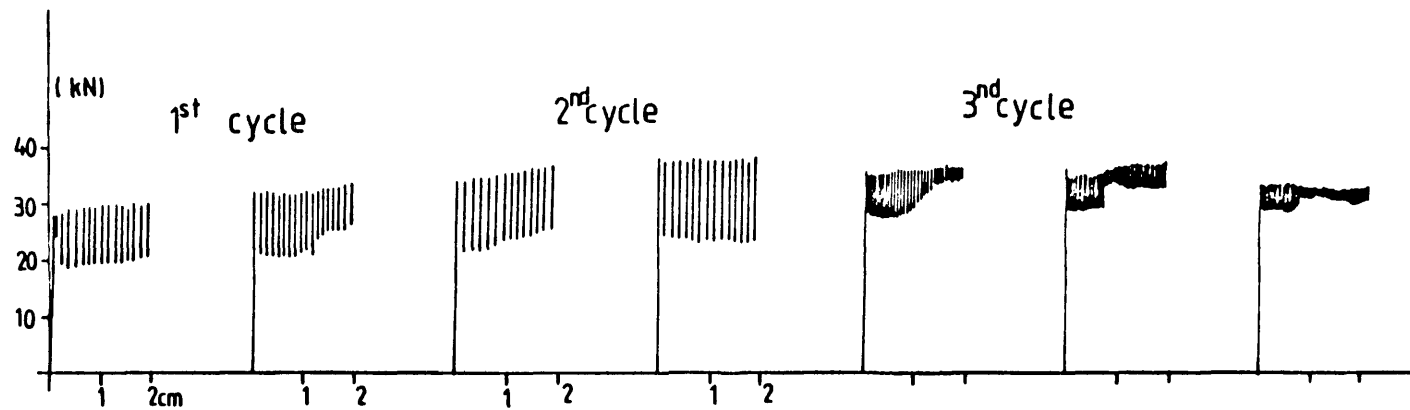


Fig. 5.9 Hysteretic response of the slipping device having high strength washers as faying surfaces

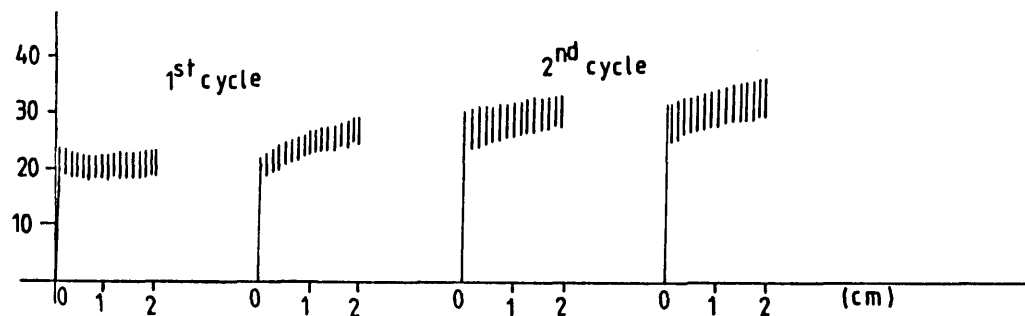


Fig. 5.10 Hysteretic response after fixing the high strength washers on the middle plate

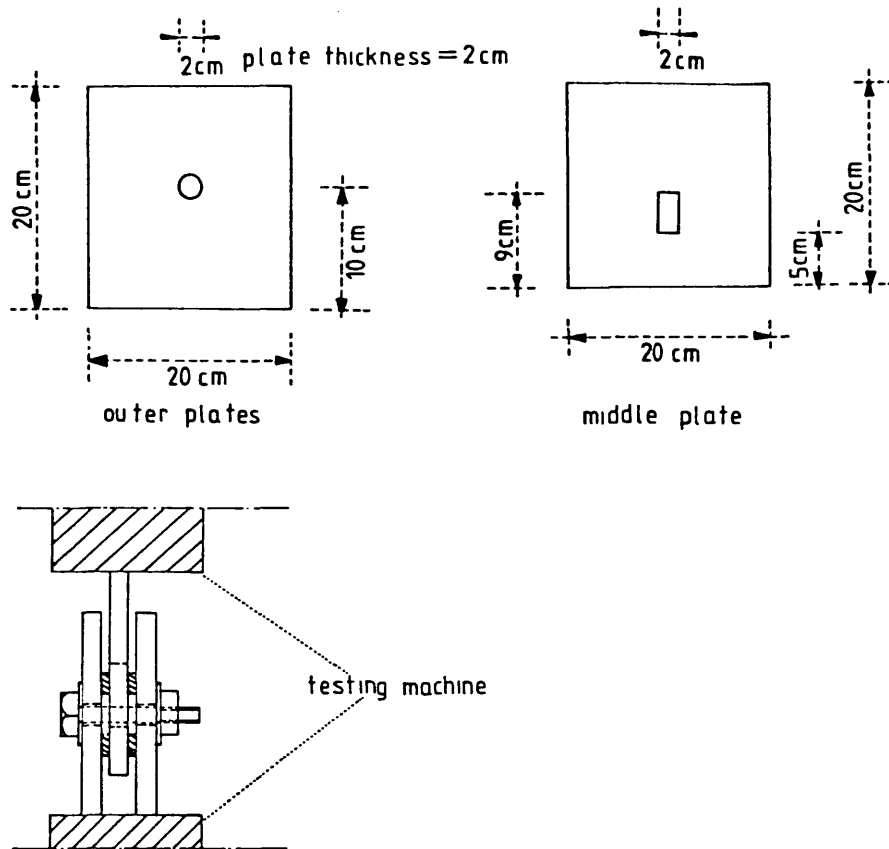


Fig. 5.11 Dimensions of the large friction joint

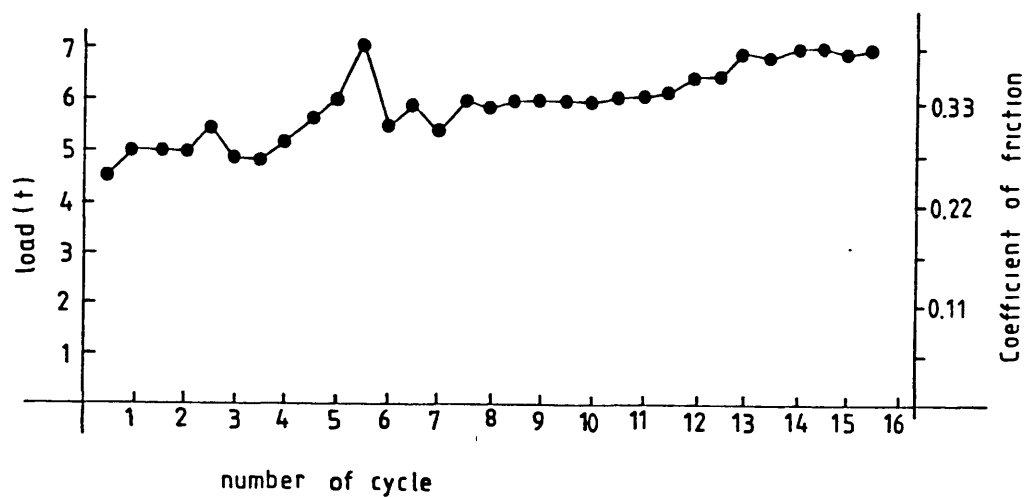


Fig. 5.12 Maximum slipping load at every half cycle of the slipping joint having ground flat stock washers as faying surfaces

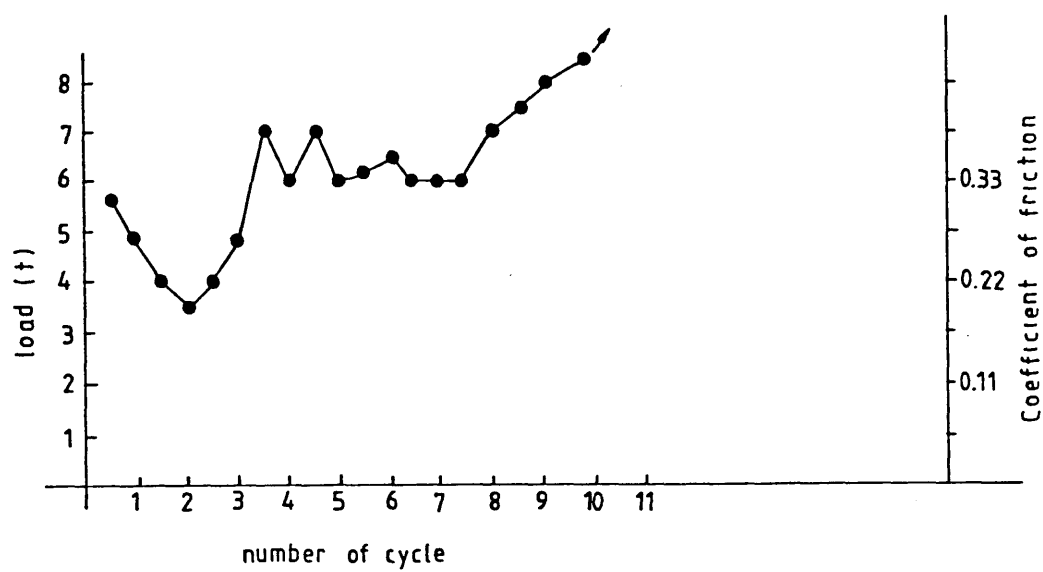


Fig. 5.13 Maximum slipping load at every half cycle of the slipping joint without introducing washers as faying surfaces

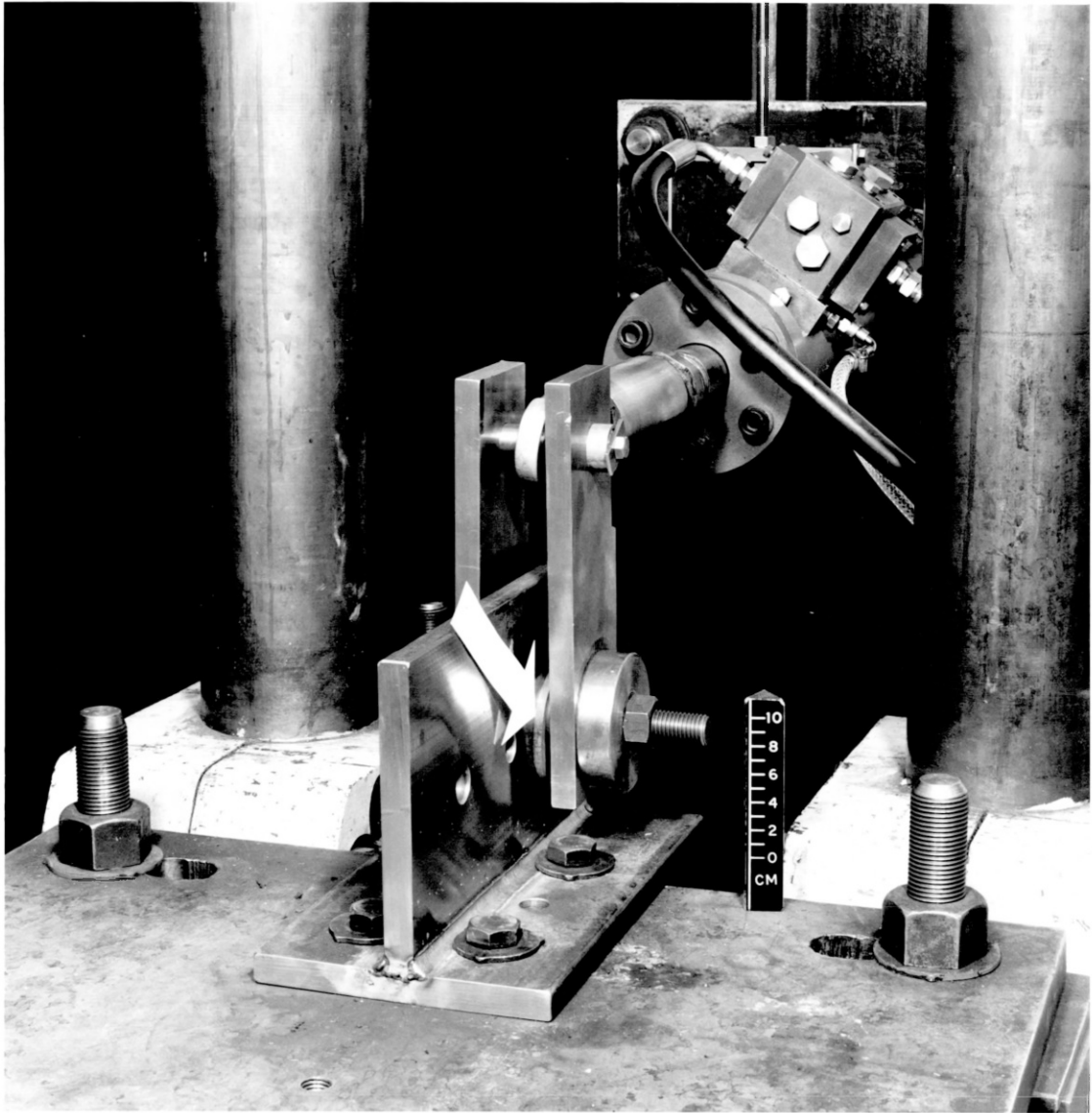
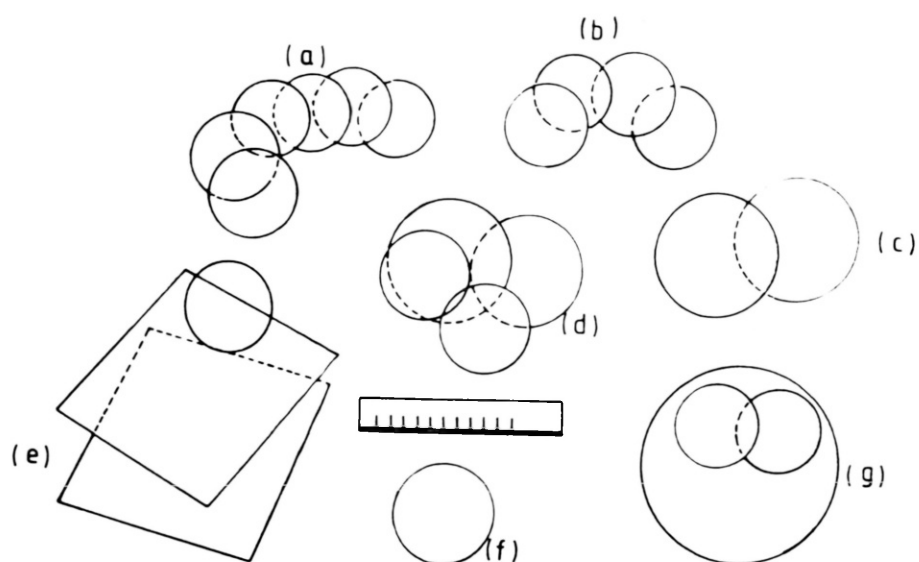


Fig. 5.14. Test rig for the friction tests.



- | | |
|-----------------------|--|
| (a) Cast iron | (e) FF Ferodo friction material |
| (b) Nitro 40B | (f) 3501F Ferodo friction material |
| (c) Stainless steel | (g) An absolute-stock Ferodo friction material |
| (d) Ground flat stock | |

Fig.5.I5. Different friction materials tested.

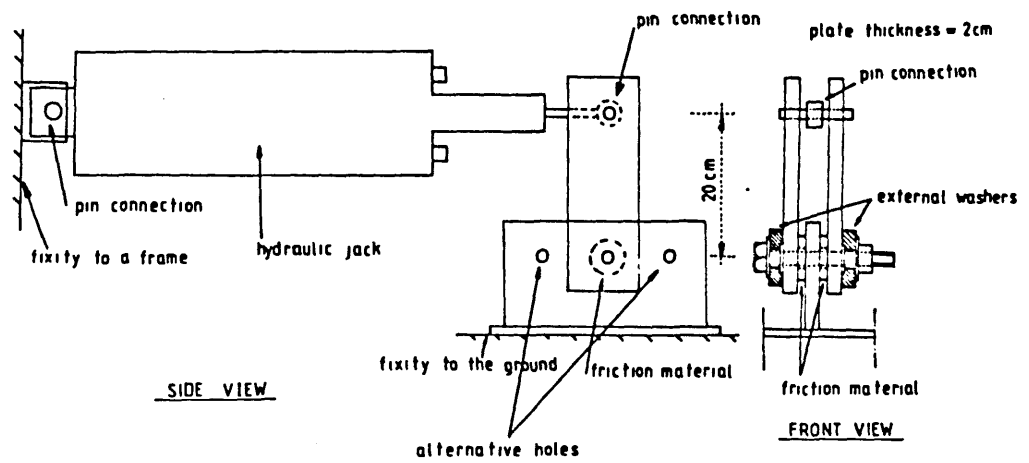


Fig. 5.16 Details of the friction tests arrangement

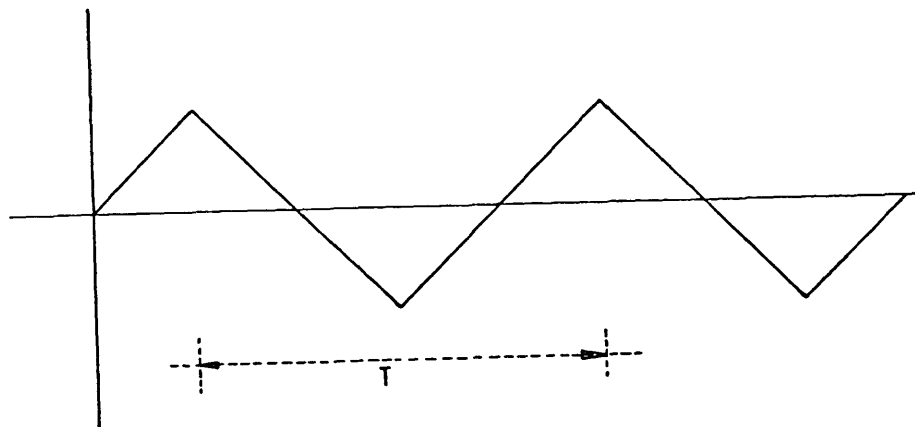
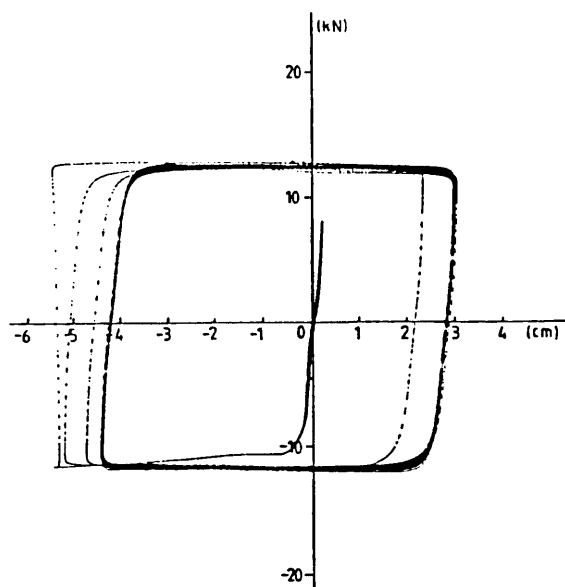
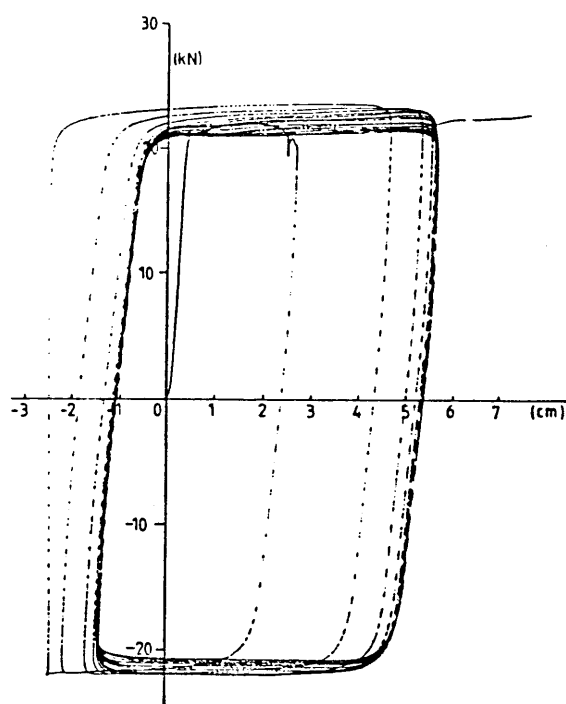


Fig. 5.17 Time history of the applied stroke

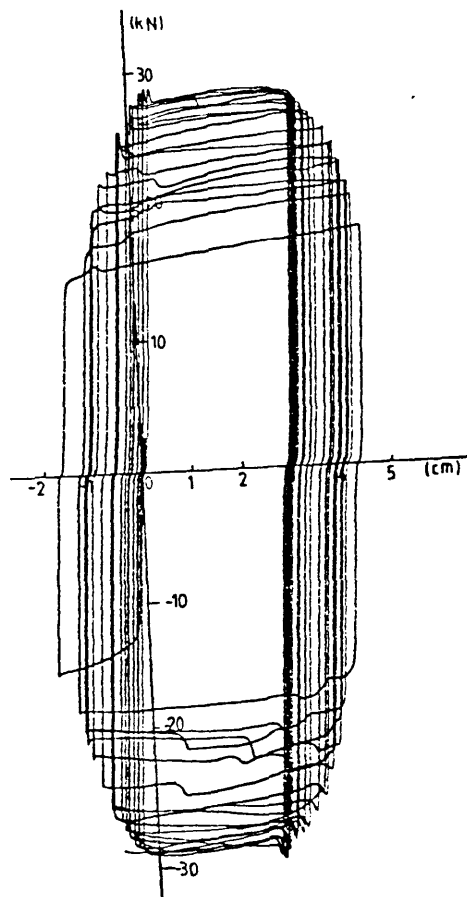


(a) Ground flat stock/ 22 N/mm^2

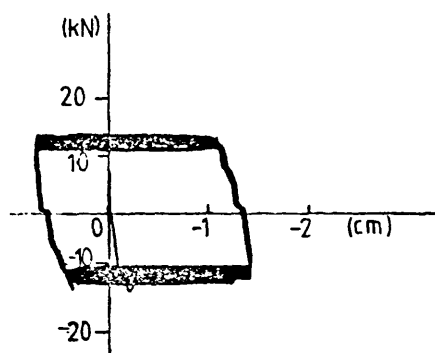


(b) Ground flat stock/ 31 N/mm^2

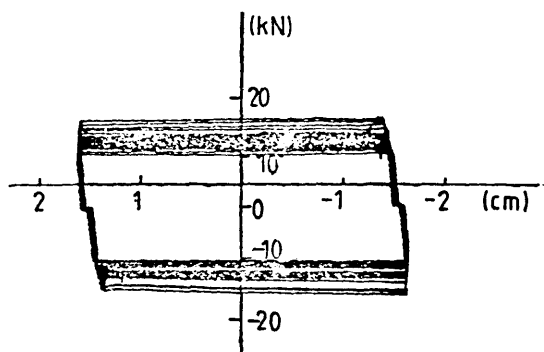
Fig. 5.18 Hysteretic response of the rotating friction joint



(c) Ground flat stock/48 N/mm²

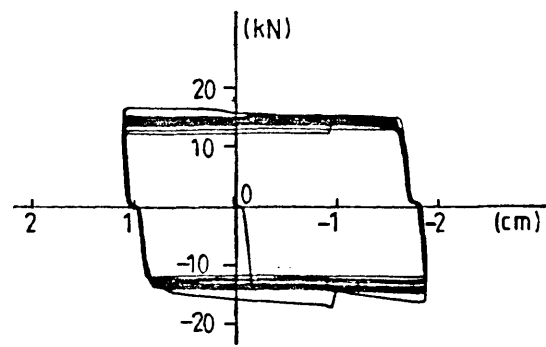
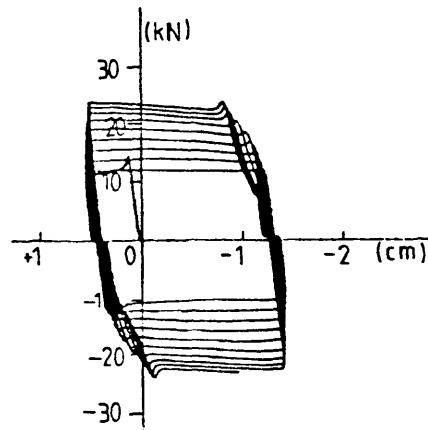
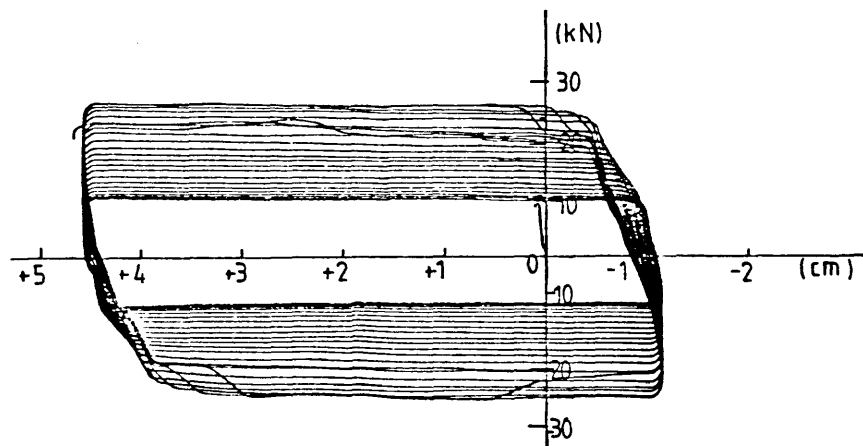
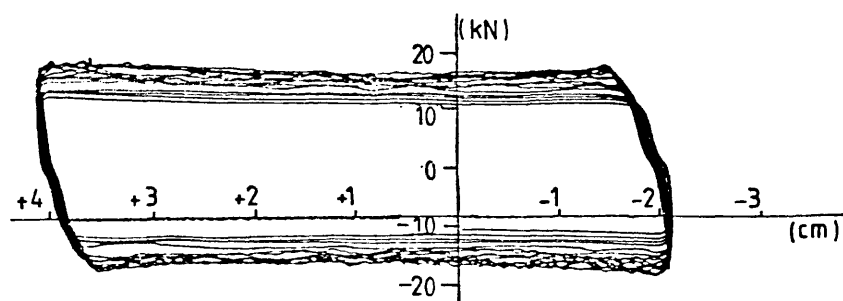


(d) Ground flat stock/48 N/mm²

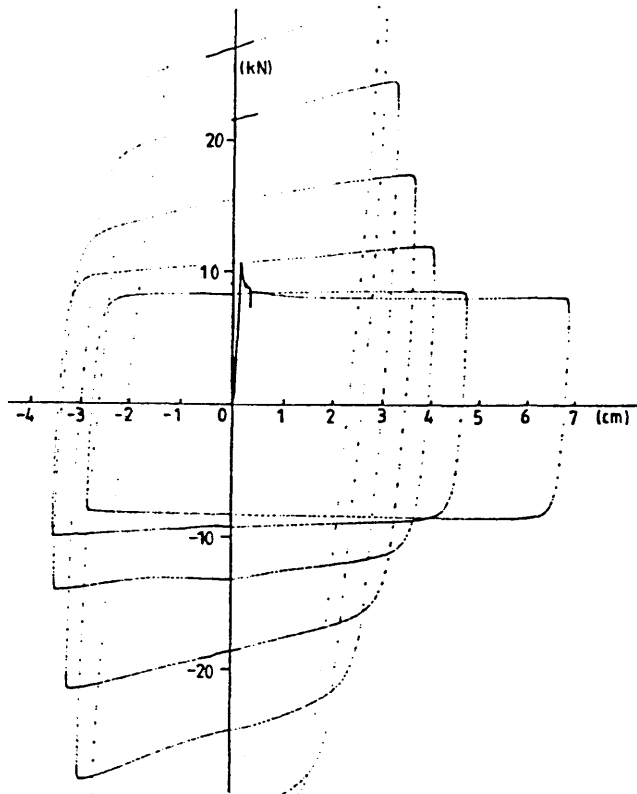
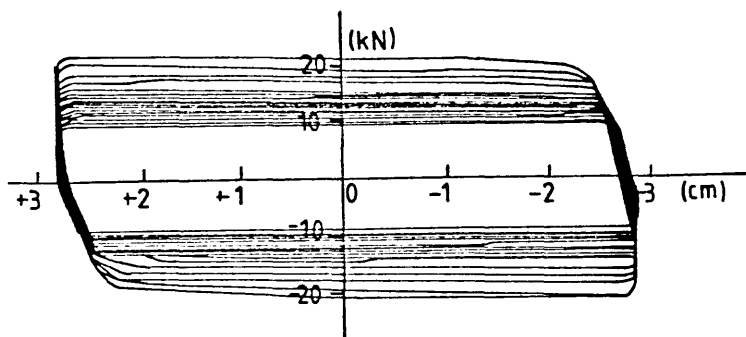
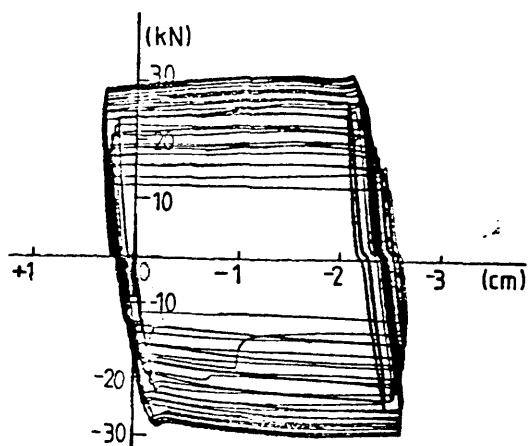


(e) Ground flat stock/48 N/mm²

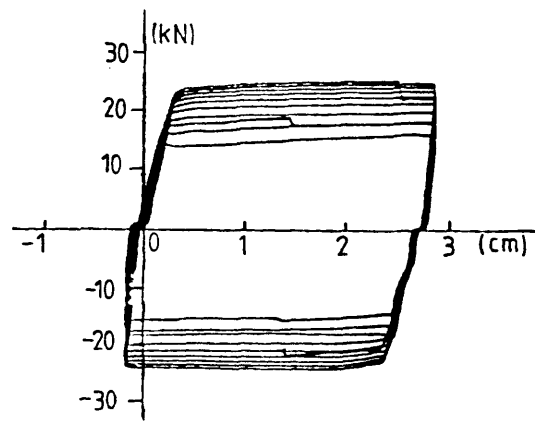
(continued)

(f) Ground flat stock/23 N/mm²(g) Ground flat stock/48 N/mm²(h) Ground flat stock/48 N/mm²(i) Ground flat stock/48 N/mm²

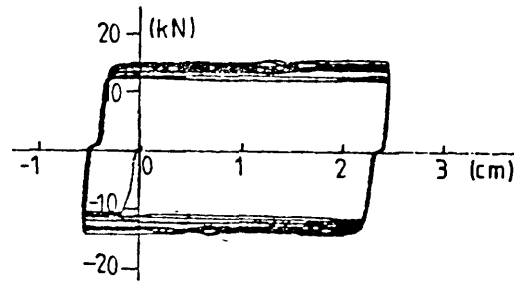
(continued)

(j) Cast iron/44 N/mm²(k) Cast iron/44 N/mm²(l) Nitroty 40B/44 N/mm²

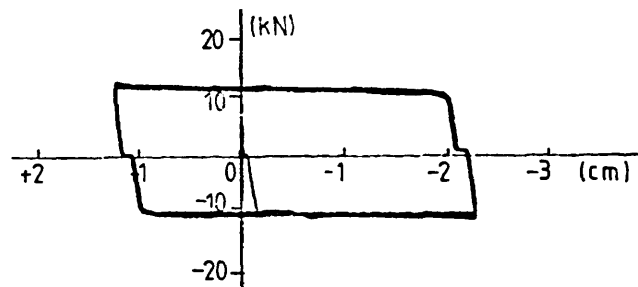
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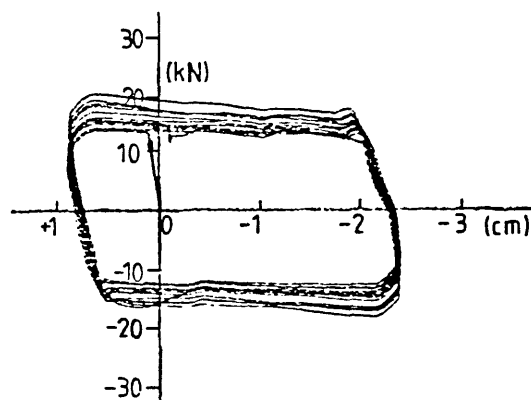
(m) Nitroy 40B/42 N/mm^2



(n) Nitroy 40B/42 N/mm^2

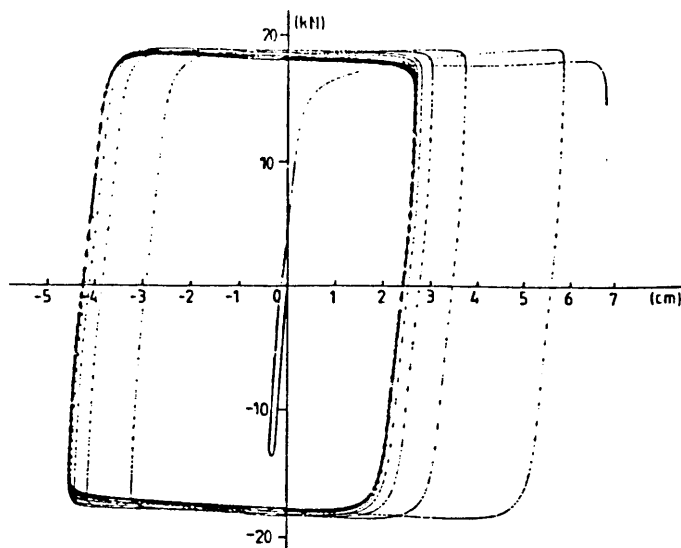
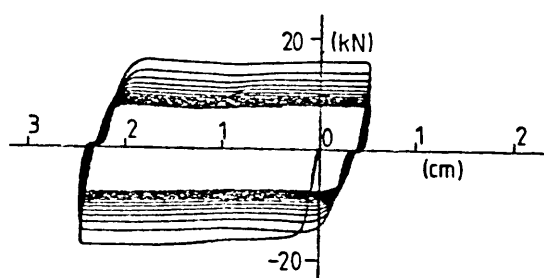
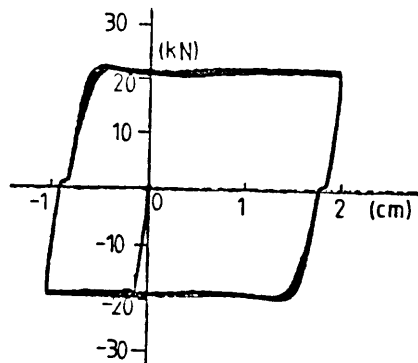
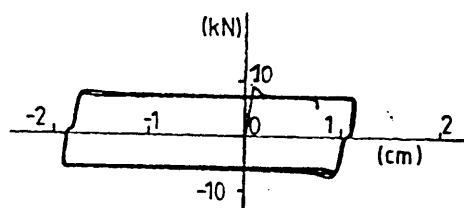


(o) 500k-Monel/23 N/mm^2

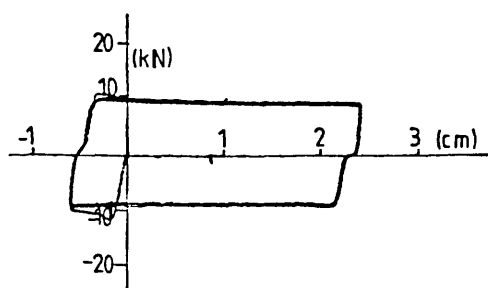


(p) 500k-Monel/23 N/mm^2

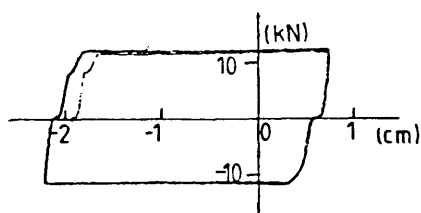
(continued)

(q) 3501F/31 N/mm^2 (r) FF/48 N/mm^2 (s) FF/23 N/mm^2 (t) Proposed Ferodo/104 N/mm^2

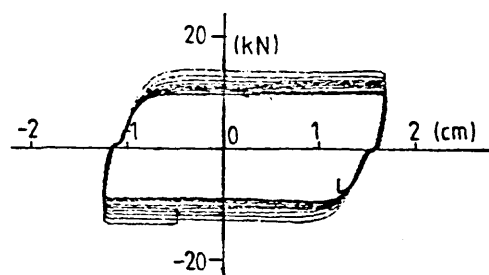
(continued)



(u) 3701F/39 N/mm²



(v) 3701F/58 N/mm²



(w) 3904F/58 N/mm²

(continued)

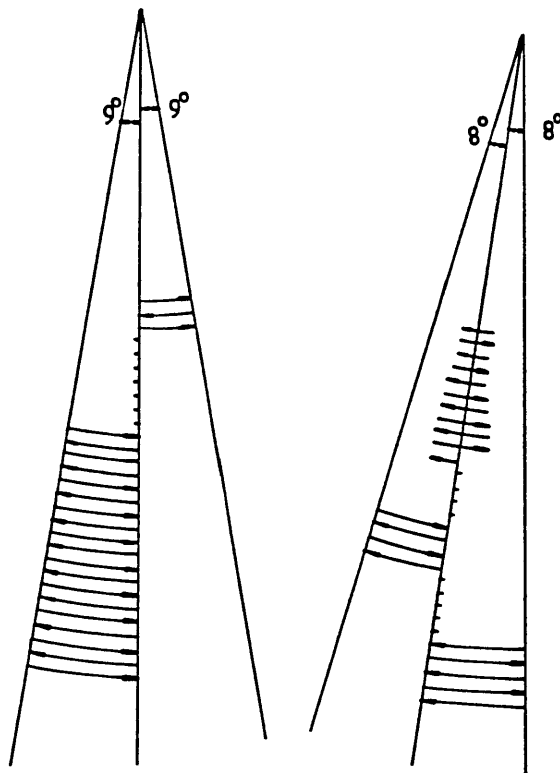


Fig. 5.19 Relative rotations between washers and plates

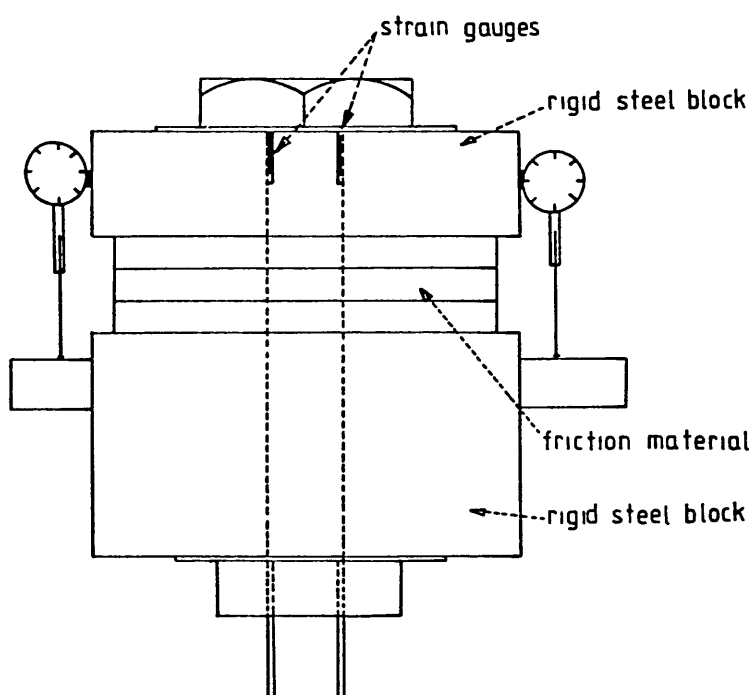


Fig. 5.20 Arrangements for the creep tests

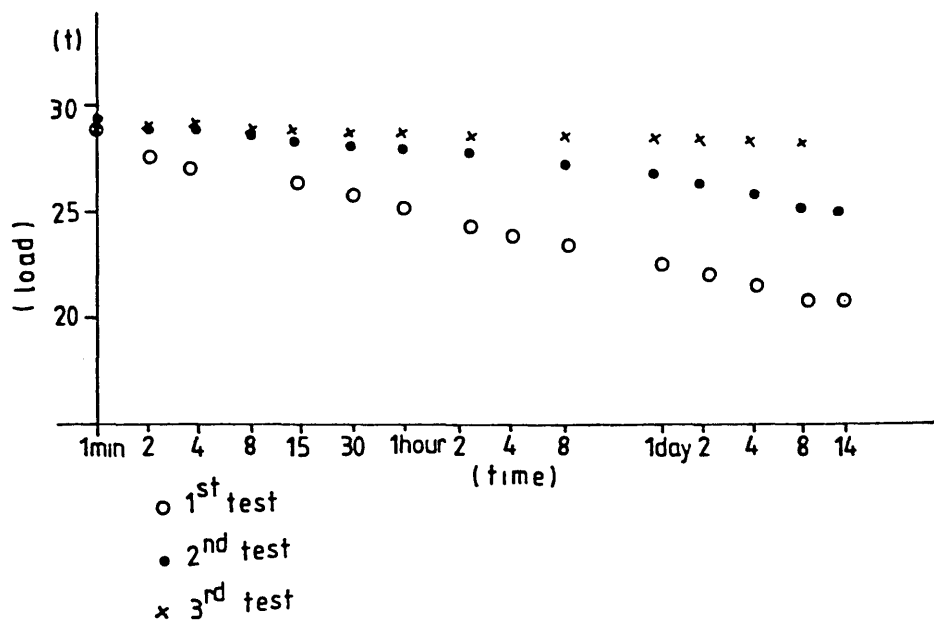


Fig. 5.21 Reduction of the load on the bolt with time in the creep tests

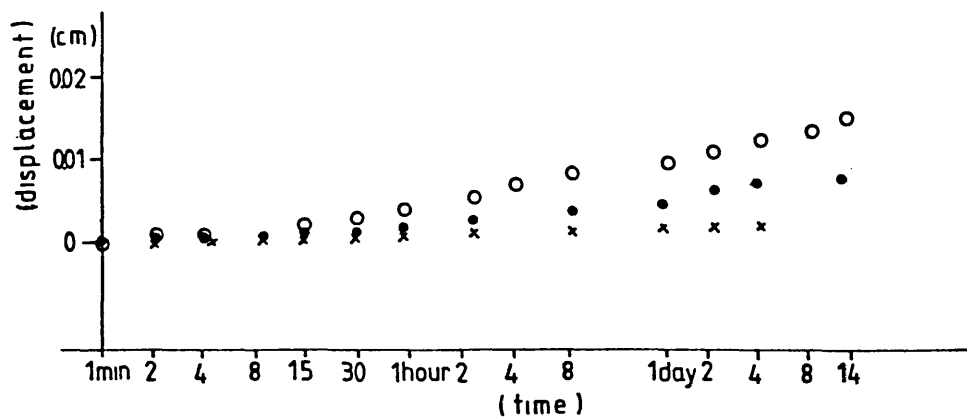
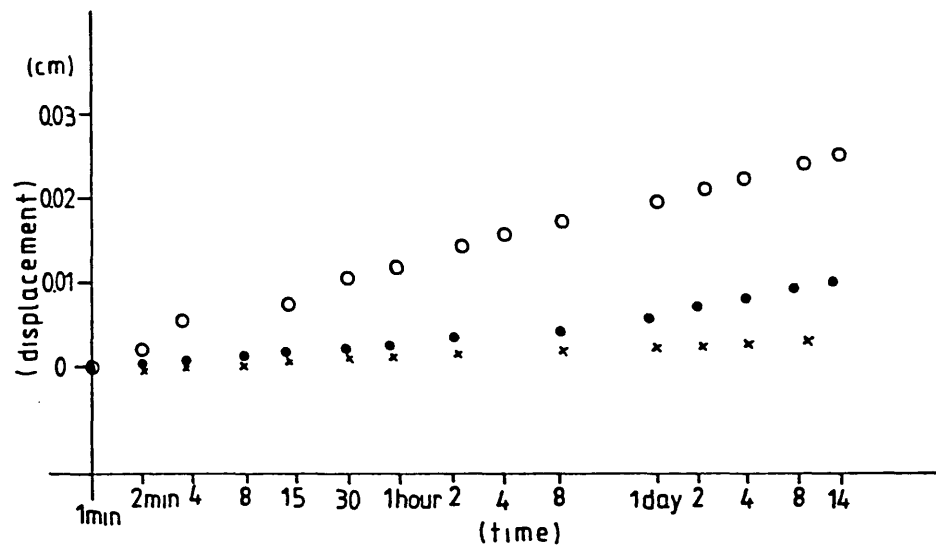


Fig. 5.22 Dial gauge readings in the creep tests

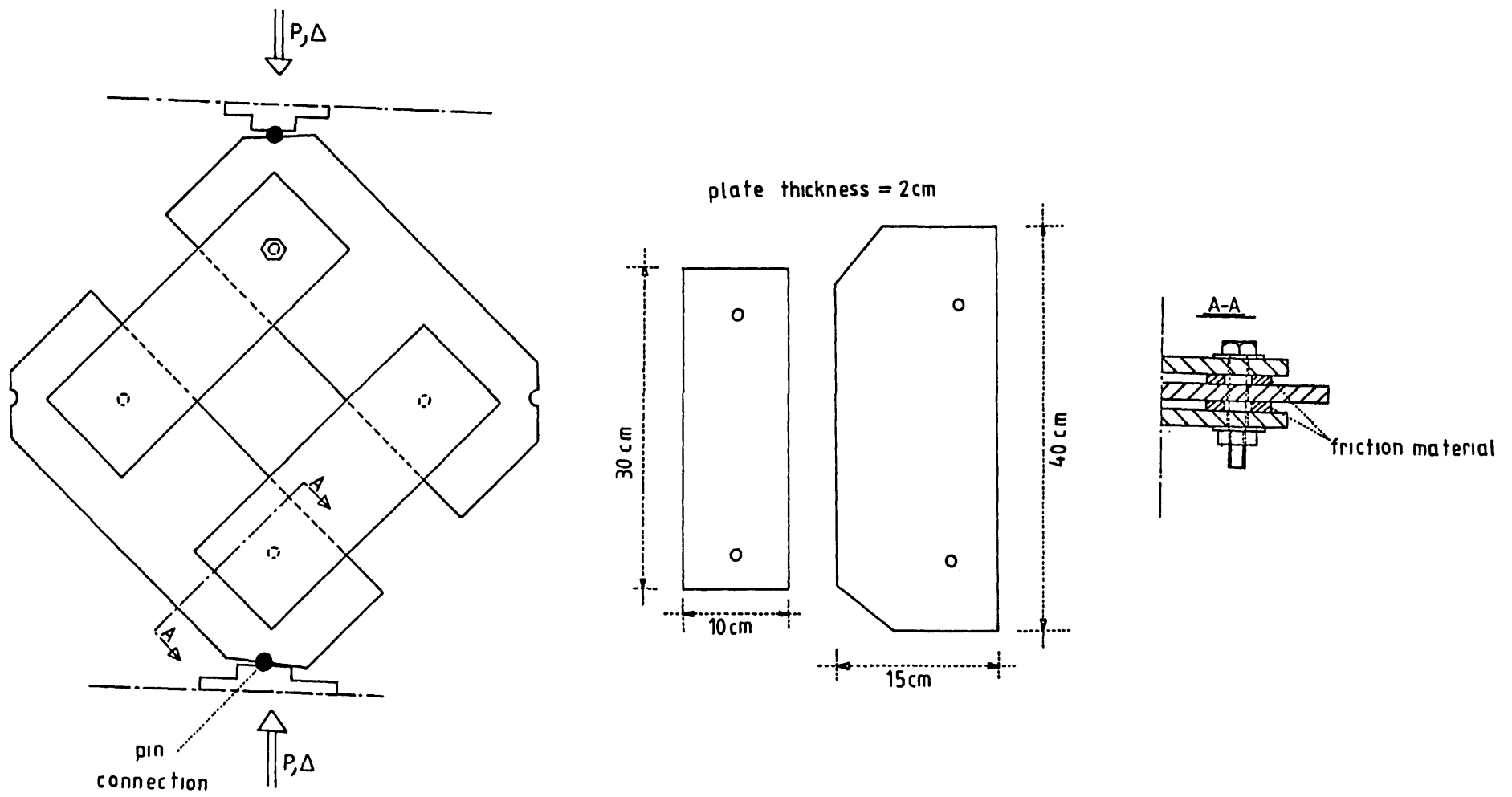
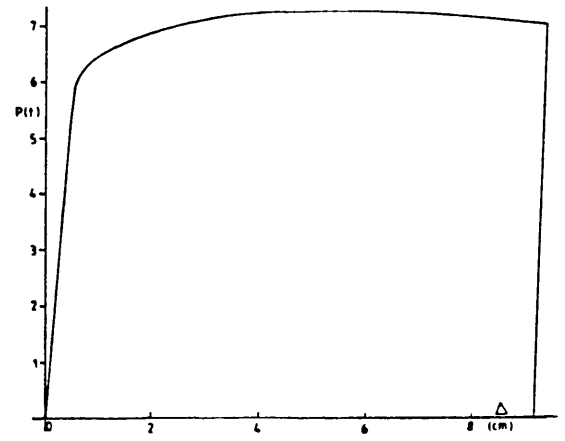
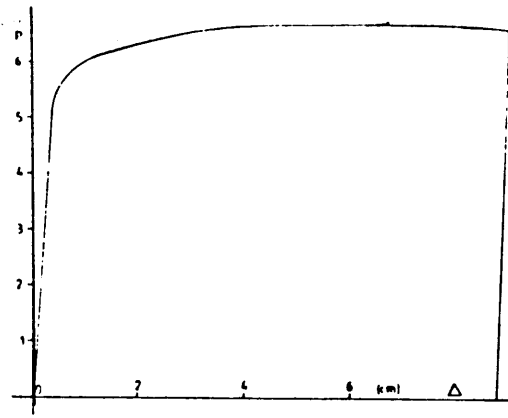


Fig. 5.23 The large rotating device tested

1st cycle

240



5th cycle

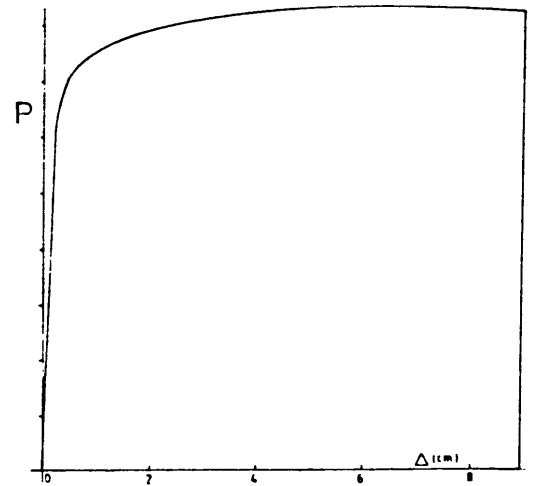
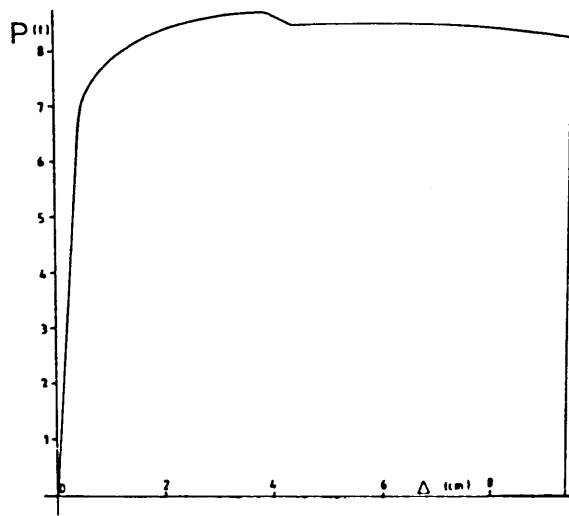


Fig. 5.24 Hysteretic response of the rotating device

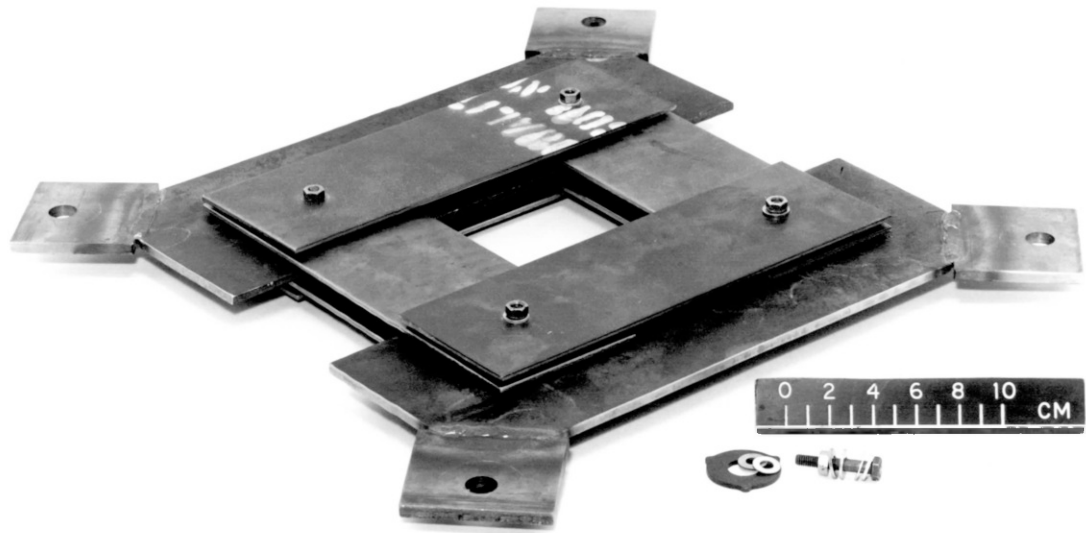


Fig.5.25. The model rotating device.

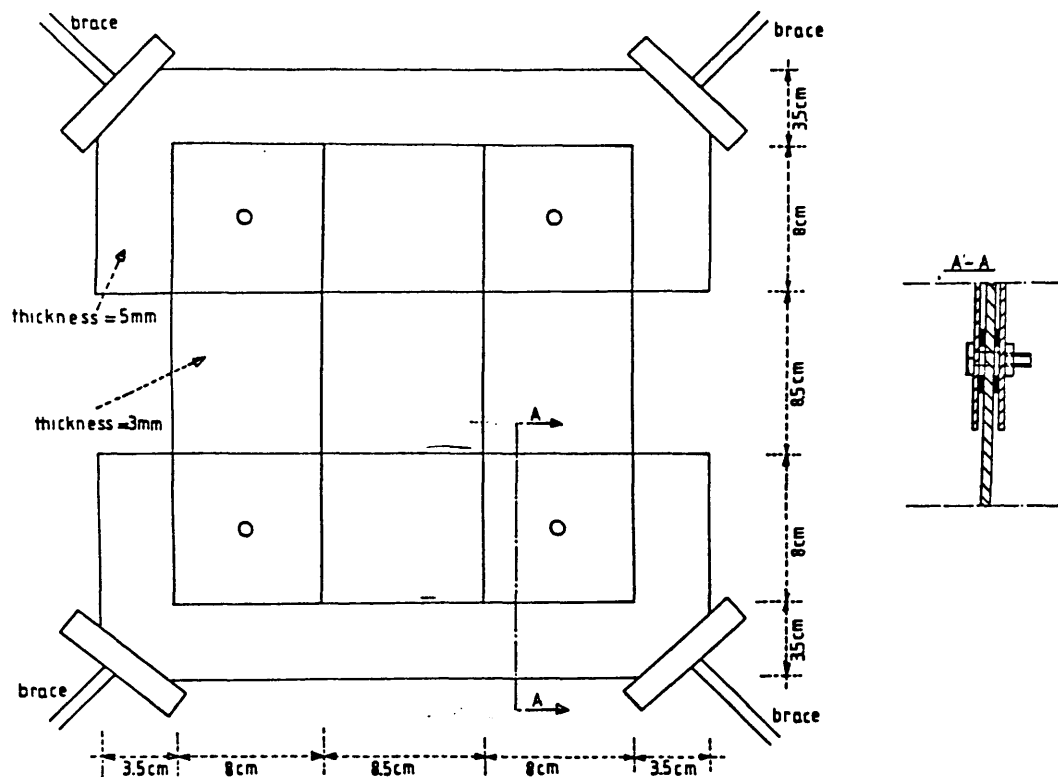


Fig. 5.26 Dimensions of the small rotating device

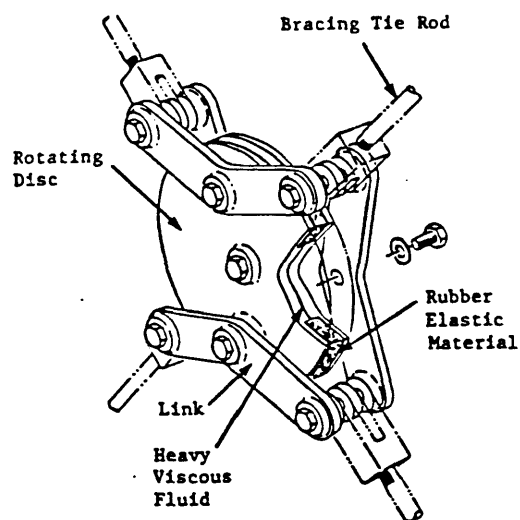


Fig. 5.27 The brace damper developed by Scholl (ref(68))

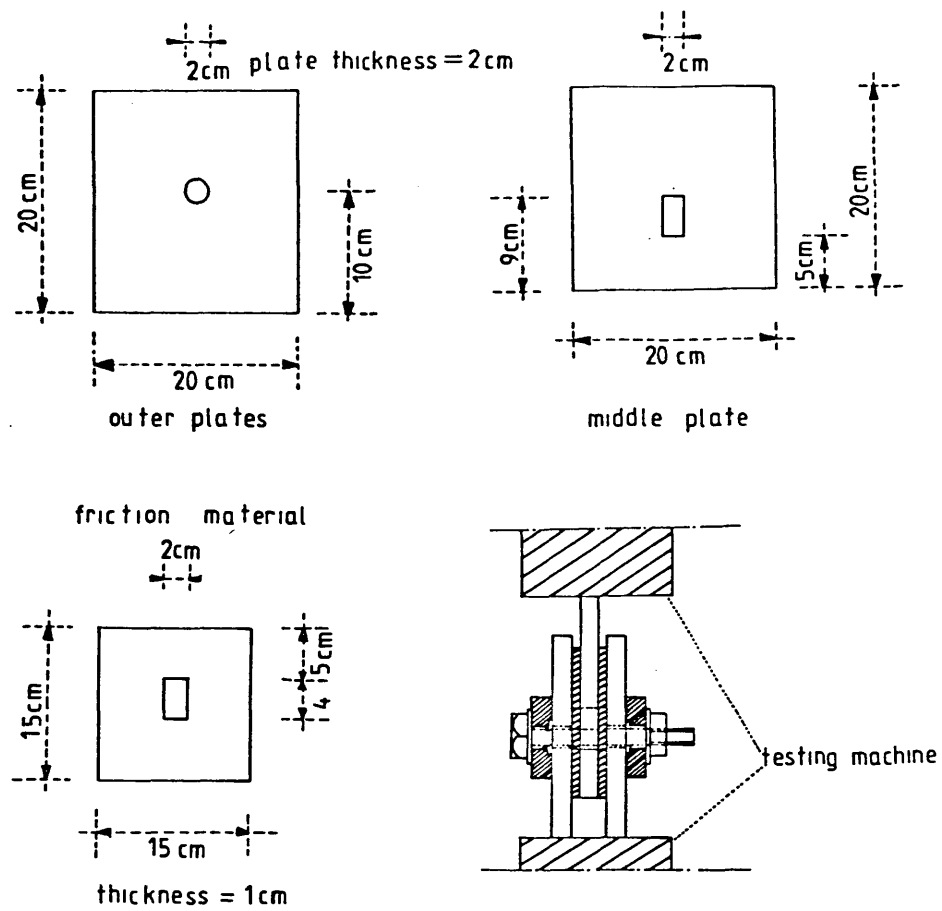


Fig. 5.28 Dimensions of the slipping joint having the 3701F Ferodo friction material

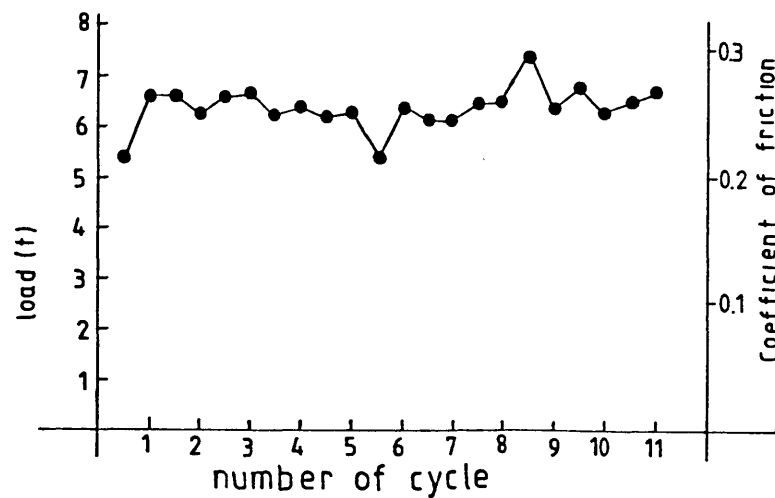


Fig. 5.29 Maximum slipping load at every half cycle on the slipping joint

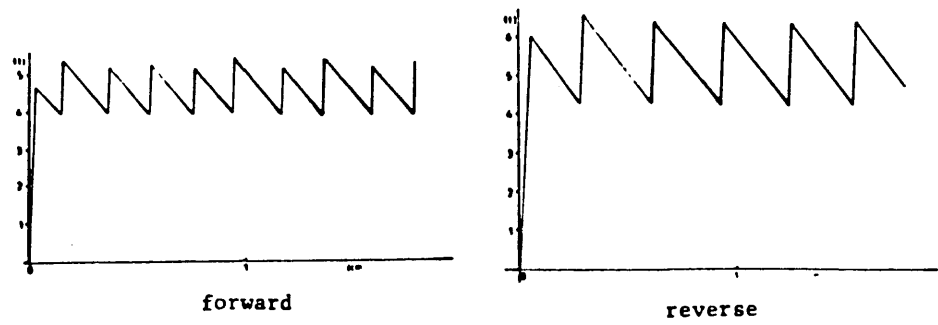
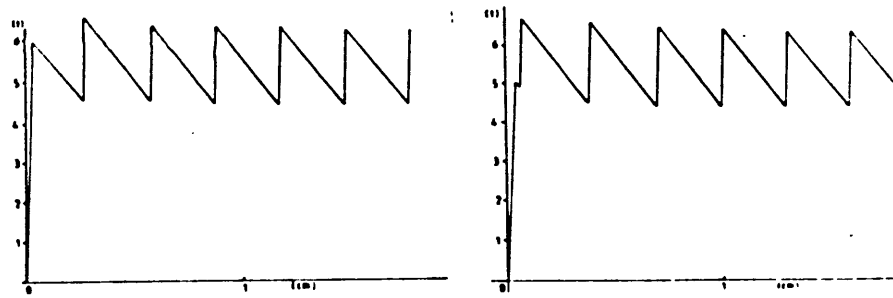
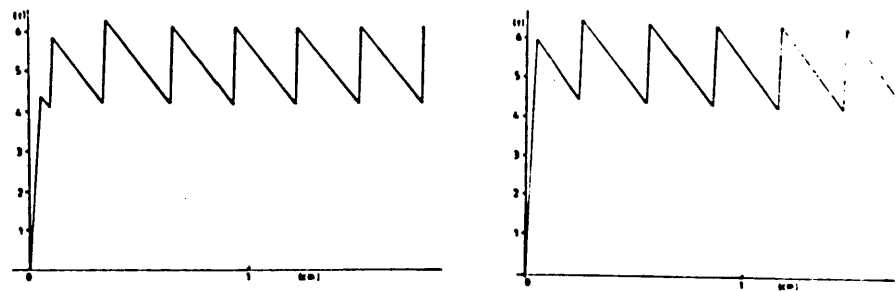
2nd cycle3rd cycle

Fig. 5.30 Hysteresis loops of the slipping joint for the first three cycles

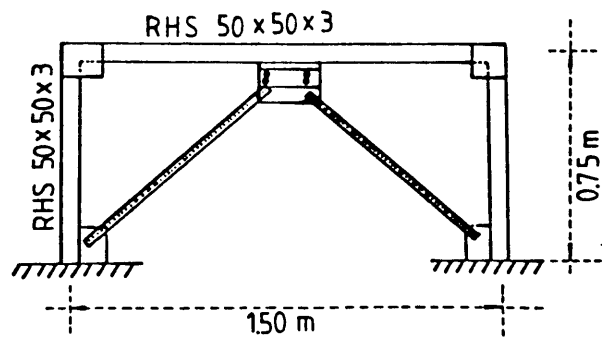


Fig. 5.31 Dimensions of the K-braced frame incorporating a slipping device

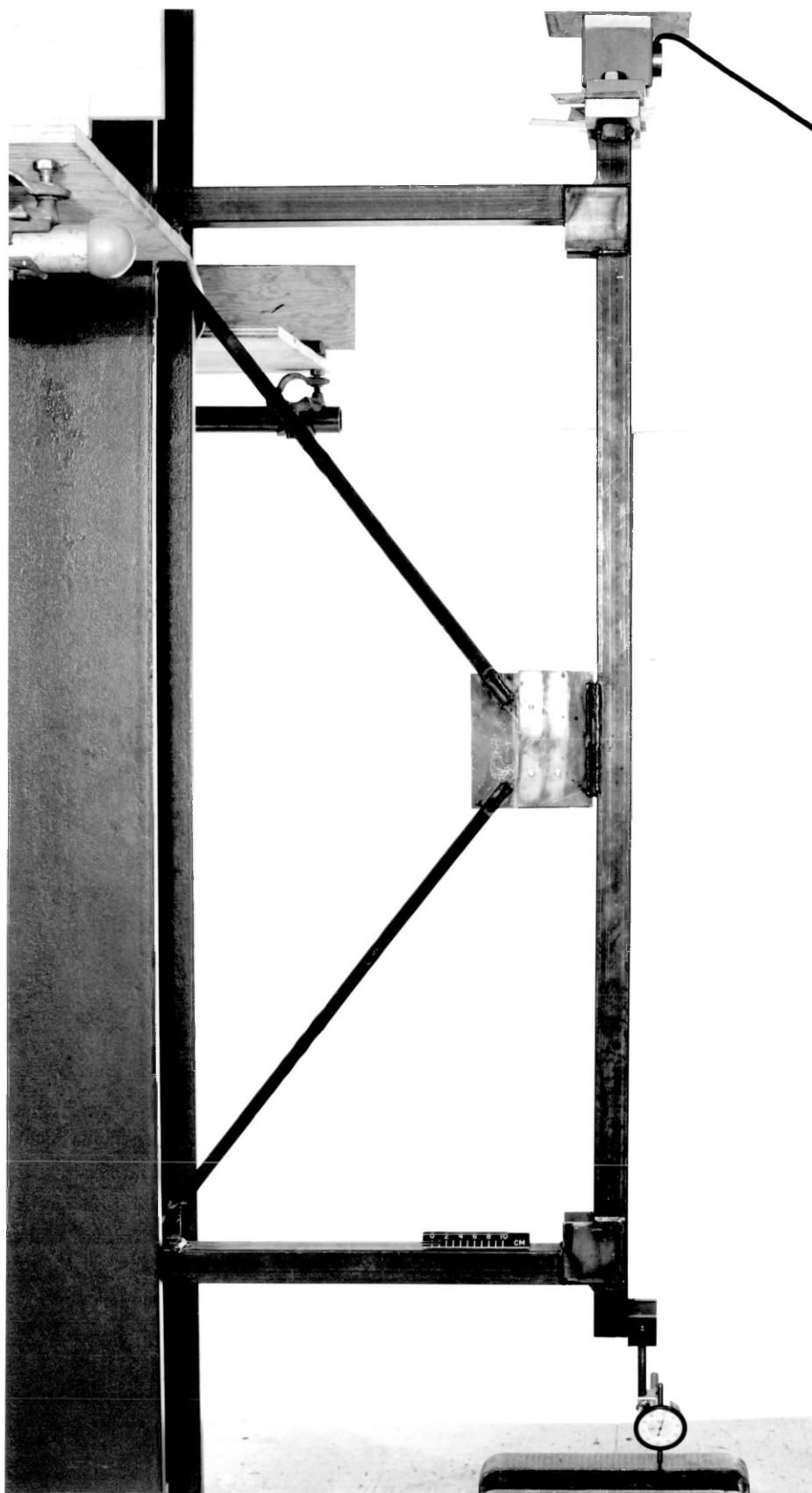


Fig.5.32. The k-braced frame incorporating a slipping device.

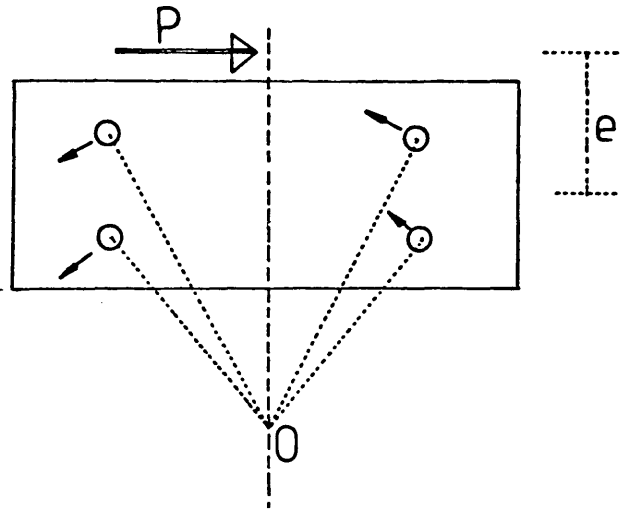


Fig. 5.33 Eccentrically applied load in fasteners

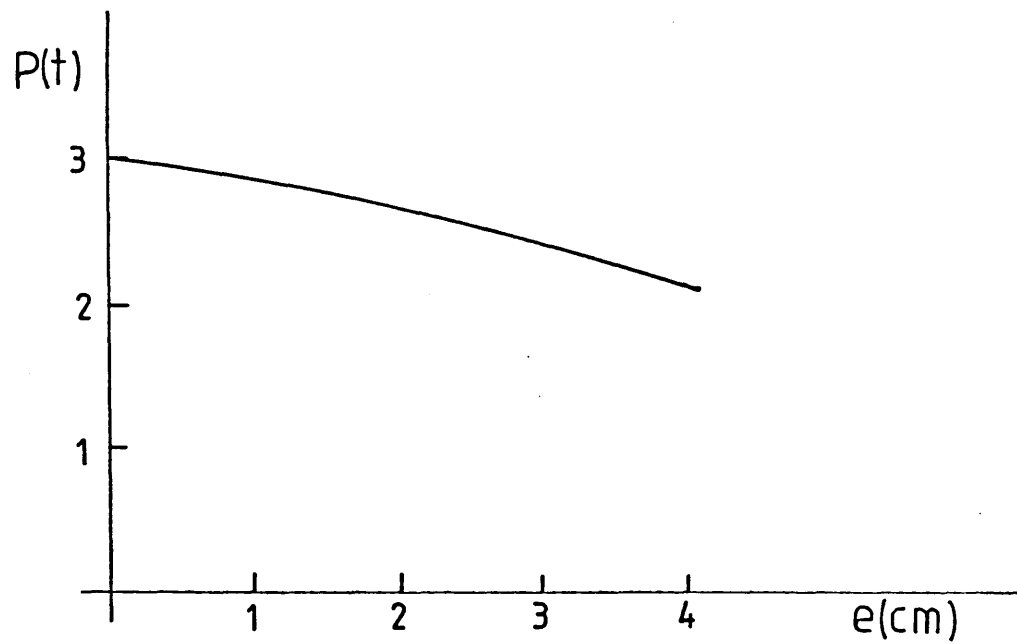


Fig. 5.34 Reduction of the slipping load of the device as a function of the eccentricity

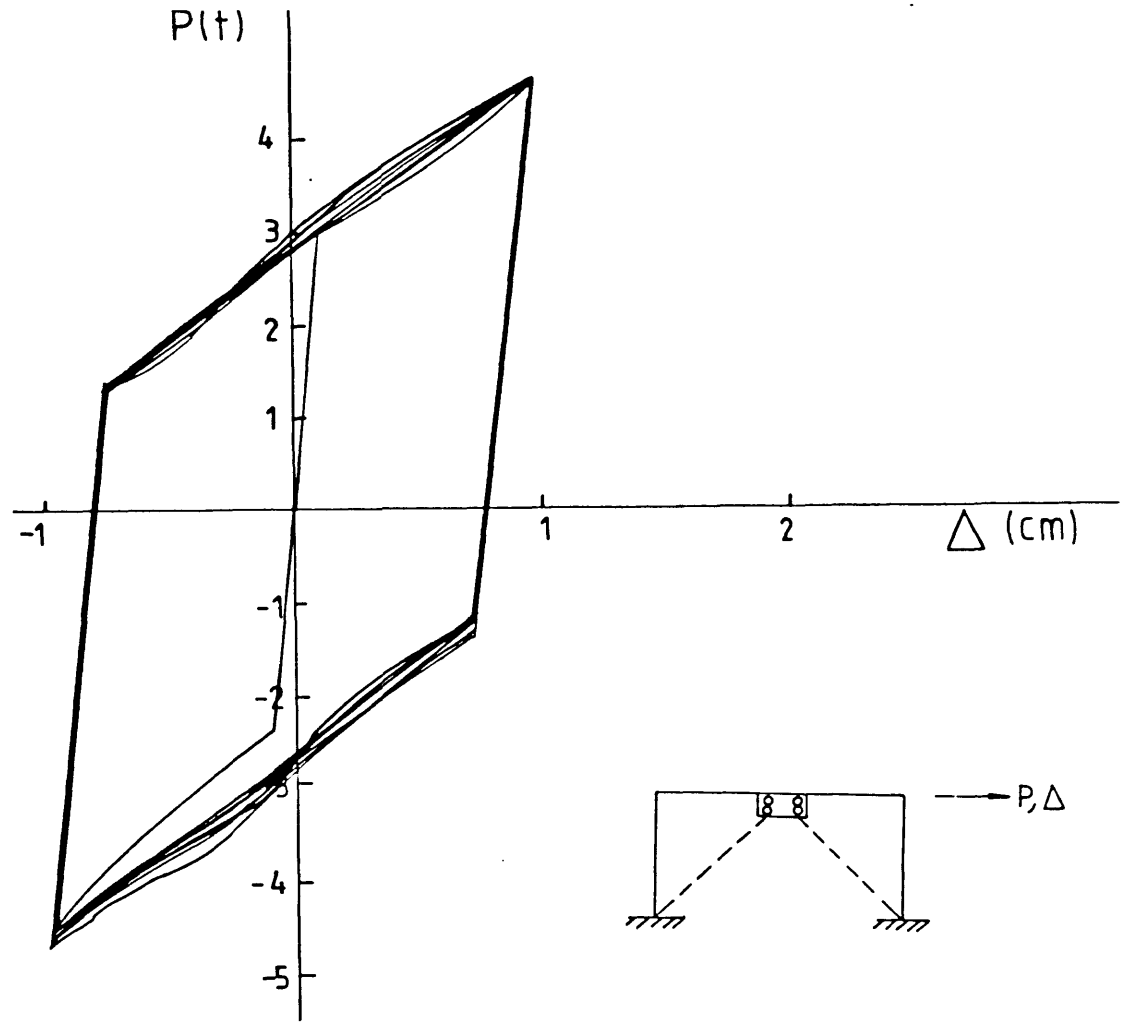


Fig. 5.35 Hysteresis loops of the K-braced frame incorporating a slipping device

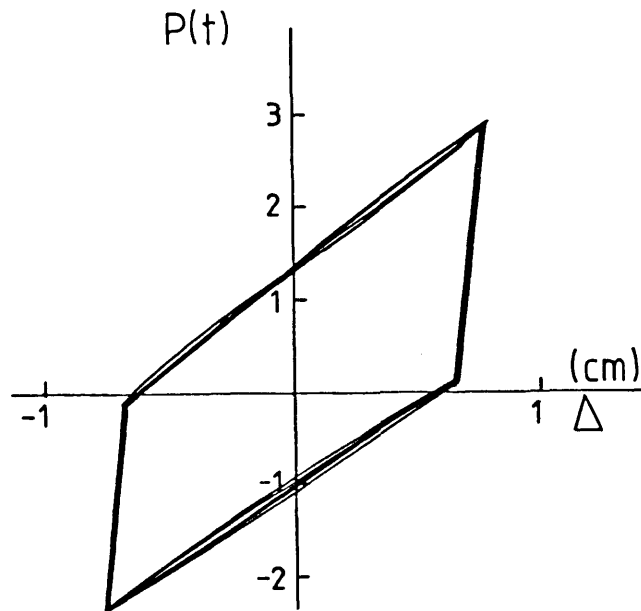


Fig. 5.36 Hysteresis loops of the K-braced frame by reducing the slipping load of the device

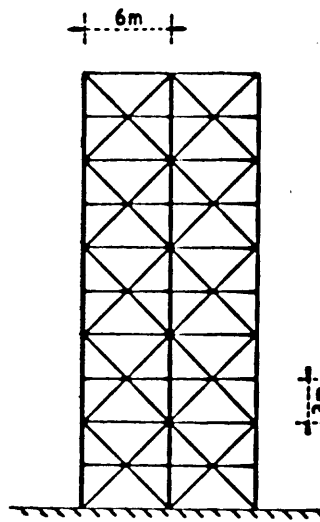


Fig.5.37 The designed ten-storey building frame

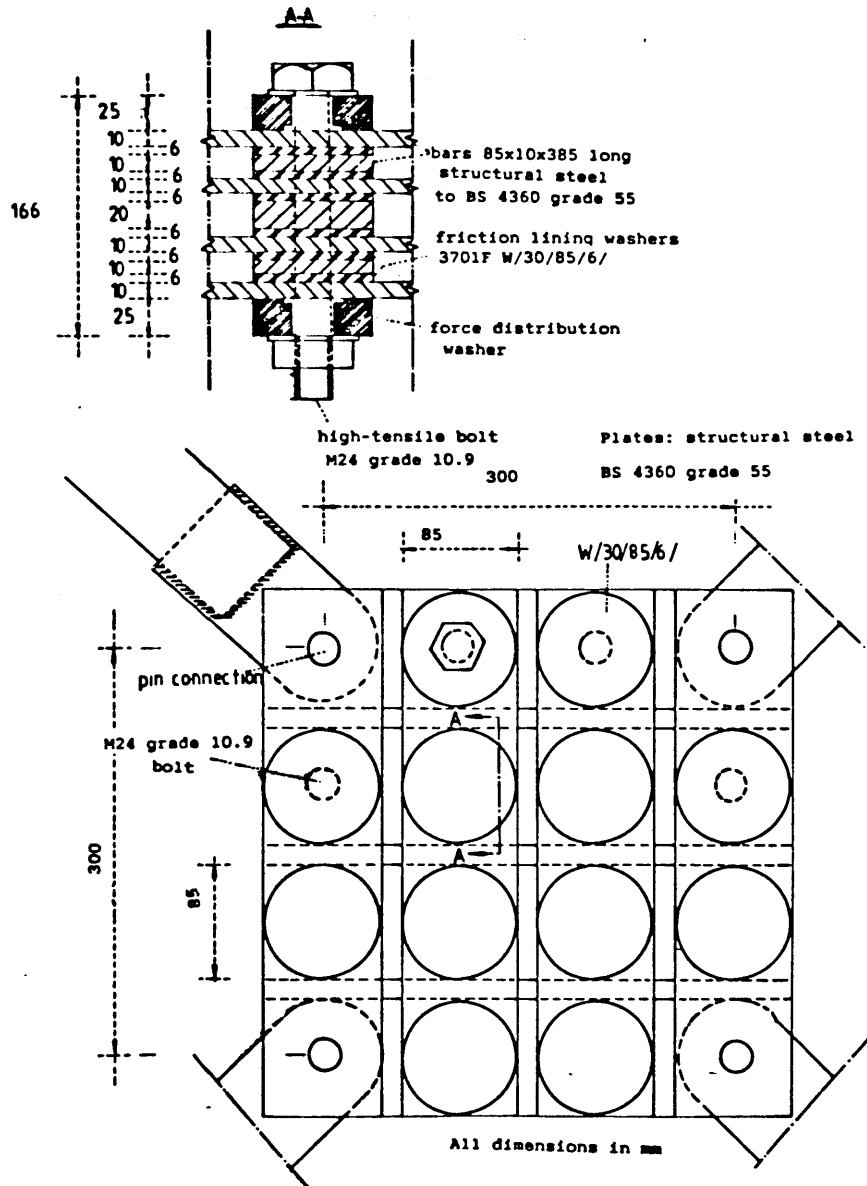


Fig.5.38 A typical full-size rotating device

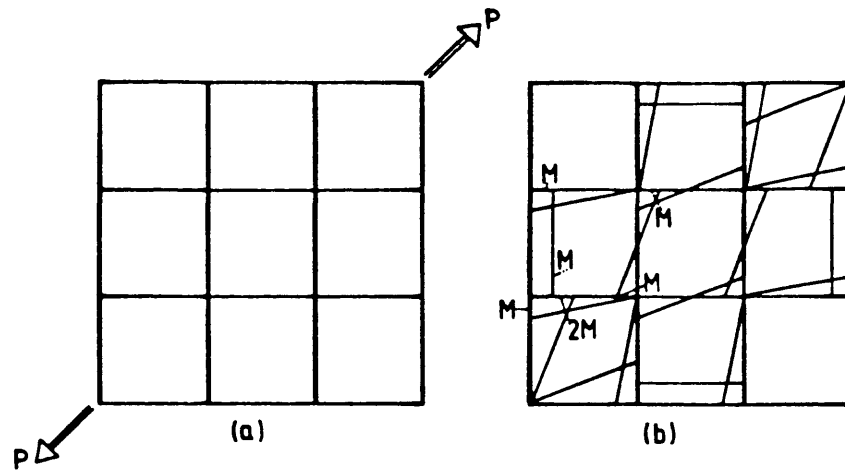


Fig. 5.39 The bending moment diagram of the device

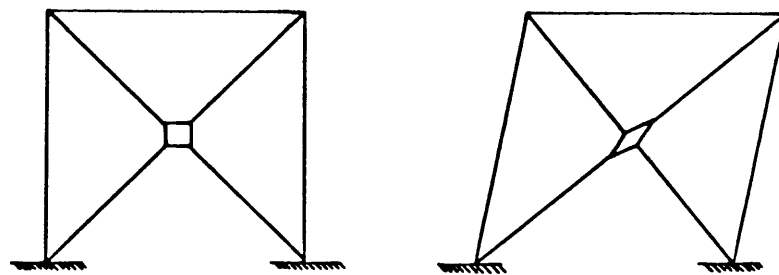


Fig. 5.40 Deformation of a single storey braced panel having the rotating device

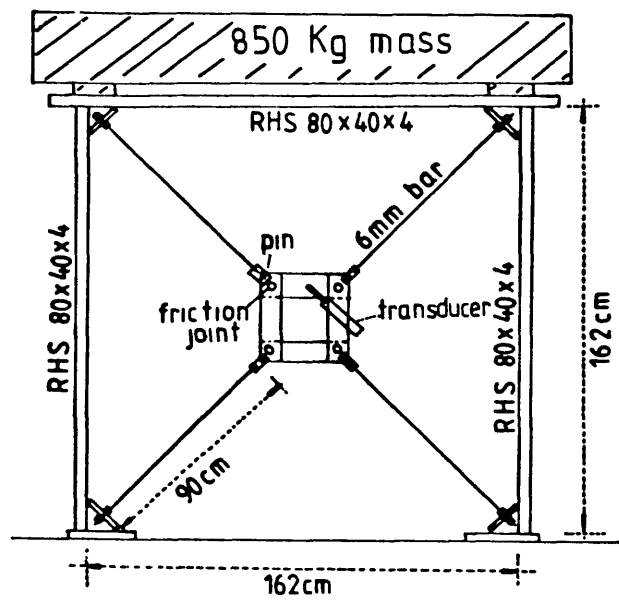


Fig. 6.1 Dimensions of the X-braced frame incorporating the rotating device

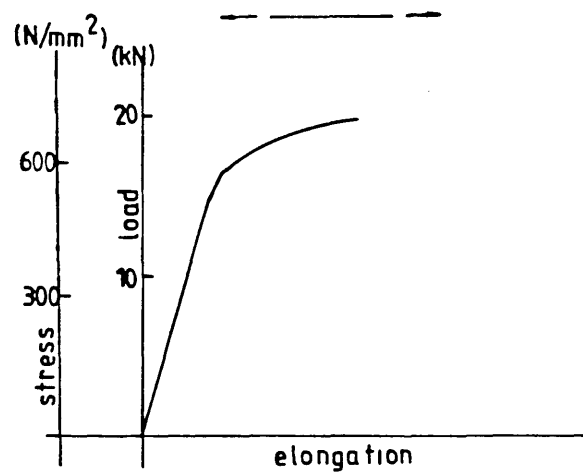


Fig. 6.2 Force-displacement monotonic curve of the bars

(A photograph, fig.6.3 is shown in the next page)

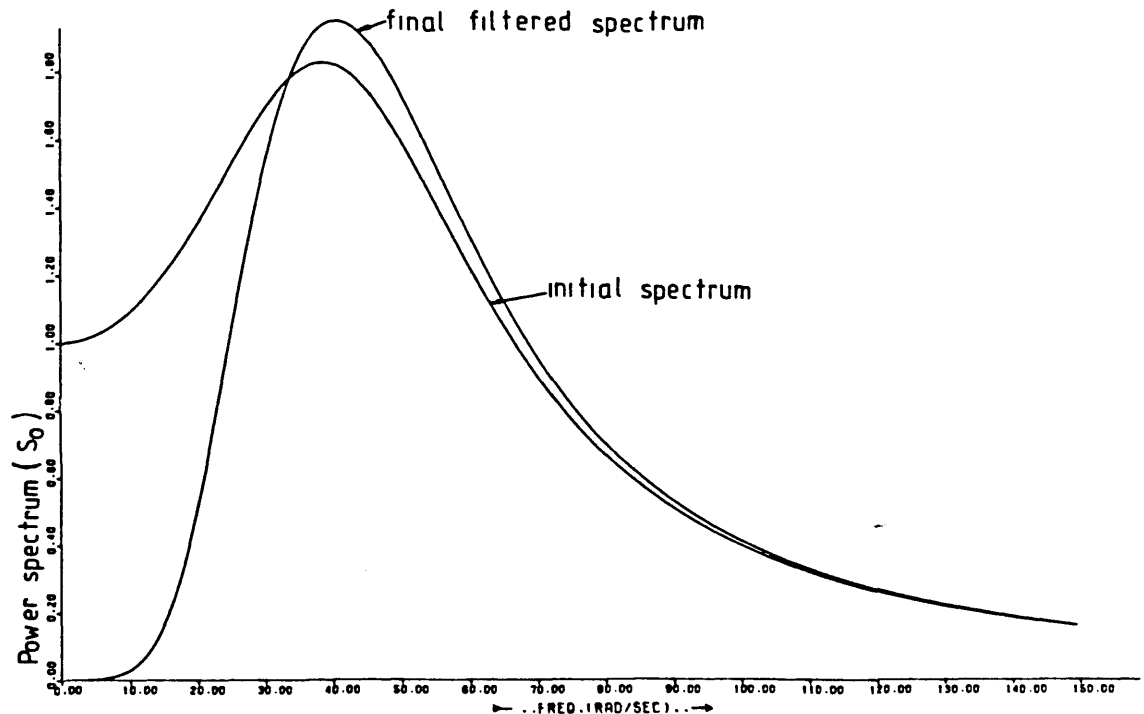


Fig. 6.4 Power spectrum of generated artificial earthquakes

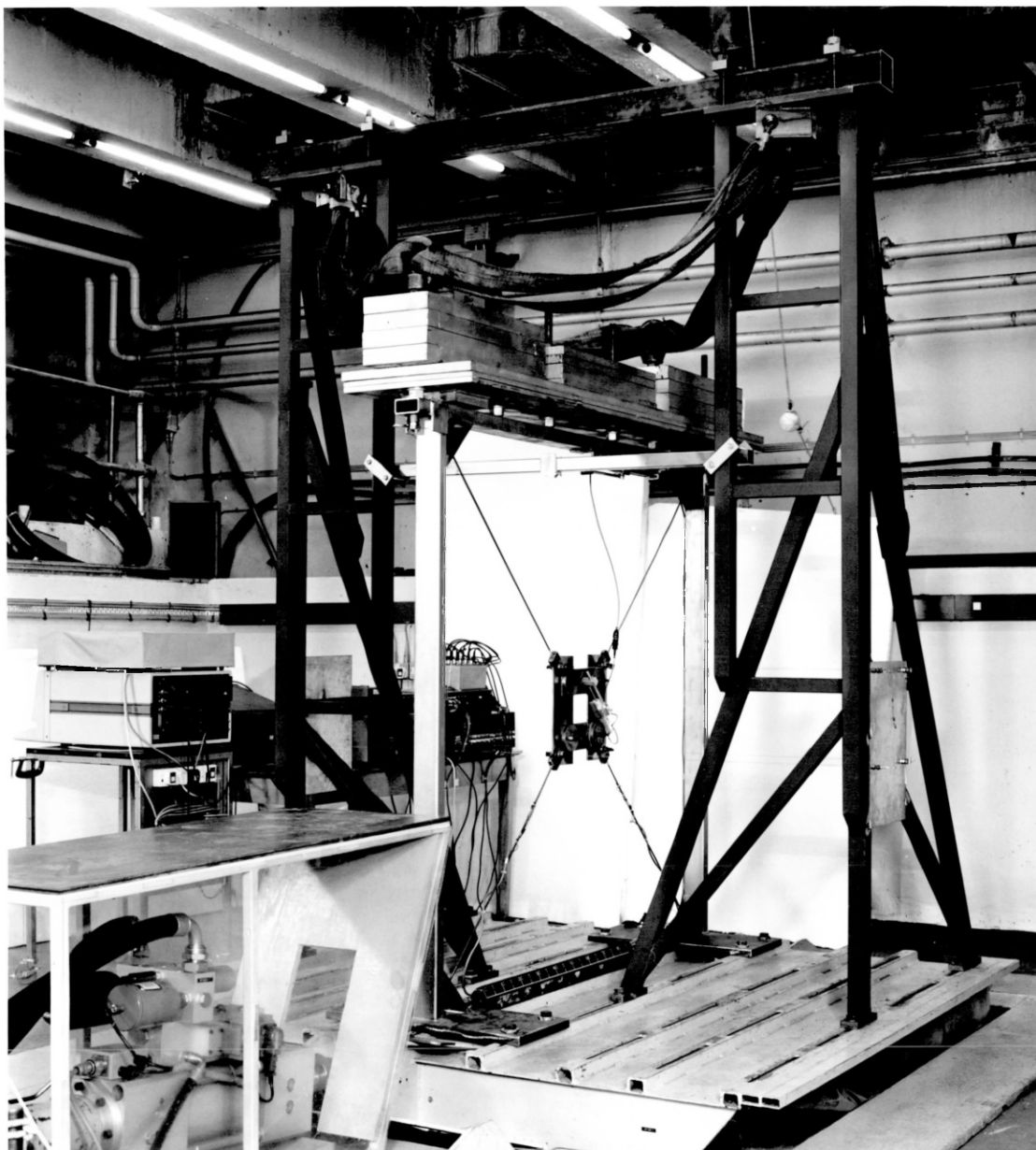


Fig.6.3. The X-braced frame with the rotating device.

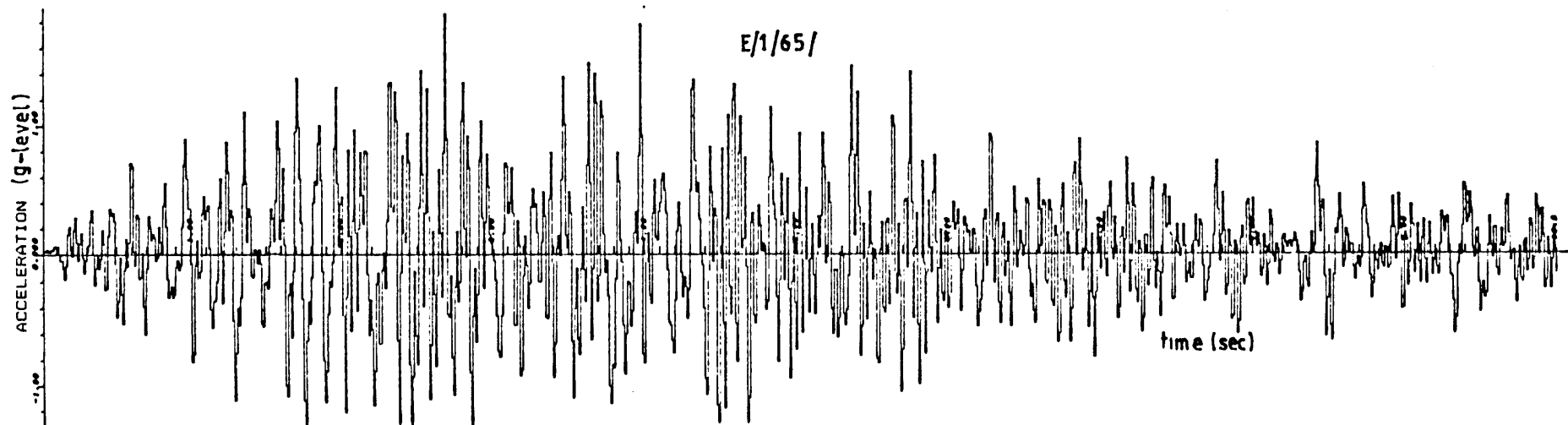


Fig. 6.5 A typical acceleration time history appearing on the platform

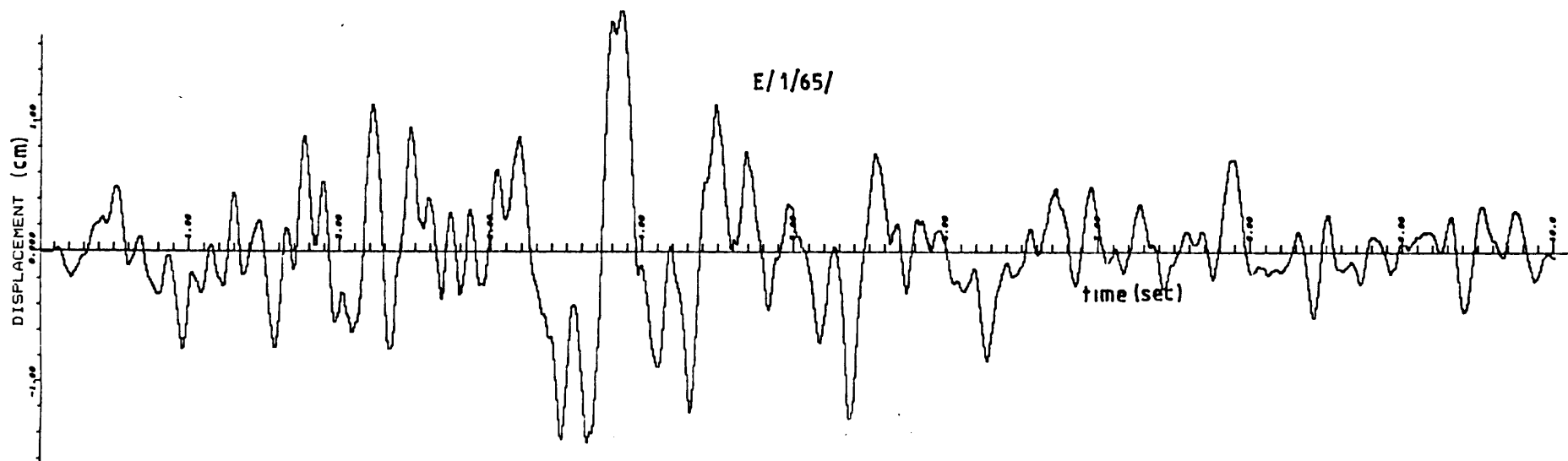


Fig. 6.6 A typical displacement time history appearing on the platform

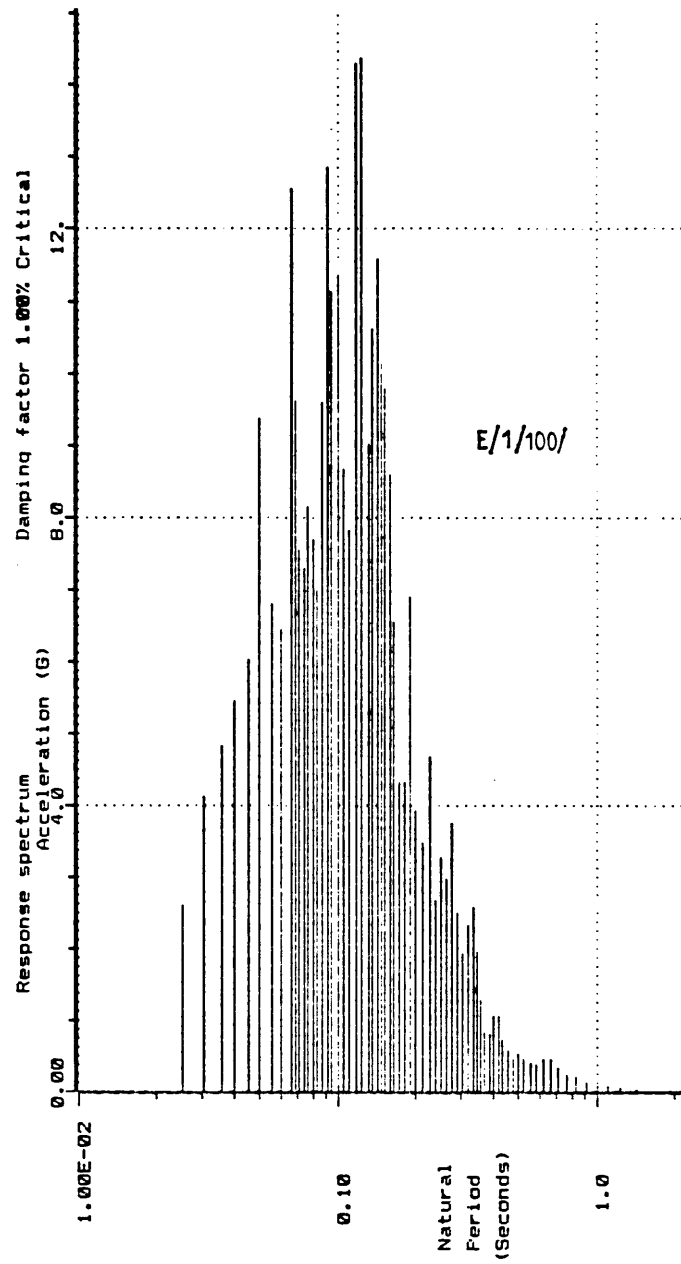


Fig. 6.7 Acceleration spectrum of the target earthquake motion

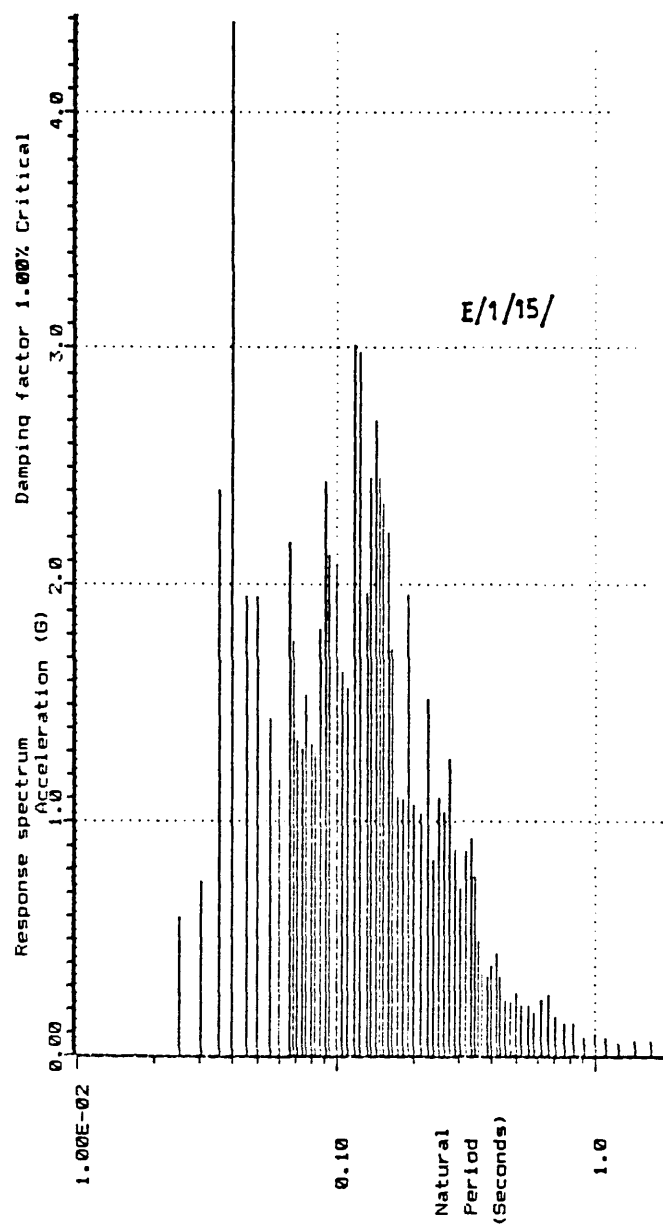


Fig. 6.8 Acceleration spectrum of the acceleration appearing on the platform without the test frame on the platform

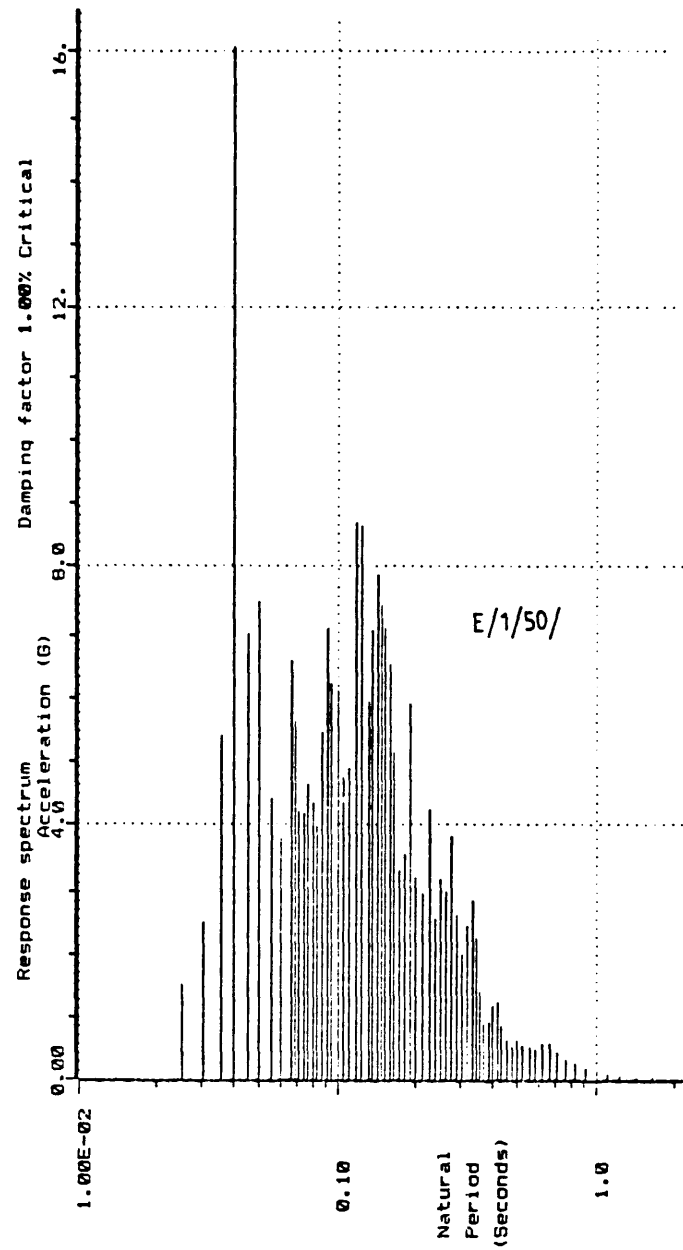


Fig. 6.9 Acceleration spectrum of the acceleration appearing on the platform of the braced frame incorporating the rotating device and 50% excitation level

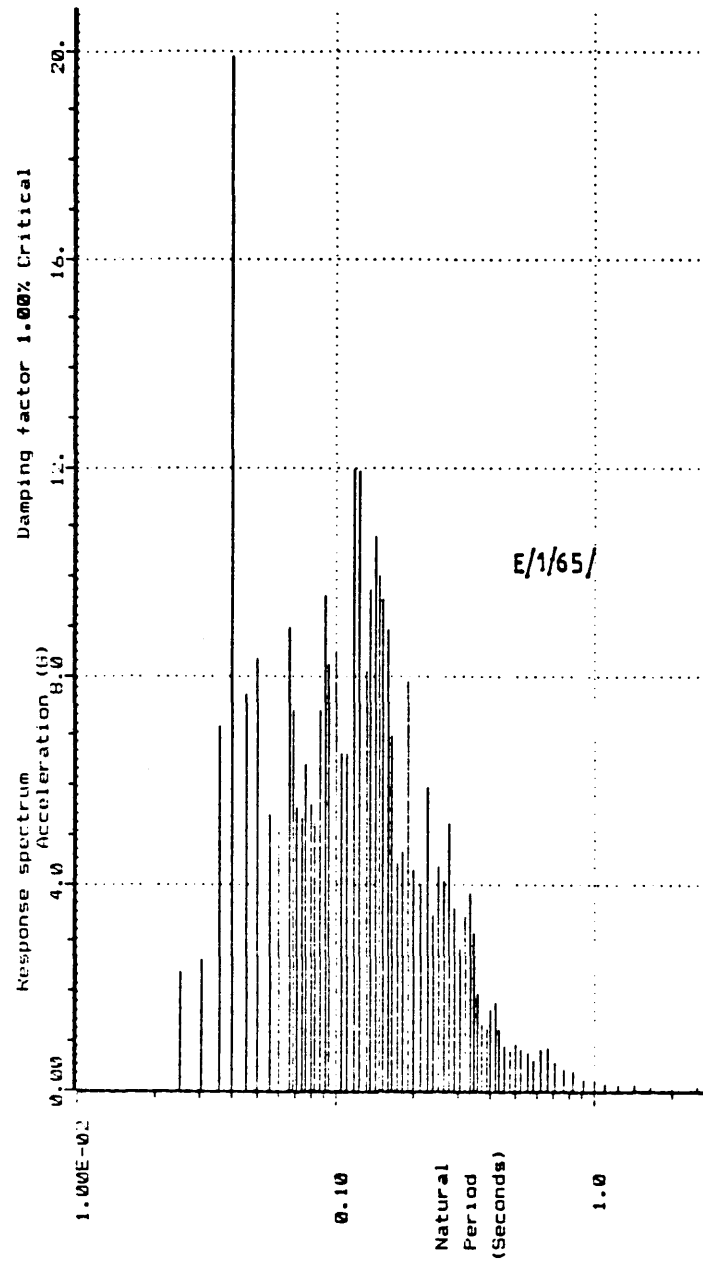


Fig. 6.10 Acceleration spectrum of the acceleration appearing on the platform of the braced frame incorporating the rotating device and 65% excitation level

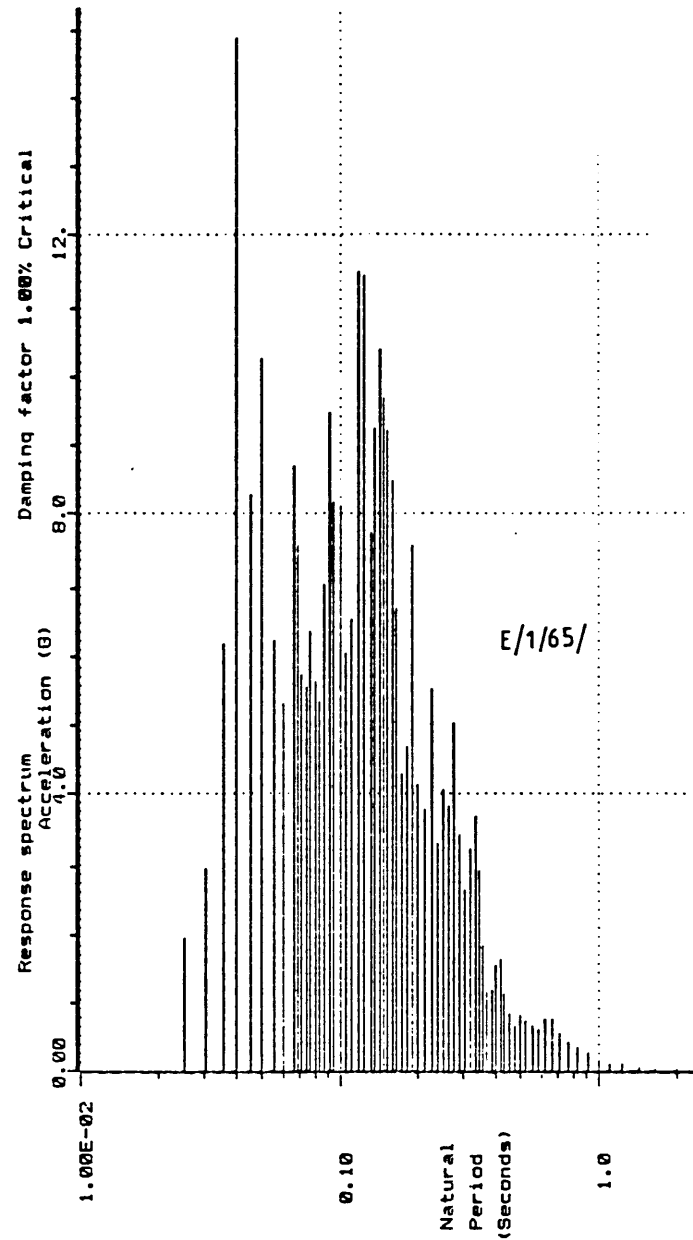


Fig. 6.11 Acceleration spectrum of the acceleration appearing on the platform of the braced frame incorporating the bending device and 65% excitation level

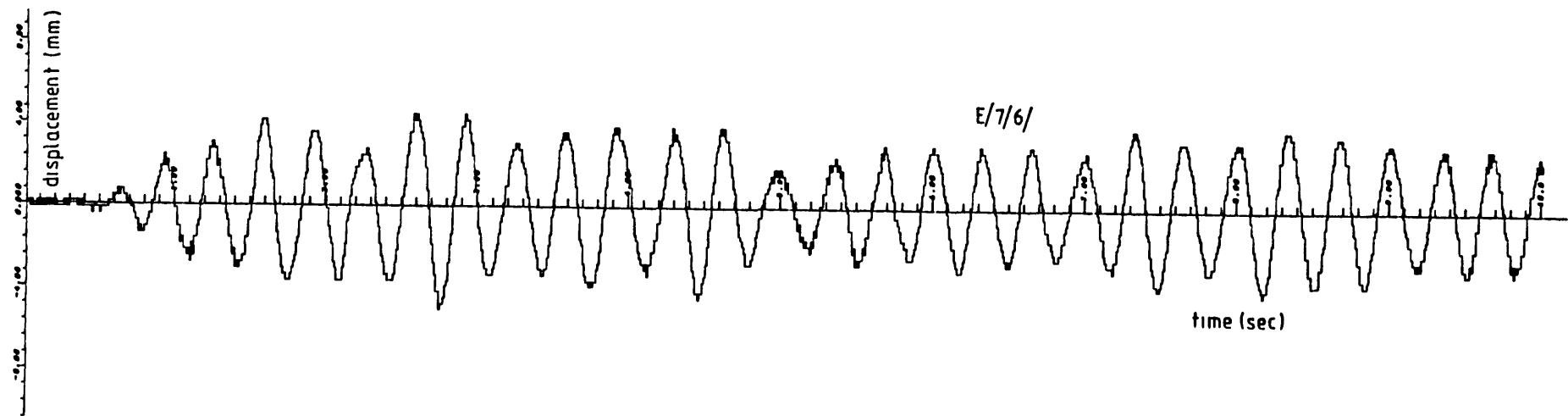


Fig. 6.12 A typical displacement time history of the unbraced frame

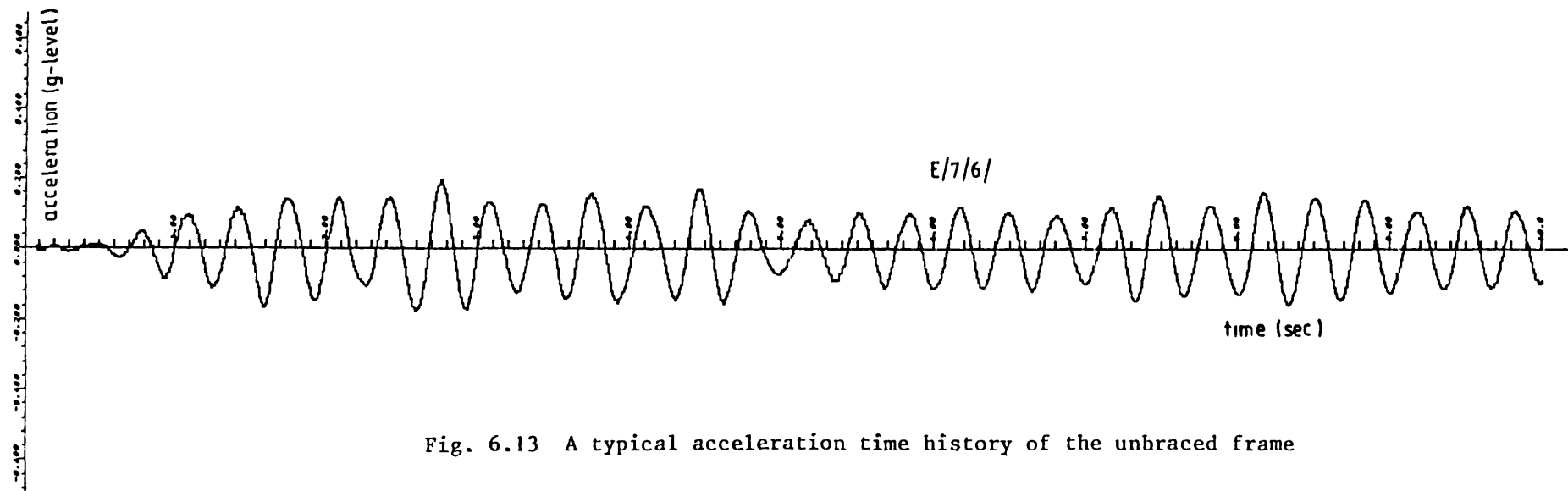


Fig. 6.13 A typical acceleration time history of the unbraced frame

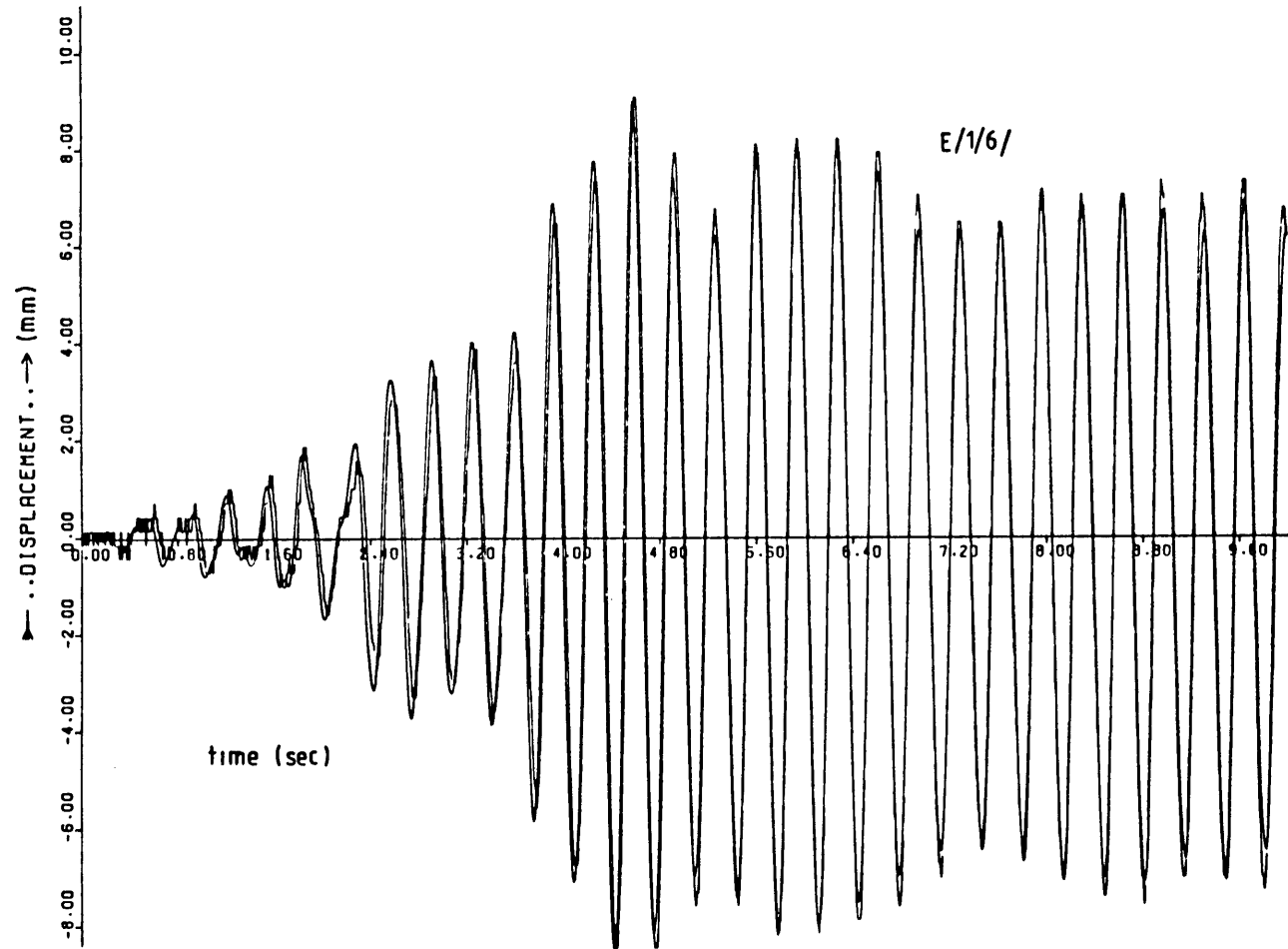


Fig. 6.14 Comparison between experimental and analytical results
in the unbraced frame

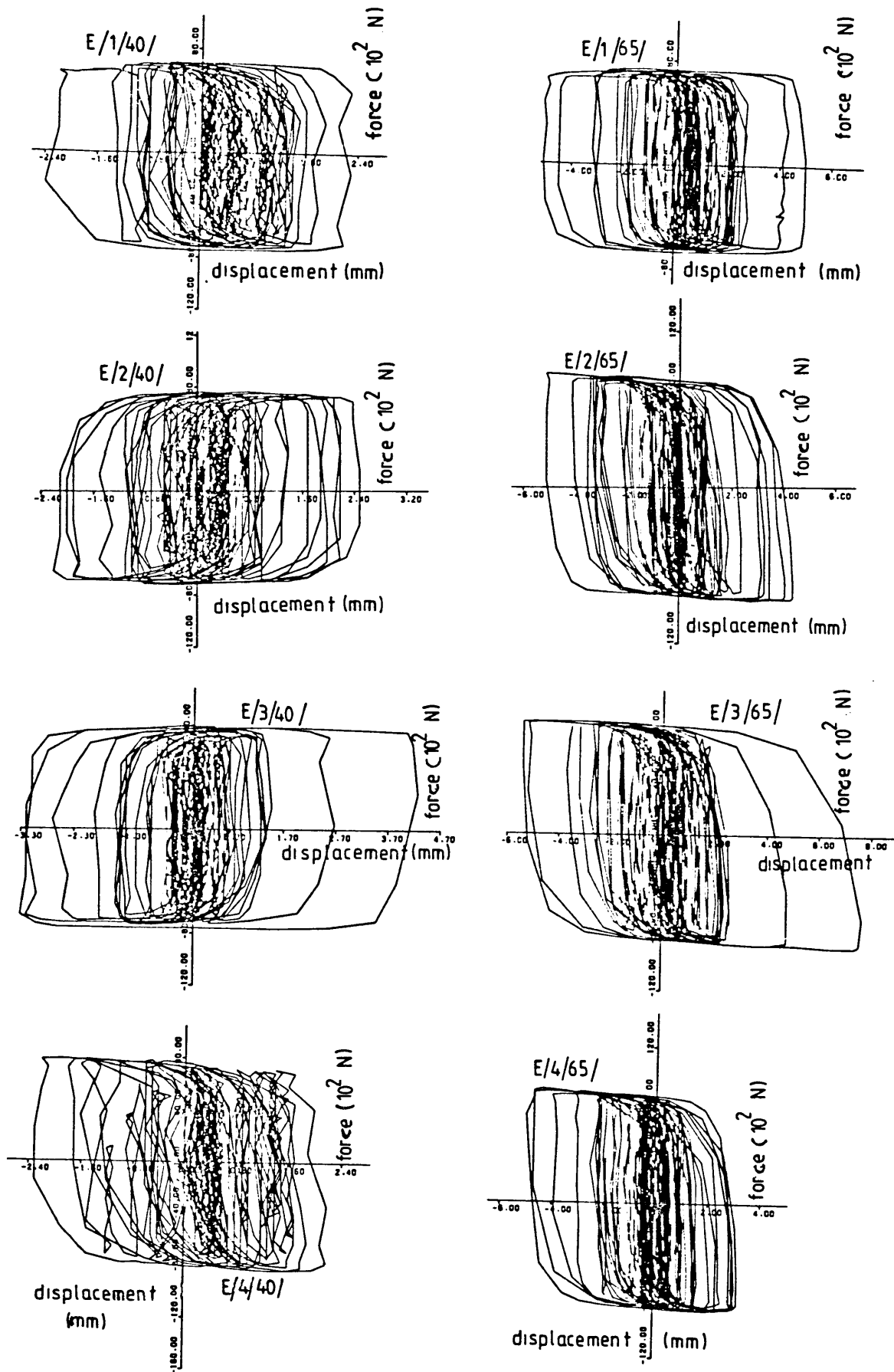
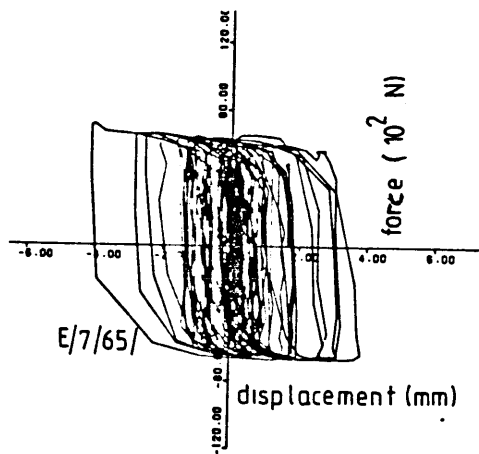
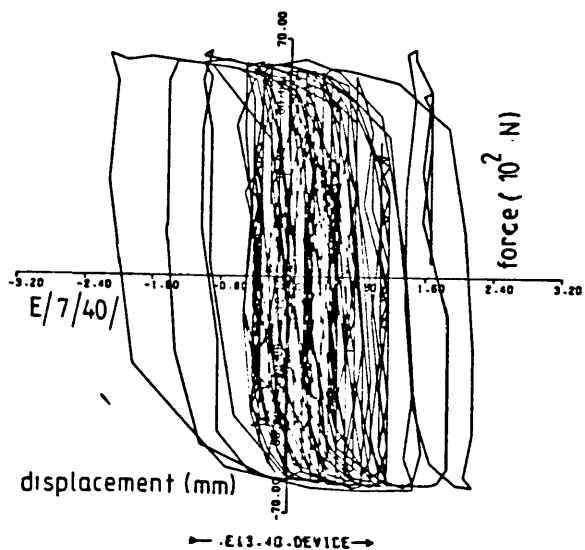
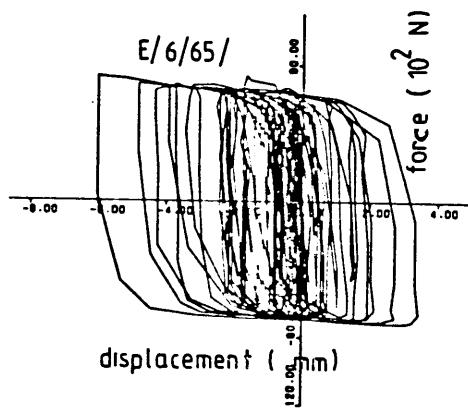
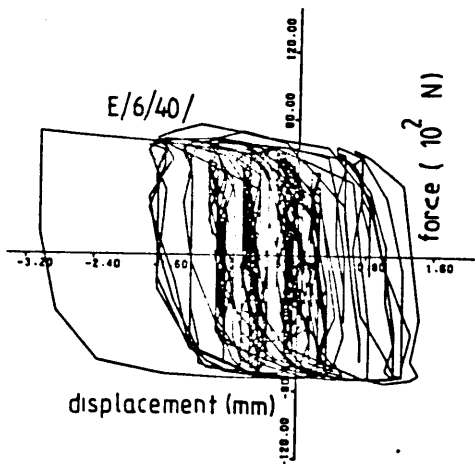
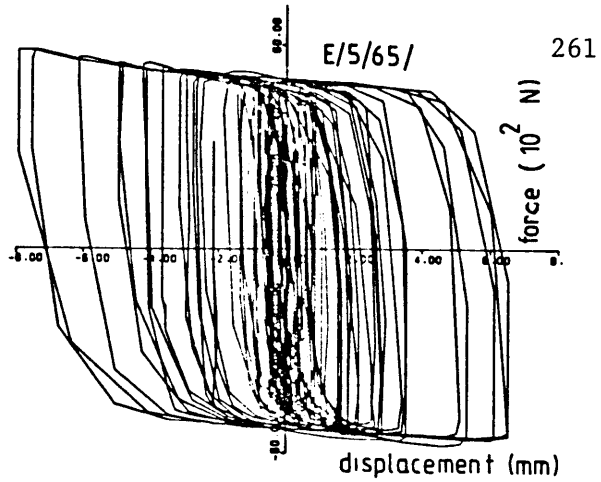
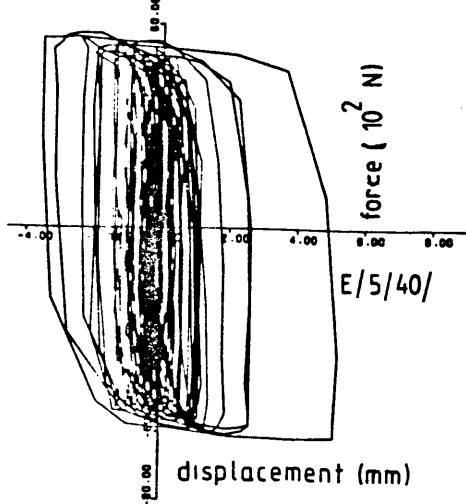
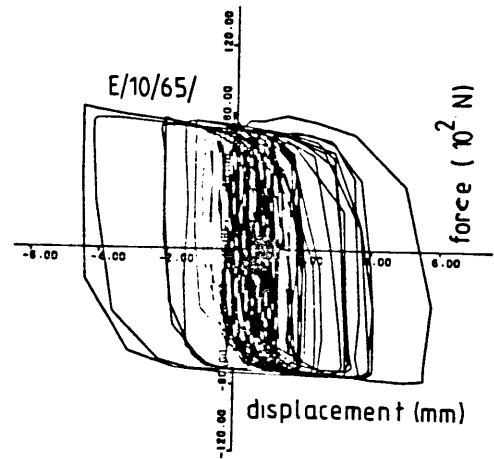
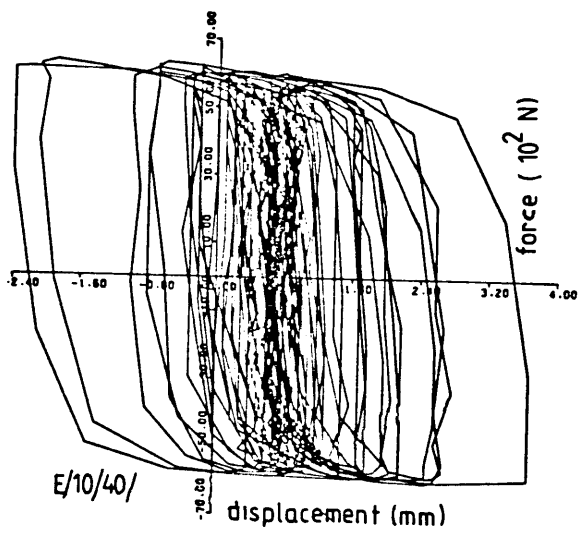
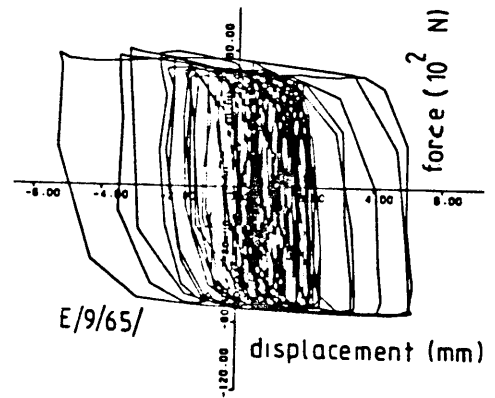
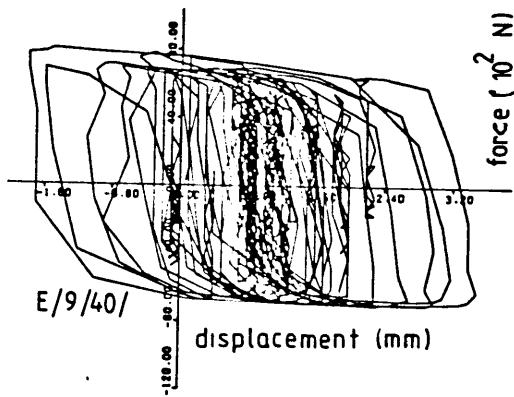
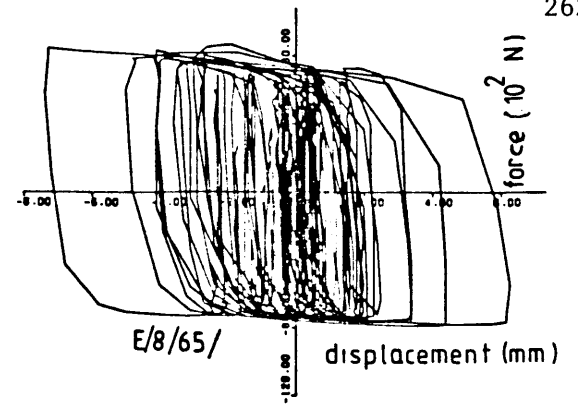
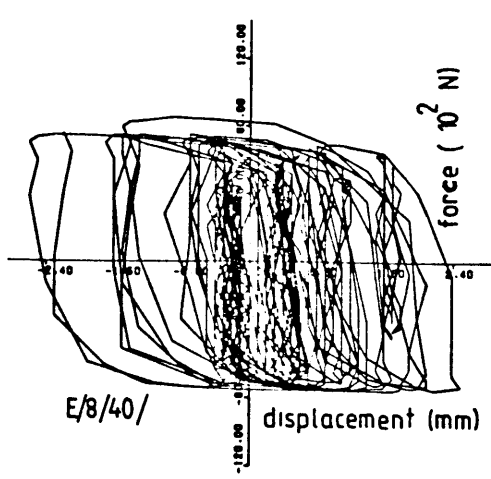


Fig. 6.15 Hysteresis loops in the friction joint of the braced frame having the rotating device



(continued)



(continued)

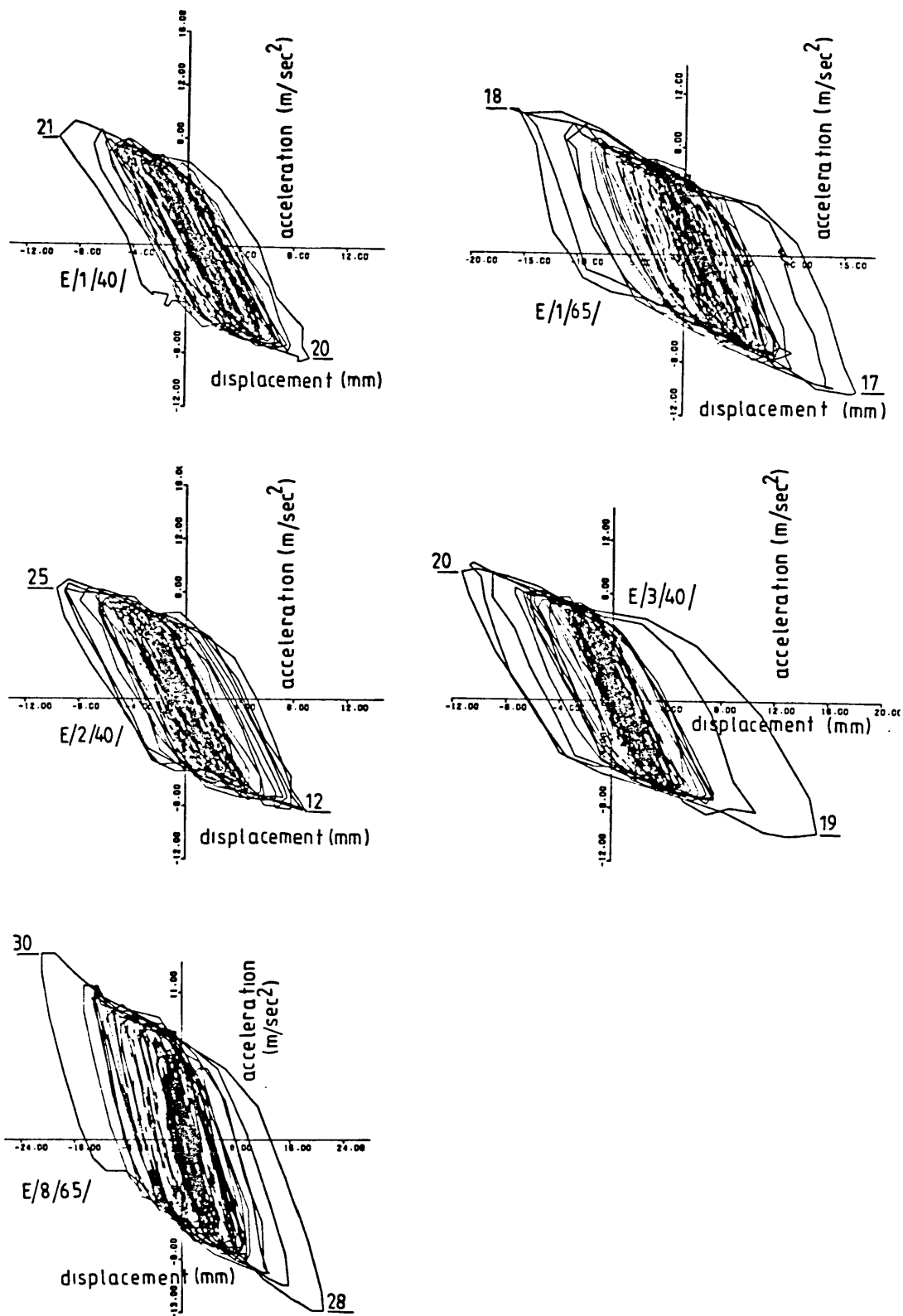
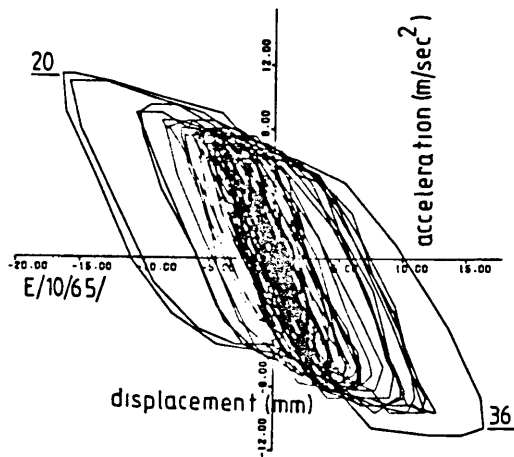
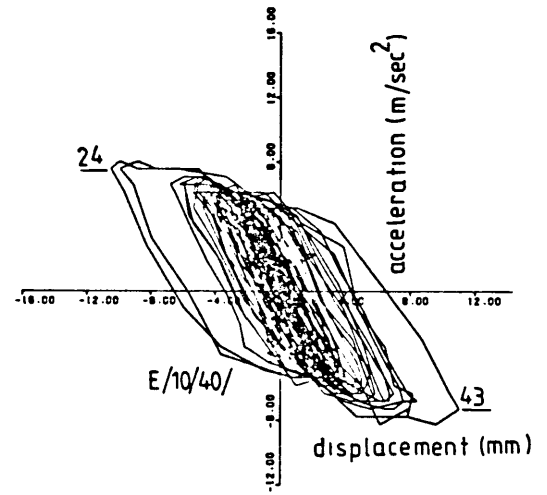
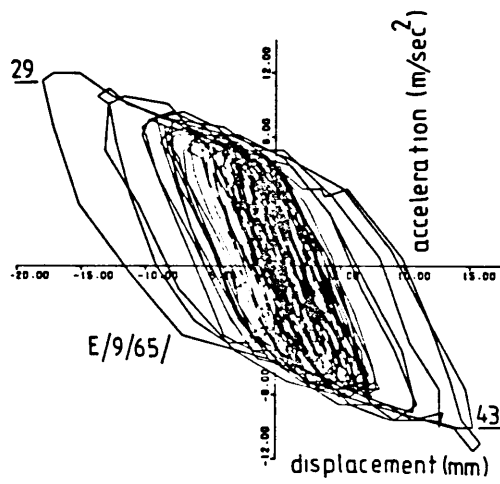
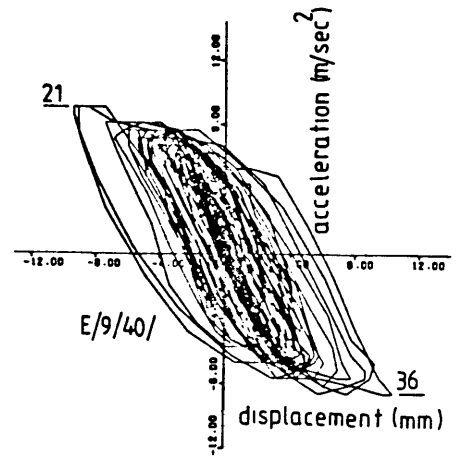
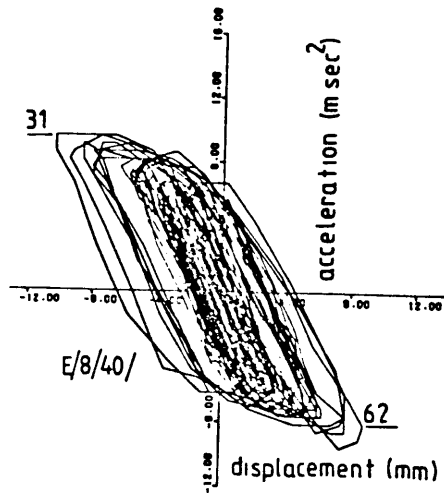


Fig. 6.16 Hysteresis loops of the braced frame incorporating the rotating device



(continued)

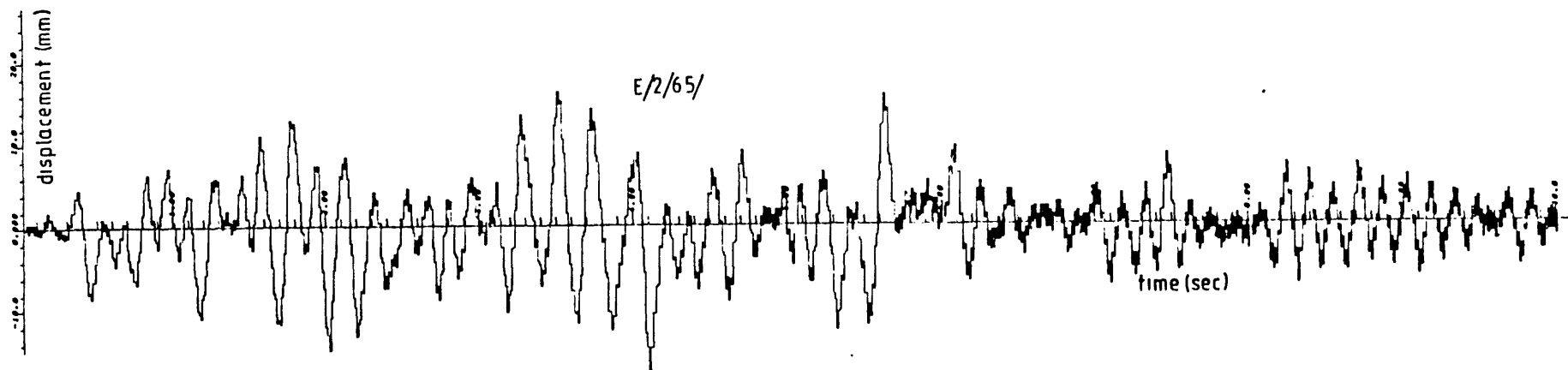


Fig. 6.17 A typical displacement time history of the braced frame incorporating the rotating device

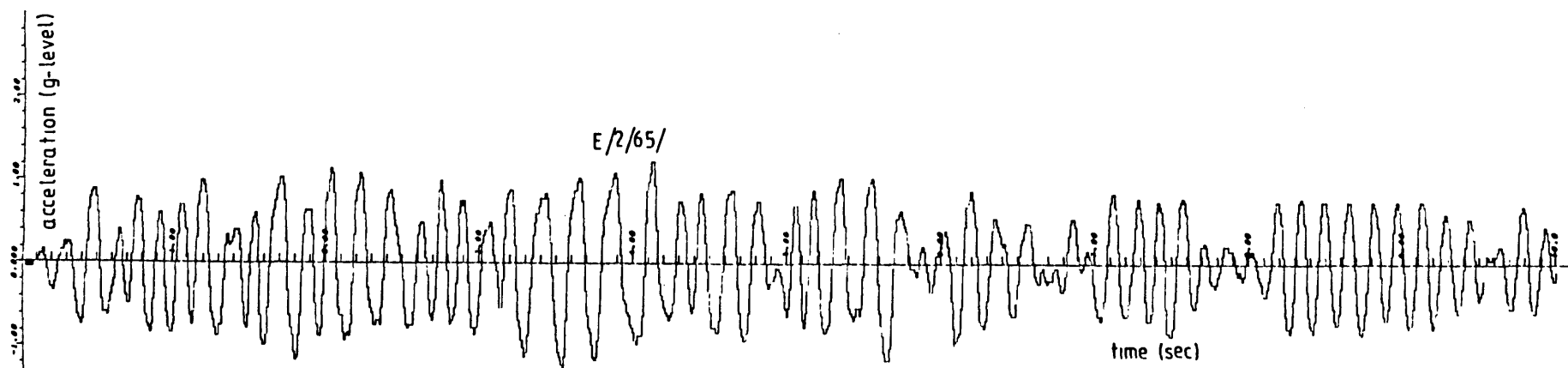


Fig. 6.18 A typical acceleration time history of the braced frame incorporating the rotating device

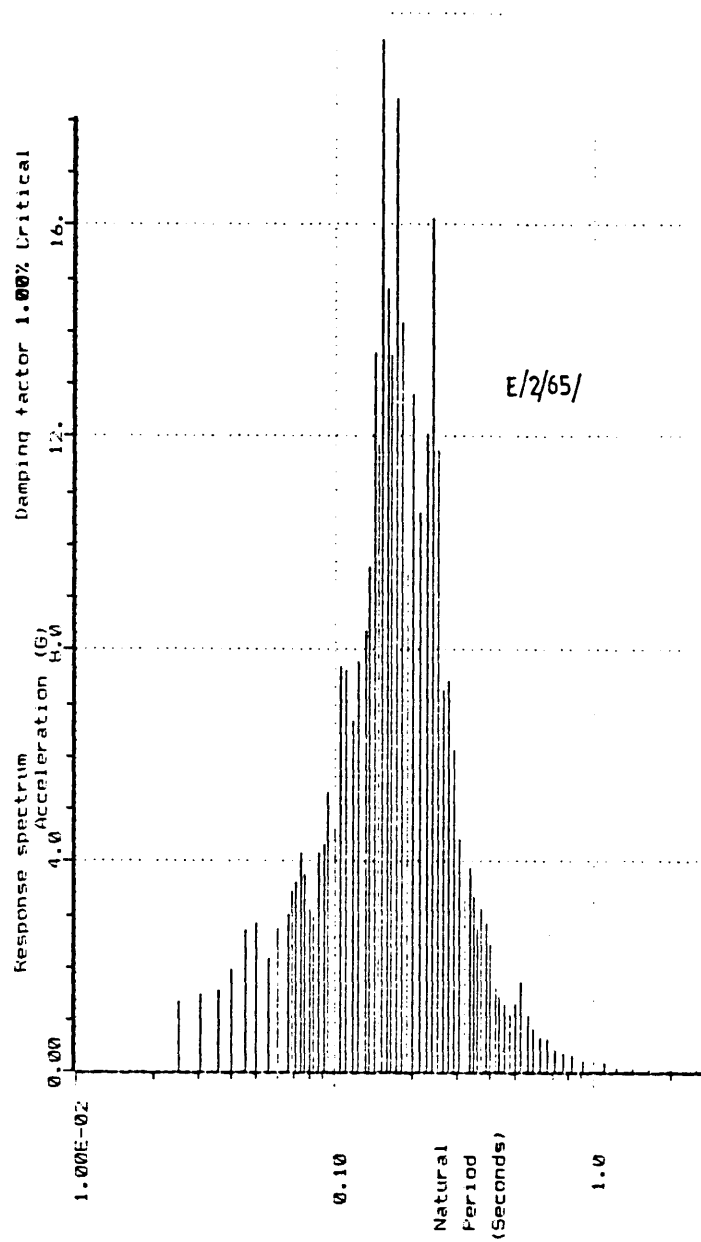


Fig. 6.19 Acceleration spectrum of the response acceleration of the braced frame incorporating the rotating device

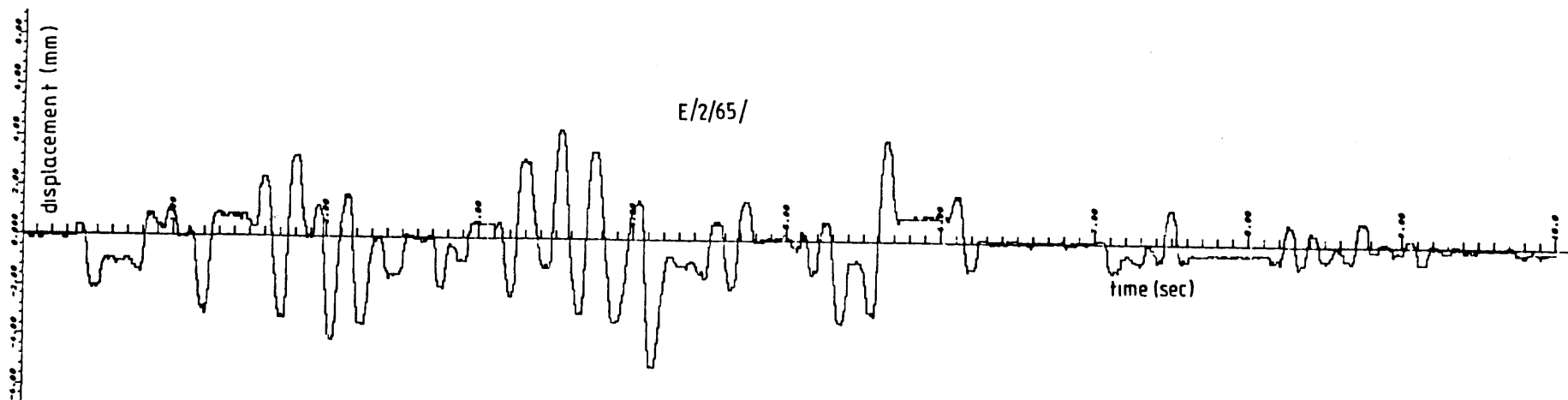


Fig. 6.20 A typical slip time history given by the transducer on the device

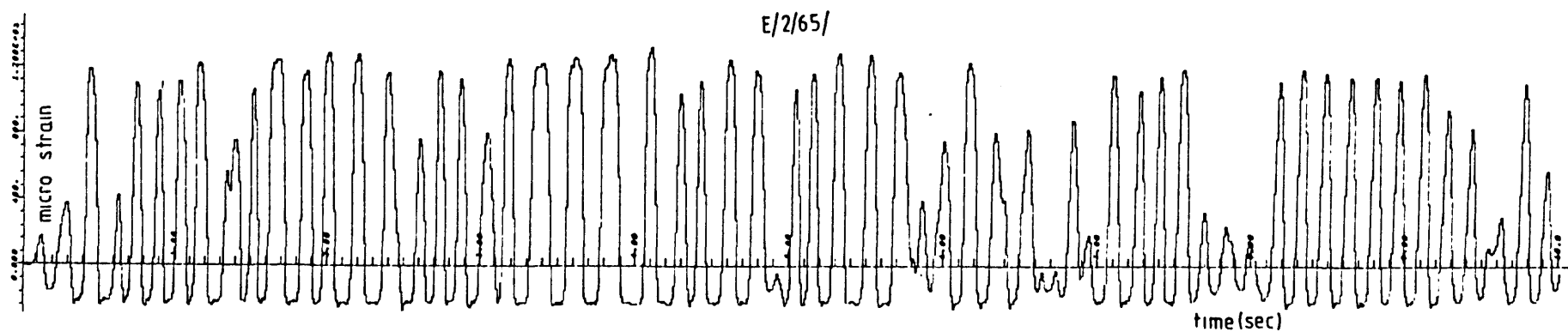
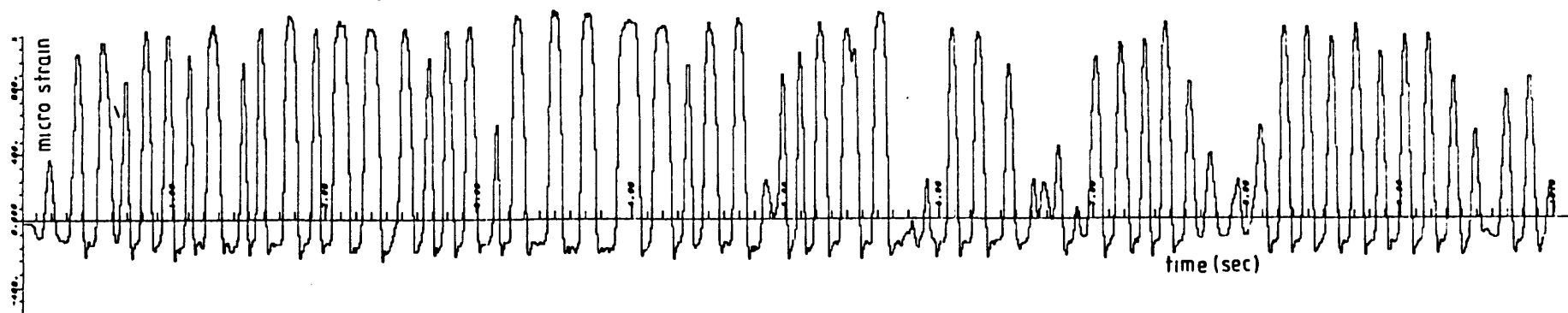


Fig. 6.21 Typical strain gauge readings for the two braces

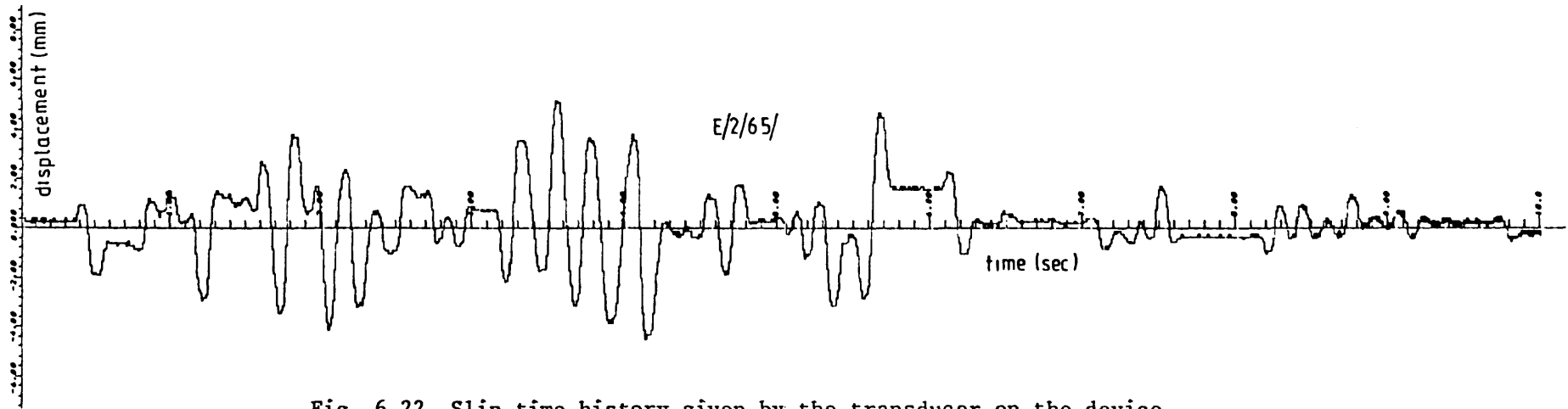


Fig. 6.22 Slip time history given by the transducer on the device
by repeating one earthquake record

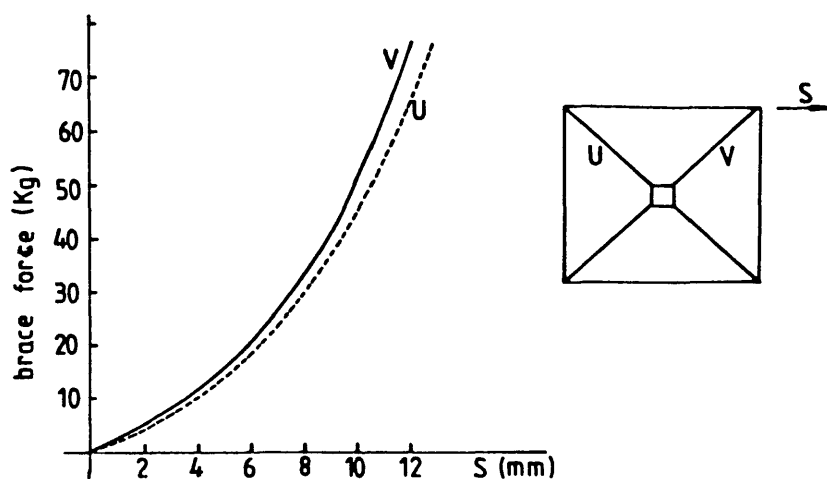


Fig. 6.23 Additional brace forces resulting from the change of the geometry of the device

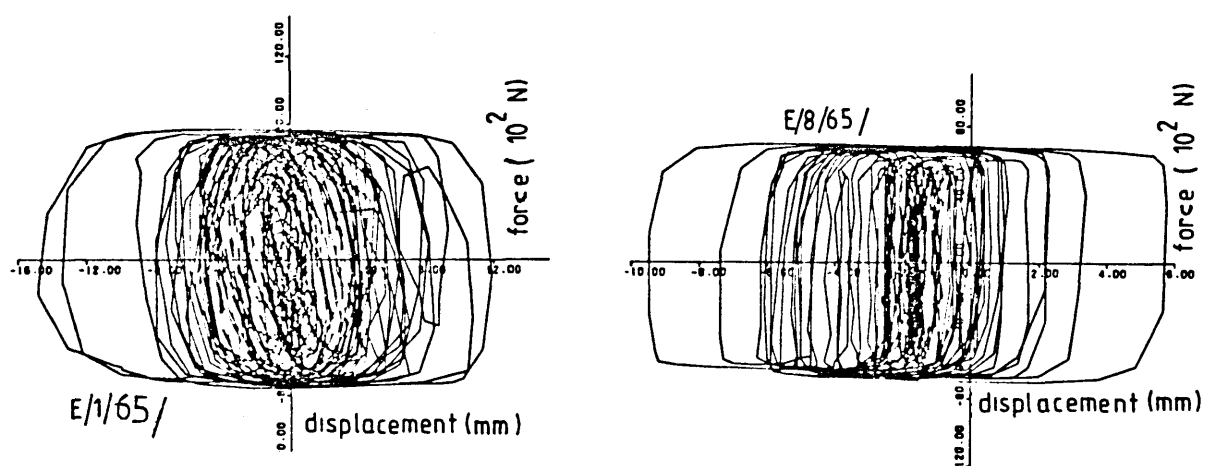


Fig. 6.24 Hysteresis loops of the friction joint by introducing pretension to the braces

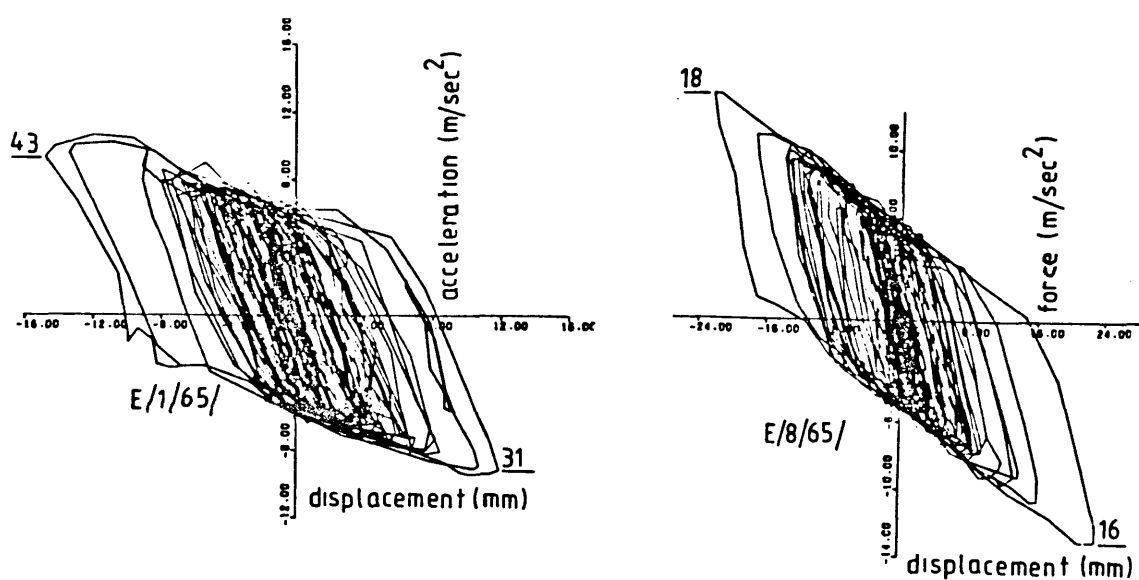


Fig. 6.25 Hysteresis loops of the frame by introducing pretension to the braces

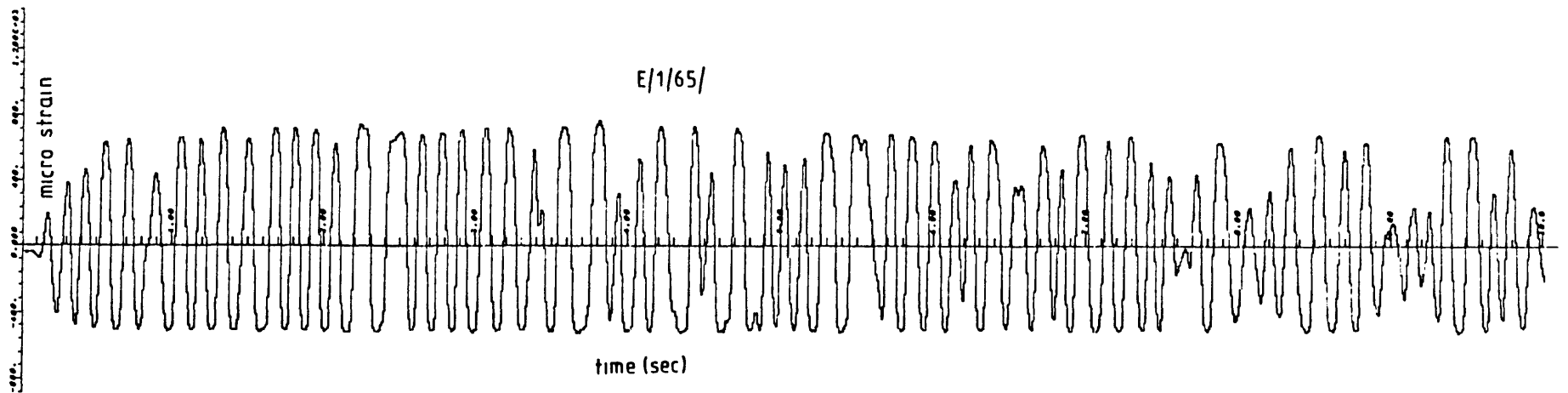
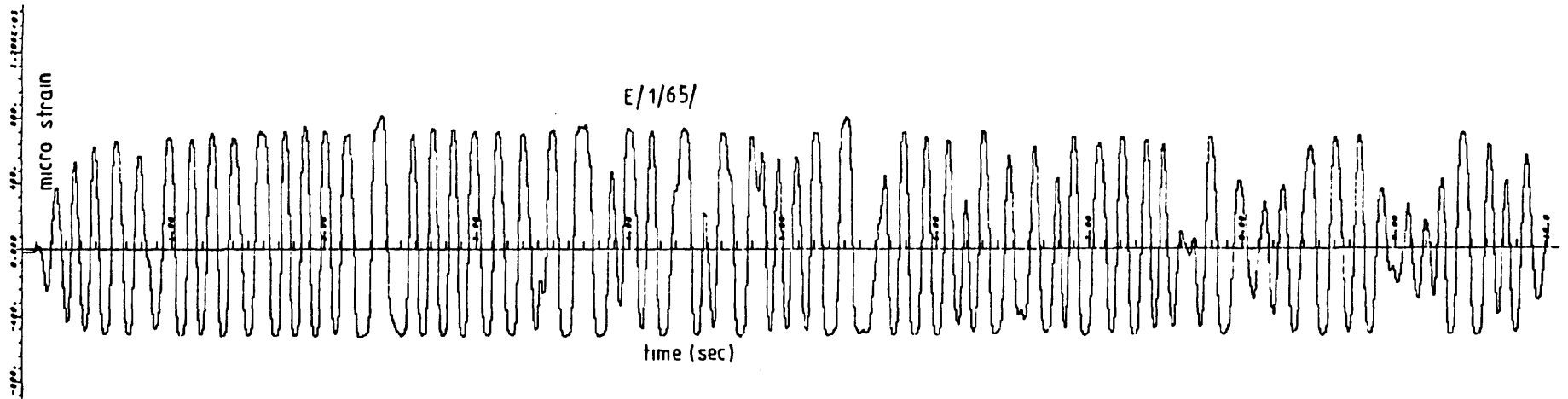


Fig. 6.26 Typical strain gauge readings for the two braces when pretension is introduced in the braces

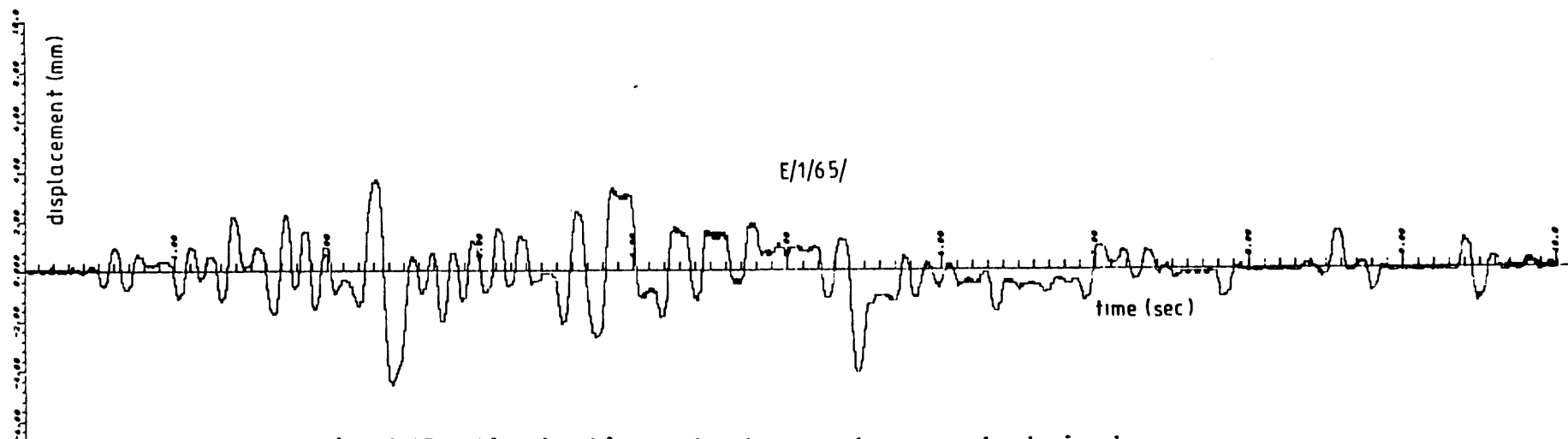


Fig. 6.27 Slip time history by the transducer on the device by introducing pretension to the braces

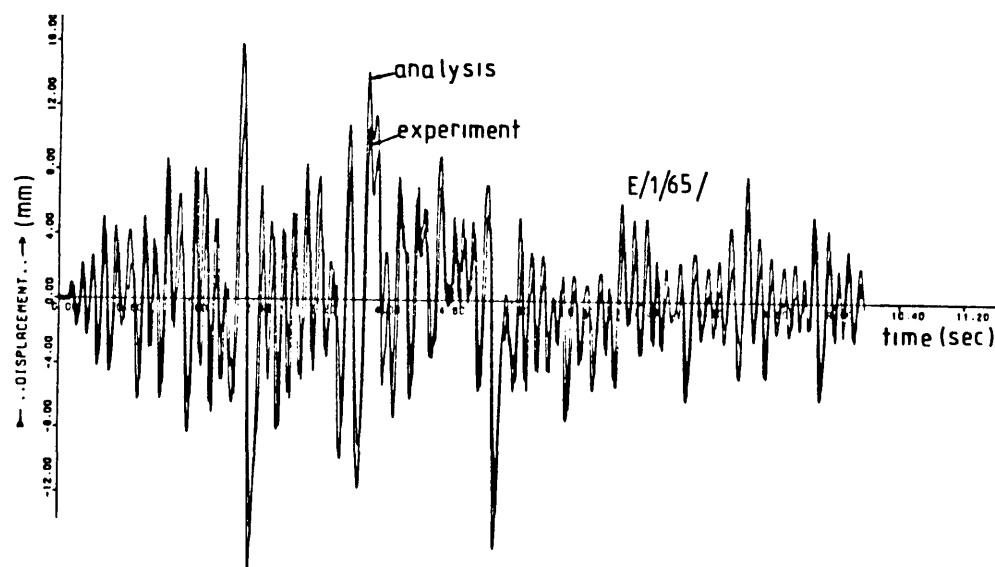


Fig. 6.28 Comparison between experimental and analytical results for the braced frame incorporating the rotating device

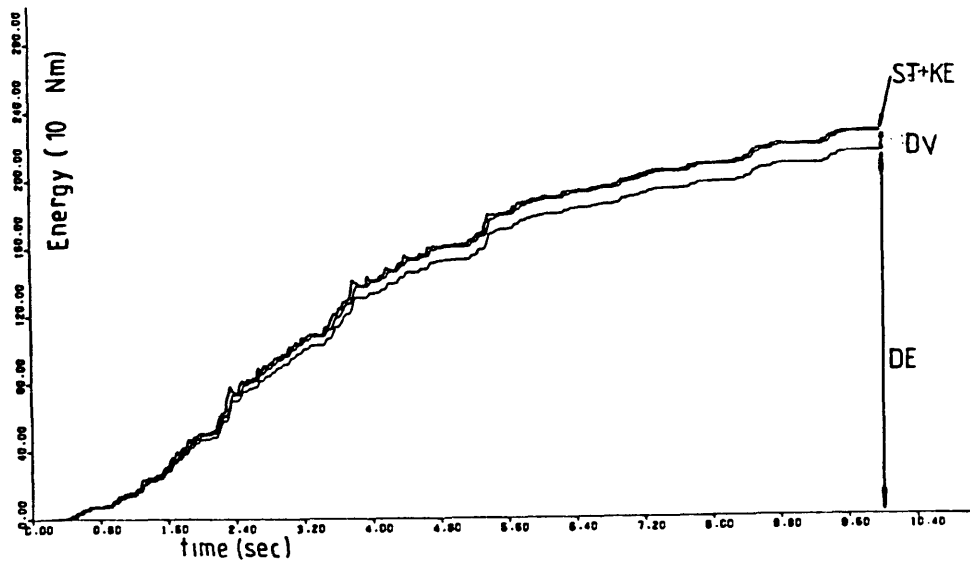


Fig. 6.29 Distribution of input energy in the braced frame

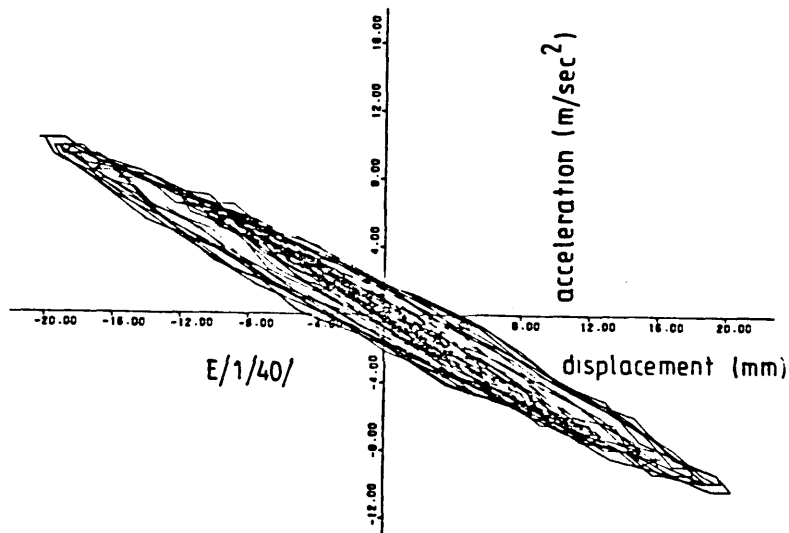


Fig. 6.30 Response of the braced frame with the bending device for a small excitation level

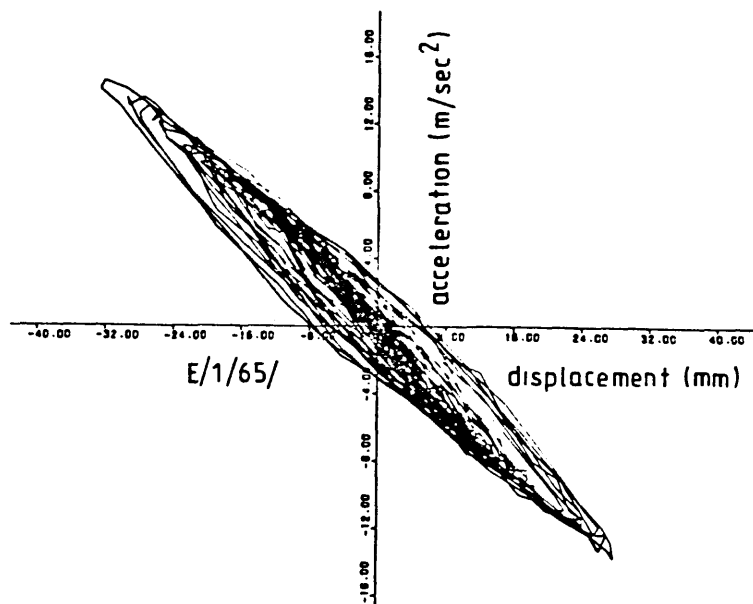


Fig. 6.31 Response of the braced frame with the bending device for a higher excitation level

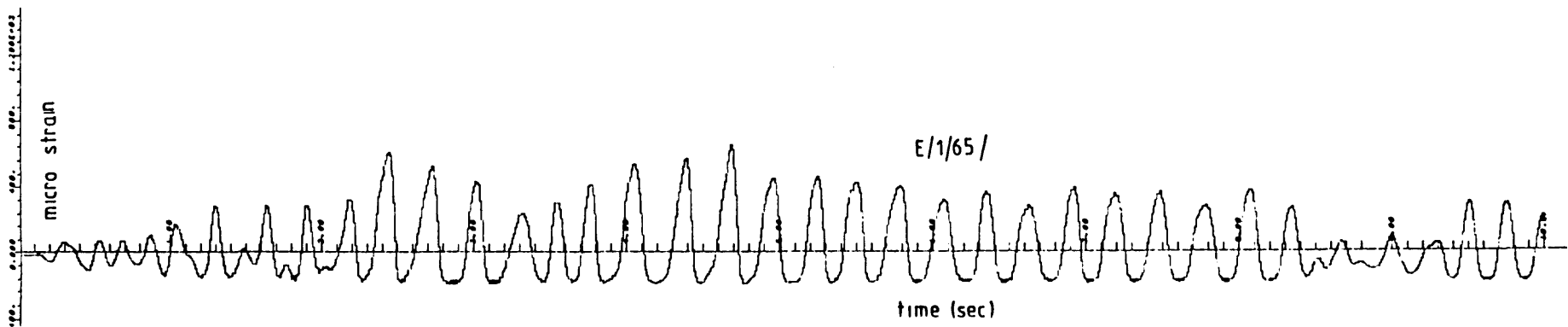
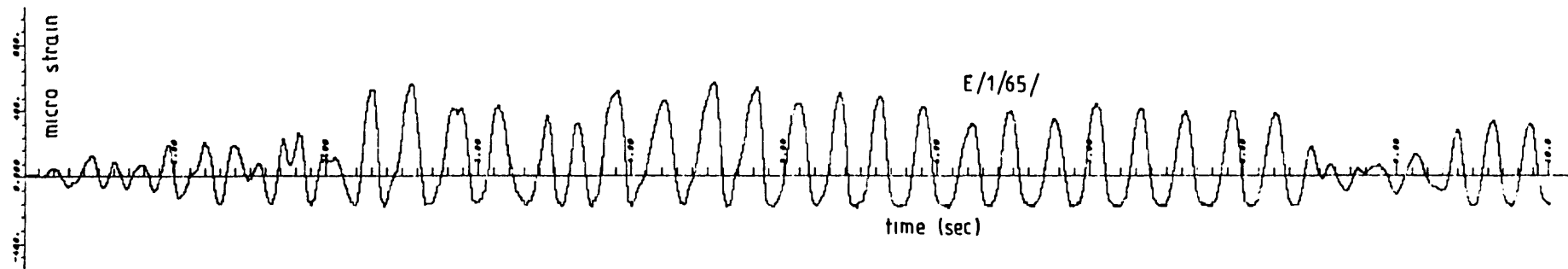


Fig. 6.32 Strain gauge readings for the two braces for the braced frame incorporating the bending device

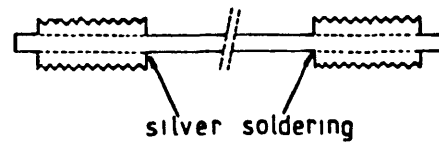


Fig. 6.33 Arrangement used to test braces along

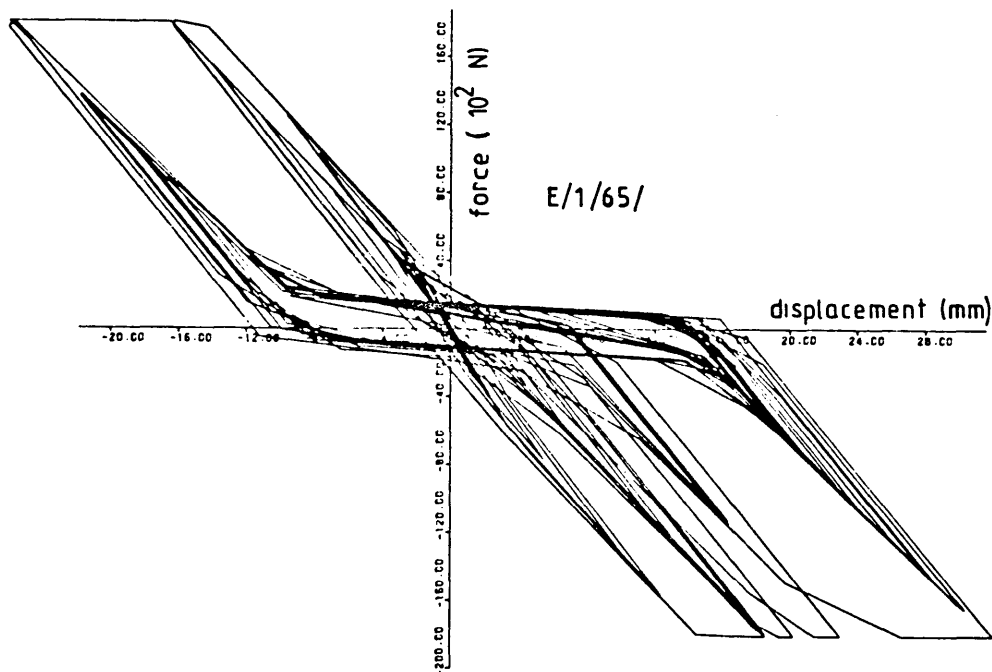


Fig. 6.34 Analytical predictions giving the hysteresis loops of the braced frame without any device

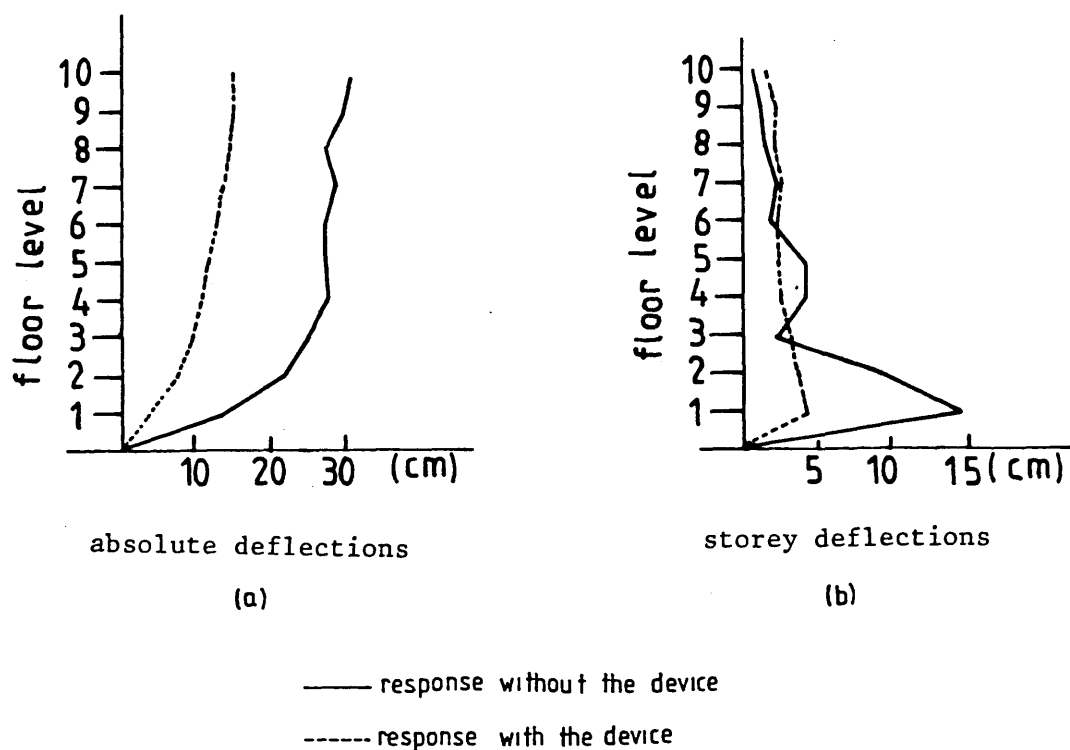


Fig. 7.1 Dynamic response of the ten-storey shear braced frame

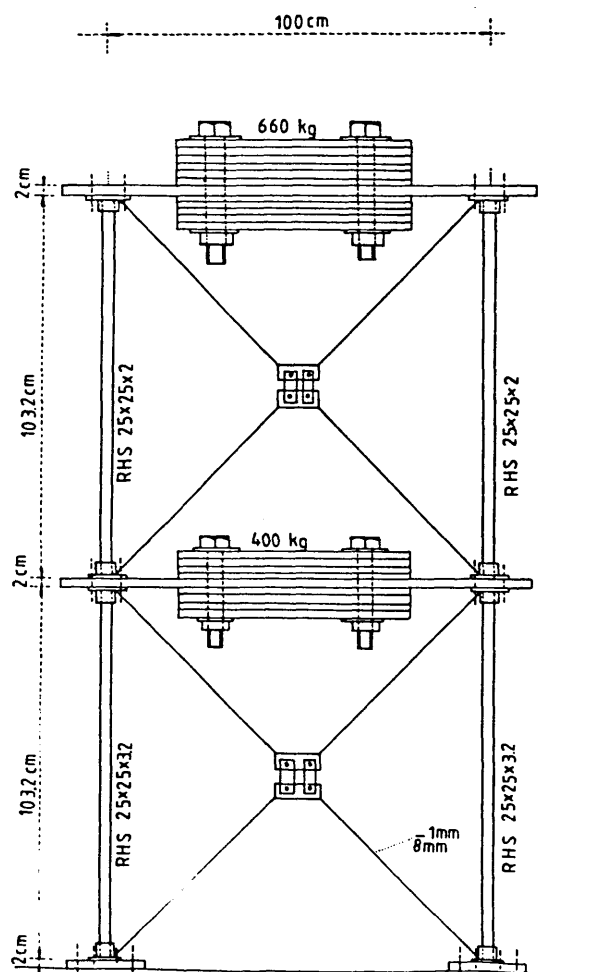


Fig.7.2 Elevation of the frame on the plane of shaking

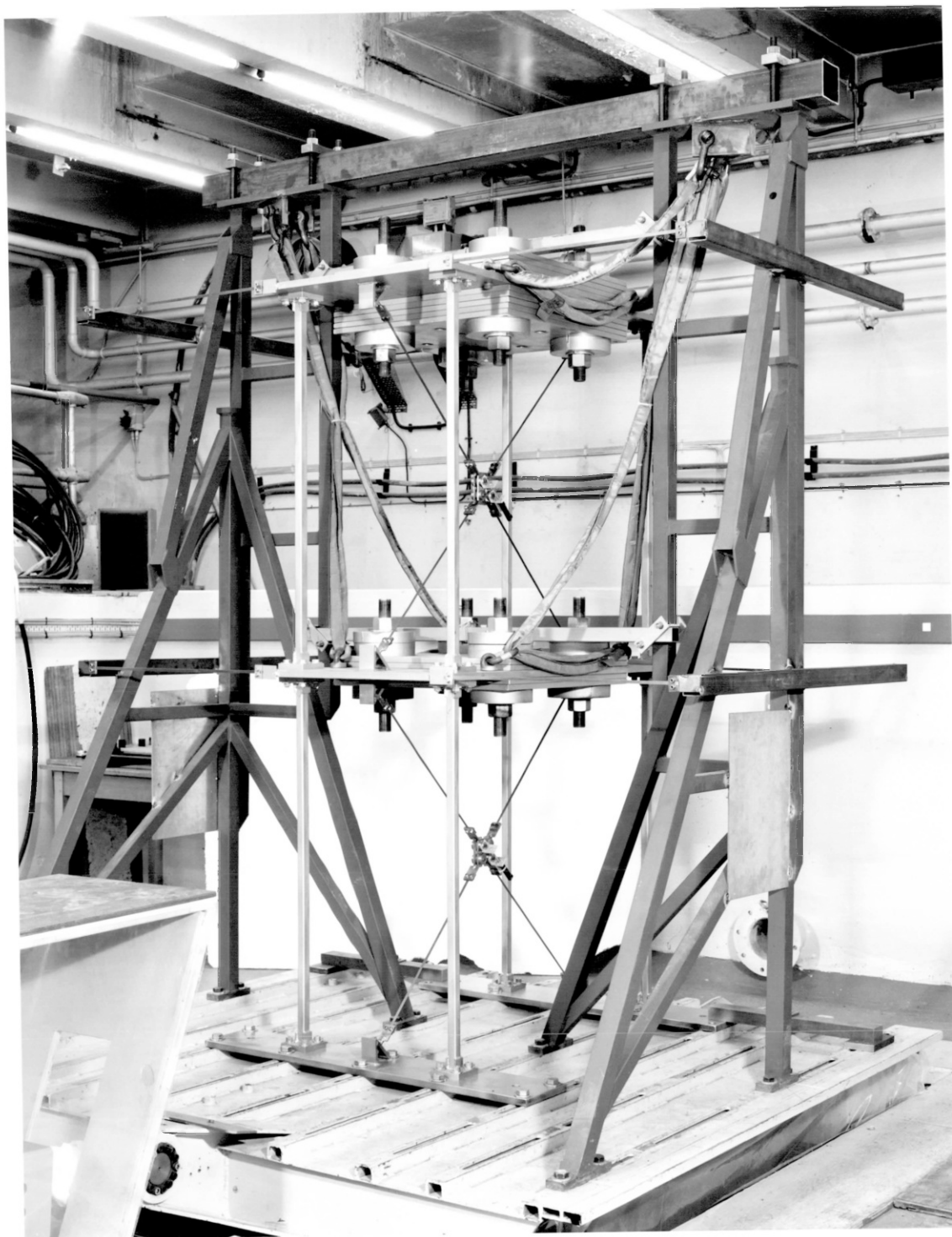


Fig. 7.3 The two-storey braced frame

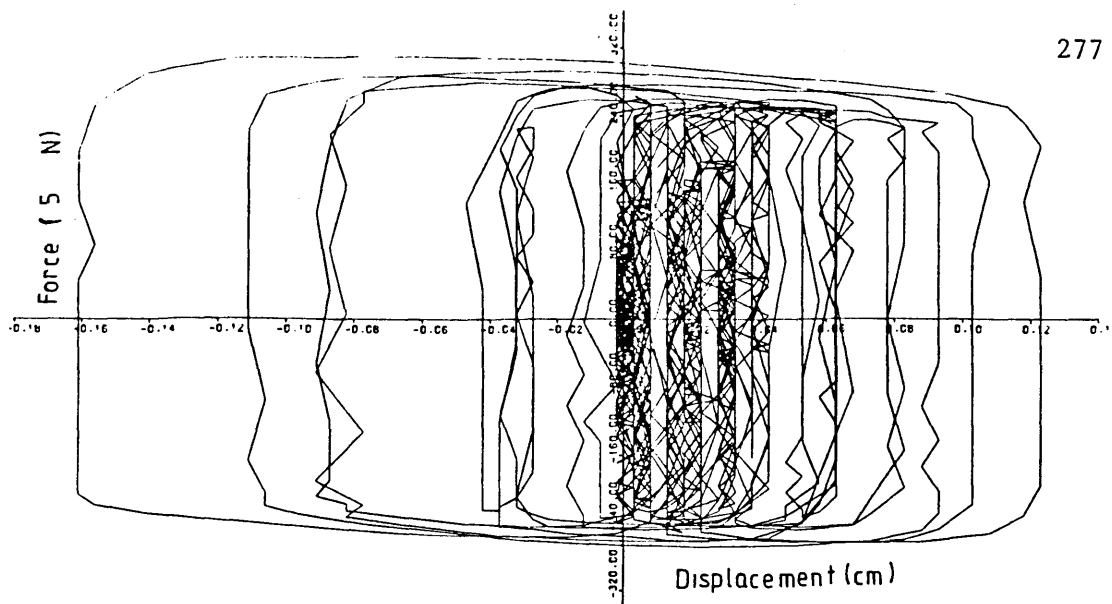


Fig. 7.4 Typical hysteretic response of the friction joint

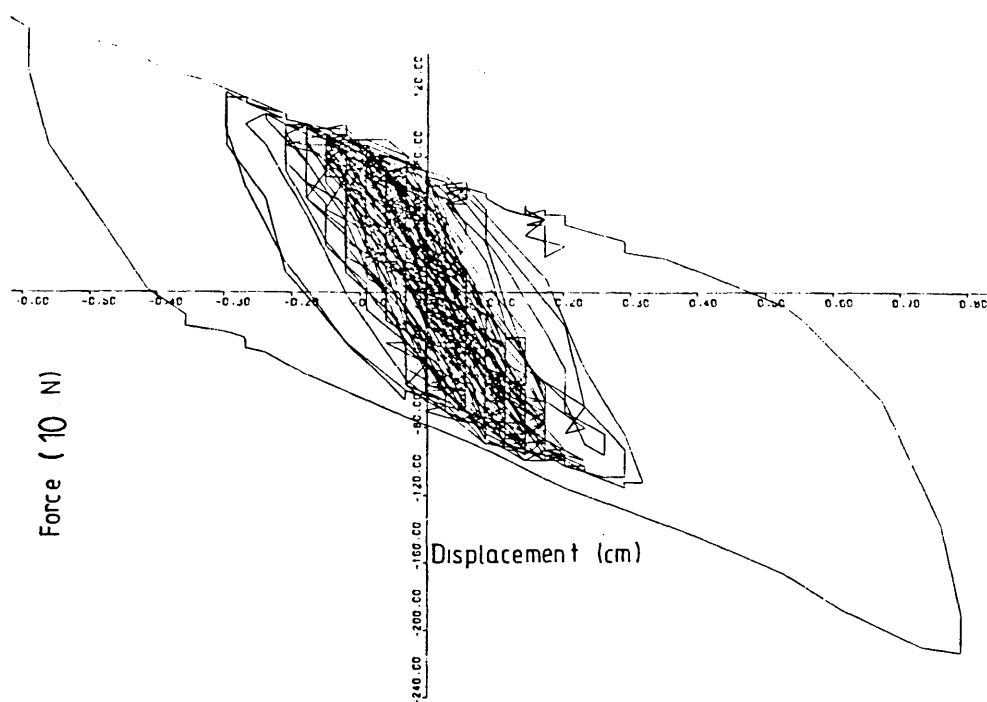


Fig. 7.5 Hysteretic response of the top storey of the frame

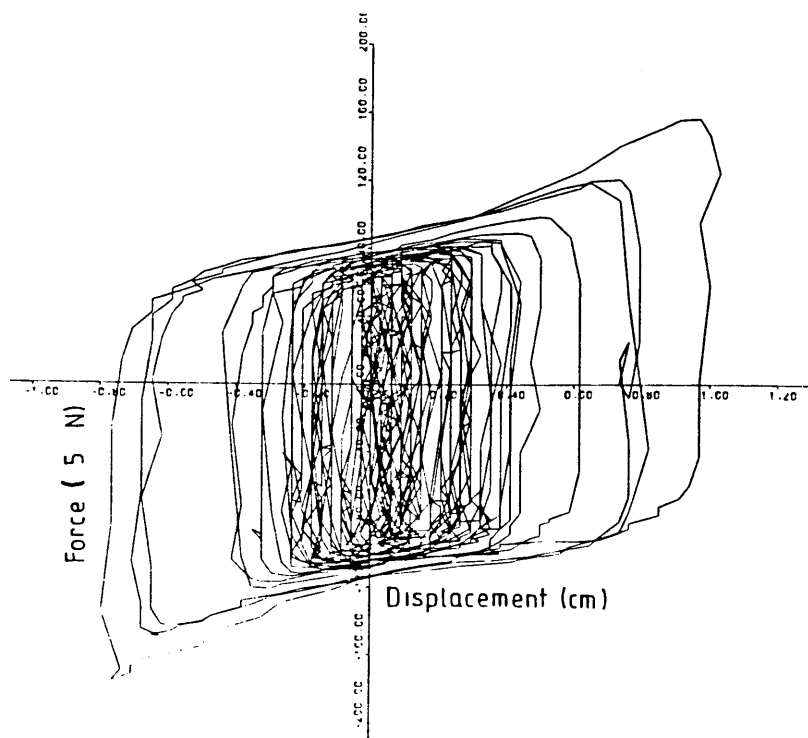


Fig. 7.6 Hysteretic response of the friction joint for higher rotations of the device

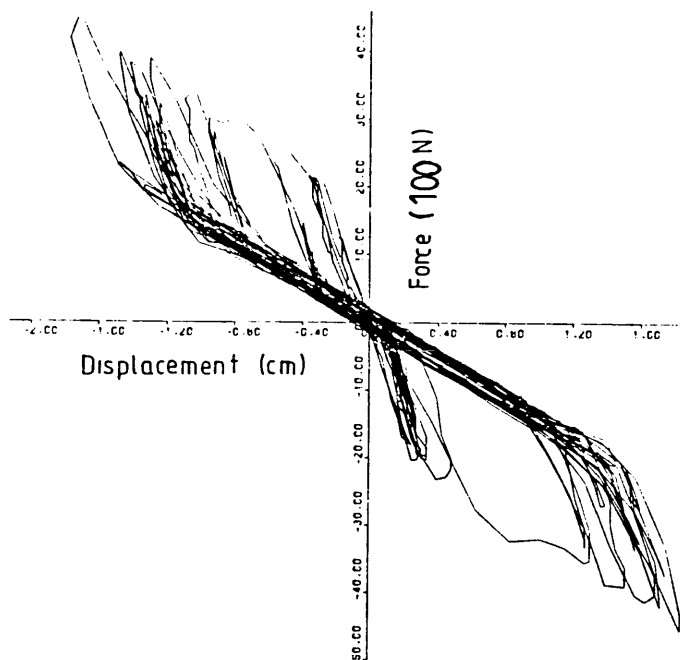


Fig. 7.7 Hysteretic response of the top storey with the devices locked

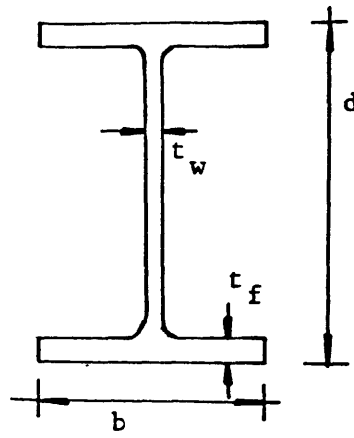


Fig. A1 An I-section beam

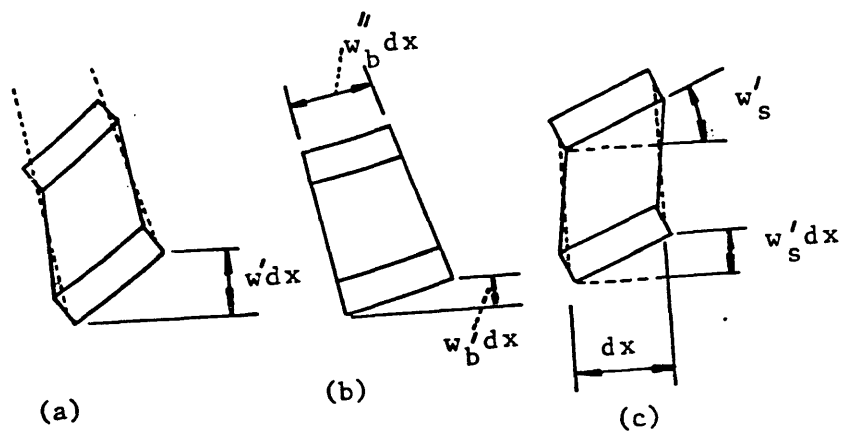


Fig. A2 Deformation of an infinitesimal sandwich beam element

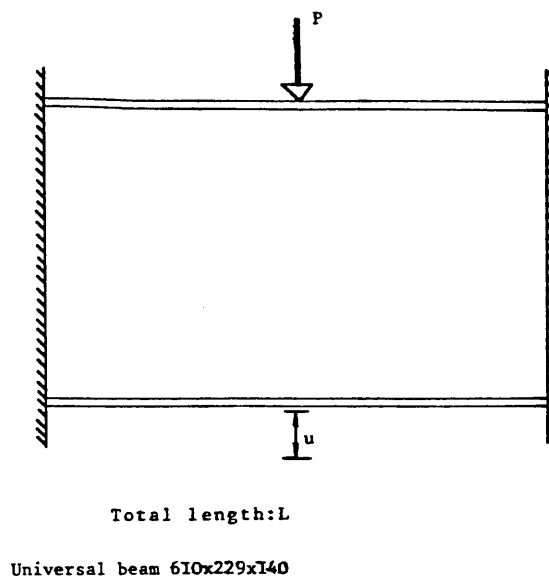


Fig. A3 A fully fixed I-section beam subjected to a point load in the middle

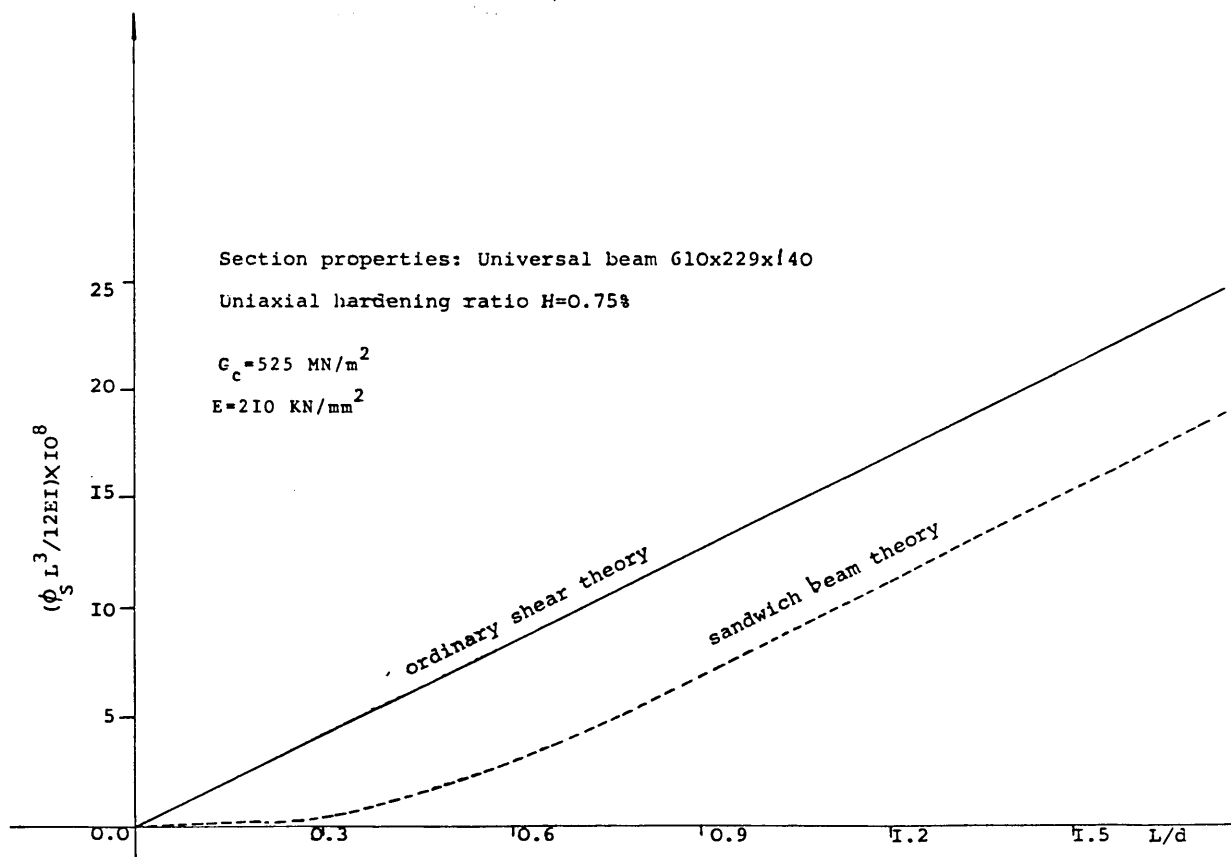


Fig. A4 Comparison between ϕ_s according to the ordinary shear deflection theory and to the sandwich beam theory

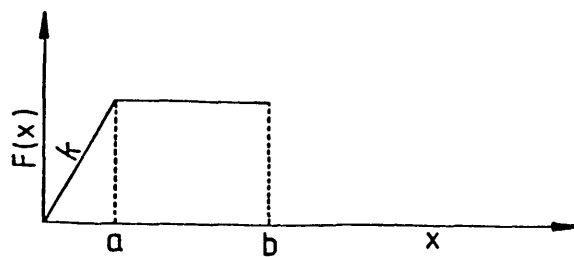


Fig. B1 An elasto-plastic restoring force

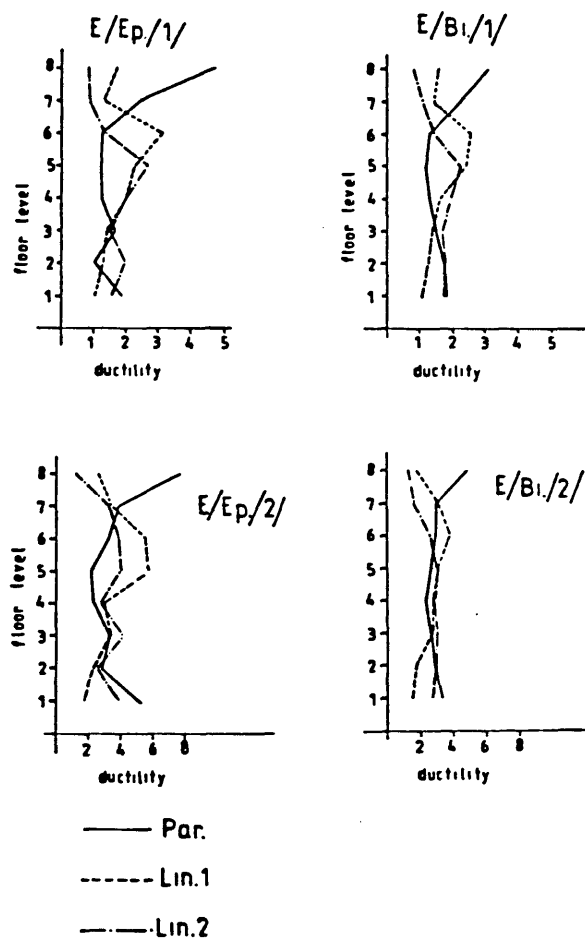


Fig. B2 Ductility distribution in the three structures

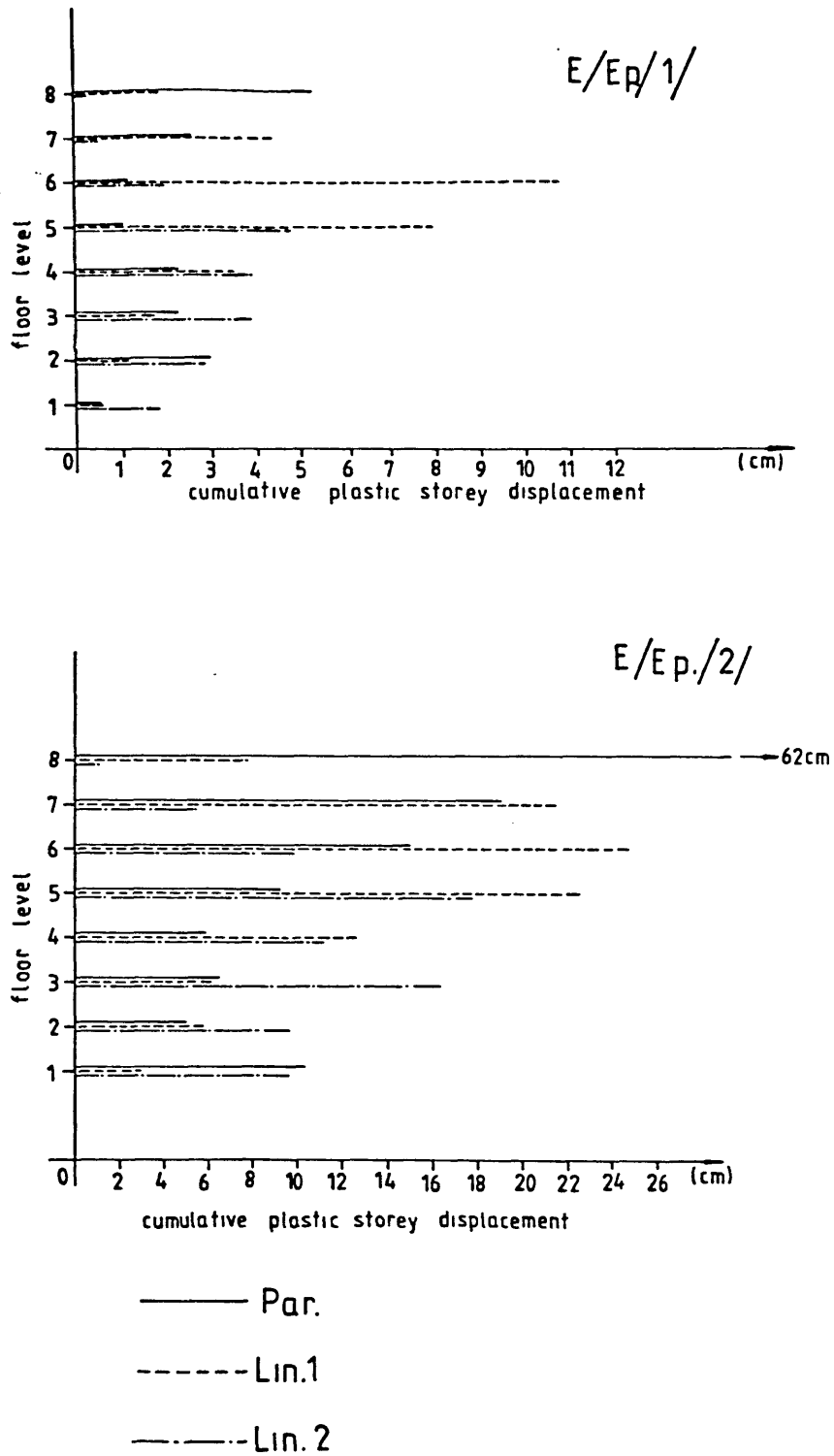


Fig. B3 Cumulative plastic displacement in the three structures

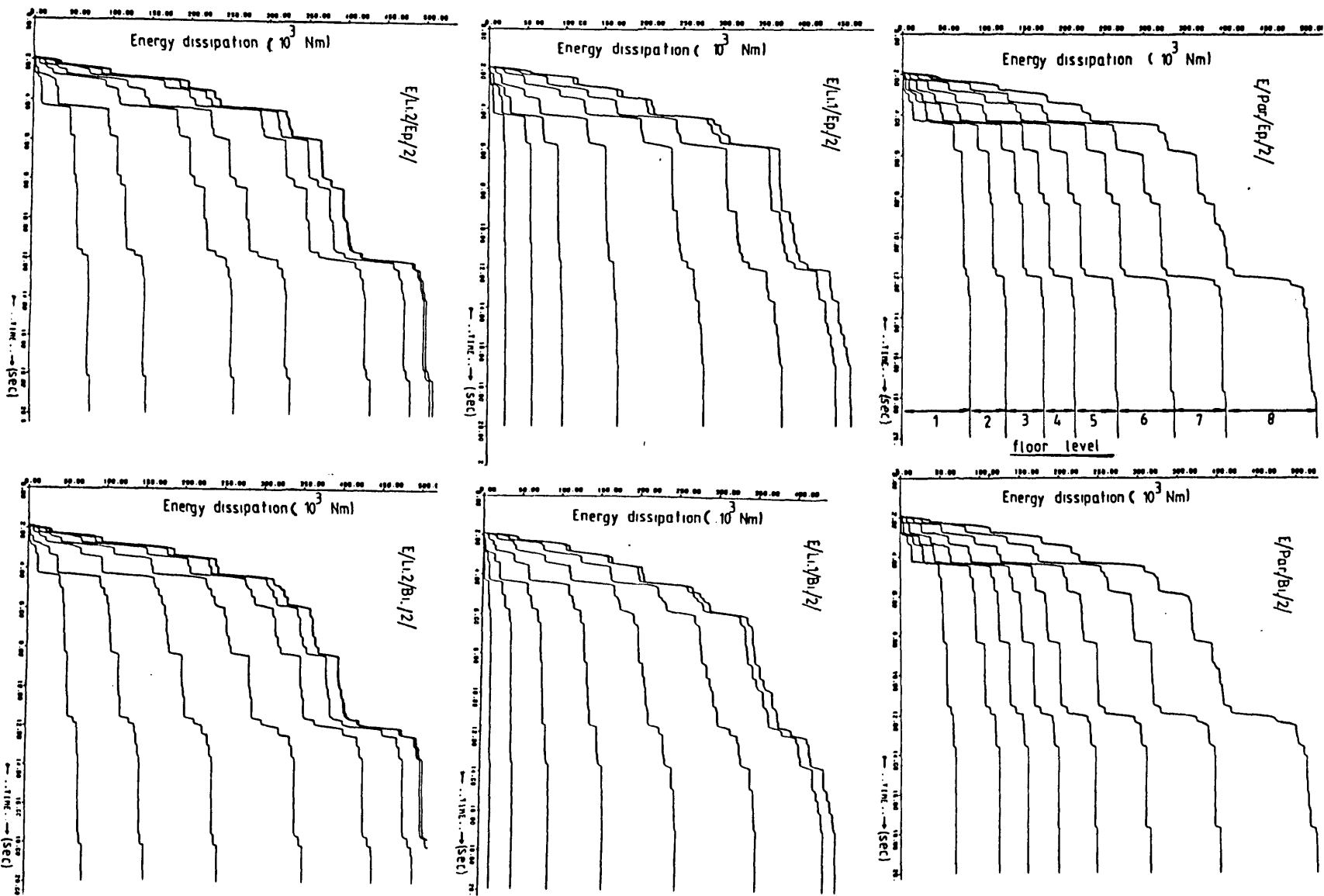
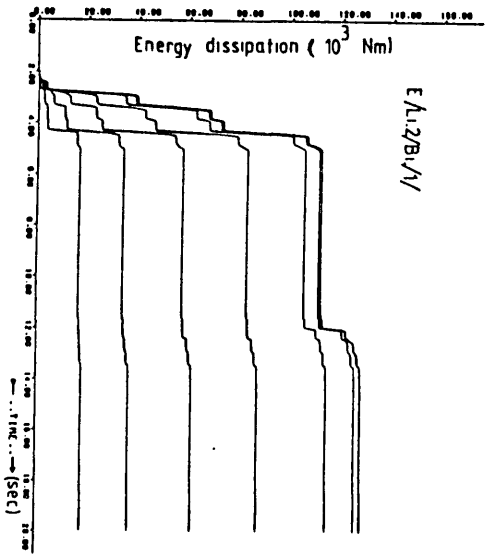
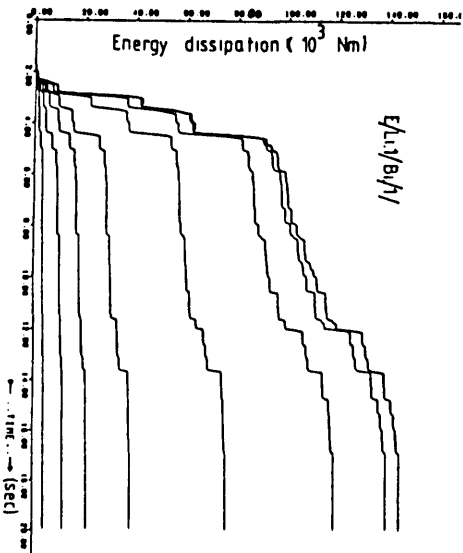
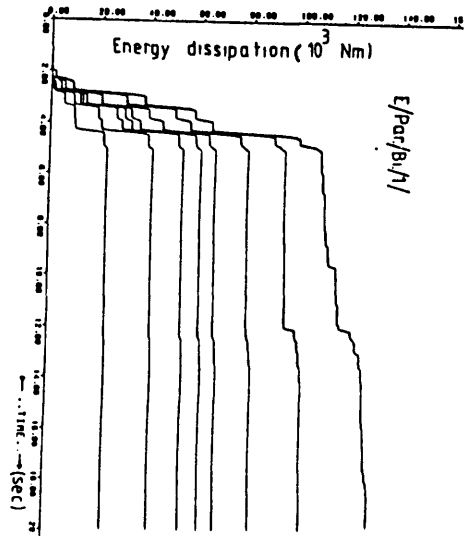
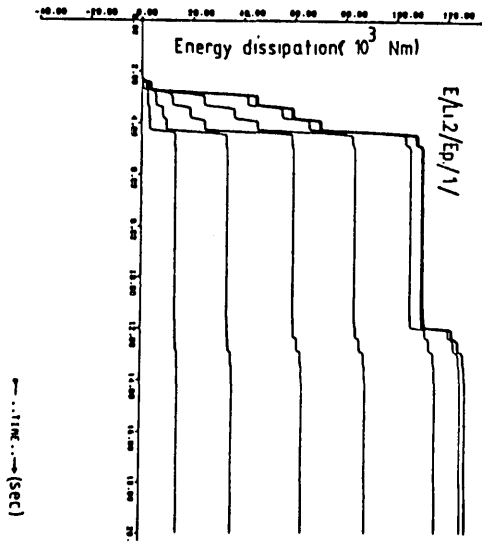
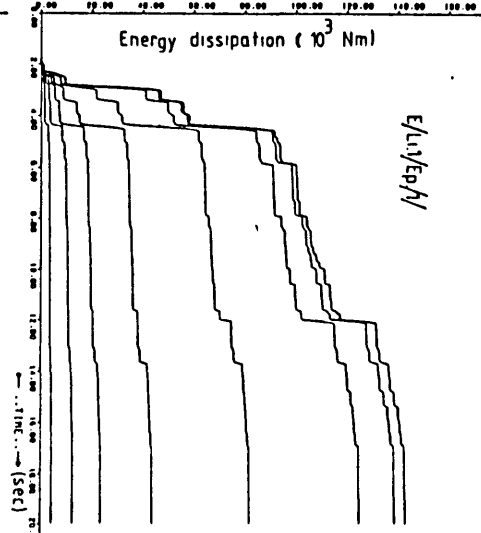
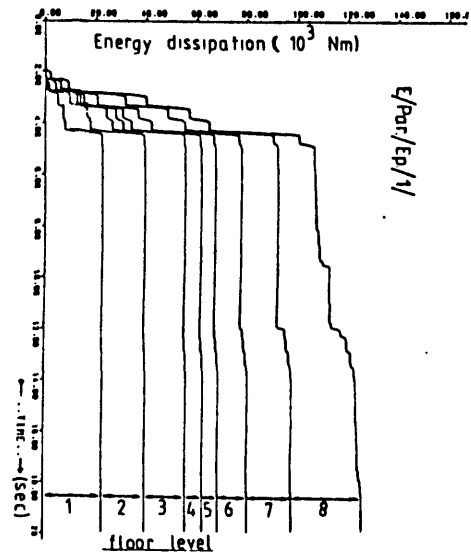


Fig. B4 Energy dissipation per floor in the three structures



(continued)

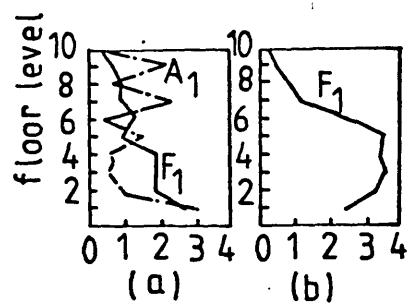


Fig. B5 (a) Column ductility
(b) Beam ductility

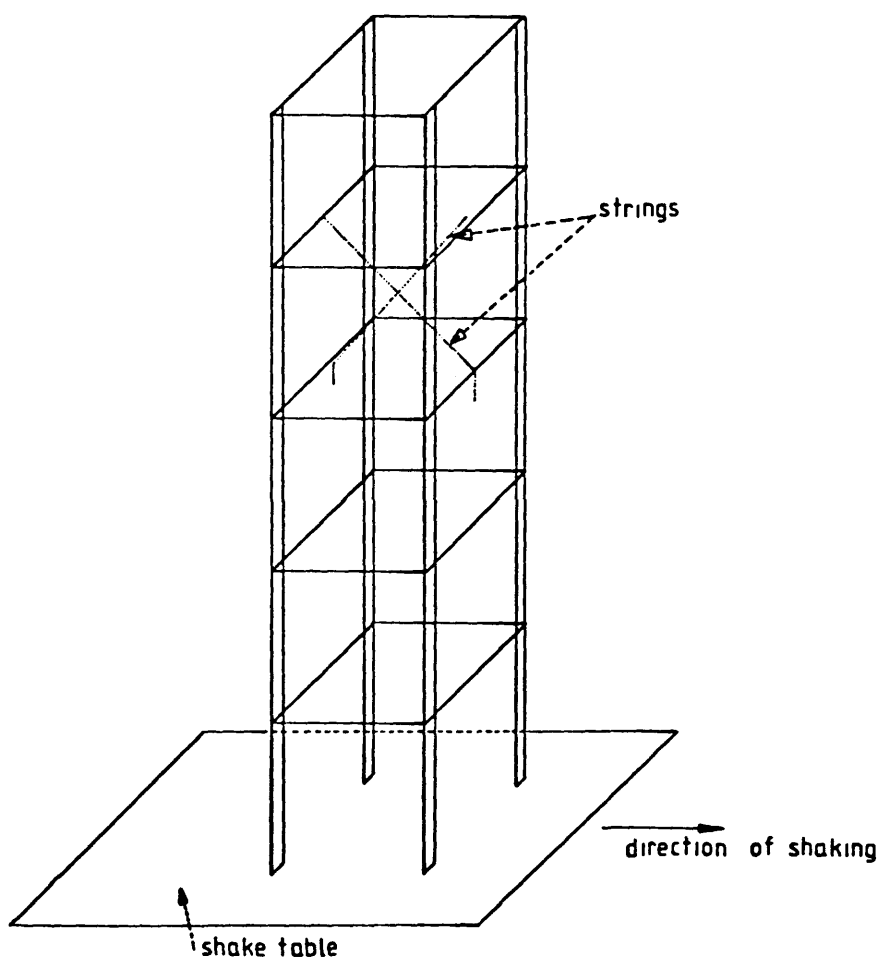


Fig. B6 Shake table experimentation on stiffness distribution

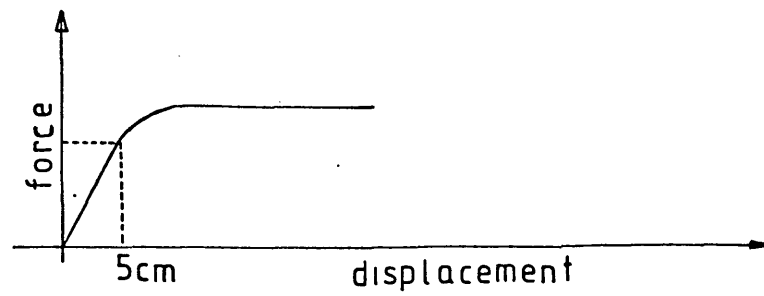


Fig. B7 Experimental restoring force per storey of the frame

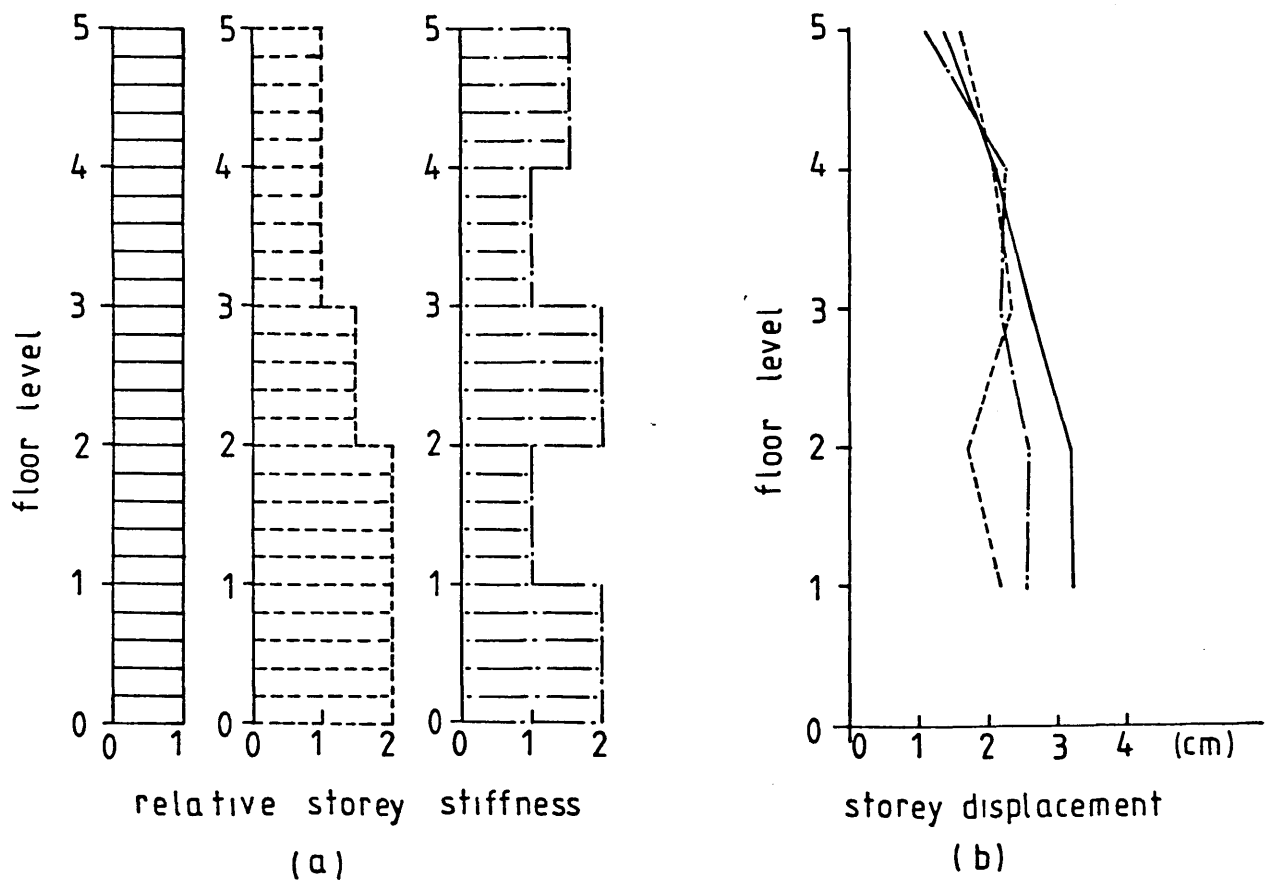


Fig. B8 Earthquake response of the model five-storey frame