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EFFECT OF VERTICAL MOTION ON THE EARTHQUAKE RESPONSE OF REINFORCED CONCRETE COLUMNS

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EFFECT OF VERTICAL MOTION ON THE EARTHQUAKE RESPONSE OF REINFORCED CONCRETE COLUMNS

by

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Synopsis:

This study aims at studying in detail the dynamic performance of reinforced concrete columns under three dimensional excitation of earthquake ground motions. Special attention was placed on the effects of vertical component on the response of the columns. The three dimensional characteristics and hysteretic behavior of RC columns were investigated for various cyclic loading by means of nonlinear fiber finite element modeling.

Key Words : seismic design, RC column, vertical ground motion,
finite element model

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ABSTRACT

This research aims at studying in detail the dynamic performance of reinforced concrete columns under three dimensional excitation of earthquake ground motions and special attention was placed on the effects of vertical component on the response of the reinforced concrete columns. The three dimensional characteristics and hysteretic behavior of RC bridge columns were investigated for various cyclic loading by means of nonlinear fiber finite element modeling.

The analysis exhibited some important phenomena which are uncommon compared with what is usually observed with the RC columns. The fluctuation in axial force and biaxial bending can significantly influence the inelastic cyclic response of RC columns. The analysis also indicates that three dimensional loading results in reduction in strength and flexural stiffness as well as the energy dissipation capacity of RC column, compared to the corresponding uniaxial response. Lack of consideration of these factors may lead to significantly different response in bridges, in cases where three dimensional effects dominate the behavior of RC columns. The simplified phenomenological nonlinear models are not suitable to obtain reasonable results when RC columns are subjected to three dimensional excitation.

The research demonstrated that severe pinching of hysteretic loops of RC column is evident under three dimensional cyclic loading. In some cases, this effects are so drastic that the area enclosed by hysteretic curves is almost zero or even negative, which means that the energy dissipation capacity is zero or negative. In general, this low energy dissipation capacity should be avoided in the design of columns because high level of energy dissipation capacity is desirable for structures.

The nonlinear time history analysis suggests that vertical ground motion affects the general response of the RC columns. Usually, the vertical component tends to aggravate the response.

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1. INTRODUCTION

1.1 General

Bridges are a vital part of the lifeline systems in metropolitan areas. It is imperative that emergency vehicles be able to gain access to the devastated areas to remove victims and to provide disaster relief immediately following a major earthquake. Bridges are important links in the transportation system that must be in service if the affected areas is to recover its economic system efficiently. The closure of transportation routes during an earthquake may further aggravate the situation and paralyze the region.

Experience of destruction of many highway bridges during the past earthquakes revealed the seismic susceptibility of this type of structures and thus stimulated both theoretical and experimental research associated with the seismic response of the highway bridges.

The collapse of several highway bridges during the San Fernando earthquake of February 9, 1971, demonstrated the potential for damage and loss of life associated with modern highway bridges during earthquakes. This was further demonstrated in the October 17, 1989 Loma Prieta earthquake which resulted in the collapse of the Cypress Street Viaduct with a loss of 70 lives(Nim et al, 1989).

As to the recent great Hanshin earthquake disaster in Japan, a large number of bridges were damaged in this earthquake and one estimate of the affected number is of the order of 60% of the total inventory in that area.

1.2 Research Significance

In current earthquake design codes for both building and bridges, it is not required to incorporate the effects of vertical component of ground motions. Such lack of consideration is based on the assumption that the maximum acceleration in the vertical direction is smaller than the maximum acceleration in horizontal directions. Perhaps the additional cost in analysis and design associated with considering the vertical component of ground motion is another reason that contributed to such justification. However, contrary to such assumptions, reports on recent earthquakes indicate that not only is the maximum acceleration in vertical direction significant, but also there have been earthquakes where the maximum acceleration in this direction has exceeded the maximum acceleration in the horizontal directions.

It is the purpose of this chapter to address this point and to present earthquake measurement

and theoretical works that indicate the importance of vertical component of earthquake motions.

1.2.1 Past Earthquakes

Past earthquakes have provided a large amount of valuable data on ground and structural motions. These data have notably increased, and will continue to increase, the knowledge of experts in the field of earthquake engineering. Investigation into some of these data signifies the importance of vertical component of earthquake motions.

On February 9, 1971, a destructive earthquake of magnitude 6.6 struck the northern parts of Los Angeles adjacent to the San Fernando Valley. The maximum vertical acceleration of San Fernando earthquake was recorded at Pacoima Dam and was 0.7g. Observed maximum displacement across the fault trace was about six feet in the vertical direction and about five feet in the horizontal direction (Housner, 1971). Indeed, reports of field investigations indicated that the vertical motion played a major part in damage sustained by bridges during this earthquake. Based on field investigations by Fung, et al. (1971), the vertical acceleration is one of the primary causes of damage during this earthquake. They concluded that the vertical force generated by the vertical acceleration may have been the crucial force that caused the collapse of some of the structures. Elliott and Nagai (1973) also indicated that pounding from vertical acceleration contributed heavily to the destruction although the initial failure may have been from horizontal forces.

The El Asnam earthquake of October 10, 1980, is considered extraordinary because of the strong movement in the vertical direction (Mallick, 1984). A maximum of 5m of vertical shift during this earthquake has been reported. Unfortunately, no acceleration records of this earthquake were available to define the basic dynamic characteristics of the motion. However, there have been indications that the vertical acceleration may have reached a maximum very close to 1.0g, while the maximum horizontal acceleration is estimated about 0.25g (Despeyroux, 1984). Inspection of buildings after the earthquake have shown many cases of failures which could be ascribed only to vertical forces. Despeyroux (1984) indicated that there was practically no example of a failure obtained through the repetition of a number of cycles of an increasing amplitude, with plastic hinging. On the contrary collapse generally appear as the consequence of brittle failures. As it is known recently, axial force in an RC pier has significant influence on its strength, stiffness and ductility. The axial load of high intensity might be induced by vertical ground motion. For example, an increase in axial load adversely affects the ductility and results in brittle failure. Furthermore, reduction in axial load decreases shear capacity of a section causing shear failure of RC members as it was observed during El Asnam earthquake. Pulling out of reinforcement, prior to reaching the section's designed

moment capacity, is another type of damage that can be attributed to tensile axial forces. This type of damage was noticed during the El Asnam earthquake and the San Fernando earthquake.

Finally, contrary to the traditional belief, records show during an earthquake of magnitude 5.9 in Southern California the vertical acceleration was significant and much higher than the horizontal acceleration. The earthquake occurred on July 8, 1986, with an epicenter 12 km northwest of Palm Springs on the San Andreas fault. Strong motion acceleration recordings indicated 0.68g horizontal and 0.78g vertical at North Palm Springs at 8 km epicentral distance, and 0.33g horizontal and 0.59g vertical at Desert Hot Springs at 10 km (EERI, 1986).

1.2.2 Recent Earthquakes

The January 17, 1995, Hyogo-ken Nanbu earthquake is the most damaging earthquake to have struck Japan since the great Kanto earthquake. The earthquake supplied a large number of recordings. Again, the importance of the vertical ground motion was demonstrated in these records. The records from the bridge of east Kobe showed a maximum vertical acceleration of 395 gal and horizontal acceleration of 327 gal. The acceleration records obtained from the Port Island indicated 556 gal vertical and 341 gal horizontal. The records from Amagasaki Bridge showed 324 gal vertical and 294 gal horizontal.

Fig. I shows some data from Hanshin Earthquake. In Fig. II some maximum acceleration measured in this earthquake are shown, with horizontal axis denoting horizontal values and vertical axis denoting the vertical ones. Fig. II also gives some data from 1994 Northridge Earthquake.

1.2.3 Structural Response

Disregarding the vertical component of earthquake motion in analysis of structures under seismic load has led to insufficient attention to recording the vibrational response of structural systems in this direction. For instance, among the 13-channel accelerograph system installed on Imperial County Service building, only one channel is in the vertical direction. It was located in the basement of the building and, therefore, nothing can be learned about vertical vibration of the structure from its recording. It is worthwhile to mention that this building was designed in conformance with the 1967 version of the Uniform Building Code. However, it was severely damaged during the Imperial Valley earthquake of October, 1979. Free field recording of the ground motions indicated 0.21g, 0.24g, and 0.24g maximum acceleration in N-S, E-W, and vertical direction, respectively. Based on seismic design

guidelines, which only consider horizontal motions, the structure should have survived the earthquake. Although there are so many recordings in different levels of the structure in the horizontal direction, with lack of recordings in the vertical direction, it is not surprising that there is not a unique opinion on the cause of the failure among those who investigated this building (Kreger, 1983)

However, there are better records of vibrational response of highway bridges to earthquake motion. Many of the 26 transducers located on or near the Meloland Road Overpass are in the vertical direction. Their recordings show that during the same earthquake (i.e., Imperial Valley, October 9, 1979), the road deck horizontal and vertical peak acceleration were comparable, and they reached acceleration levels of 0.51g and 0.50g, respectively, indicating the possibility and importance of vertical excitation of structural systems caused by ground motions.

Due to extensive damage that occurred to the highway bridges during the San Fernando earthquake, a program was initiated in the mid-1970's within the California Strong Motion Instrumentation Program (CSMIP) to instrument bridges and other transportation structures. The Rio Del Overpass (two-span RC box girder bridge) was instrumented in September 1977 with 20-channels of accelerometers (Shakal, et al., 1984). Since then it has been subjected to three earthquakes for which records are available. During the Cape Mendocino Offshore earthquake of August 1983, the peak vertical acceleration at the center of a span was larger than that at the abutment by nearly a factor of ten. In contrast with the dramatic difference seen for the vertical response of the bridge, the transverse acceleration shows only a moderate increase in amplitude at the center of the bridge compared to the abutment.

1.2.4 Previous Analytical Work

Similar to the measurement of the vertical component of earthquake motions, limited analytical work has been performed on structural systems to assess the effect of this motion on their seismic performance. This is especially true regarding nonlinear behavior of structures. Newmark and Resenbluth (1971) argued the importance of the vertical motion, specifically when it acts simultaneously with other components of ground motion. They mentioned that the importance is usually much greater when inelastic behavior is considered. Actually, Shih and Lin (1982) have shown that for softer structures with a small post-yielding to pre-yielding stiffness ratio, the aggravating effect of vertical motion in causing greater structural response is more pronounced.

Work by Anderson and Bertero (1971) on a 10-story unbraced steel frame indicated that inclusion of the vertical motion can result in significant increase in the ductility requirements

of the upper story girders and columns and lower story columns. Results of a study on a 6-story braced steel frame by Geol (1977) indicated similar effects. Sakurai's (1977) conclusion based on response observations, vibrational tests and numerical work on a 10-story building is that the vertical responses of the building are as important as the horizontal responses to dynamic stability of the structure during right-under earthquakes. Numerical results by Iyengar and Saha (1977) revealed that it is essential to consider the vertical ground motion in analyzing stacks and towers. Finally, elastic analysis of a 6-story reinforced concrete frame has shown that the vertical component of ground motion caused large forces in the upper story columns. Furthermore, axial force in the interior columns of lower stories were considerably affected by this component of ground motion (Kikuchi and Yoshimura, 1984). Hence, analytical work on effects of the vertical motion, although limited, indicates the importance of this motion on seismic performance of buildings.

Due to the relatively low value of the vertical frequencies for bridges (within 2.5 to 6 Hz for the first two modes) the effects of the vertical motion is even more pronounced, since the greatest amount of energy from an earthquake is in this frequency range.

1.2.5 Concluding Remarks

From measurement of ground accelerations during the past earthquake, it is apparent that contrary to the customary belief, the vertical acceleration can reach values comparable to horizontal accelerations or may even exceed these accelerations. Moreover, history of damage and failure of some structures during those earthquake exhibits the possibility of vertical motion as one of the primary causes. Also, considering only horizontal motions, analytical studies have not been able to reveal the sources of all structural failures during past earthquakes. Some previous analyses showed that significant forces might be generated via vertical vibrations, and they can be a potential factor in causing damage or failure of structures. Therefore, it is important to study the effects of vertical seismic excitation.

1.3 Objective and Scope

To date most research works consider only the effect of horizontal components of ground motion. As a result, the current design specifications for highway bridges in Japan, considers only the two horizontal components of motion in the analysis.

The purpose of this study is to investigate the effects of vertical motion, in conjunction with transverse motion, on the behavior of RC highway bridges. Because columns and piers are a major source of energy dissipation through nonlinear behavior, special emphasis will be placed

on the study of these elements. Therefore, it is important to model these components with as realistic of a nonlinear model as practical that can predict the complex behavior of RC members under arbitrary loading conditions and histories.

The main objectives of this study are as follows:

- 1) To establish an analytical model of RC beam-column to handle complex loading pattern and to check the analytical results against experimental test results.
- 2) To conduct a detailed study of nonlinear response of RC columns under 3-D cyclic lateral load and varying axial load.
- 3) To perform detailed study of RC columns to examine the relative importance of vertical motion.
- 4) To make adequate recommendations to the current seismic design and analysis methods for RC columns regarding the effects of vertical earthquake motion.

1.4 The Research Layout

The research work now being performed is composed of four stages as follows:

- (1) Literature review.
- (2) Modification of available computer software to meet the needs of present study.
- (3) Fundamental studies on the behaviors of nonlinear response of RC columns under three dimensional cyclic lateral load and axial load.
- (4) Nonlinear time history analysis of RC columns to examine the relative importance of vertical motion.

2. ESTABLISHMENT OF MODEL AND NUMERICAL ANALYSIS

2.1 Introduction

In highway bridge structures the three dimensional motion of an earthquake has greatest effects on bridge piers. The piers are subjected to biaxial bending from combined longitudinal, transverse, and torsional motion of the structure, with added axial loads due to vertical ground motion. It is well known that the bending strength in any particular directions decreased by the existence of a simultaneous moment along another axis. Recently, a number of papers have been published on the earthquake damage to building structures during the 1971 San Fernando earthquake. In these studies, the effects of two dimensional earthquake motion on the response of structural members was recognized to be substantially more significant than had been previously anticipated.

In order to obtain true evaluations of seismic safety, strong motion response analyses which consider the dynamic structural properties in the inelastic range are essential. Such analyses must be able to trace the damage process of the structural members in detail. A detailed analysis of the dynamic evolution of a structure subjected to intense ground motion requires realistic modeling of the restoring force characteristics of the constituent members. One of the difficulties in this regard is the idealization of the three dimensional interaction of the restoring forces in members subjected to biaxial bending with varying axial force. An identification of the characteristics of this class of dynamic behavior is important to the understanding of the nature of severe damage in structures of a variety of types.

Two basic procedures may be used for modeling the inelastic behavior of beams and columns: fiber type and section type. In the fiber type of model, the member cross section is divided into a number of small areas (fibers). Each fiber is assumed to be uniaxially stressed and to have behavior governed by the hysteretic stress-strain characteristics of the material it simulates. Detailed and accurate results can be obtained from this type of models, but the computational effort required makes them expensive for practical application. In the section type of model, it is assumed that inelastic behavior is defined for the cross section as a whole, not for individual fibers. Force-deformation relationships for the cross section must be specified, each governed by the cross section dimensions and the hysteretic force-deformation characteristics of the member material. Models of this type tend to be more difficult to use and less accurate than fiber models, but more efficient computationally.

The research described in this paper has used the fiber type model.

2.2 Historical Background

Many studies of inelastic frames under earthquake forces have been described in recent years. A brief history of more recent studies is presented here.

The action-deformation relationships assigned to a member can have a significant influence on the calculated response. As a result, nonlinear analysis has concentrated on the modeling of stiffness changes in the members and the establishment of realistic hysteretic behavior.

Hidalgo and Clough investigated a number of analytical models for the response prediction of a two-story, single bay frame, which they also tested on a shaking table. Starting with a two-component, elasto-plastic element, they attempted to improve the correlation between analysis and experiment by adding degradation effects to the model. One method of including degradation effects was to impose empirical changes in the value of the elastic modulus at specified times during the excitation. A second technique was based on degradation of the generalized stiffness of the first mode of vibration of the structure. Although these techniques can provide accurate results for specific frames, they are not convenient for general purpose application.

Takeda examined the experimental results from cyclic loading of a series of reinforced concrete connections, and proposed a hysteresis model which was in agreement with these results. Several investigators have used this model, in both its original and modified forms. Litton adopted a modified Takeda model for a reinforced concrete beam element for the DRAIN-2D computer program. This element consists of an elastic beam element with inelastic rotational springs at each end. A similar type of element has been suggested by Otani. This element consists of a bilinear beam element with an inelastic rotational spring and a rigid link at each end. Neither of these models considered biaxial interaction effects.

Riahi and Powell have described a 3D beam-column element and incorporated it into the ANSR computer program. The element is assumed to be made up of three parallel components, two elasto-plastic components to represent yielding and one elastic component to model strain hardening. Interaction for biaxial bending and axial force are considered, but the element does not have stiffness degradation characteristics.

Takizawa and Aoyama have developed the basic formulation for a reinforced concrete column model acted upon by biaxial bending moments. This model incorporates a two dimensional extension of various nonlinear models for one-dimensional response analysis, in particular a degrading trilinear stiffness model. The theory demonstrates how degradation effects can be considered, but it does not account for axial forces and was not applied in a complete beam-column element.

Morris presented a procedure for three dimensional frames by employing an approximate interaction equation by Tebedge and Chen for I section column under biaxial bending. However, complete loading–unloading cycles and hysteretic behavior at the plastic hinges were not studied. Uzgider adopted this method to study the hysteretic behavior at plastic hinges for three–dimensional dynamically loaded frames. The elasto–plastic action–deformation relationships at the ends of the frame member was represented by an equation which corresponds essentially to the inverse of the Ramberg–osgood representation. Inelastic interaction of biaxial end moment and axial force was included.

Chen and Powell developed a 3D beam–column element using generalized plastic hinge concepts. The element belongs to section type of model, considering both the distributed plasticity and lumped plasticity approaches. In developing the element, inelastic interaction between bending moments, torque and axial force was considered by means of yield interaction surfaces and a flow–rule type of plasticity theory.

2.3 Fiber Element Used in this Study

A flexibility–based fiber model is employed in the present research, which is implemented in the DRAIN family program: DRAIN–3DX.

2.3.1 Element Features

The reinforced concrete beam–column fiber element used in this research is based on the flexibility method. The behavior of the element is established at a number of control sections along the element span. The element sections are subdivided into steel and concrete fibers whose behavior determine the response. The section subdivision into fibers naturally accounts for the coupling between axial force and bending moments. The nonlinear nature of the element derives entirely from the constitutive relations of the steel and concrete fibers.

2.3.2 Concrete and Steel Constitutive Relationships

The concrete and steel constitutive relations are shown in Fig. III.

2.4 Behavior of Single Column under Various Loading Conditions

In order to gain a better understanding of the behaviors of a single RC column. Numerical

analysis of RC columns under various loading conditions needs to be conducted. A cantilever beam-column is used to perform the analysis. The unit used in the analysis is kip and inch(1 kip=4.448kN, 1 inch=25.4mm) and compressive axial load is defined to be positive.

2.4.1 Response of Column under Uniaxial Cyclic Loading with Constant Axial Force

In this analysis, the column is loaded at the top point and the input is cyclic displacement while a constant axial force is added simultaneously. A single load cycle is applied so as to capture the main characteristics of the hysteretic curves in terms of the moment-curvature at the bottom section.

Fig.1 to Fig.5 show the moment curvature relations at the bottom of the column under different constant axial loads, in which Fig.1 gives those under tensile axial loads and the rest give those under compressive axial loads. In those figures, P stands for constant axial force and compressive values are taken as positive.

Fig.6 Shows the typical hysteresis curves under tensile axial load. For the convenience of illustration, the beam face on the tensile side at initial loading is defined as side A, while on the compressive side as side B. It can be seen that the curve goes through the following points where the steel and the concrete change state during the one cycle loading.

- o) starting point where the concrete on the whole section has already cracked before loading, due to the existing tensile axial force;
- a) concrete crack on side B closes, hence, the stiffness of the whole section increases;
- b) steel on side A begins to yield in tension, which leads to significant reduction in the section stiffness;
- c) unloading;
- d) concrete crack on side B re-opens, thus, the section stiffness decreases;
- e) steel on side B yields in tension;
- f) unloading in opposite direction;
- g) steel on side A yields in tension again.

Fig.7 shows the strain histories of steel and concrete on the two outer sides A and B of the bottom section, on which the points corresponding to a through g have also been represented. Fig.8 gives the stress-strain histories of steel and concrete. These figures provide a help to understand the steel and the concrete behaviors in the beam during the cyclic loadings under tensile axial force.

Fig.9 Shows the typical hysteresis curves under compressive axial load. It can be seen that

the curve traces the following points where the steel and the concrete change state.

- o) starting point where the whole section is in uniform compression under the existing compressive axial force;
- a) concrete cracks on side A, hence, the stiffness of the whole section begins to decrease;
- b) steel on side A begins to yield in tension, which leads to significant stiffness reduction;
- c) unloading;
- d) concrete cracks on side B, thus, the section stiffness decreases further;
- e) steel on side A yields in compression;
- f) concrete crack on side A closes, the stiffness of the whole section increases;
- g) steel on side B yields in tension.
- h) unloading in opposite direction;
- i) concrete crack on side A re-opens, the section stiffness decreases;
- j) steel on side B yields in compression.
- k) concrete crack on side B closes, the stiffness of the whole section increases;
- l) steel on side A yields in tension again.

Fig.10 shows the strain histories of steel and concrete on the two outer sides of the bottom section, on which the points corresponding to a through l have also been labeled. Fig.11 gives the stress-strain histories of steel and concrete. These figures can serve as a help to understand the steel and the concrete behaviors in the beam during the cyclic loadings under compressive axial force. Here, the controlling factors to the curve are concrete crack opening and closing, steel yielding in tension and in compression.

It should be mentioned that the features shown in Fig.6 and Fig.9 only exist when the material models adopted in this analysis are employed. Different material models will demonstrate different characteristics on the hysteresis curves.

Fig.12 through Fig.29 show detailed plots for axial load P of -5.0kip , -10.0kip , -15.0kip , -20.0kip , -30.0kip , -40.0kip , 0.0kip , 5kip , 10.0kip , 20.0kip , 30.0kip , 40.0kip , 50.0kip , 60.0kip , 70.0kip , 80.0kip , 90.0kip , and 100.0kip . In each figure, (a) is moment-curvature hysteresis curve; (b) and (c) are steel stress-strain histories for side A and B; (d) and (e) are concrete stress-strain histories for side A and B; (f) and (g) are strain histories for steel and concrete; (h) and (i) are stress histories for steel and concrete. Compressive axial load is defined to be positive and compressive stress and strain of concrete are treated as positive values.

For those figures of tensile axial load cases, steel yields only in tension. It can be seen that the concrete affects the behavior during the first loading and unloading when the axial load is tensile; beyond that the section reduces to a steel couple. Only the concrete on side B works

a little during the initial loading and unloading stages, e.g. portion a-b and c-d in Fig.6. As the tensile axial load increases, so less does the concrete affect the hysteretic behavior and portion a-b and c-d tend to disappear. The strength of the section also reduces. Only the steel works and the concrete does not work any more when the tensile axial force has increased to relatively large values. It is interesting to note that the section stiffens during its initial elastic loading. Since a tensile load is applied first, thus the section is cracked, and as the moment is applied the crack closes on one side(side B) increasing the effective stiffness of the section until yielding of the tensile steel occurs. In the plot corresponding to zero axial load, the portions corresponding to initial loading and unloading are stiffer due to the concrete action. In the subsequent loading or unloading portions the concrete is incapable of carrying any tension because it has already cracked. Therefore, there is a drop in stiffness as well as moment capacity compared to the corresponding values in the initial loading.

For the figures of compressive axial load cases, steel yields both in tension and in compression. Because of the compressive load acting on the symmetrically reinforced section, the crack on the compression side must close so that the concrete fibers can participate again, before the steel on the tension side can yield. The compression yielding of steel results in a lower yield plateau (portion e-f and j-k in Fig.9) and the closing of the crack causes a sudden increase in the section stiffness (portion f-g and k-l in Fig.9) thereby producing the pinched hysteresis loops, which results in the squeezing of the hysteresis loops upon unloading and reloading. It is worth mentioning that the larger the compressive axial force is, the earlier this yield plateau occurs. Moreover, it can even occur when the bending moment is positive during unloading. This indicates that the steel yields earlier under higher compressive axial force than under lower axial force. As the axial load increases, so does the pinching effect. Another fact that should be noted is that the hysteresis curves become more smooth under higher axial loads. Note also that moment capacity also increase with the compressive axial load.

2.4.2 Response of Column under Uniaxial Cyclic Loading with Varying Axial Force

In this analysis, a column with a constant compressive axial force of 33.4kip is loaded at the top point and the input is cyclic displacement while a cyclically varying axial force of different periods is added simultaneously. P_a is the amplitude of varying axial load.

In order to study the effects of varying axial load on the hysteretic performance of columns, different period ratios T_a/T_b between the period T_a of varying axial force and the period of bending T_b are considered. Also, the effects of phasing between the bending and axial loading is studied; in which

2.4.2.a Response under different period ratio T_a/T_b

In this section, analyses under period ratios T_a/T_b of 1.0, 0.75, 0.5, and 0.25 have been conducted, with amplitudes of varying axial load P_a of 20.0kip, 30.0kip, 40.0kip, 50.0kip, 60.0kip and 70.0kip, respectively.

Fig.30 to Fig.33 show the moment curvature relations at the base of the column under different period ratios. Fig.34 through Fig.37 show detailed plots for those loading cases with $P_a=40.0$ kip. In each figure, (a) is moment–curvature hysteresis curve; (b) and (c) are steel stress–strain histories for side A and B; (d) and (e) are concrete stress–strain histories for side A and B; (f) and (g) are strain histories for steel and concrete; (h) and (i) are stress histories for steel and concrete. Compressive stress and strain of concrete are treated as positive values.

An interesting behavior that is totally different from what is normally observed from RC columns can be seen. These figures show obvious departures from conventional hysteresis curves, among which the most radical case is when $T_a/T_b=0.5$. Fig.38 gives the typical hysteresis curve when $T_a/T_b=0.5$ and $P_a=40.0$ kip. The hysteresis loop demonstrates almost zero energy dissipation capacity. The moment–curvature curve indicates that as the axial force decreases the column softens and releases its load. After crossing the loading branch, unloading takes place with such a severe pinching that the enclosed area is very small or even negative. It can be said that the energy dissipation capacity is nearly zero. Generally speaking, this low energy dissipation should be avoided in the design of bridges, because high energy dissipation capacity is beneficial for bridges during an earthquake. Another feature worth noting is that the hysteretic loops tend to be complicated as the period ratios decrease, i.e. as the frequency of axial force becomes higher than that of lateral force.

In Fig. 38, the curve traces the following points where the steel or the concrete changes state.

- o) starting point where the whole section is in uniform compression under the existing compressive constant axial force;
- a) concrete cracks on side A, hence, the stiffness of the whole section begins to decrease;
- b) steel on side A begins to yield in tension, which leads to significant stiffness reduction;
- c) unloading of varying axial load;
- d) unloading of bending moment and reloading of varying axial load in inverse direction;
- e) steel on side A yields in compression;
- f) unloading of varying axial load;
- g) concrete cracks on side B, hence, the stiffness of the whole section begins to decrease;
- h) steel on side B begins to yield in tension, which leads to significant stiffness reduction;
- i) unloading of varying axial load;

- j) unloading of bending moment and reloading of varying axial load in inverse direction;
- k) steel on side A yields in compression;
- l) unloading of varying axial load.

Fig. 39 gives the strain histories of steel and concrete on the two outer sides of the base section, on which the points corresponding to a through l have also been shown. Fig.40 gives the stress-strain histories of steel and concrete. These figures can help to understand the steel and the concrete behaviors in the beam during the cyclic loadings with varying axial load.

From these plots, the following features on the hysteresis loops of RC columns can be detected for $T_a/T_b=0.5$:

1. The section yields at a lower moment due to the increasing axial force in tension(point b in Fig. 39).
2. During unloading from point d to e, the stiffness of section at portion d-e is smaller than that at portion c-d. This can be explained by the fact that the strain and stress of steel reduce quicker at this portion than at portion c-d,
3. Unloading of steel coincides with the unloading of the varying axial force, while unloading of concrete coincides with the unloading of bending moment. In this case, it means that the behavior of steel is mainly controlled by the varying axial load, while the behavior of concrete is mainly controlled by the moment reversal.
4. As a results of varying axial force, more alternate yielding of steel in tension and in compression occurs compared to the case of constant axial force.
5. On the one hand, compressive axial force makes portion f-e higher. On the other hand, tensile axial force makes portion b-c lower. Therefore, the area between b-c and f-e becomes smaller, as the varying axial force amplitude P_a increases. Furthermore, this area becomes negative when P_a increases to a large value, resulting in substantial pinching in the hysteresis loop as shown in Fig 38. The same phenomenon occurs to portion h-i and k-l.

It can be said that it is clearly apparent that the variation in axial force can significantly affect the inelastic cyclic response of RC columns. The characteristics of RC columns under varying axial load is important factors for seismic performance in design practice for bridges, particularly, when the resulting low energy dissipation is considered. As is known to all, high levels of energy dissipation lead to small displacement in the structure which is desirable for bridges where control of displacements is important during the earthquakes. Lack of consideration of these factors may result in significantly different response in bridges, in cases where fluctuation of axial force affects the behavior of RC piers and columns.

The analysis performed here reveals some important phenomena which are truly

unconventional compared with what is commonly observed. However, this work is only a kind of numerical simulation, validation against experimental work is needed.

2.4.2.b Response under different phasing between bending and axial loading

In order to investigate the effects of different phasing between bending and axial loading on the hysteretic behavior of columns, the previous column is now analyzed under bending and axial force of different phase angles and period ratios. In this analysis, numerical computation for phase angle of 90° and 45° with period ratios T_a/T_b of 1.0, 0.75, 0.5, and 0.25 have been conducted, the amplitudes of varying axial load P_a being 20.0kip, 30.0kip, 40.0kip, 50.0kip, 60.0kip and 70.0kip, respectively.

Fig.41 to Fig.44 show the hysteretic loops at the base of the column with phase angle equal to 90° under period ratios T_a/T_b of 1.0, 0.75, 0.5, and 0.25, respectively. Fig.45 through Fig.48 show the hysteretic loops with phase angle equal to 45° under period ratios T_a/T_b of 1.0, 0.75, 0.5, and 0.25, respectively.

It can be seen that the phasing has significant influence on the hysteretic loops. Different phasings lead to different loops, even when the period ratios and the amplitudes of varying axial load are the same.

The loops are more sensitive to the phase angle for lower period ratios, i.e. for higher frequencies of axial forces.

2.4.3 Response of Column under Biaxial Cyclic Bending

In this analysis, the column is loaded at the top point and the input is cyclic displacement in one direction along the principal axis of the section with a cyclic displacement of different periods perpendicular to the same axis.

2.4.3.a Response under different period ratio T_y/T_z

In this section, the previous column is analyzed under simultaneous input of biaxial cyclic lateral tip displacements and constant axial load. The axial load considered equals one tenth of the ultimate axial load. The strong axis herein is referred to as Z axis, while weak axis as Y axis. The amplitude of cyclic load along Z axis is referred to as D_z , while that along Y axis as D_y . The period of cyclic load along Z axis is referred to as T_z , while that along Y axis as T_y . Numerical analyses have been conducted for period ratios T_y/T_z equaling 1.0, 0.75, 0.5, 0.25, 1.5, 2.0, 4.0, with amplitude ratios of lateral load D_y/D_z equaling 0.75, 0.375, 0.075, 0.0375, respectively. For comparison, analysis was also performed separately for uniaxial cyclic loading

along the two principal axes.

Fig. 49 and Fig. 50 give the hysteretic curves at the base of the column for uniaxial cyclic loading along Z axis and Y axis.

Figs. 51, 53, 55, 57, 59, 61, 63 show the hysteretic loops at the base of the column along strong axis Z, for period ratios T_y/T_z equaling 1.0, 0.75, 0.5, 0.25, 1.5, 2.0, 4.0, and amplitude ratios of lateral load D_y/D_z equaling 0.75, 0.375, 0.075, 0.0375, respectively. Figs. 52, 54, 56, 58, 60, 62, 64 show the hysteretic loops along weak axis Y, for period ratios T_y/T_z equaling 1.0, 0.75, 0.5, 0.25, 1.5, 2.0, 4.0, and amplitude ratios of lateral load D_y/D_z equaling 0.75, 0.375, 0.075, 0.0375, respectively.

It can be seen that there are obvious interactions between the biaxial bending on the hysteretic behaviors along the two principal directions. The analysis also indicates that biaxial bending results in reduction of strength and flexural stiffness compared to the corresponding uniaxial response. Biaxial bending affects the hysteretic characteristics of the column. Pronounced pinching of the hysteresis loops is still evident though it may be different from the uniaxial behavior under similar axial force.

Hysteretic curves in the principal directions differ for different period ratios, particularly in the Y direction. The initial stiffnesses in the two orthogonal directions are different. Thus, the period ratio is critical in the Y direction, since the strength of the column in this direction is less.

When the amplitude in the weak axis is less than a certain value, the effects of bending along the weak axis on the strong axis are very small or zero. On the other hand, the bending along the Z axis has stronger effects on the weak axis. The bending along weak axis exhibits hysteretic behavior even when the bending moment is below the yielding moment of the corresponding uniaxial bending, as long as the bending along the strong axis exists.

2.4.3.b Response under different phasing between biaxial bending

The effects of different phasing between biaxial bending on the hysteretic behavior are studied in this section. The previous column is analyzed under simultaneous input of biaxial cyclic lateral tip displacements of different phase angles and period ratios. The constant axial load considered equals one tenth of the ultimate axial load.

In this analysis, numerical computation is performed for phase angle of 90° and 45° with period ratios T_y/T_z of 1.0, 0.75, 0.5, 0.25, 1.5, 2.0, 4.0, and amplitude ratios of lateral load D_y/D_z equaling 0.75, 0.375, 0.075, 0.0375, respectively.

Figs. 65, 67, 69, 71, 73, 75, 77 show the hysteretic loops at the base of the column along strong axis Z, for phase angle of 90° and period ratios T_y/T_z of 1.0, 0.75, 0.5, 0.25, 1.5, 2.0, 4.0, and amplitude ratios of lateral load D_y/D_z of 0.75, 0.375, 0.075, 0.0375, respectively. Figs. 66, 68, 70, 72, 74, 76, 78 show the hysteretic loops along weak axis Y, for phase angle of 90° and period ratios T_y/T_z of 1.0, 0.75, 0.5, 0.25, 1.5, 2.0, 4.0, and amplitude ratios of lateral load D_y/D_z of 0.75, 0.375, 0.075, 0.0375, respectively.

Figs. 79, 81, 83, 85, 87, 89, 91 show the hysteretic loops at the base of the column along strong axis Z, for phase angle of 45° and period ratios T_y/T_z equal to 1.0, 0.75, 0.5, 0.25, 1.5, 2.0, 4.0, and amplitude ratios of lateral load D_y/D_z equal to 0.75, 0.375, 0.075, 0.0375, respectively. Figs. 80, 82, 84, 86, 88, 90, 92 show the hysteretic loops along weak axis Y, for phase angle of 45° and period ratios T_y/T_z equal to 1.0, 0.75, 0.5, 0.25, 1.5, 2.0, 4.0, and amplitude ratios of lateral load D_y/D_z equal to 0.75, 0.375, 0.075, 0.0375, respectively.

It can be seen that the phasing also has significant influence on the hysteretic loops. Different phasings result in different loops, even when the biaxial period ratios and the amplitude ratios are the same.

The loops in Y direction are more sensitive to the phase angle. The loops in Z direction are less so, since the strength and stiffness in this direction are larger compared with those in Y direction.

2.5 Nonlinear Time History Analysis of A Model RC Columns Under Combined Excitations of Both Horizontal and Vertical Ground Motions

In this part, the nonlinear time history analysis of a model reinforced concrete model bridge under three dimensional seismic excitation is performed. The model bridge is idealized as a single column with a mass at the top simulating the dead load of deck(see reference 11). The bottom of column is fixed on the ground. The column has a height of 2.55m and a rectangular cross section of 52cm by 42cm. Longitudinal reinforcing bars are arranged by 5 reinforcing bars with diameter of 16mm(D16) in one direction and 3 reinforcing bars with diameter of 16mm(D16) in the other direction. The tie reinforcement is arranged by 17.3cm pitch by reinforcing bars with diameter of 10mm(D10). The average yielding stress of the longitudinal and the tie reinforcement is 3,608kgf/cm/cm for D16, 3,552kgf/cm/cm for D10. The average compression strength of the concrete is 335kgf/cm/cm. The weight of the superstructure supported by the column is 49.1tf. Axial stress by the dead weight of the superstructure is 9.2kgf/cm/cm. Fig. 2.5.1 shows some details of the model bridge column. Fig. 2.5.2 gives the material properties of the column.

In this analysis, the acceleration records triggered at Higashi Kobe Bridge during the Hyogo-ken Nanbu Earthquake are used as input motion. The peak acceleration is scaled to be 0.6G and 0.8G respectively in the two seismic load cases. Fig. 2.5.3 through Fig. 2.5.5 show the unscaled input acceleration histories in X, Y, and Z directions. The damping ratio is equal to 5% for the first two natural vibration modes.

To demonstrate the relative effect of vertical ground motion, two cases are studied for the model bridge: (1) only horizontal ground motion is considered; (2) both horizontal and vertical motions are considered.

Fig. 2.5.6 through Fig. 2.5.9 show the acceleration histories at top point of the model in X and Y directions under horizontal input and combined input of both horizontal and vertical ground motions with peak ground acceleration equal to 0.6G. Fig. 2.5.10 through Fig. 2.5.13 show the displacement histories at top point of the model in X and Y directions under horizontal input and combined input of both horizontal and vertical ground motions with peak ground acceleration equal to 0.6G. Fig. 2.5.14 through Fig. 2.5.17 show the hysteresis histories in terms of moment versus curvature at the base of the model in X and Y directions under horizontal input and combined input of both horizontal and vertical ground motions with peak ground acceleration equal to 0.6G.

Fig. 2.5.18 through Fig. 2.5.21 show the acceleration histories at top point of the model in X and Y directions under horizontal input and combined input of both horizontal and vertical ground motions with peak ground acceleration equal to 0.8G. Fig. 2.5.22 through Fig. 2.5.25 show the displacement histories at top point of the model in X and Y directions under horizontal input and combined input of both horizontal and vertical ground motions with peak ground acceleration equal to 0.8G. Fig. 2.5.26 through Fig. 2.5.29 show the moment-curvature hysteresis loops at the base of the model in X and Y directions under horizontal input and combined input of both horizontal and vertical ground motions with peak ground acceleration equal to 0.8G.

From these figures, it can be seen that the vertical ground motion does have certain effect on the response of the model bridge. Usually, the response acceleration and displacement tend to be larger under excitation of combined input than those under only horizontal excitation. The shapes of the response histories and the moment-curvature hysteresis curves are also different for the two cases.

2.6 Conclusions

The purpose of this research is to study in detail the dynamic performance of RC columns under excitation of earthquake ground motions and special attention was focused on the effects of vertical component on the response of the RC columns. The three dimensional characteristics and hysteretic behavior of RC bridge piers were investigated for various cyclic loading by means of finite element modeling. Nonlinear numerical analysis was performed by utilizing a kind of fiber element, which can monitor the details of damage of RC columns.

The analysis revealed some important phenomena which are totally unusual compared with what is commonly observed with the RC columns. The fluctuation in axial force and biaxial bending can significantly affect the inelastic cyclic response of RC columns. The analysis also indicates that three dimensional loading results in reduction in strength and flexural stiffness as well as the energy dissipation capacity of RC column, compared to the corresponding uniaxial response. Lack of consideration of these factors may lead to significantly different response in bridges, in cases where three dimensional effects dominate the behavior of RC columns. The simplified phenomenological nonlinear models are not suitable to obtain reasonable results when RC columns are subjected to three dimensional excitation.

The research showed that severe pinching of hysteretic loops of RC column is evident under three dimensional cyclic loading. In some cases, this effects are so drastic that the area enclosed by hysteretic curves is almost zero or even negative, which means that the energy dissipation capacity is zero or negative. In general, this low energy dissipation capacity should be avoided in the design of RC columns, because high level of energy dissipation capacity is desirable, where the control of displacement is important during the earthquakes.

The nonlinear time history analysis indicates that vertical ground motion affects the general response of the columns. In most cases, the vertical component tends to aggravate the response.

The work finished in this research is only a kind of numerical one for the limited configurations of models. It is therefore suggested that validations against experimental work be done in the future, to provide experimental basis for the updating of the knowledge concerning about three dimensional excitations of RC columns.

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FIGURES

		Kobe (1995) M = 7.2		
		The weather station of Kobe Ocean	The bridge of east Kobe	Port Island
Max. acc. (gal)	Horizontal	818	327	341
	Vertical	332	395	556
Max. Vel. (cm/sec)	Horizontal	90	91	91
	Vertical	40	35	62
Max. Disp (cm)	Horizontal	21	49	40
	Vertical	11	15	28

Fig. 1 Data from Kobe Earthquake

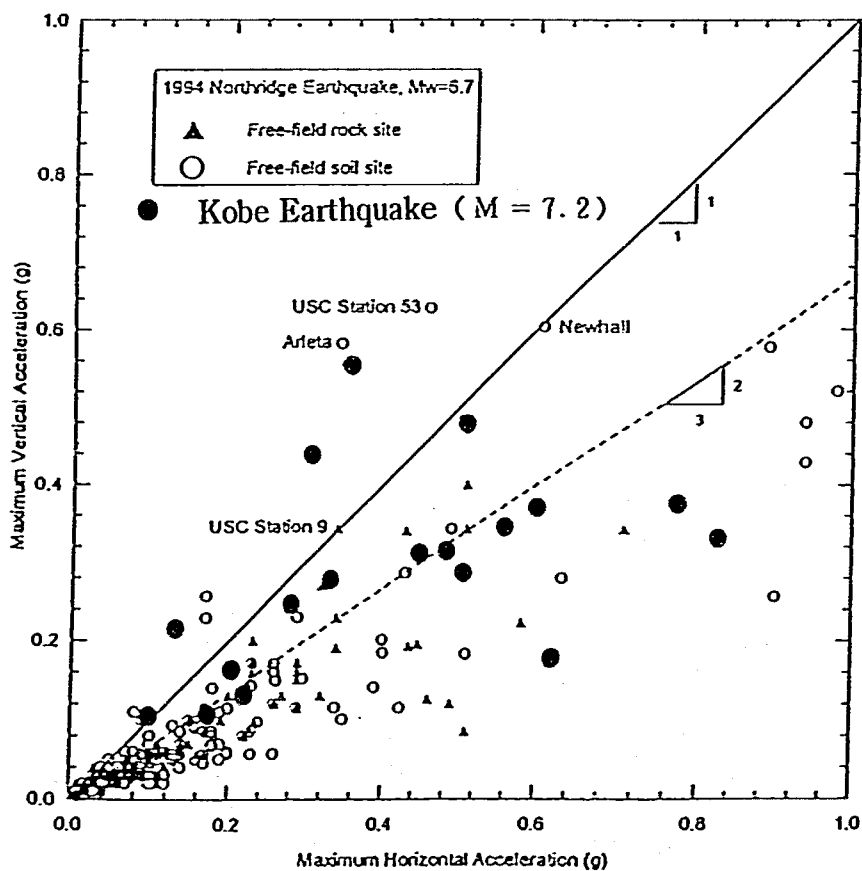


Fig. II Distribution of Maximum Acceleration

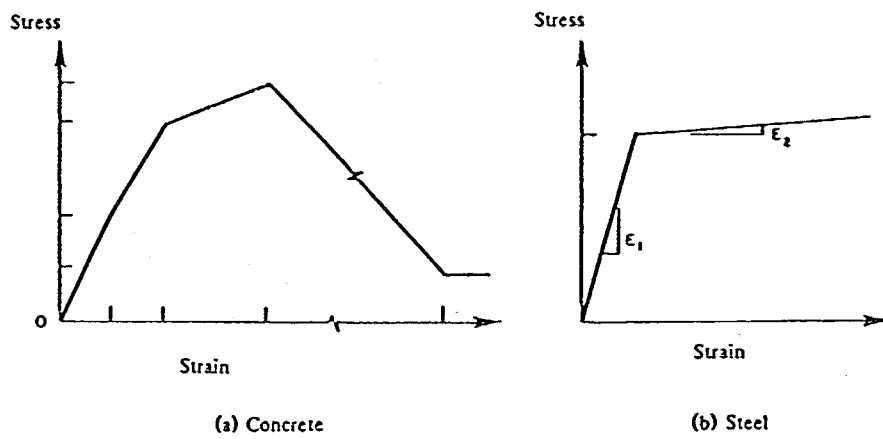


Fig. III Idealized Material Properties

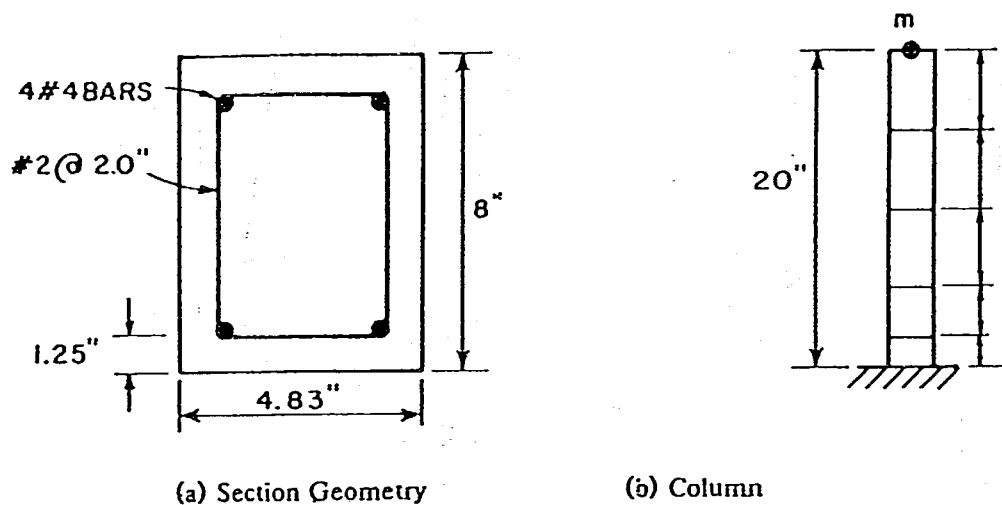
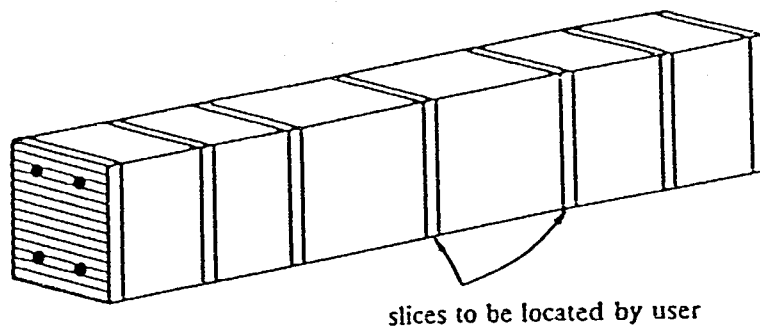


Fig. IV Example Column

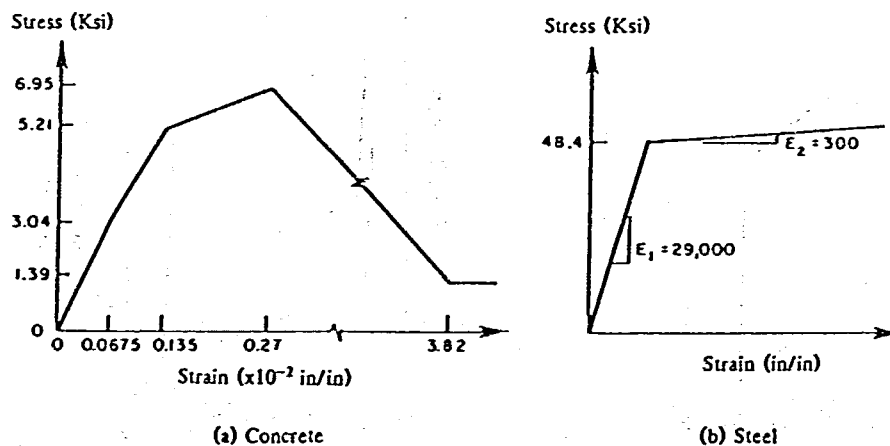


Fig. V Idealized Material Properties of Example Column

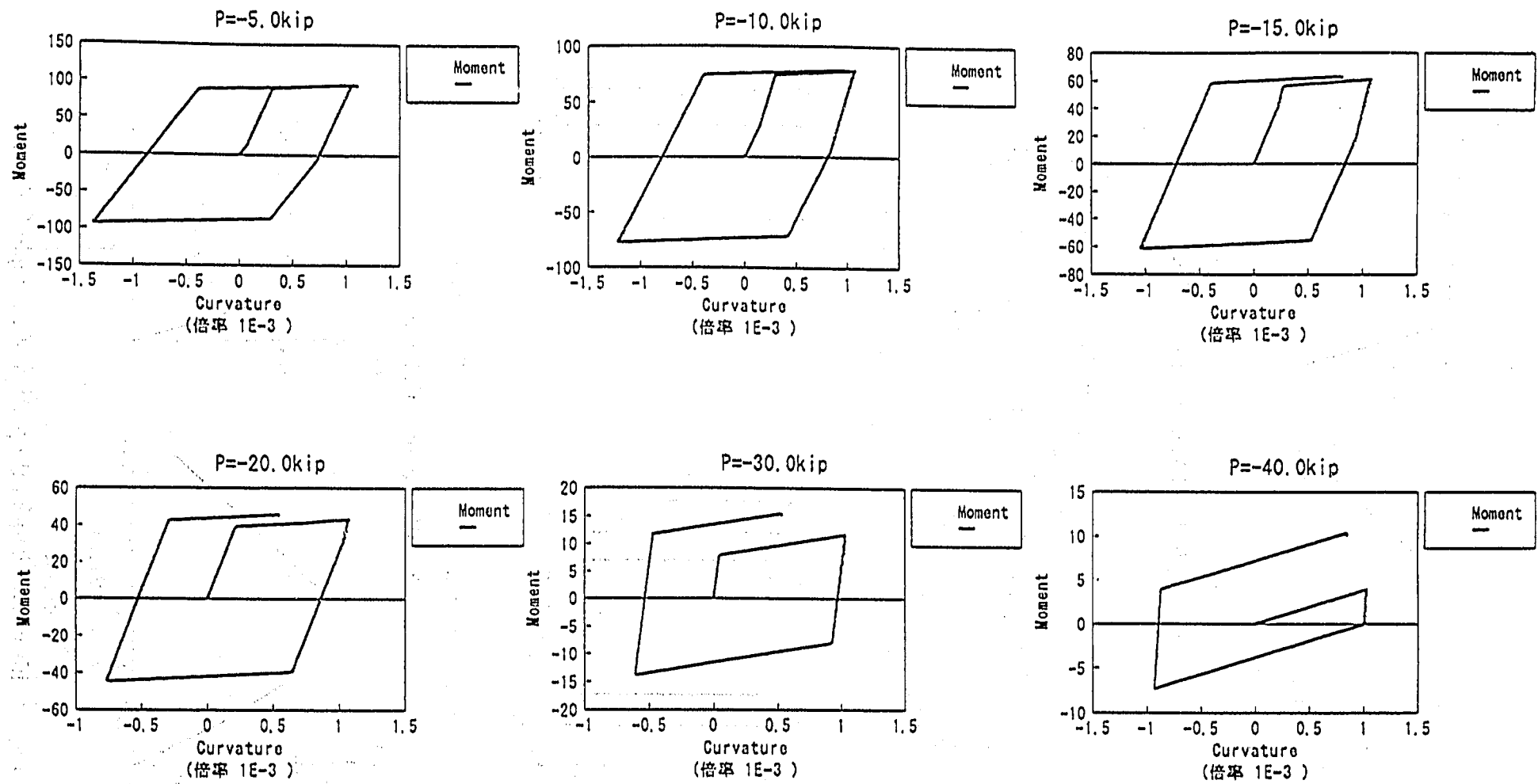


Fig. 1 Base moment-curvature histories under tensile axial load

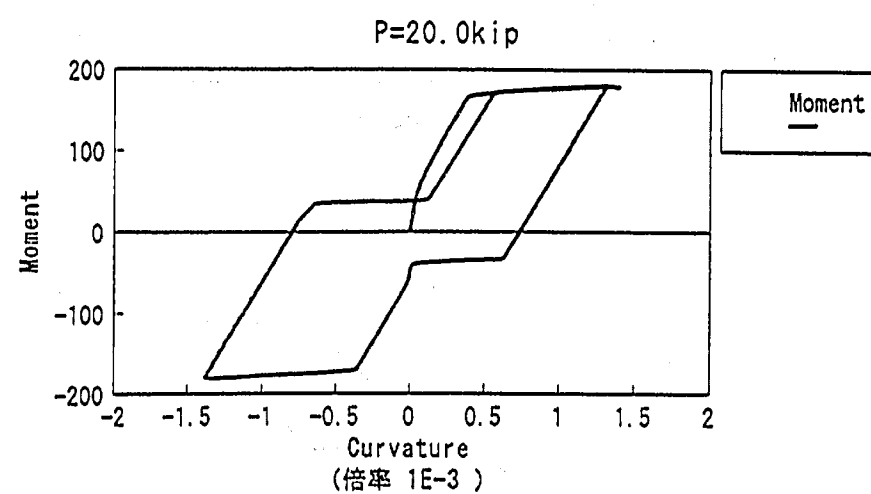
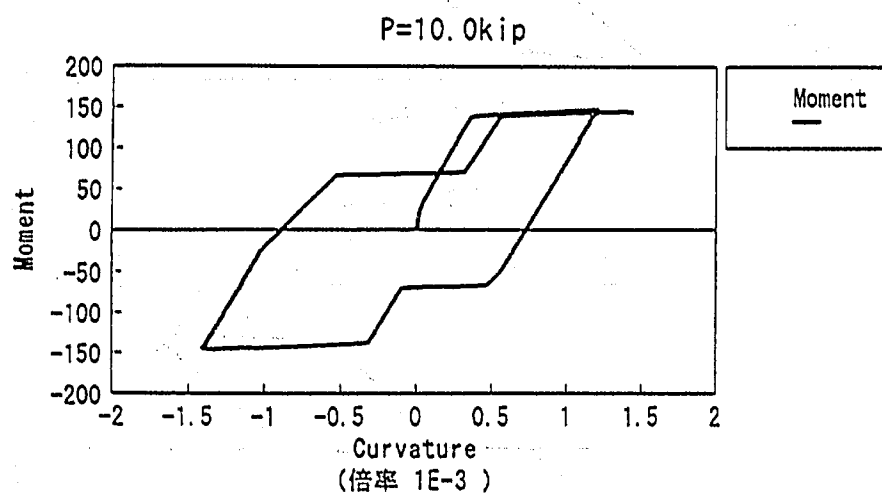
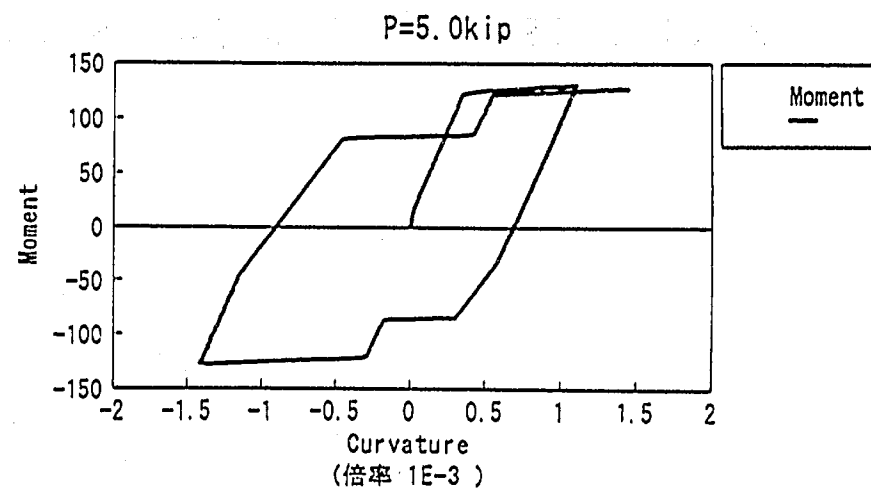
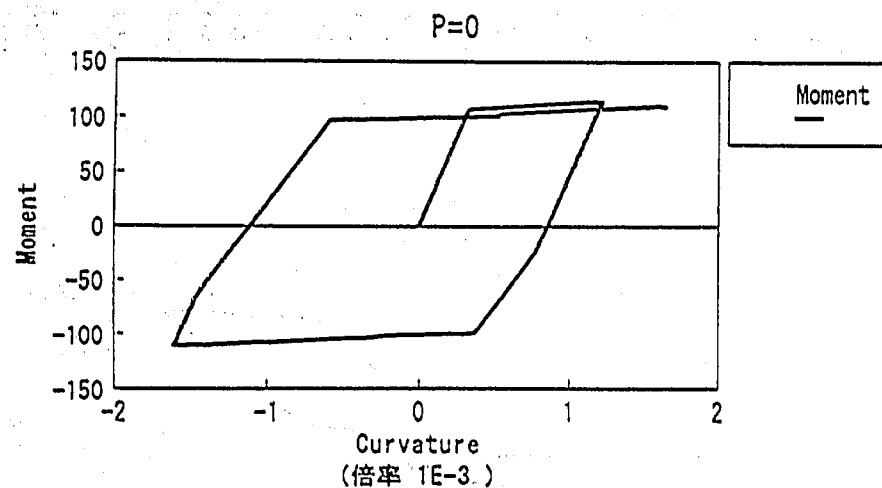


Fig. 2 Base moment-curvature histories under compressive axial load

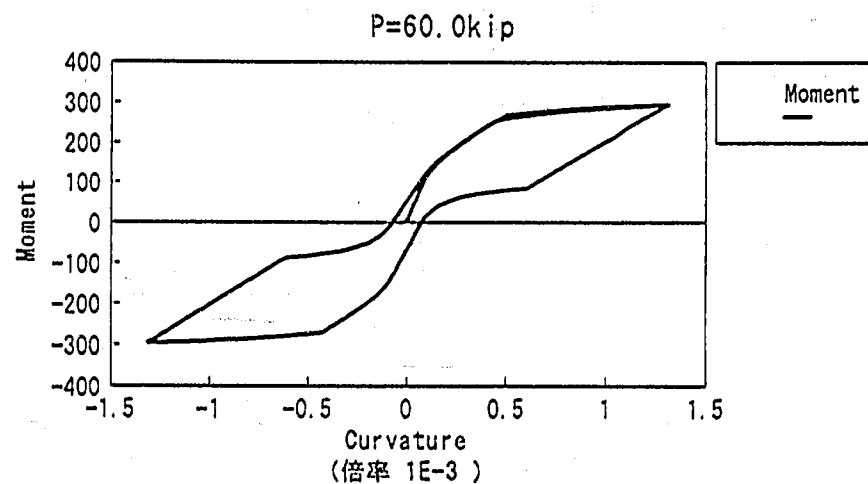
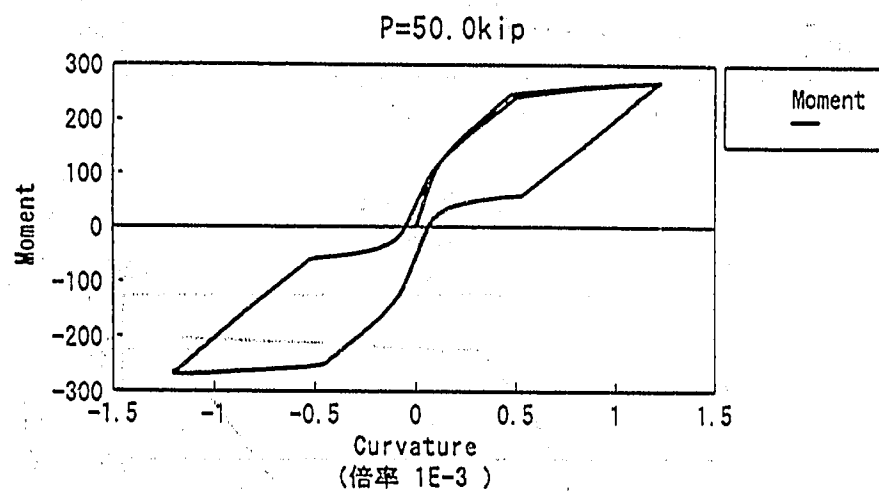
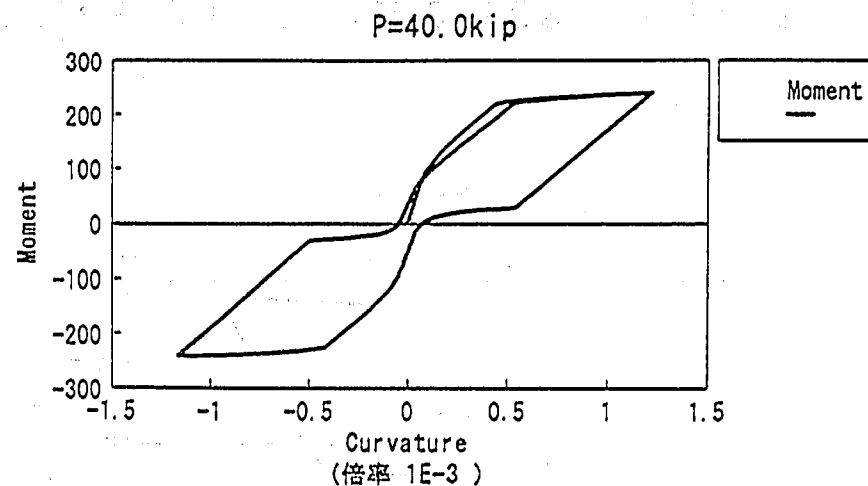
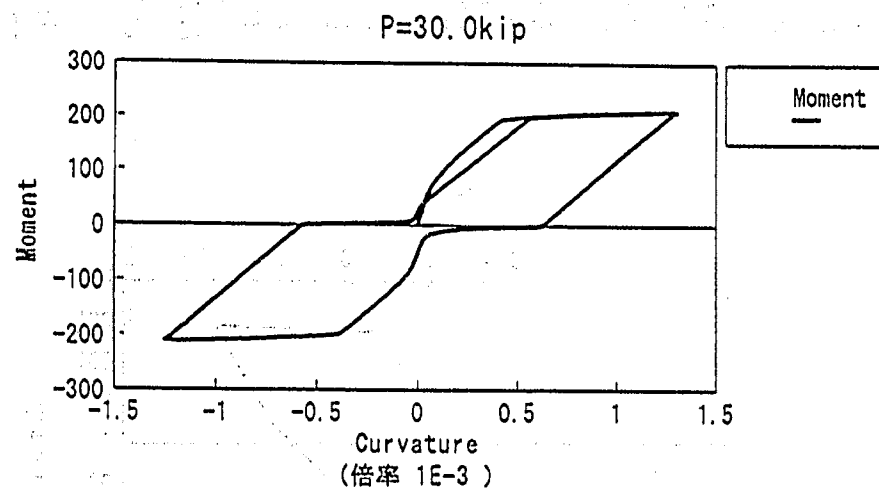


Fig. 3 Base moment-curvature histories under compressive axial load

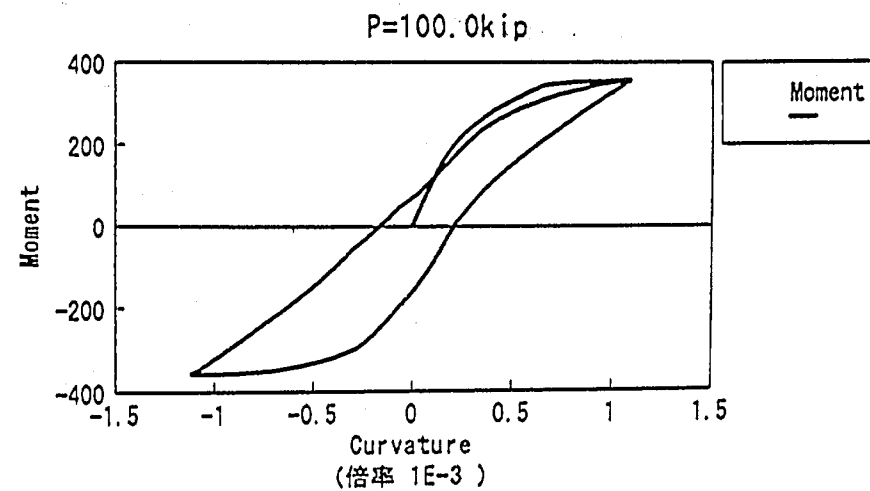
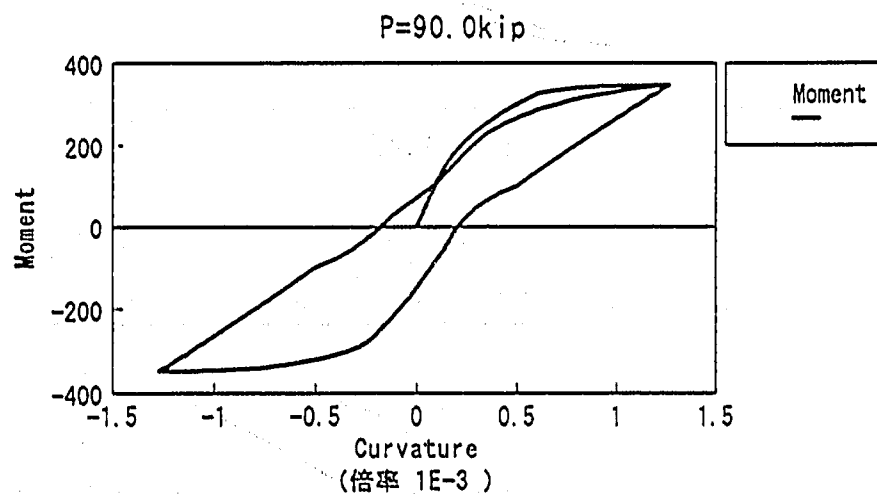
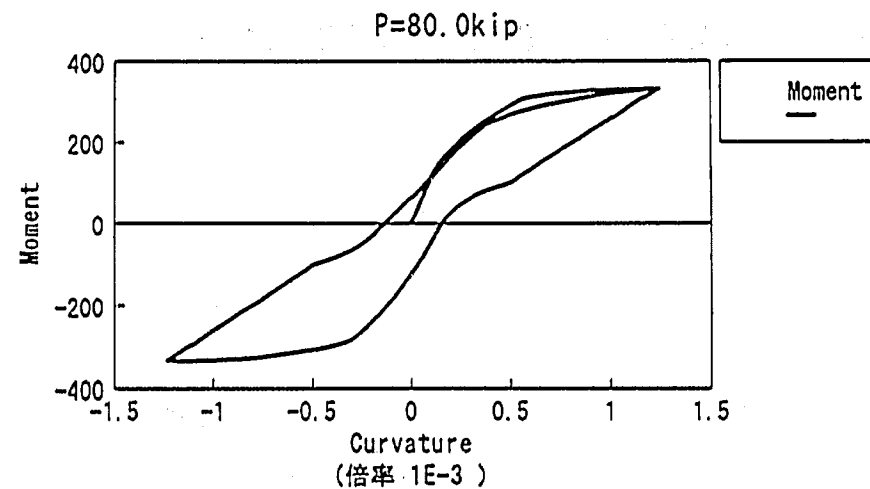
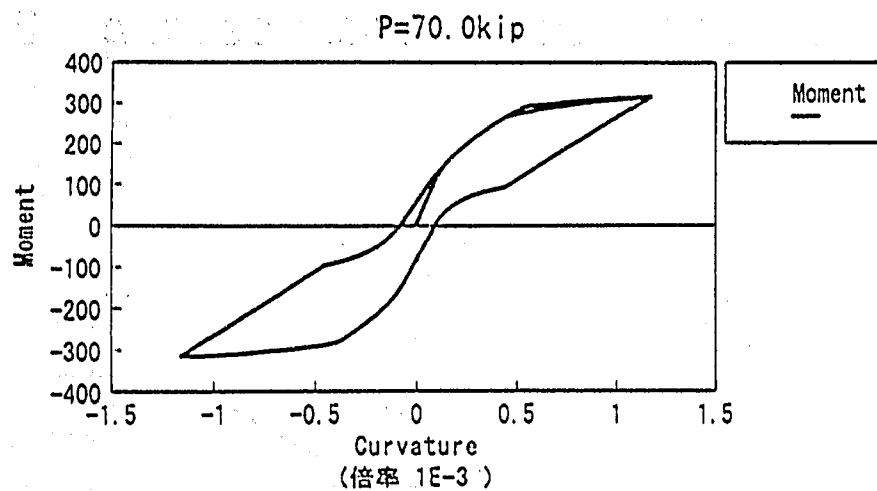


Fig. 4 Base moment-curvature histories under compressive axial load

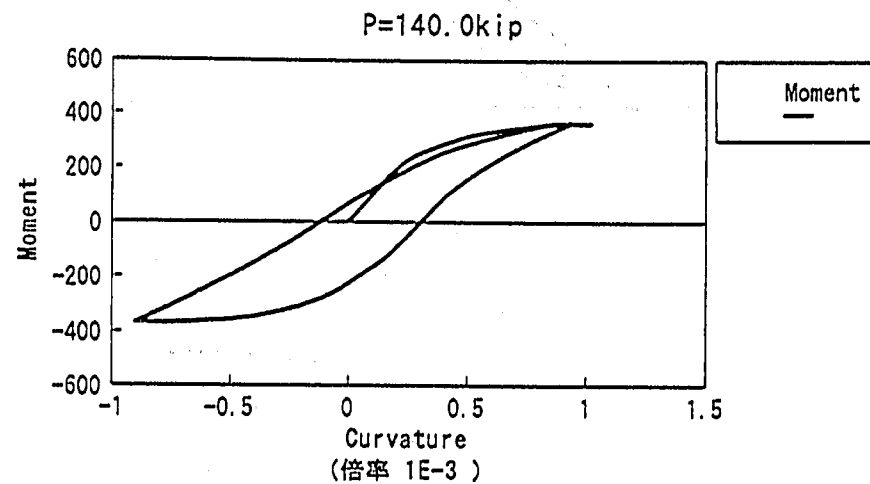
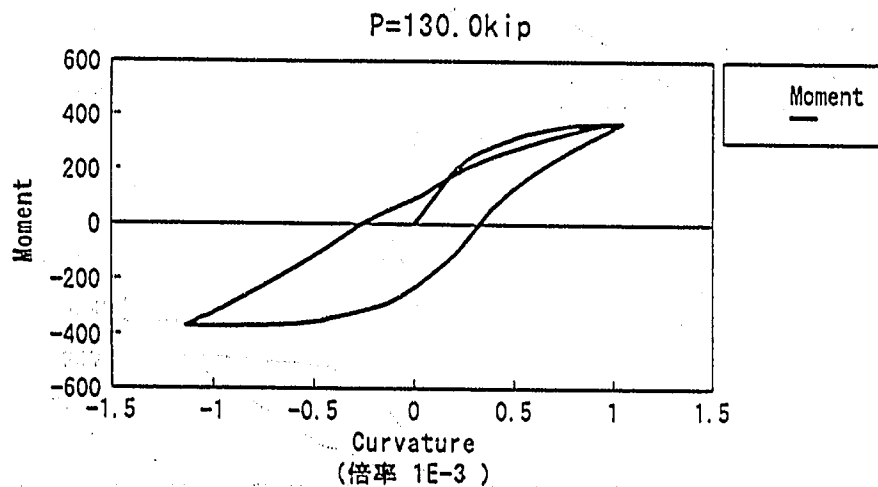
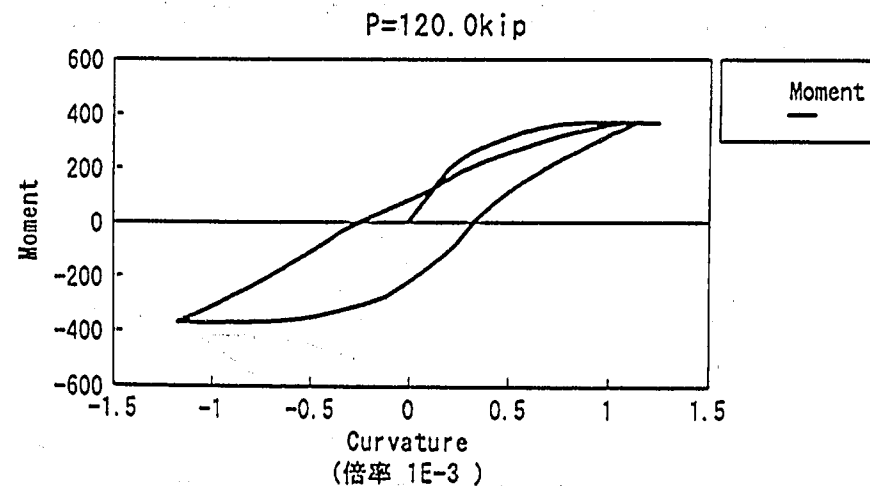
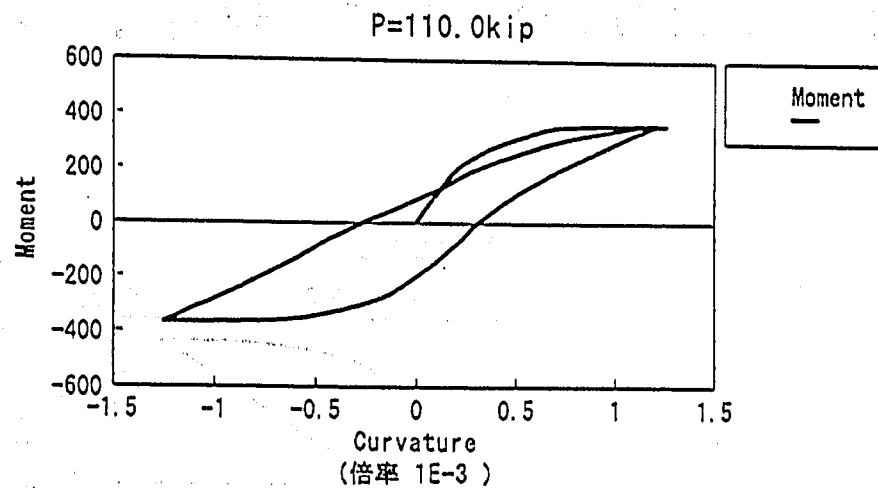


Fig. 5 Base moment-curvature histories under compressive axial load

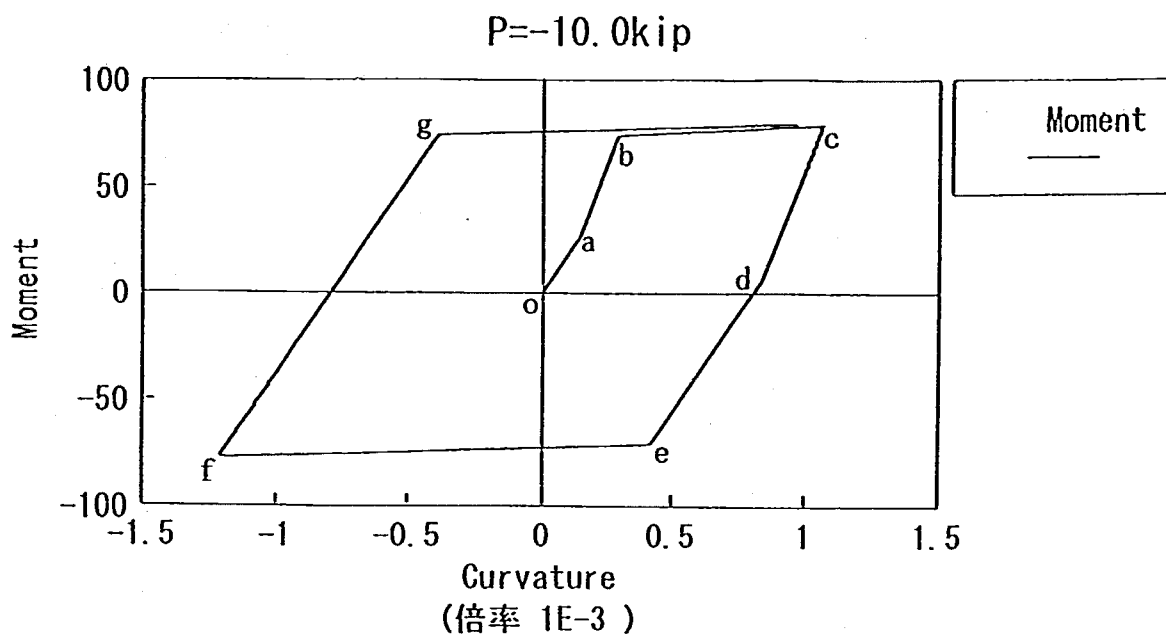


Fig. 6 Typical hysteresis curve under tensile axial load

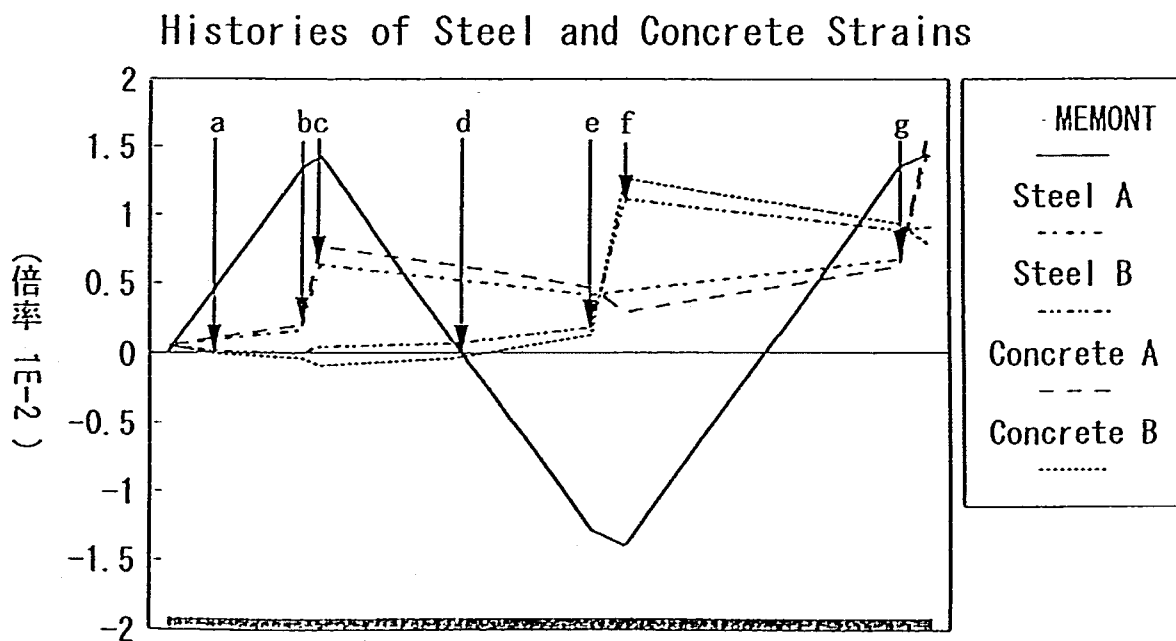


Fig. 7 Histories of steel and concrete strains ($P = -10.0 \text{ kip}$)

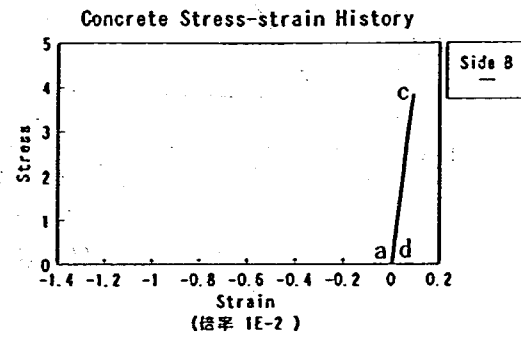
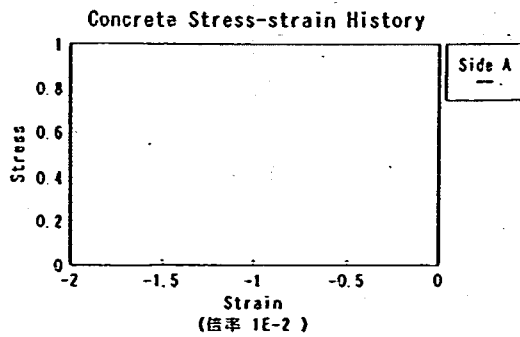
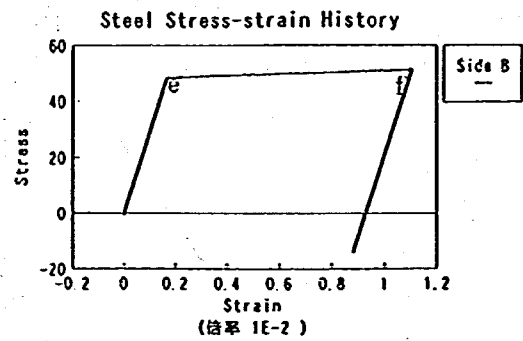
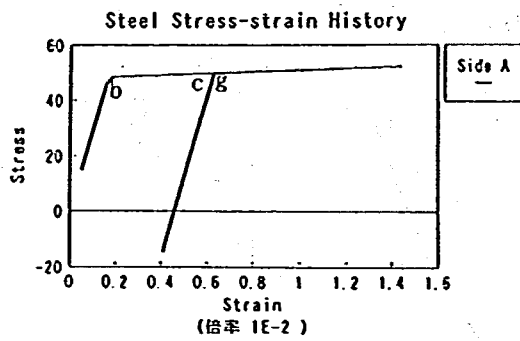


Fig. 8 Stress-strain histories of steel and concrete ($P = -10.0 \text{ kip}$)

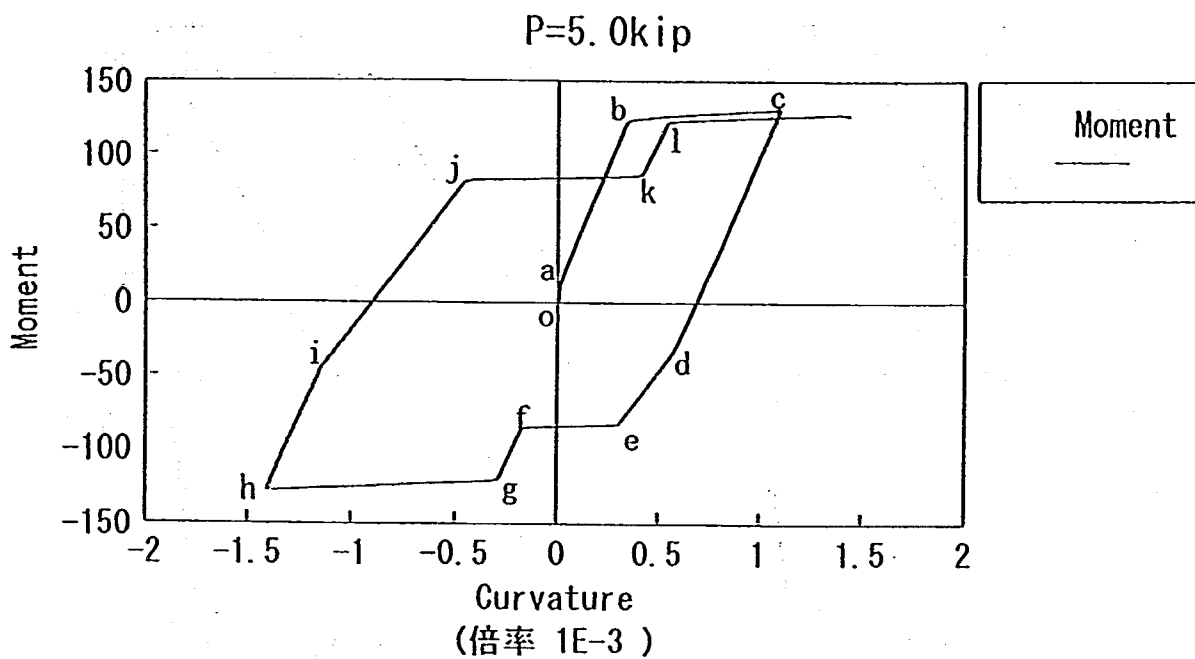


Fig. 9 Typical hysteresis curve under compressive axial load

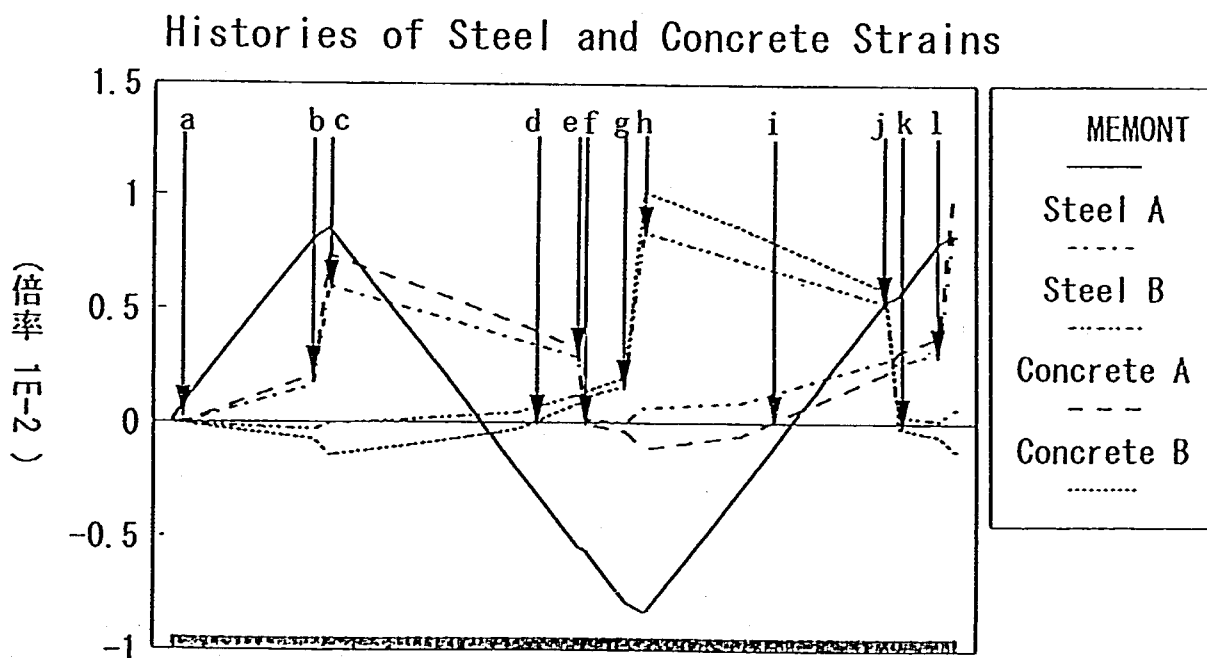


Fig. 10 Histories of steel and concrete strains (P=5.0kip)

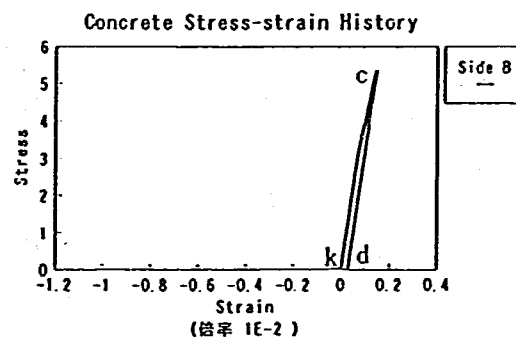
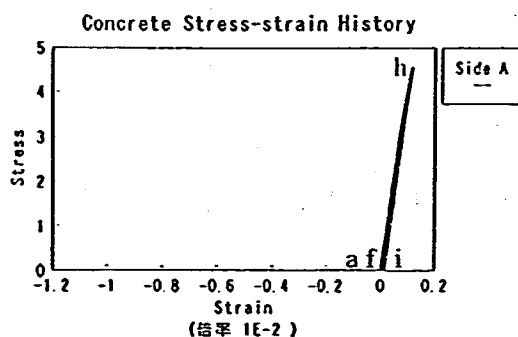
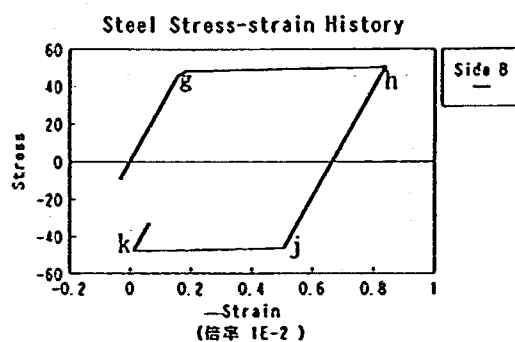
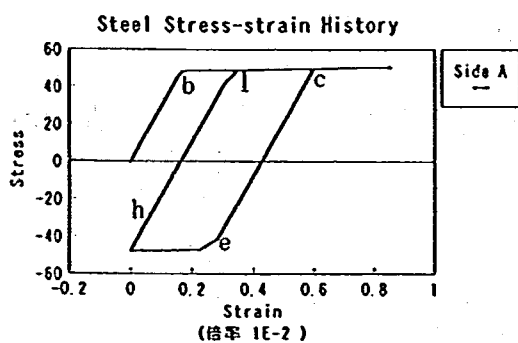
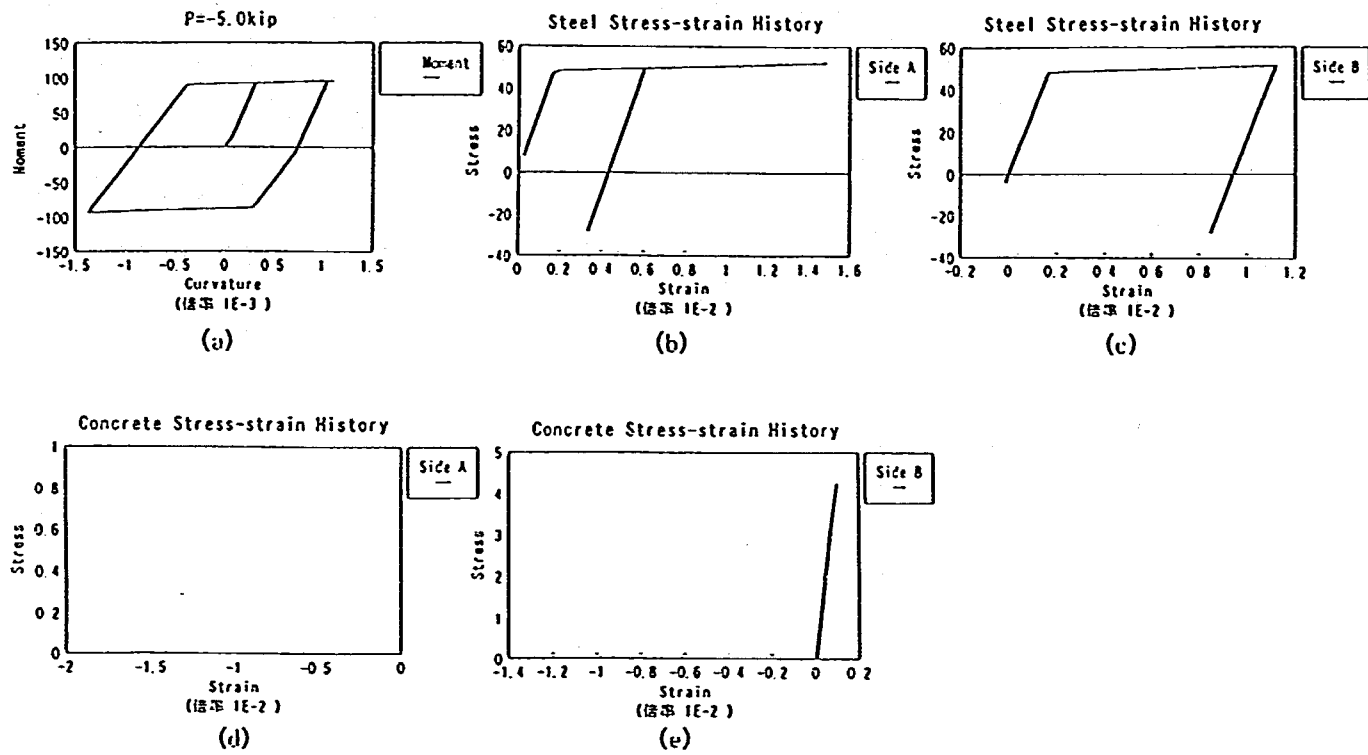


Fig. 11 Stress-strain histories of steel and concrete ($P=5.0\text{kip}$)

$P = -5.0 \text{ kip}$ T11



$P = -5.0 \text{ kip}$ T12

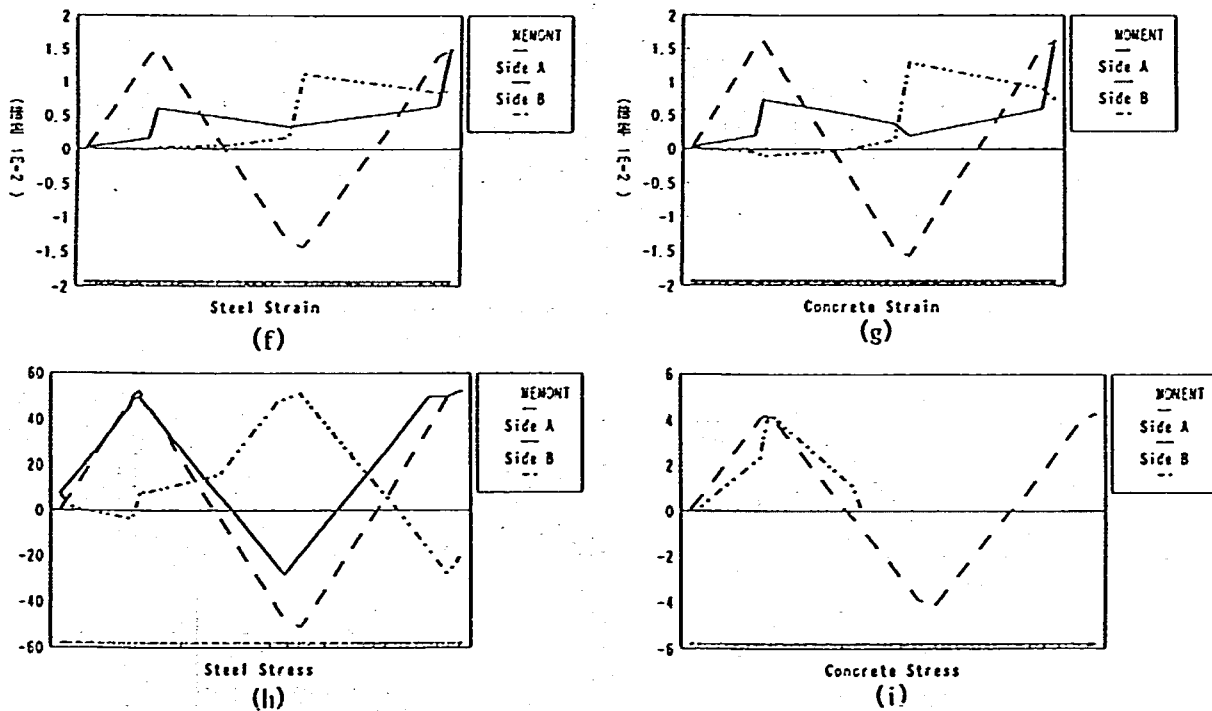
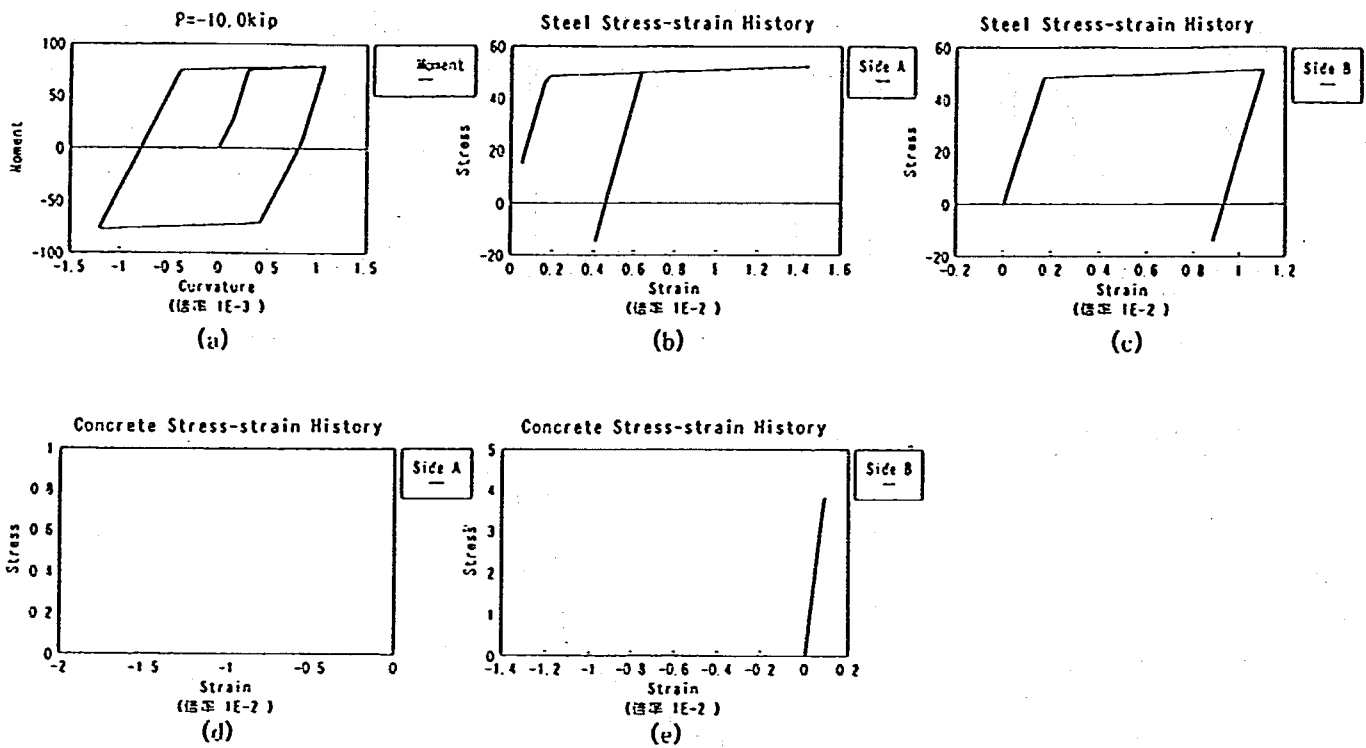


Fig. 12 Plots for axial force $P = -5.0 \text{ kip}$

$P = -10.0 \text{ kip}$ T21



$P = -10.0 \text{ kip}$ T22

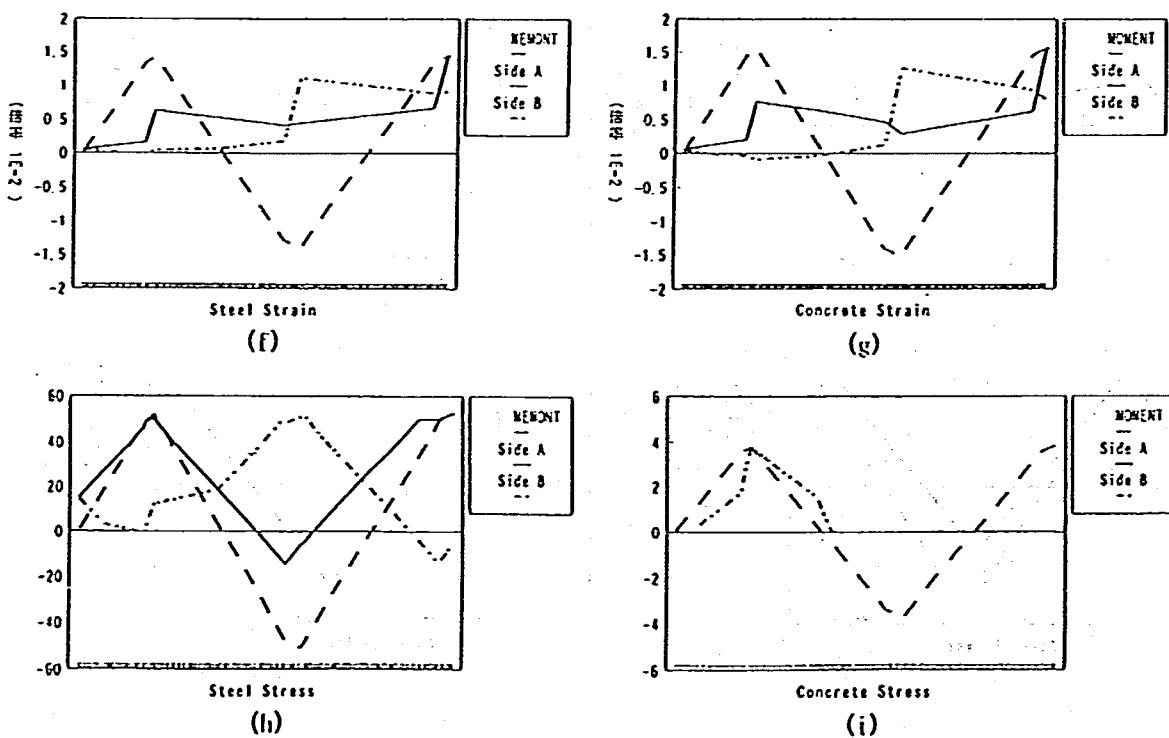
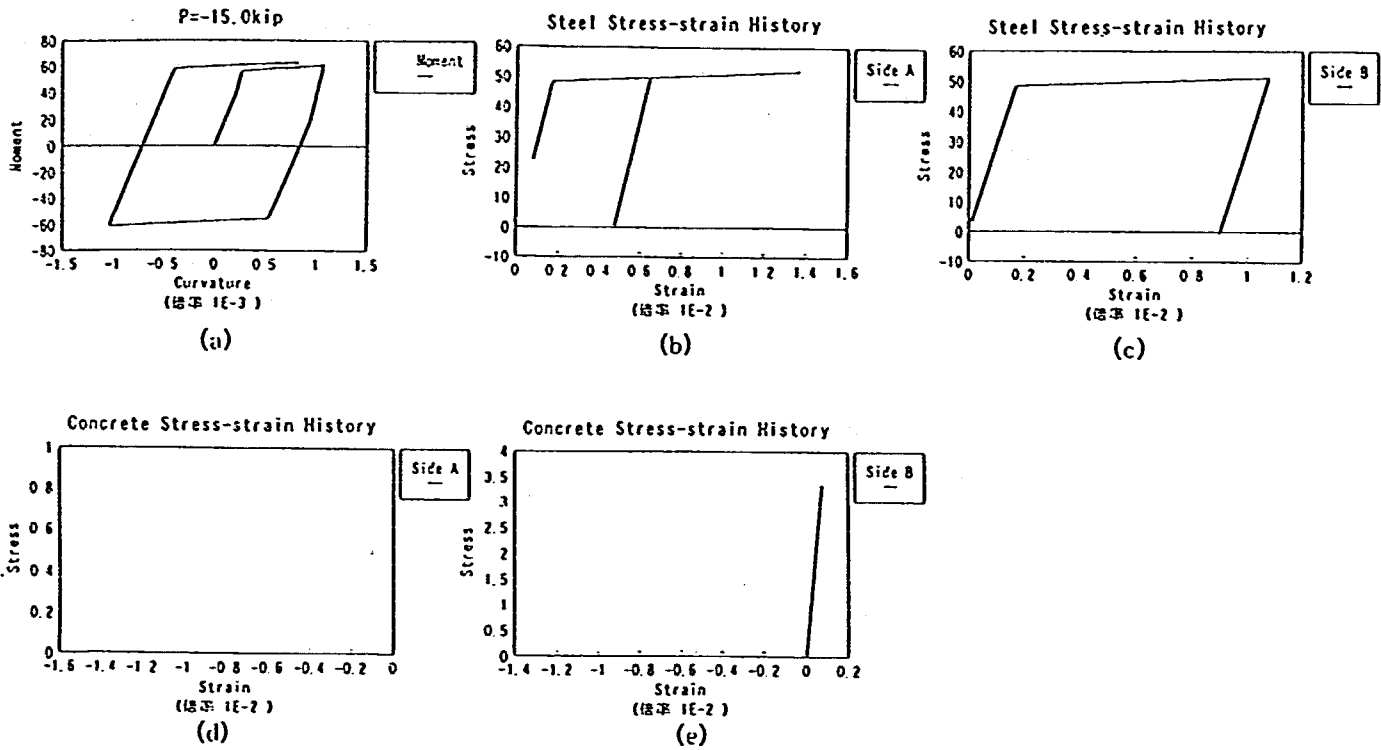


Fig. 13 Plots for axial force $P = -10.0 \text{ kip}$

$P = -15.0 \text{ kip}$ T31



$P = -15.0 \text{ kip}$ T32

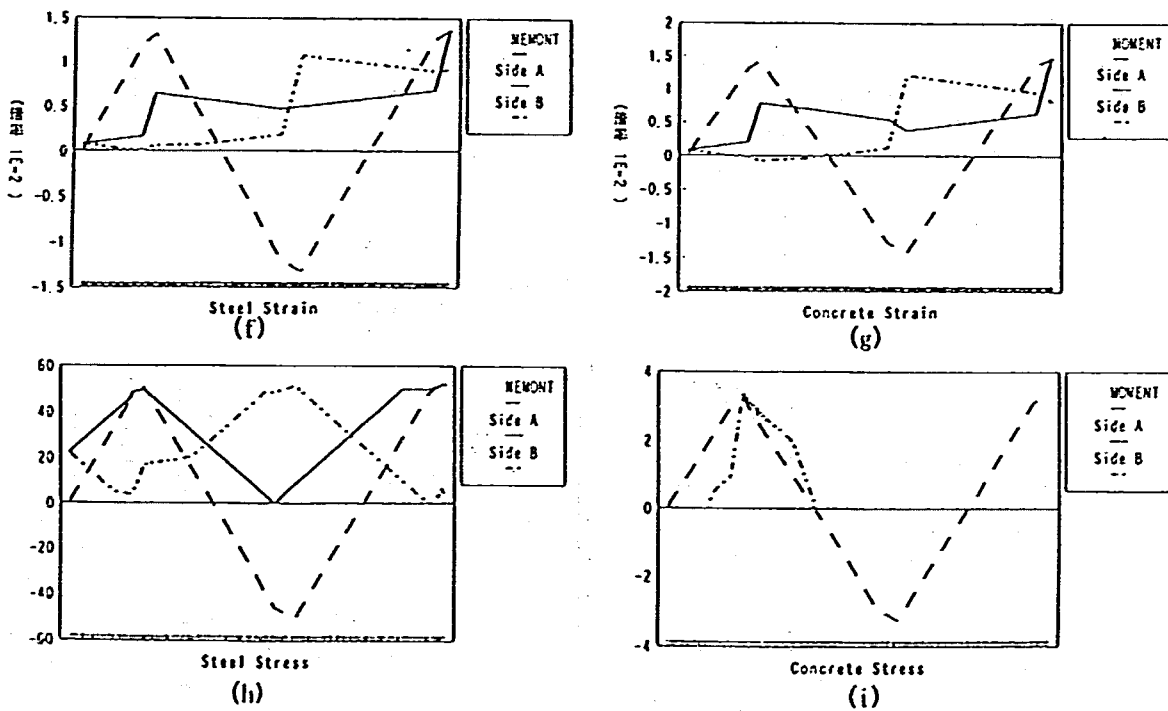
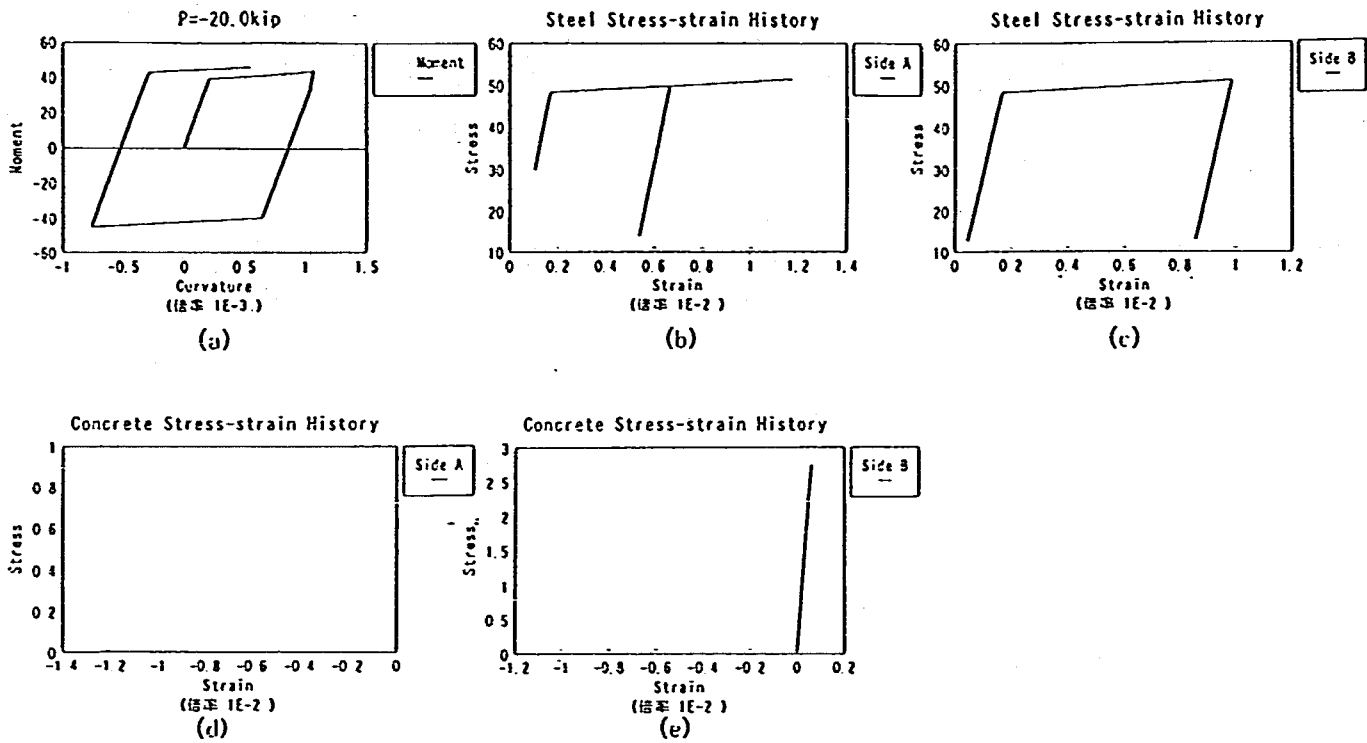


Fig. 14 Plots for axial force $P = -15.0 \text{ kip}$

$P = -20.0 \text{ kip}$ T41



$P = -20.0 \text{ kip}$ T42

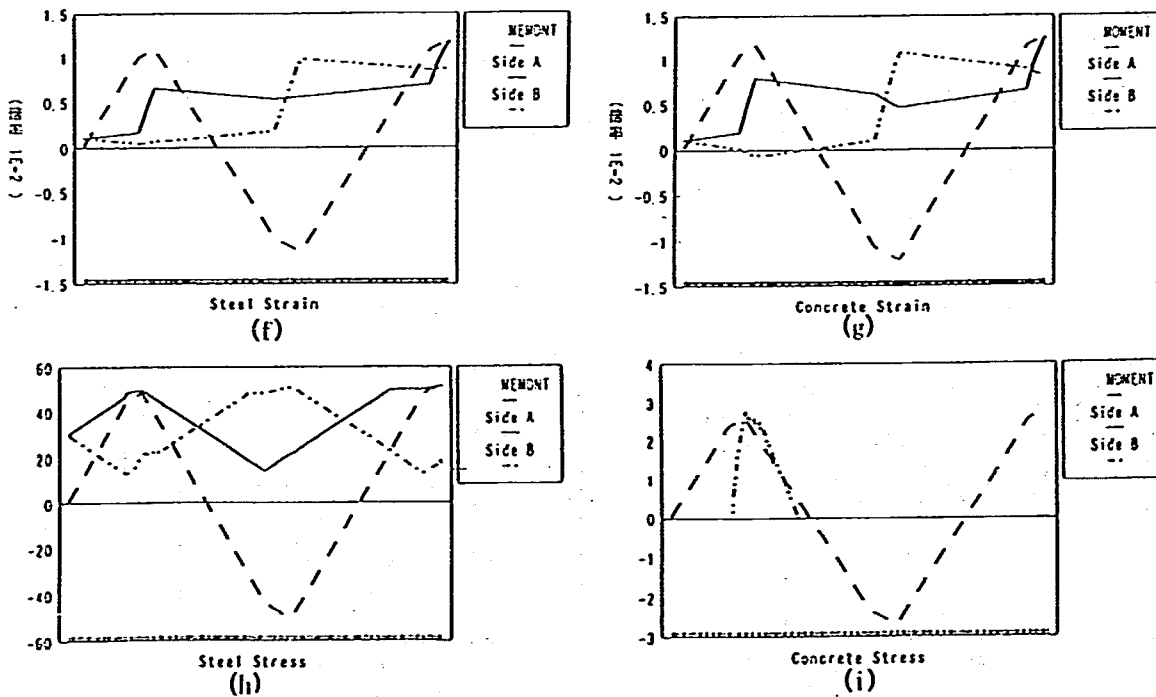
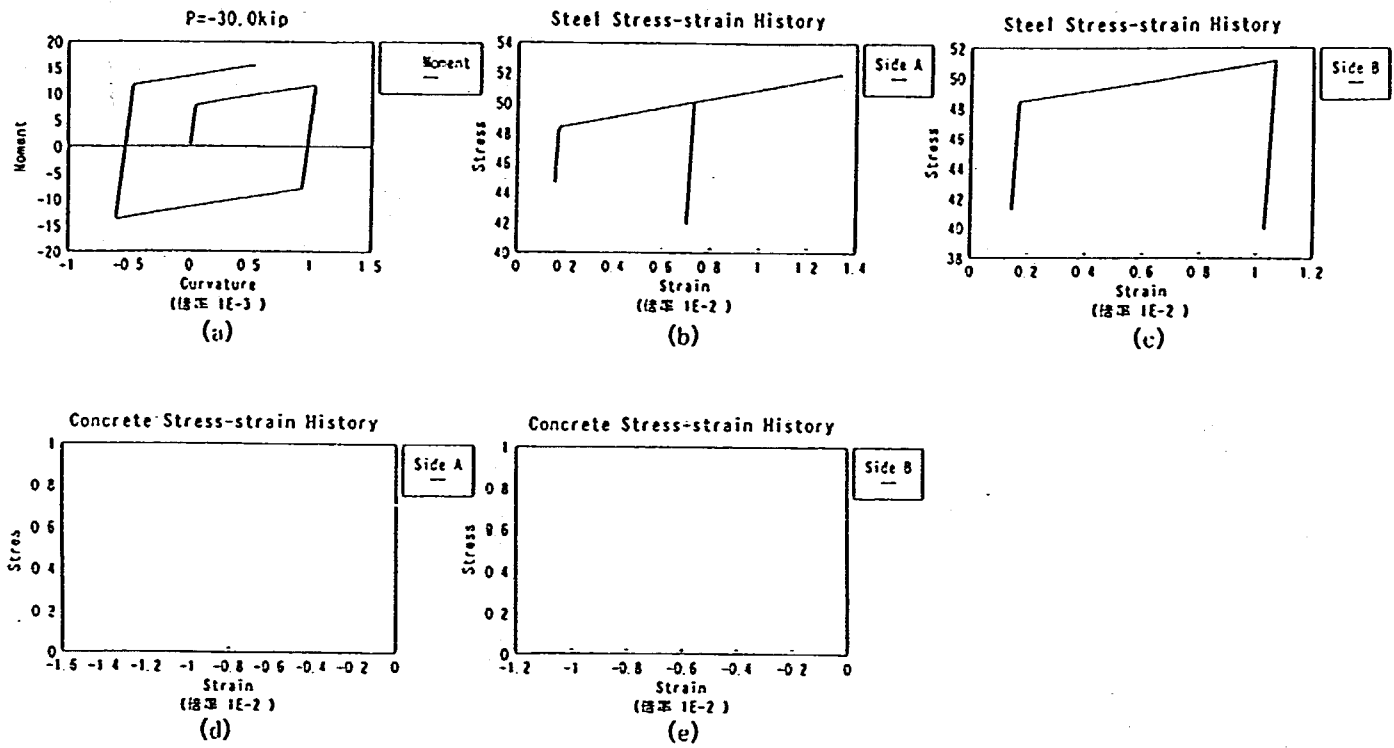


Fig. 15 Plots for axial force $P = -20.0 \text{ kip}$

$P = -30.0 \text{ kip}$ T51



$P = -30.0 \text{ kip}$ T52

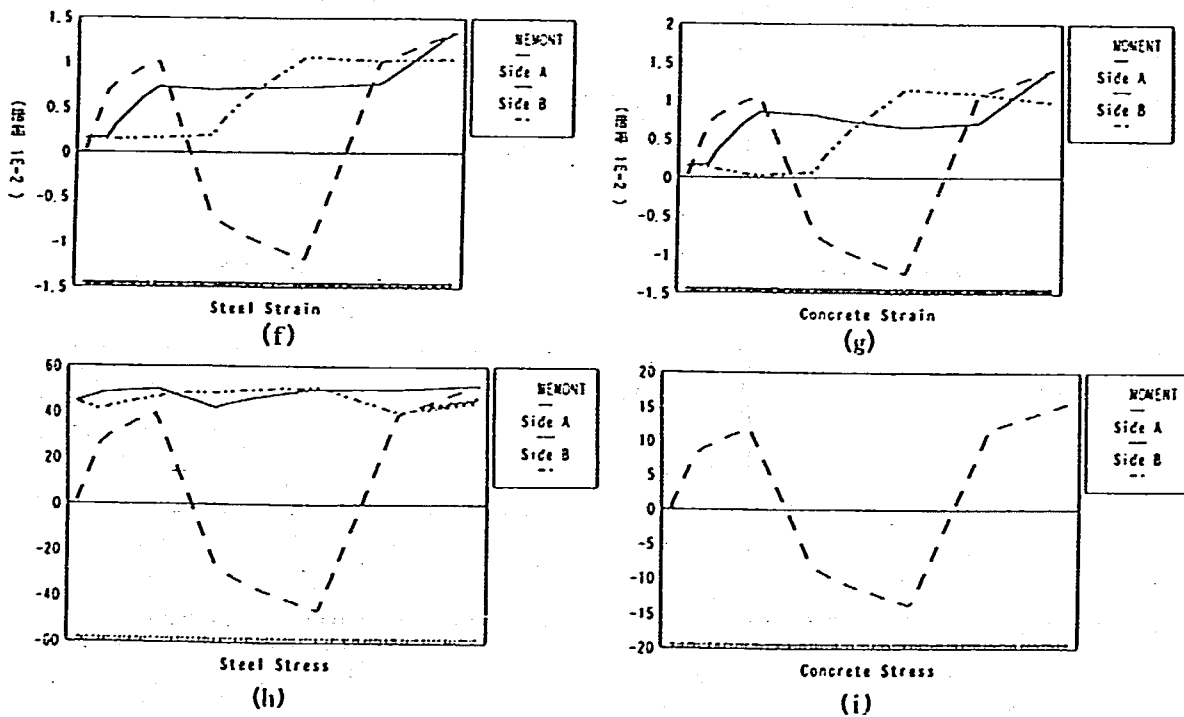
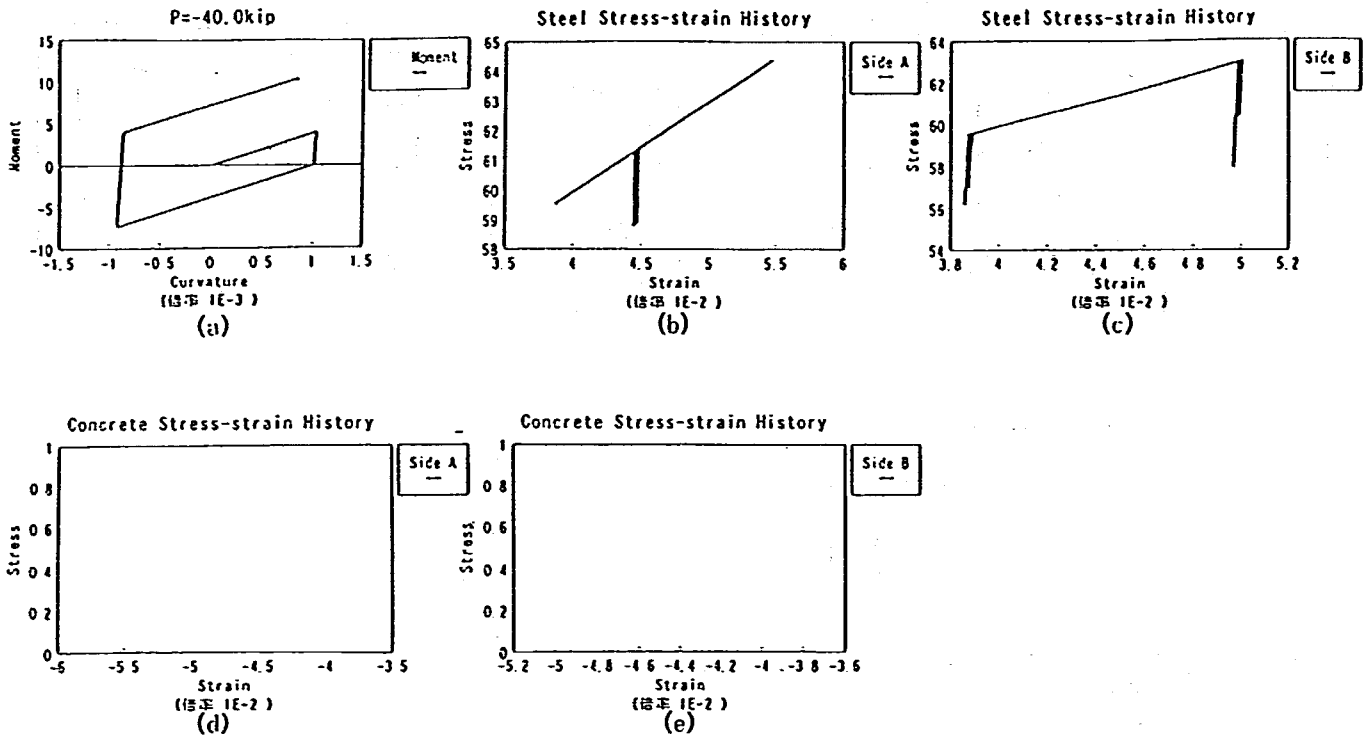


Fig. 16 Plots for axial force $P = -30.0 \text{ kip}$

$P = -40.0 \text{ kip}$ T61



$P = -40.0 \text{ kip}$ T62

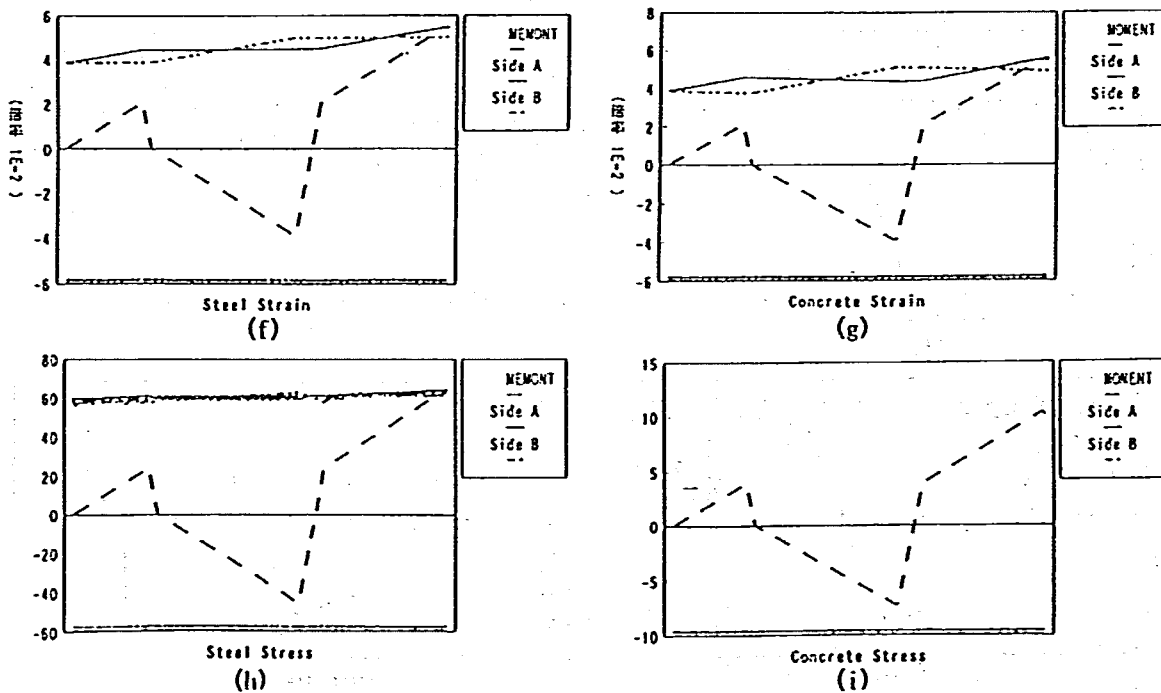
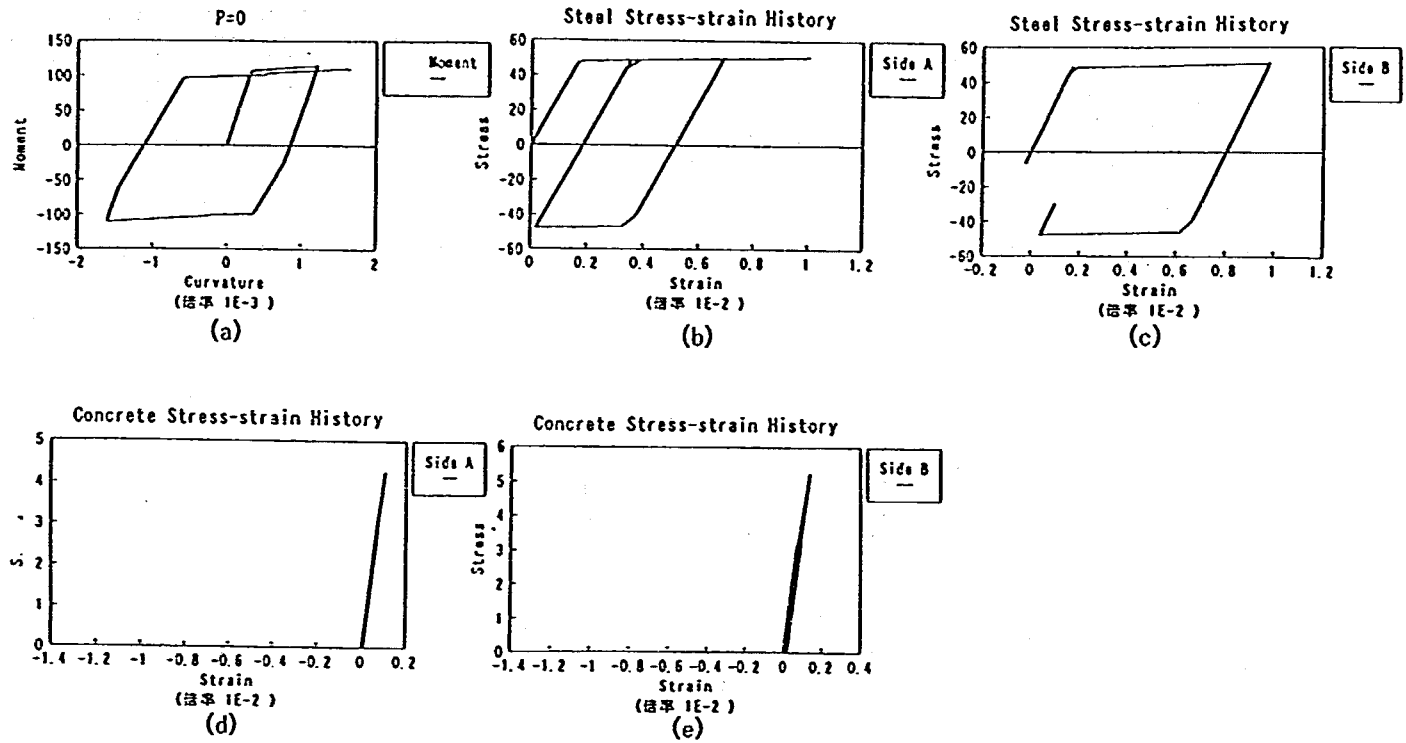


Fig. 17 Plots for axial force $P = -40.0 \text{ kip}$

P=0 C01



P=0 C02

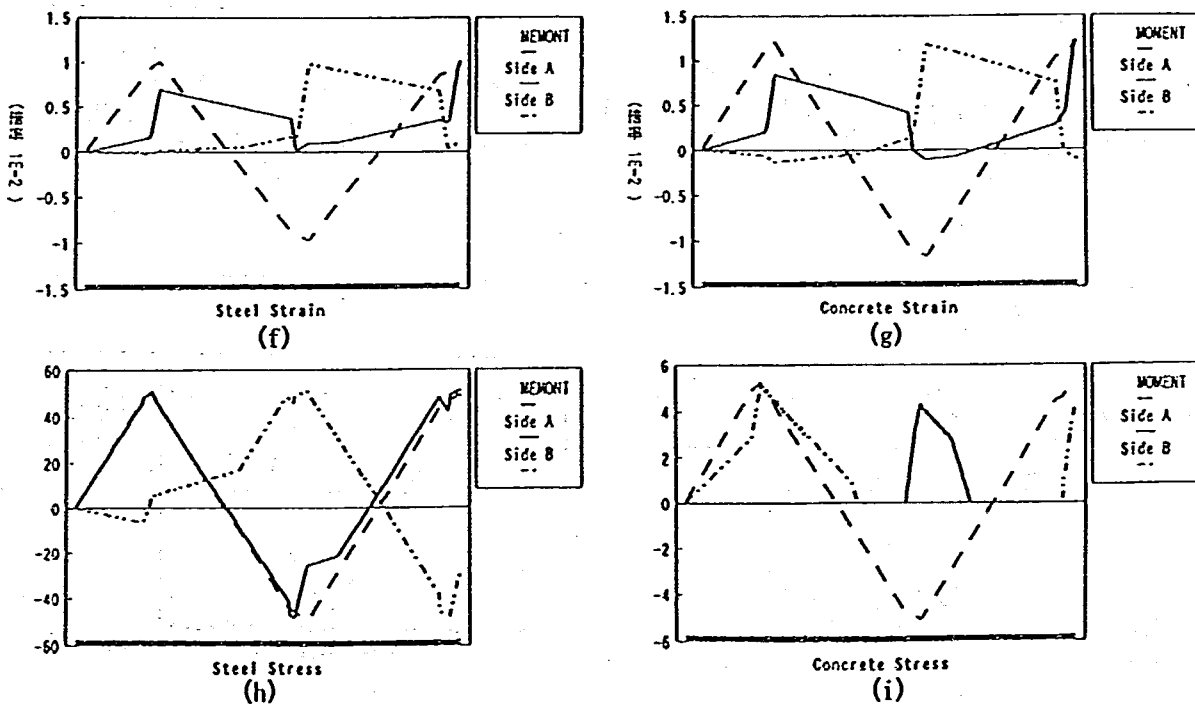
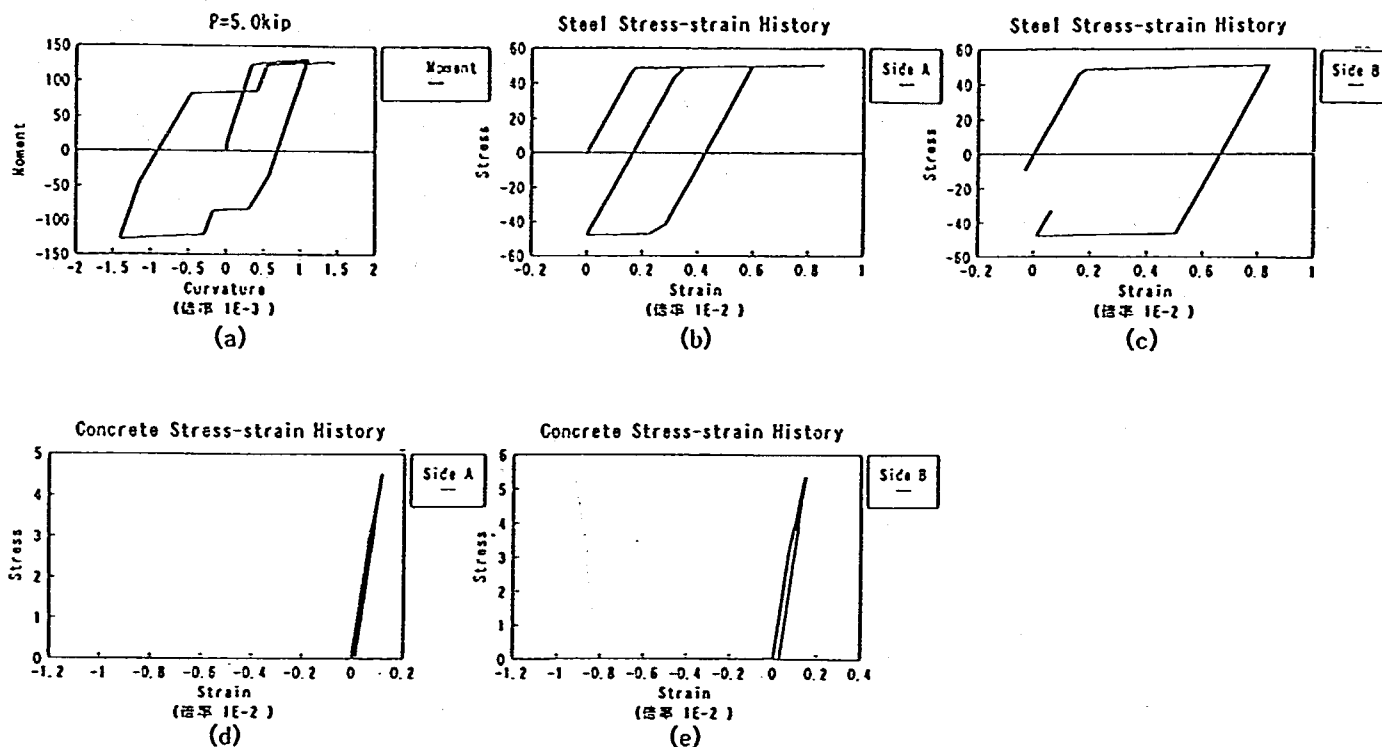


Fig. 18 Plots for axial force P=0

P=5.0kip C21



P=5.0kip C22

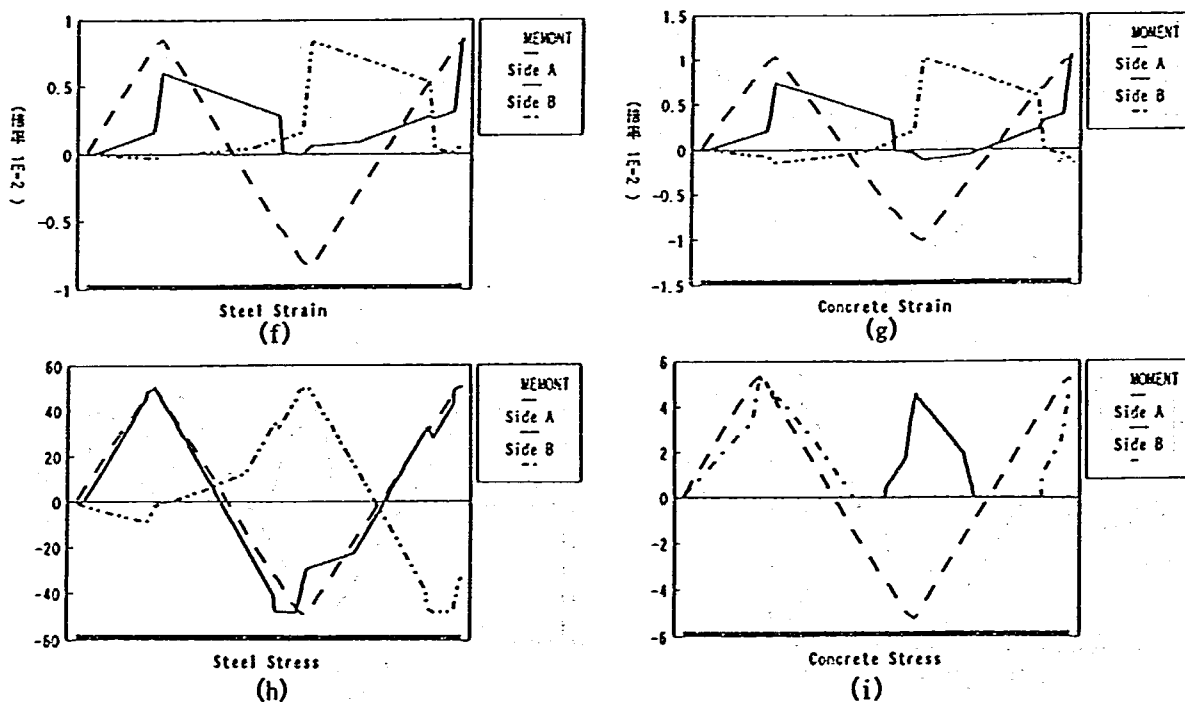
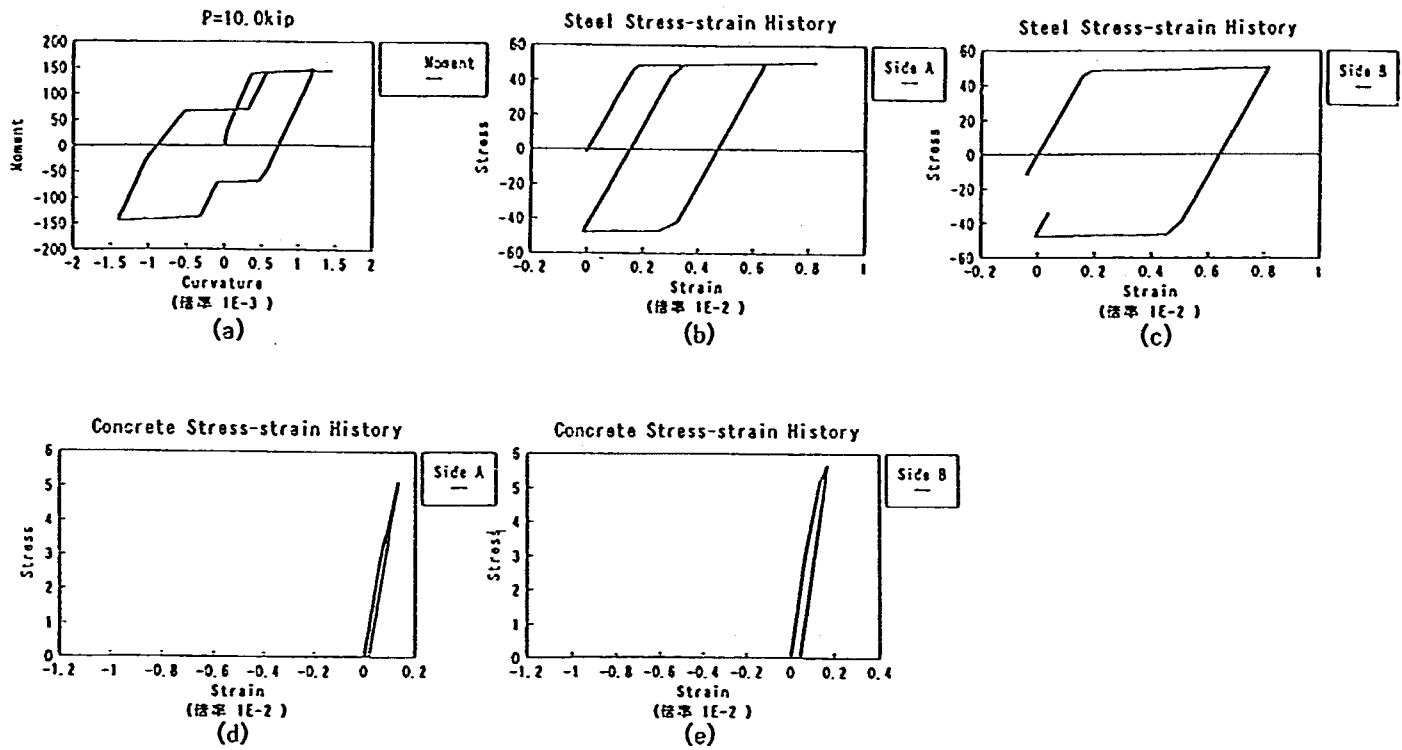


Fig. 19 Plots for axial force P=5.0kip

P=10.0kip C31



P=10.0kip C32

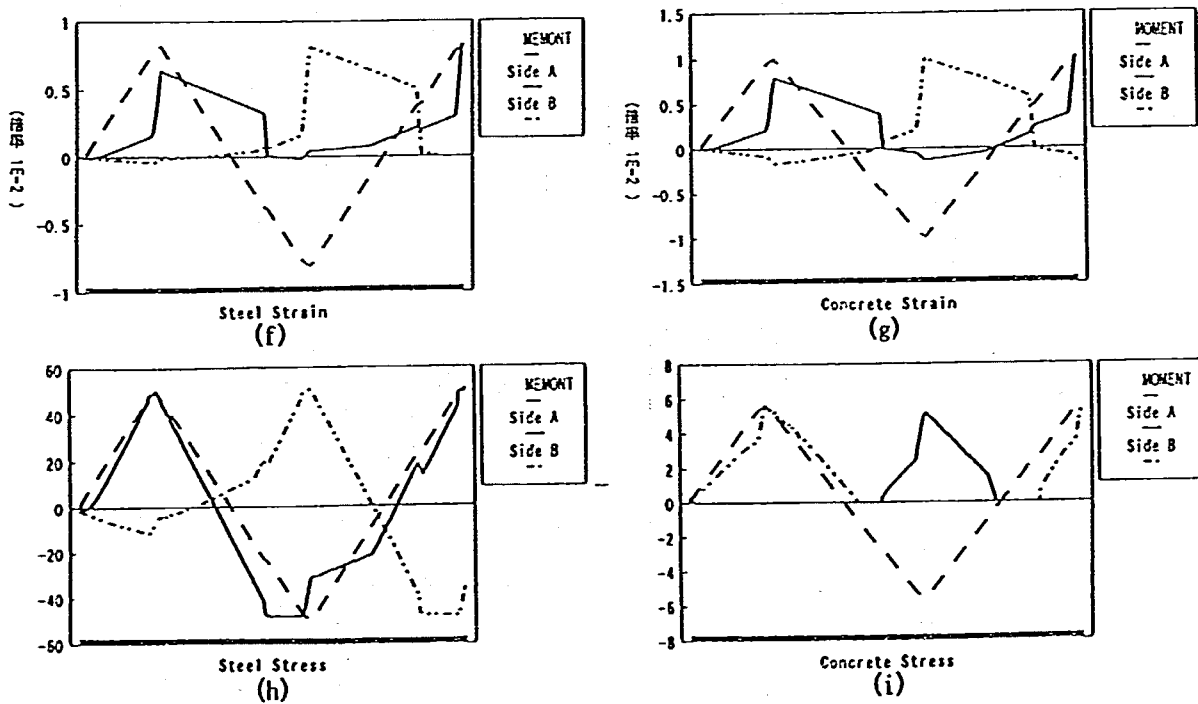
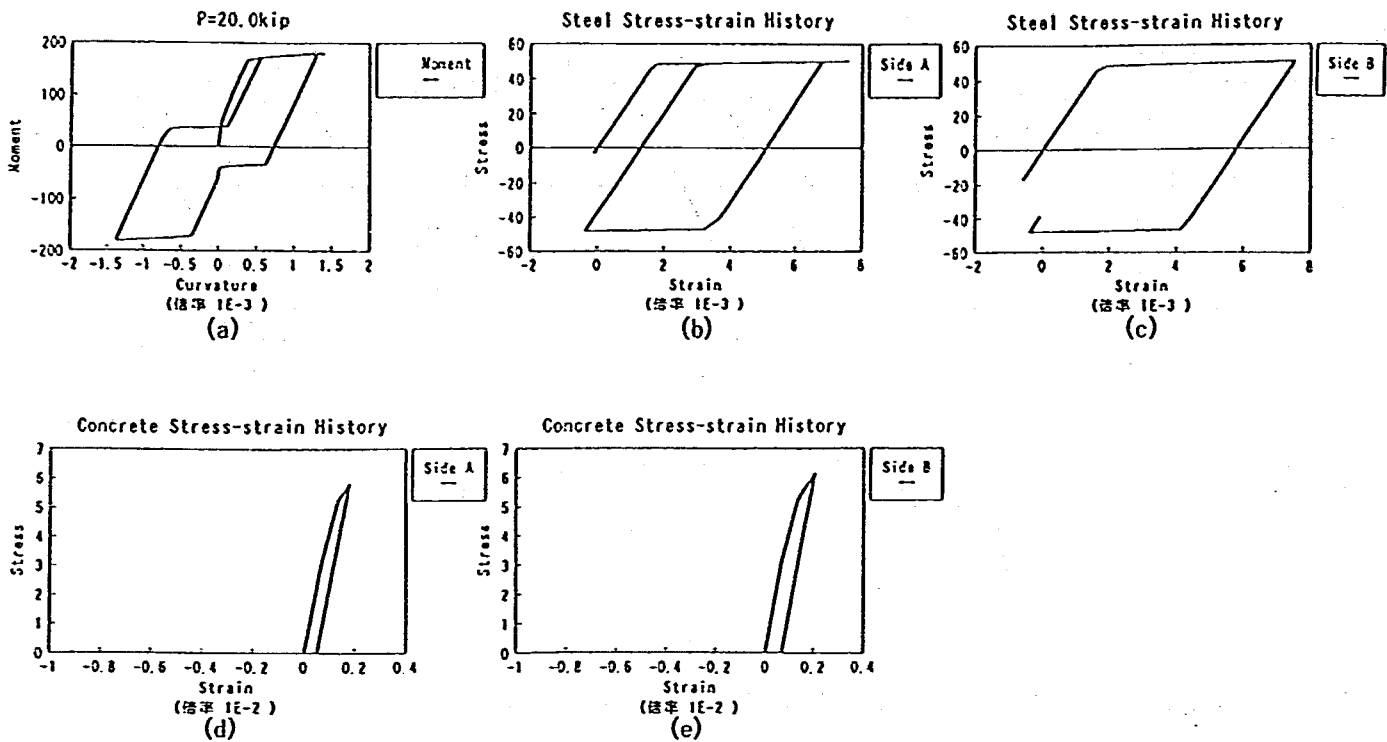


Fig. 20 Plots for axial force P=10.0kip

P=20.0kip C41



P=20.0kip C42

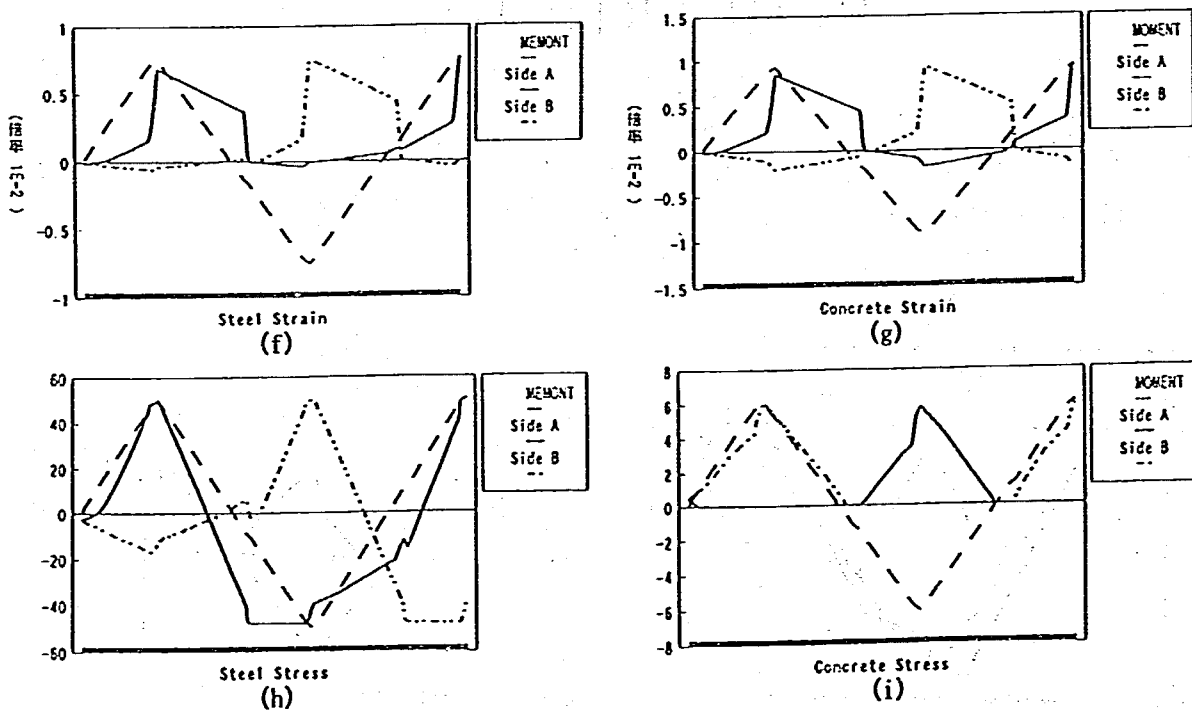
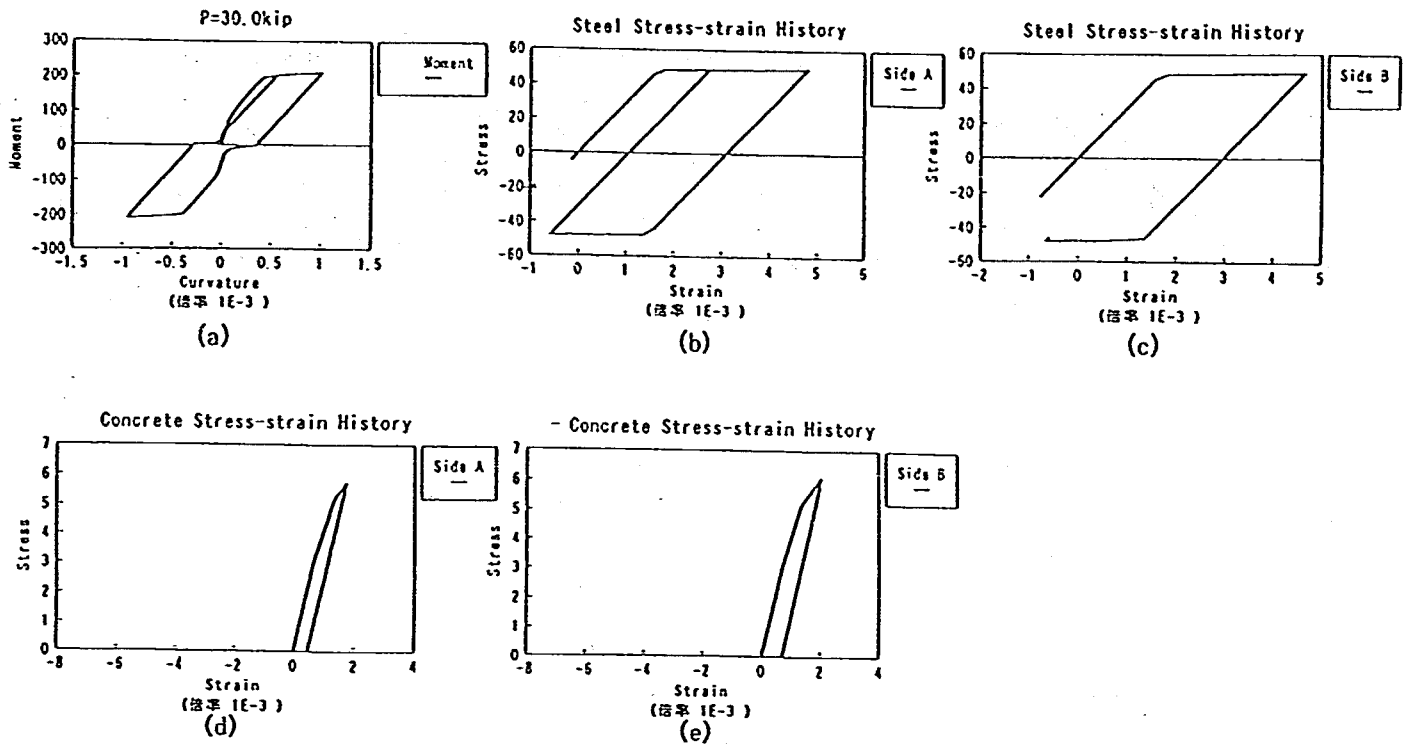


Fig. 21 Plots for axial force P=20.0kip

$P=30.0\text{kip}$ C51J



$P=30.0\text{kip}$ C52J

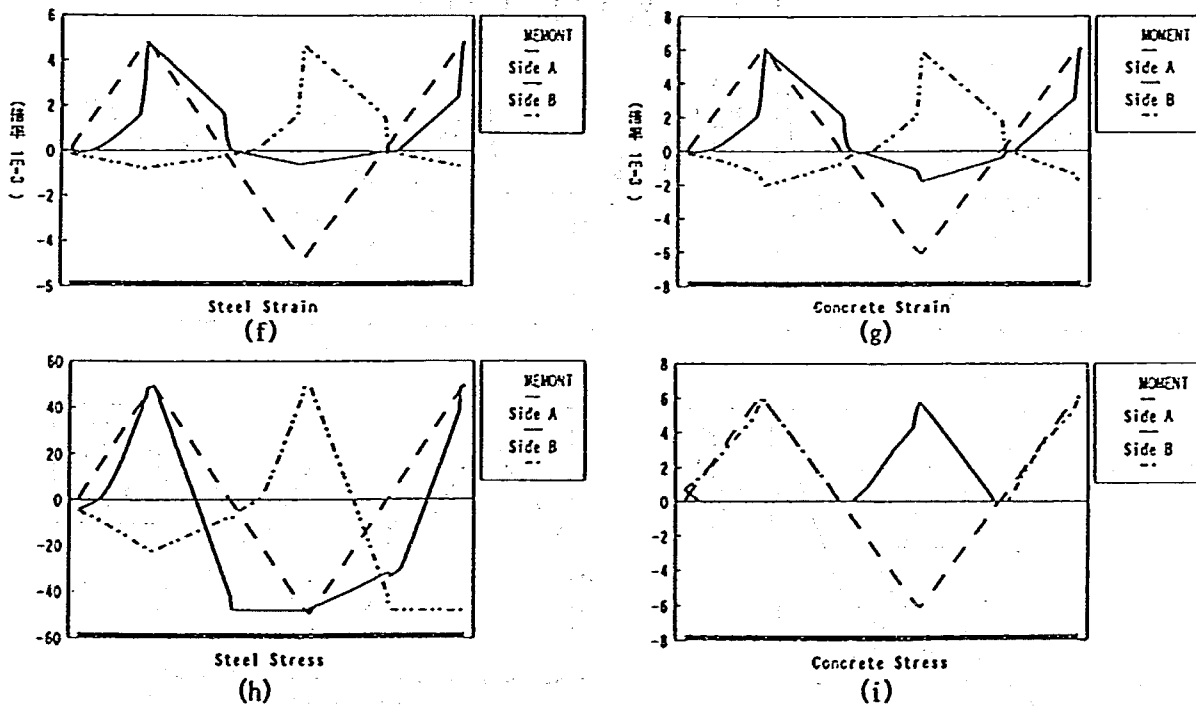
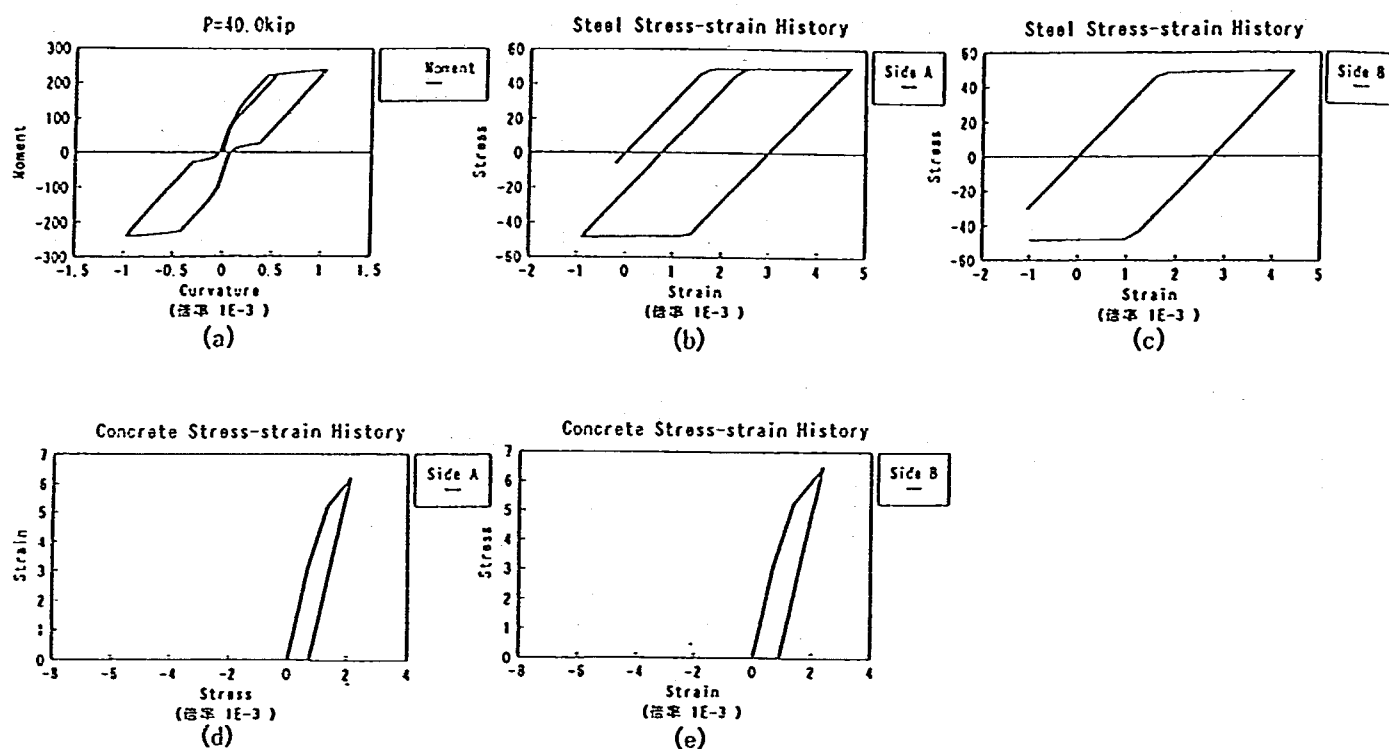


Fig. 22 Plots for axial force $P=30.0\text{kip}$

P=40.0kip C61



P=40.0kip C62

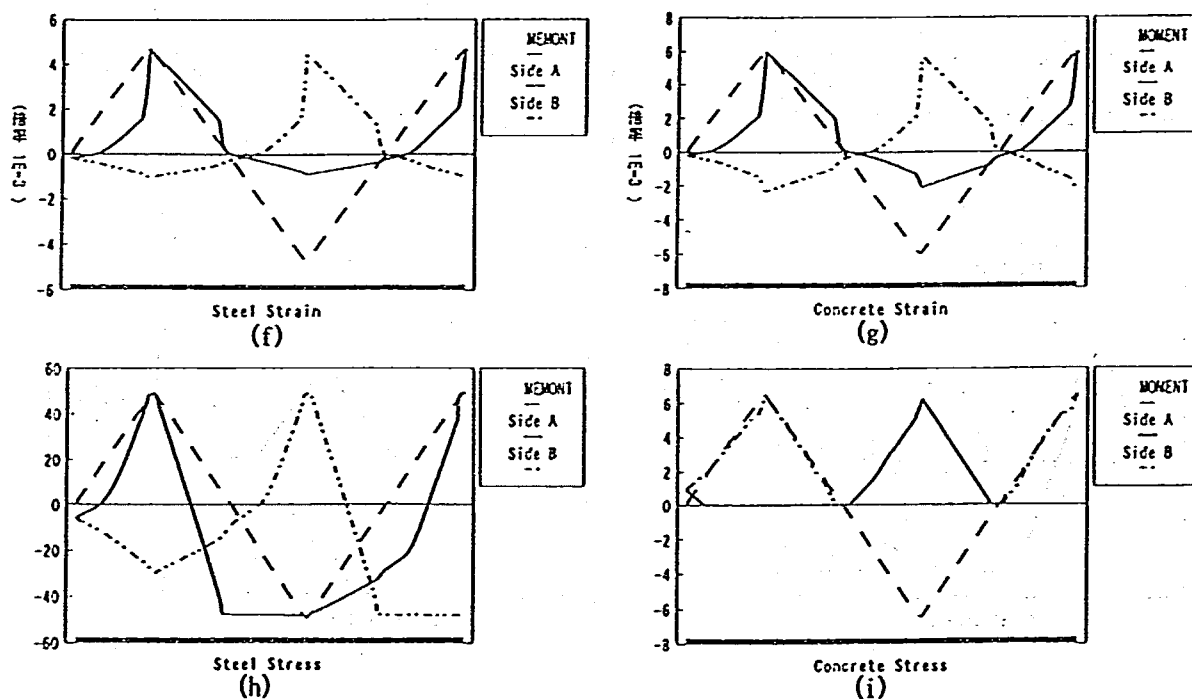
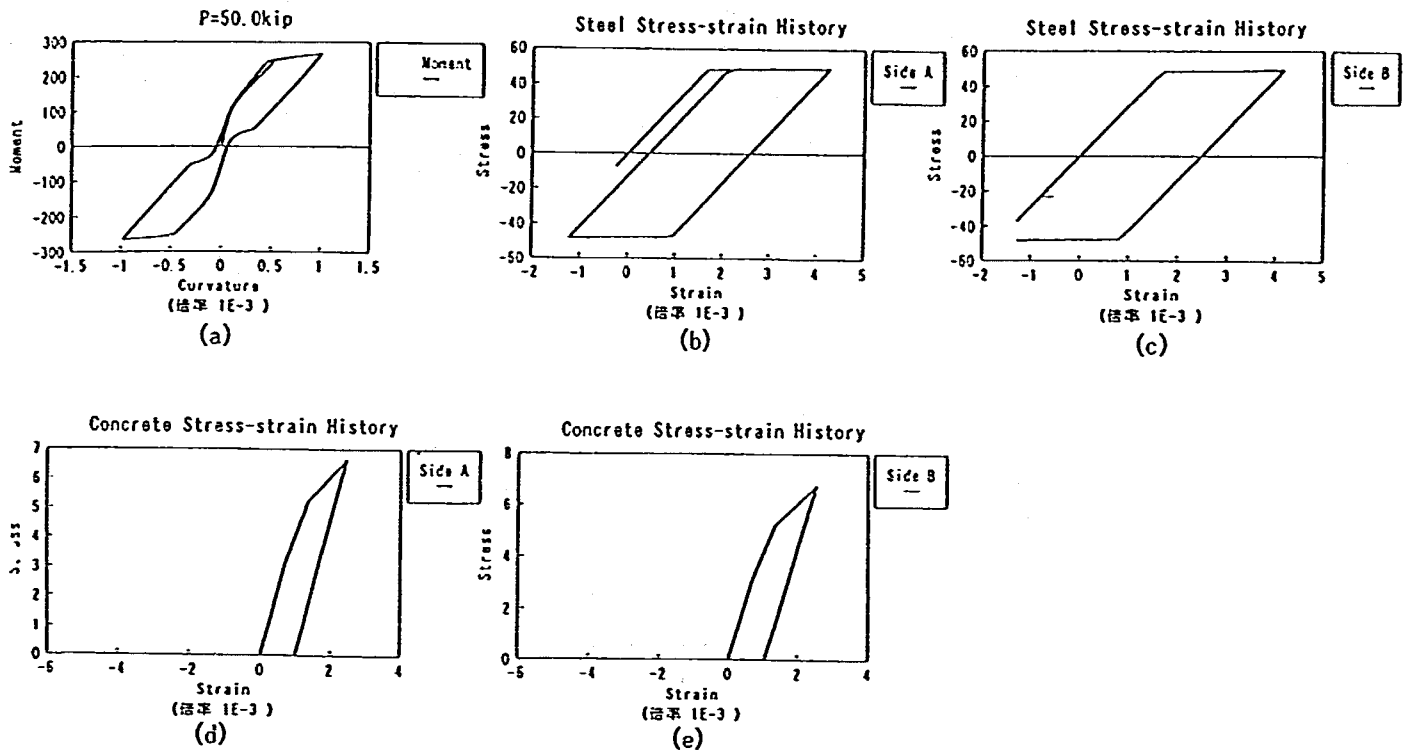


Fig. 23 Plots for axial force P=40.0kip

P=50.0kip C71



P=50.0kip C72

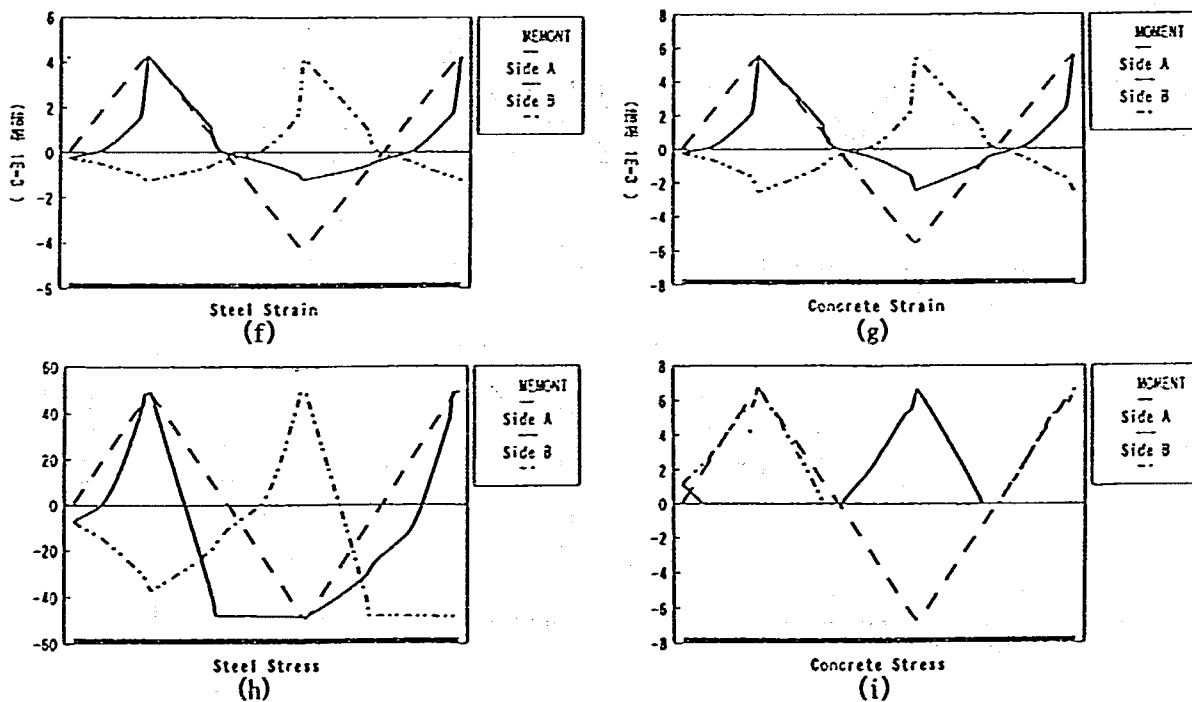
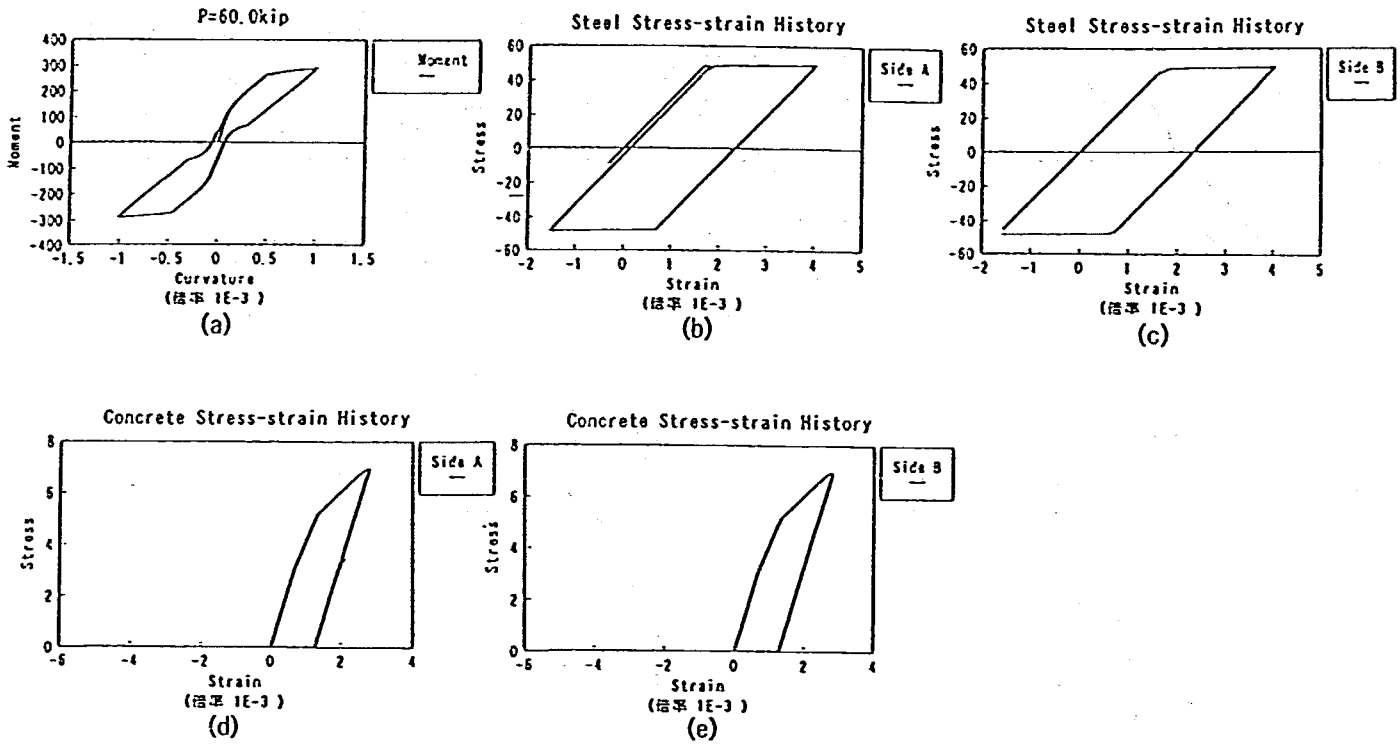


Fig. 24 Plots for axial force P=50.0kip

P=60.0kip C81



P=60.0kip C82

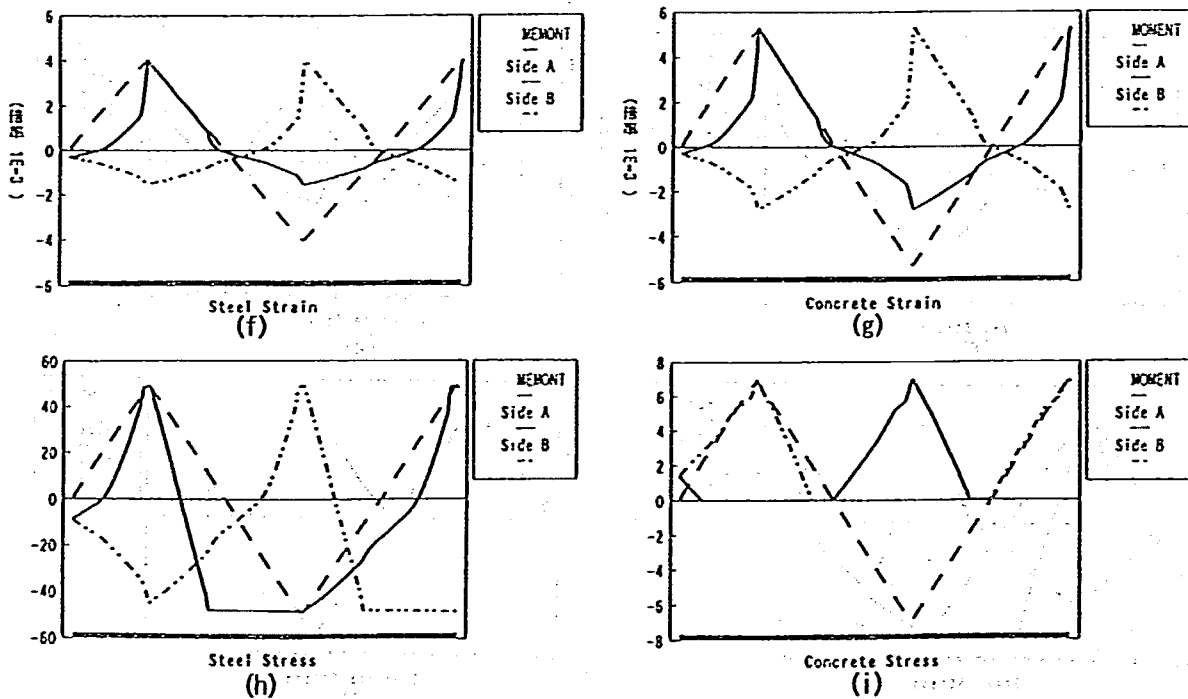
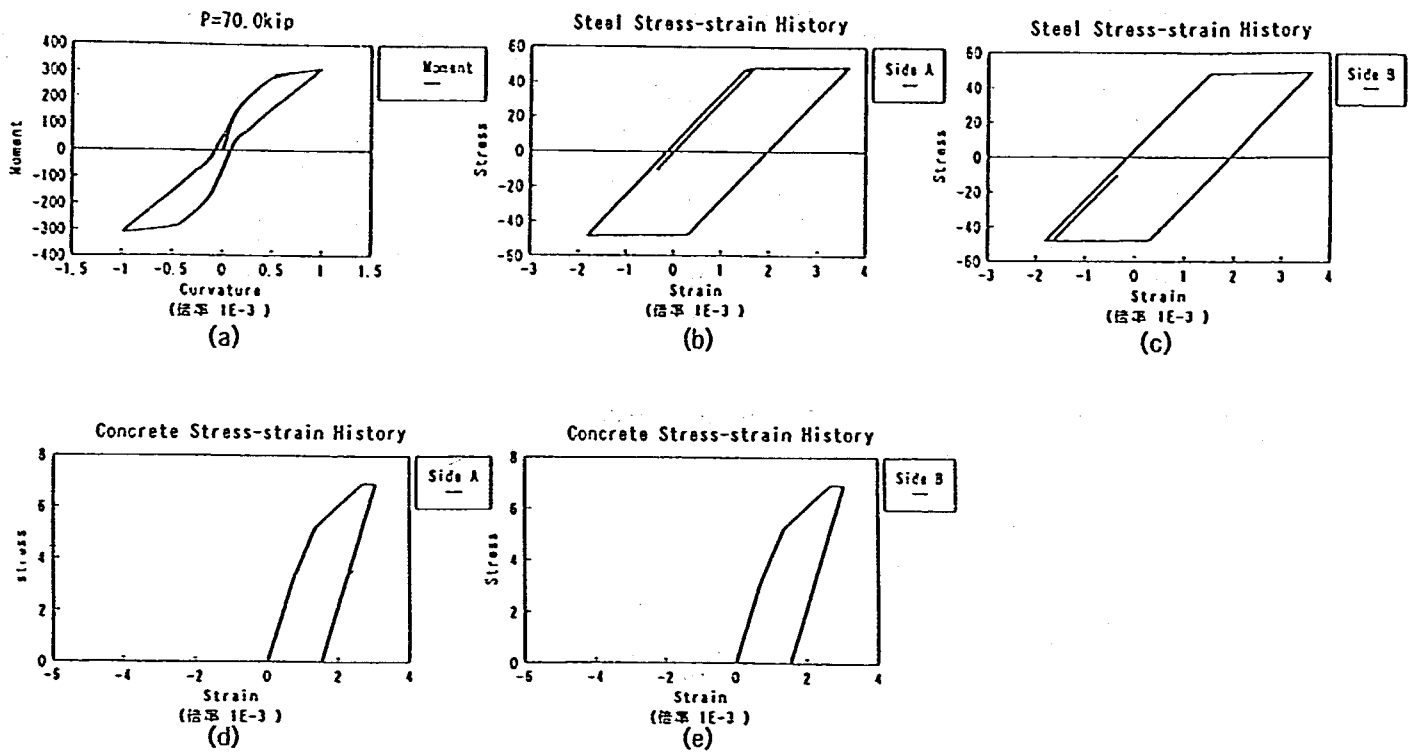


Fig. 25 Plost for axial force P=60.0kip

P=70.0kip C91



P=70.0kip C92

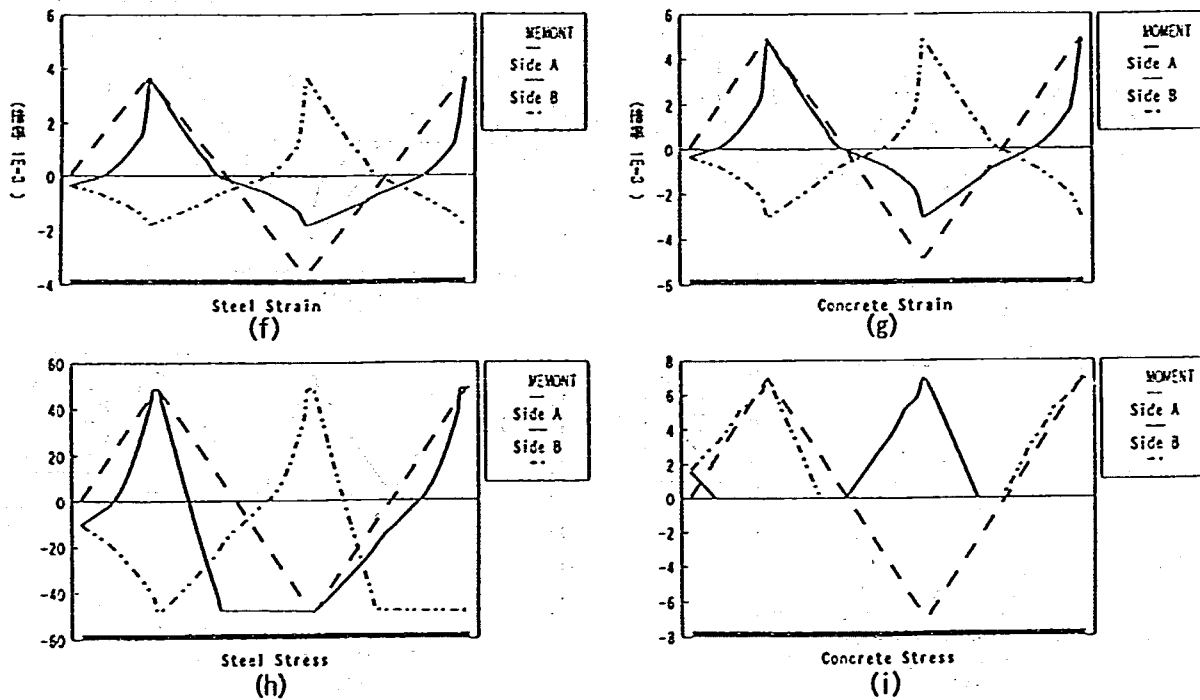
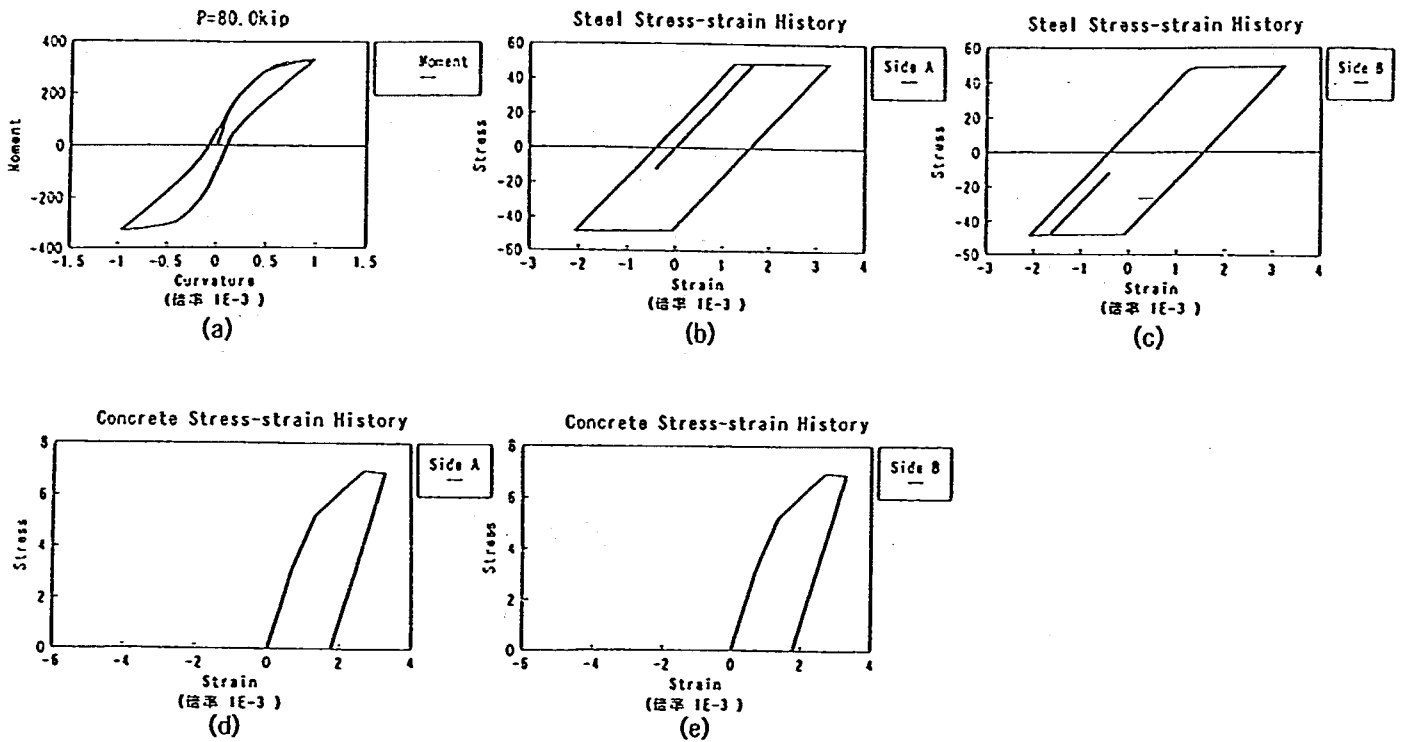


Fig. 26 Plots for axial force P=70.0kip

P=80.0kip C101



P=80.0kip C102

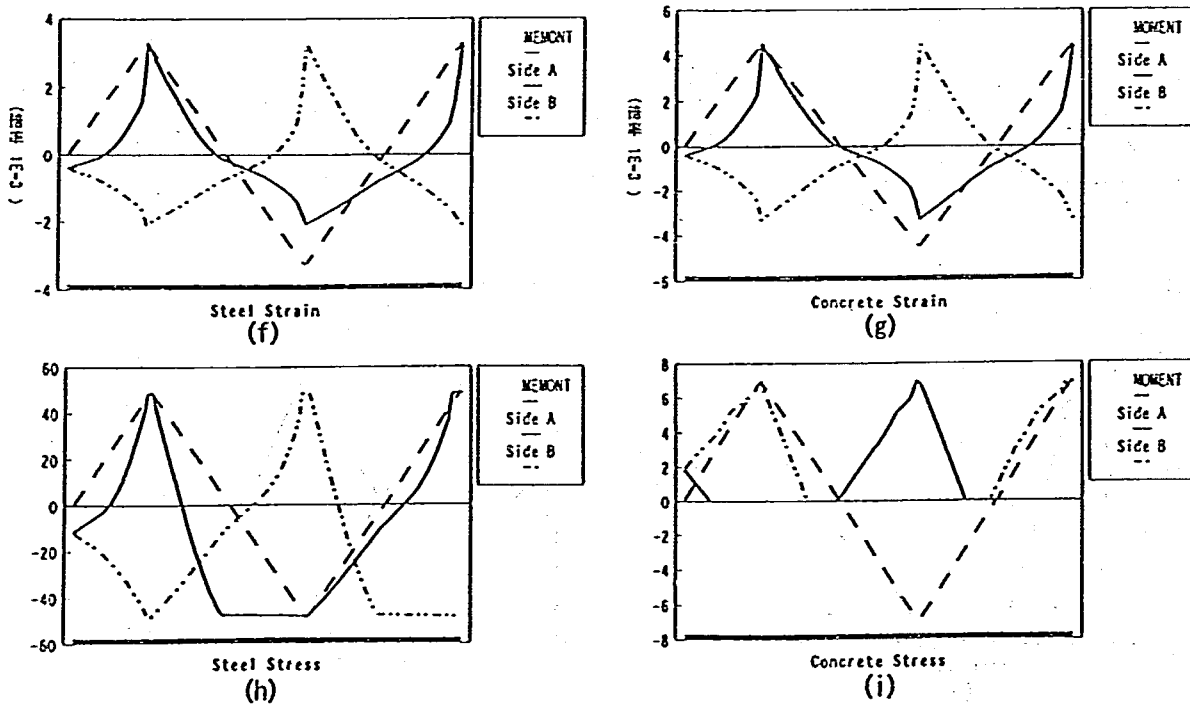
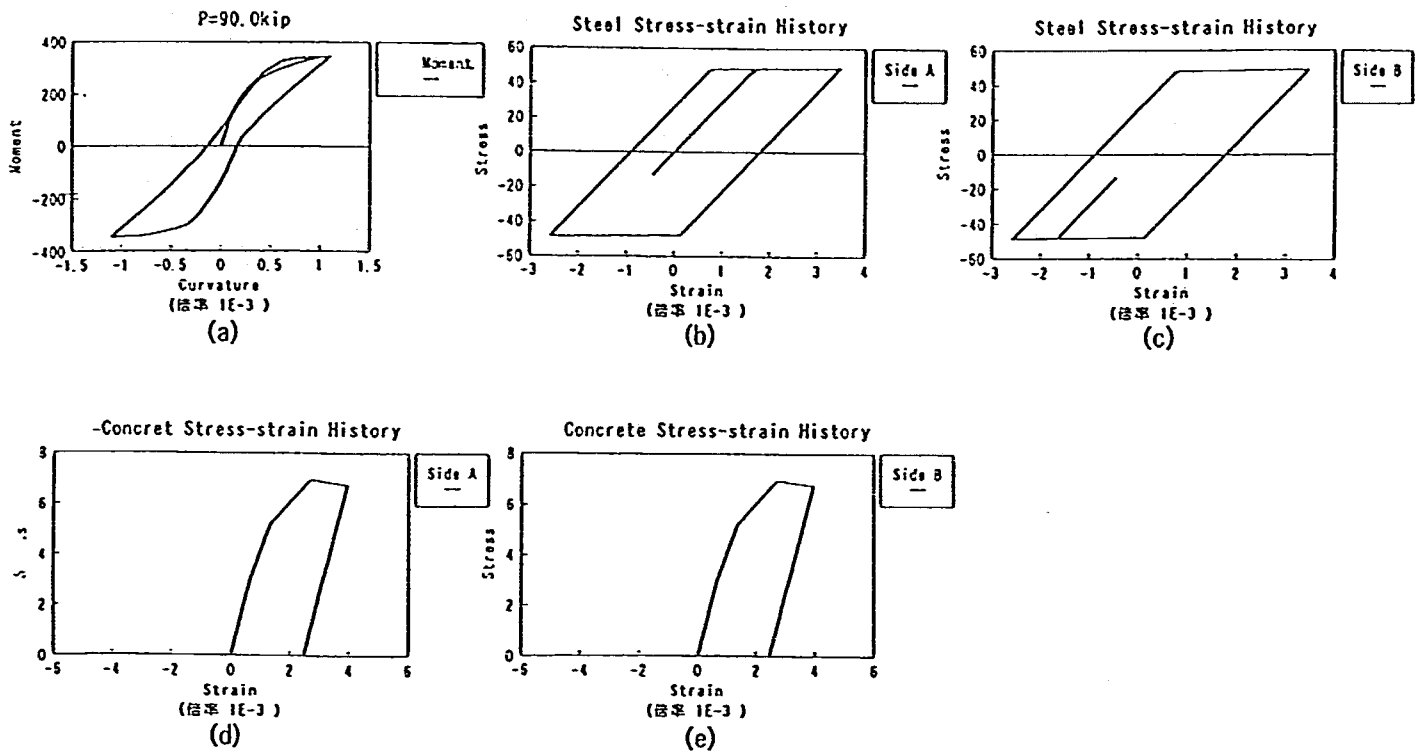


Fig. 27 Plots for axial force $P=80.0\text{kip}$

P=90.0kip C111



P=90. kip C112

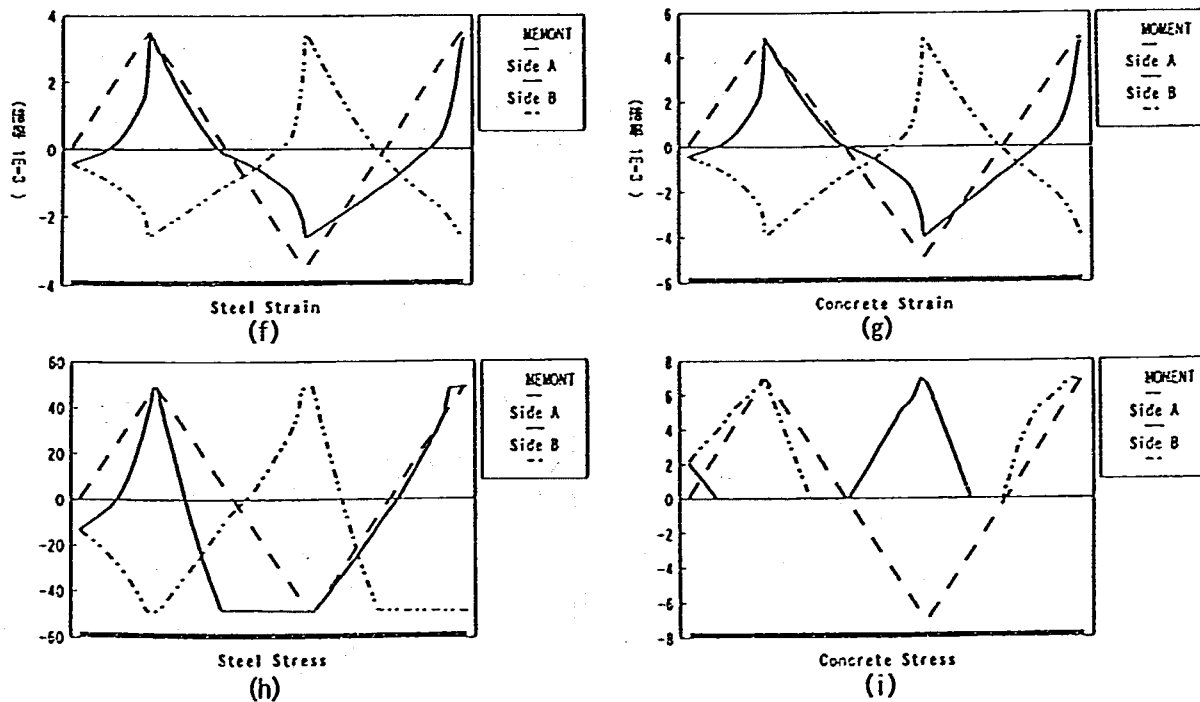
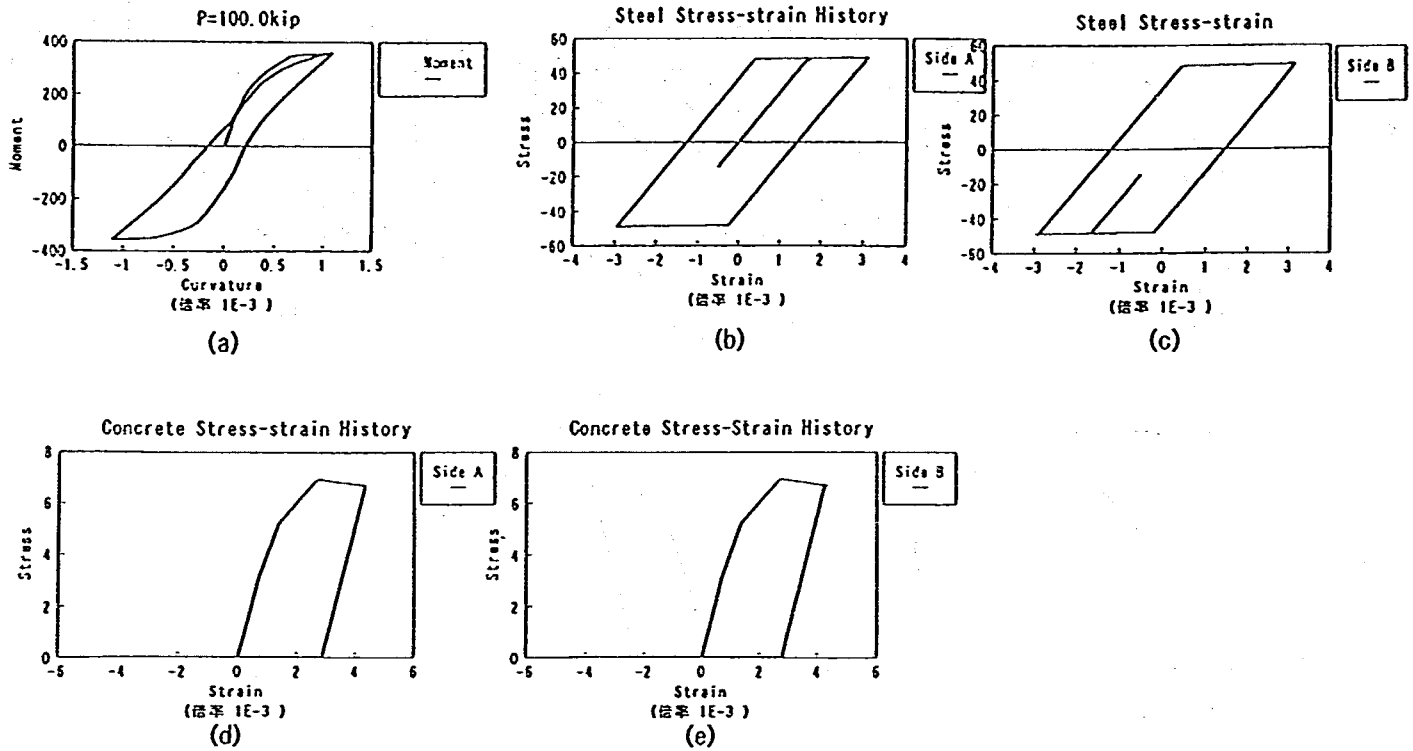


Fig. 28 Plots for axial force P=90.0kip

P=100.0kip C121



P=100.0kip C122

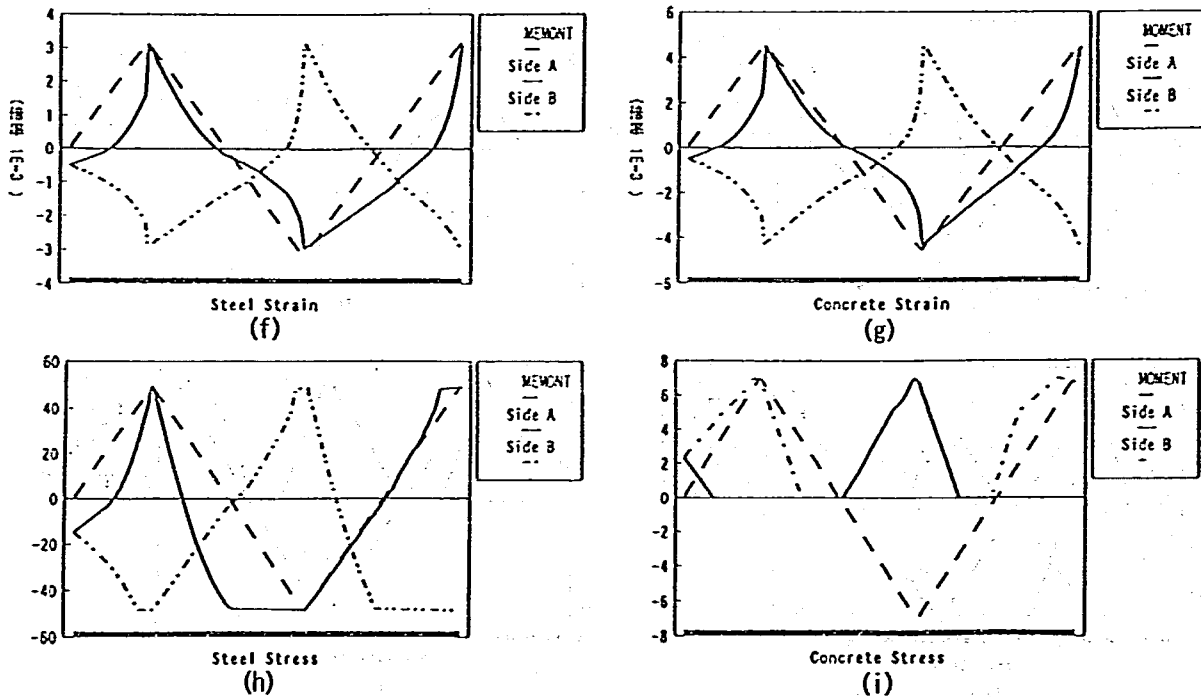
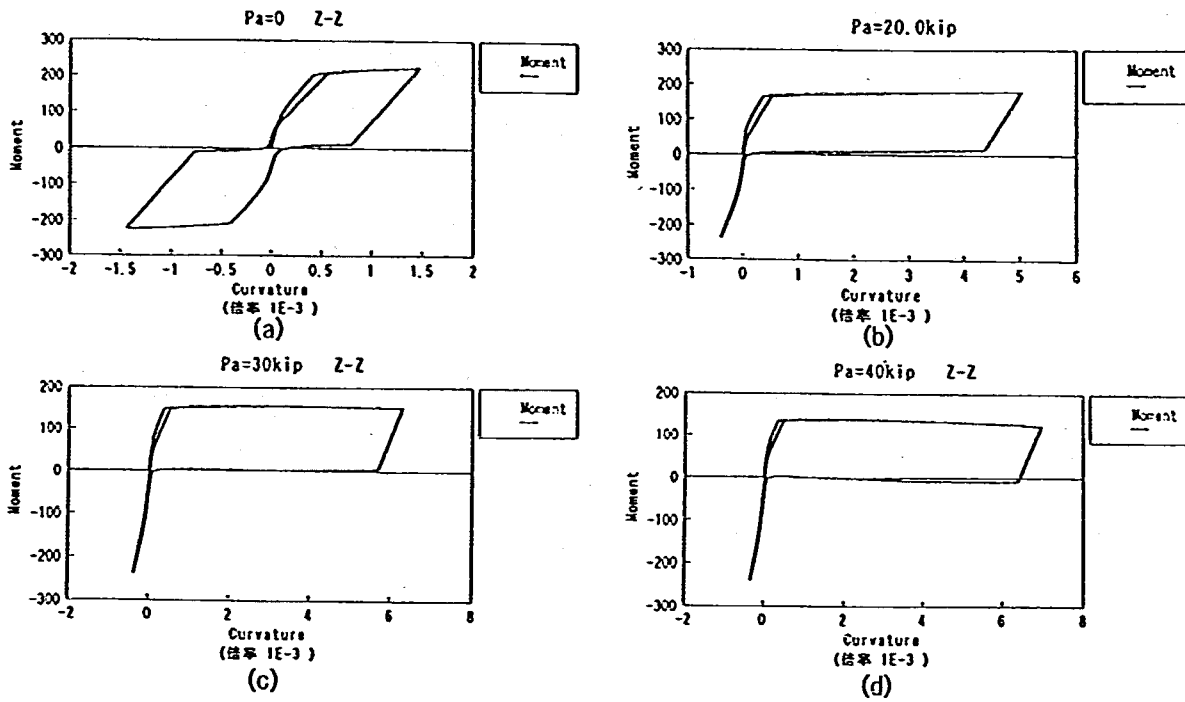


Fig. 29 Plots for axial force P=100.0kip

$$T_a/T_b=1.0$$



$$T_a/T_b=1.0$$

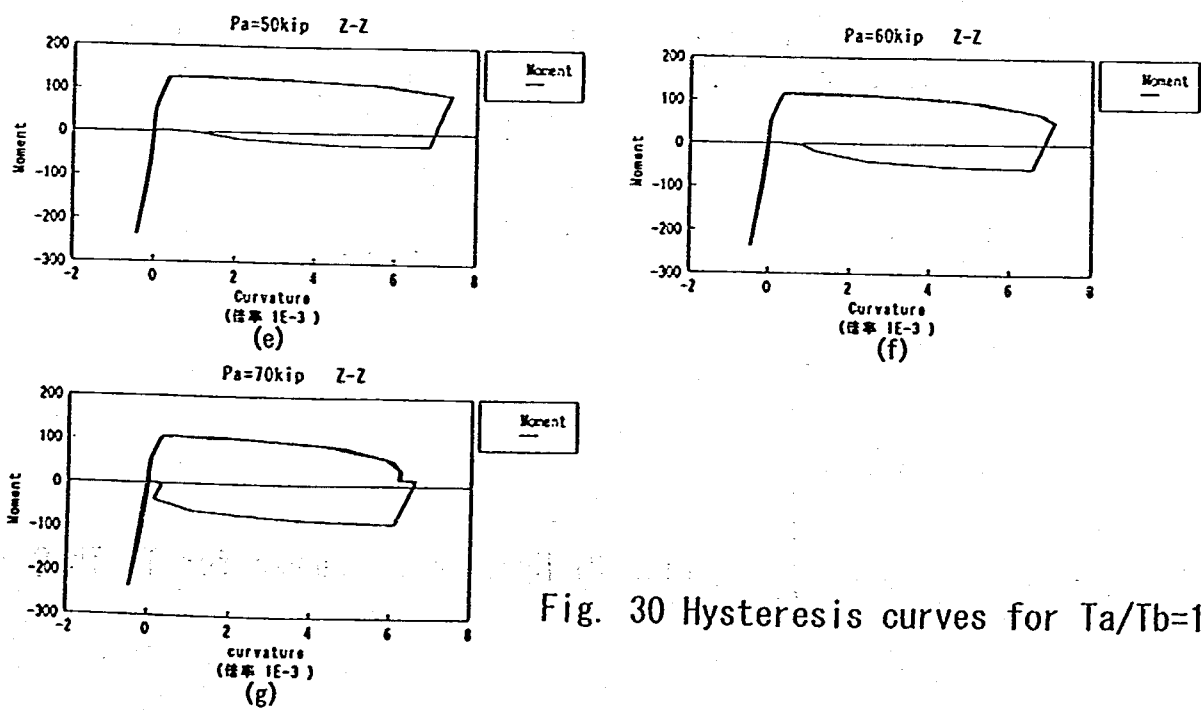
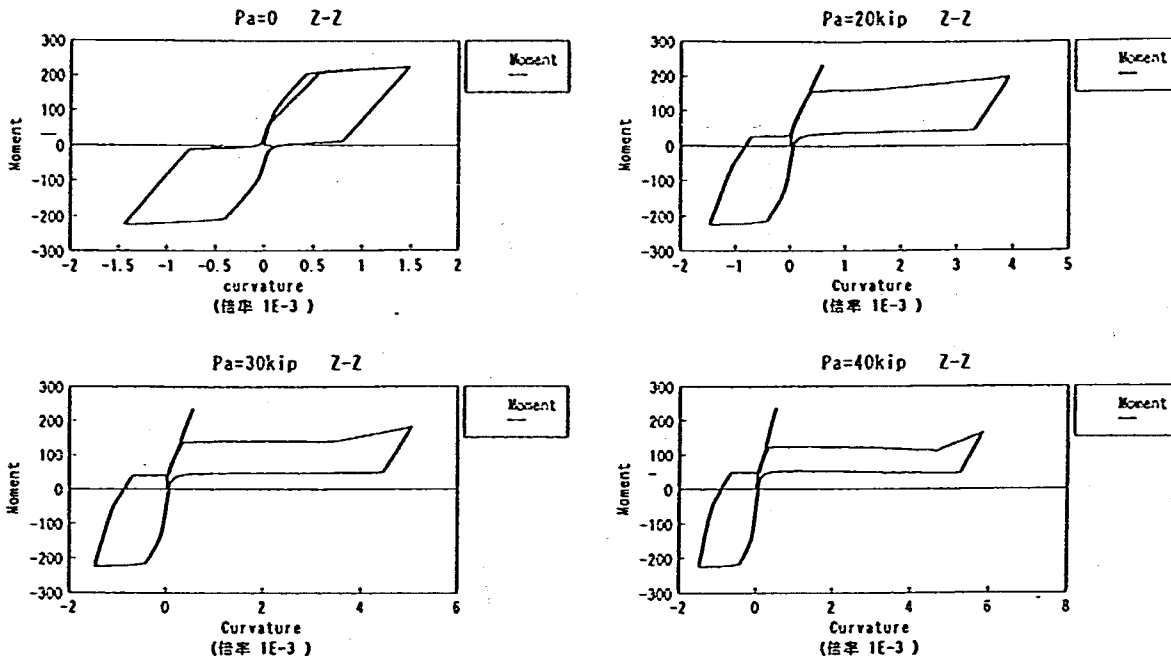


Fig. 30 Hysteresis curves for $T_a/T_b=1.0$

$$T_a/T_b=0.75$$



$$T_a/T_b=0.75$$

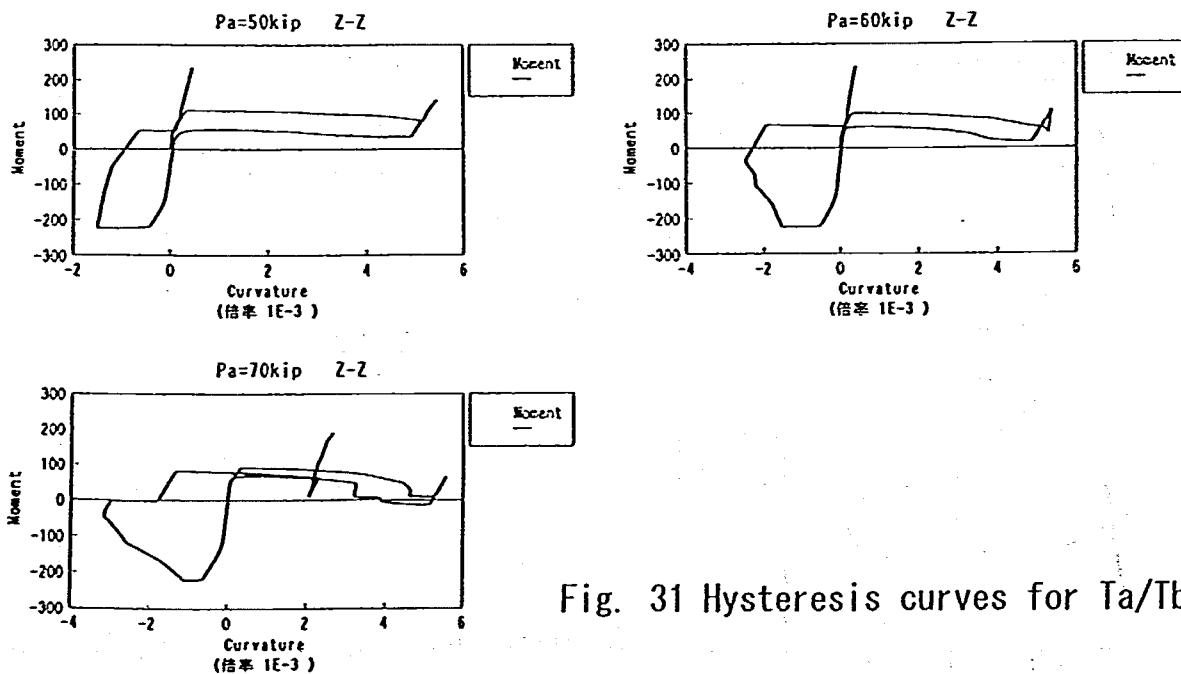
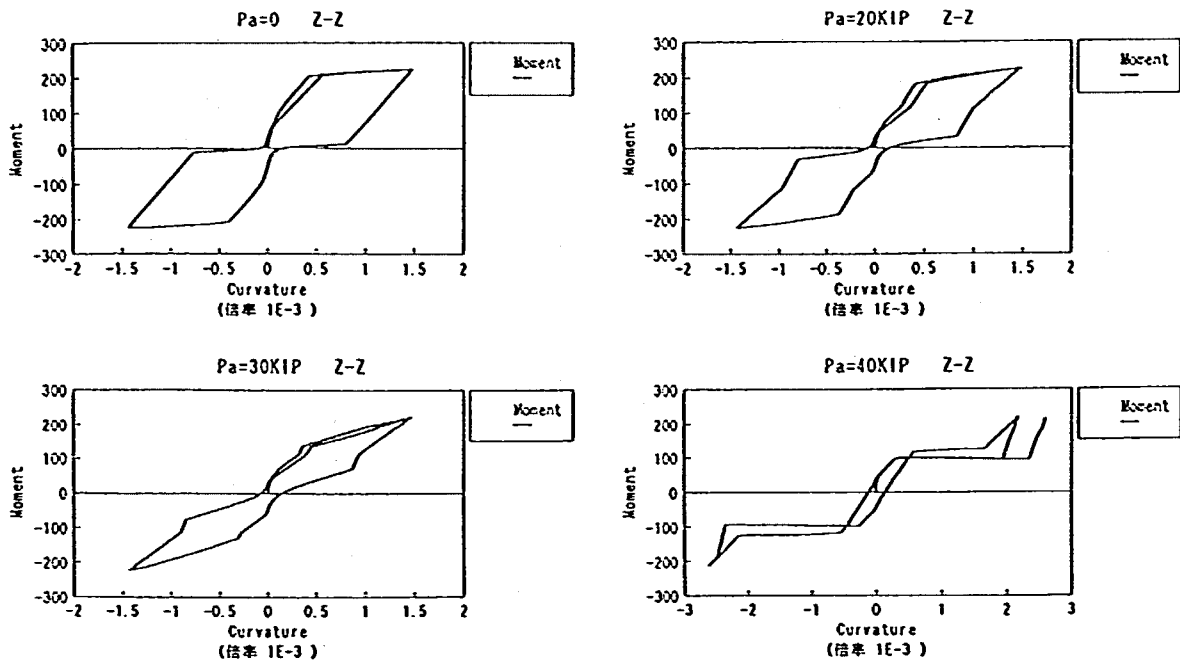


Fig. 31 Hysteresis curves for $T_a/T_b=0.75$

$$T_a/T_b=0.5$$



$$T_a/T_b=0.5$$

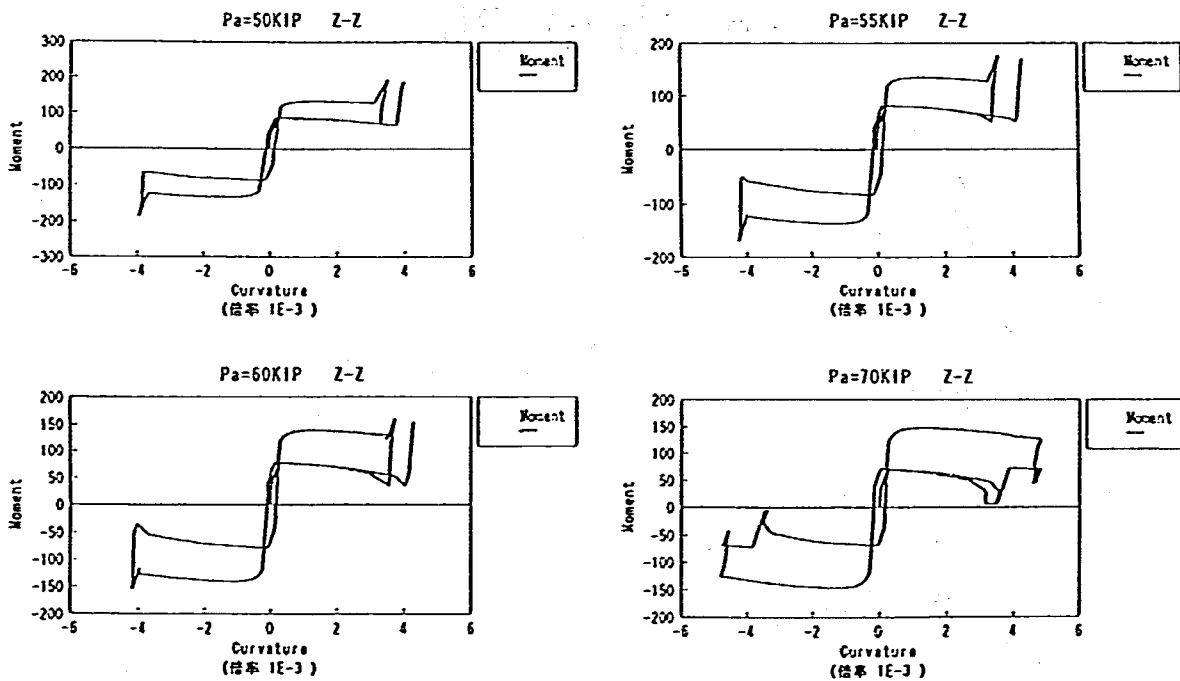
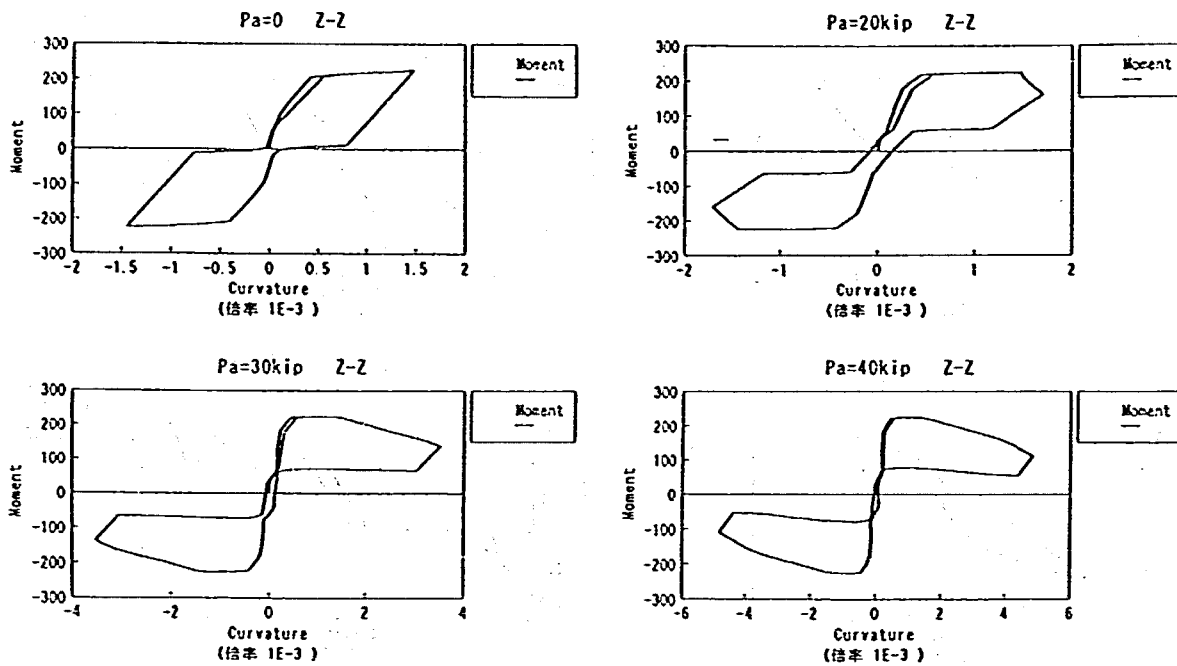


Fig. 32 Hysteresis curves for $T_a/T_b=0.5$

$$T_a/T_b=0.25$$



$$T_a/T_b=0.25$$

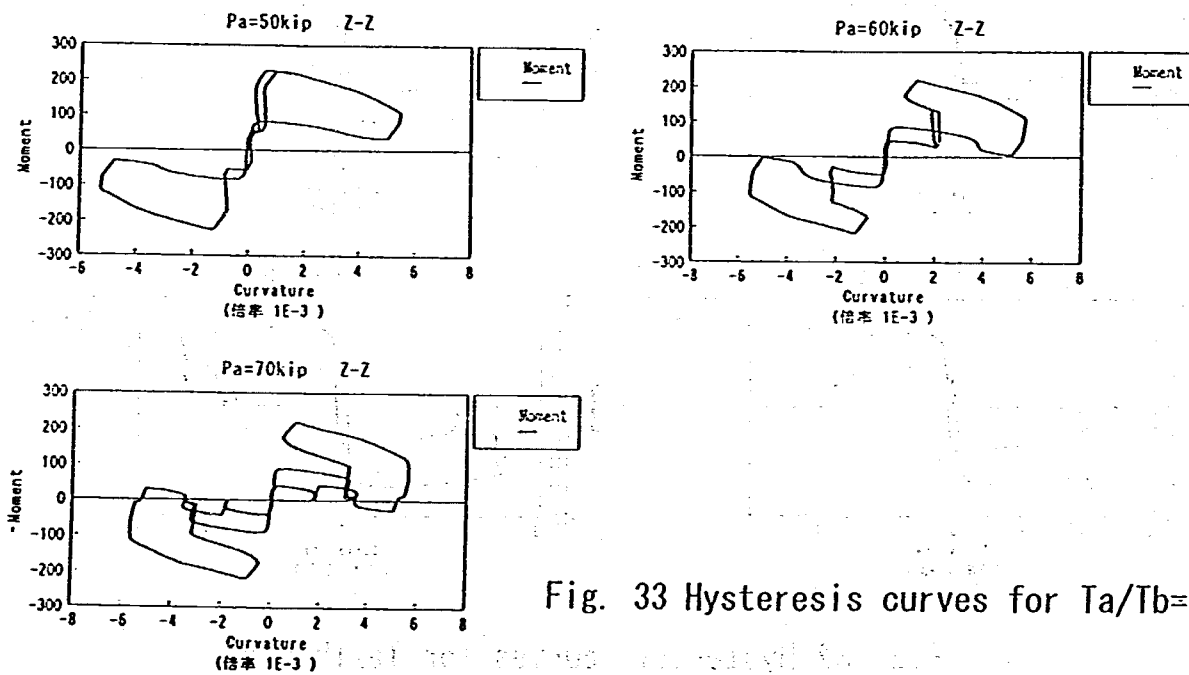
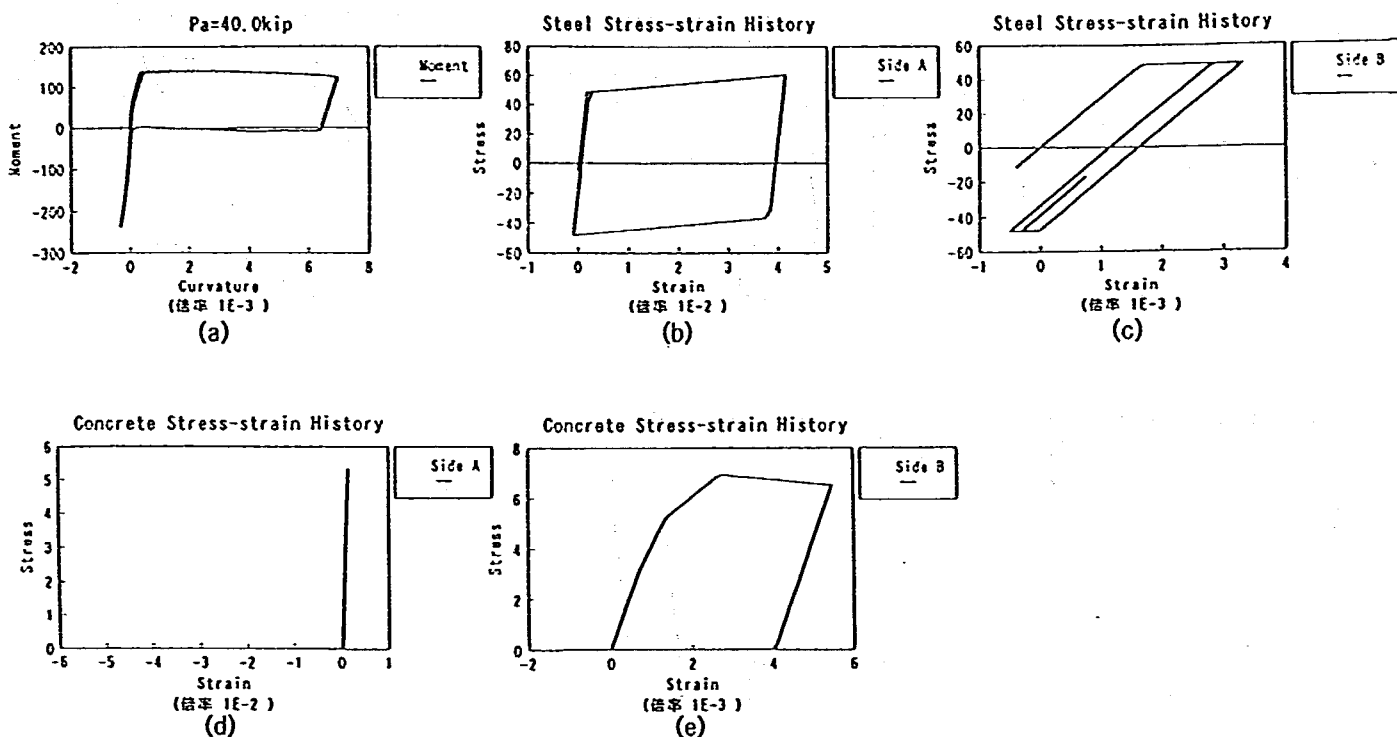


Fig. 33 Hysteresis curves for $T_a/T_b=0.25$

$$T_a/T_b=1.0$$



$$T_a/T_b=1.0$$

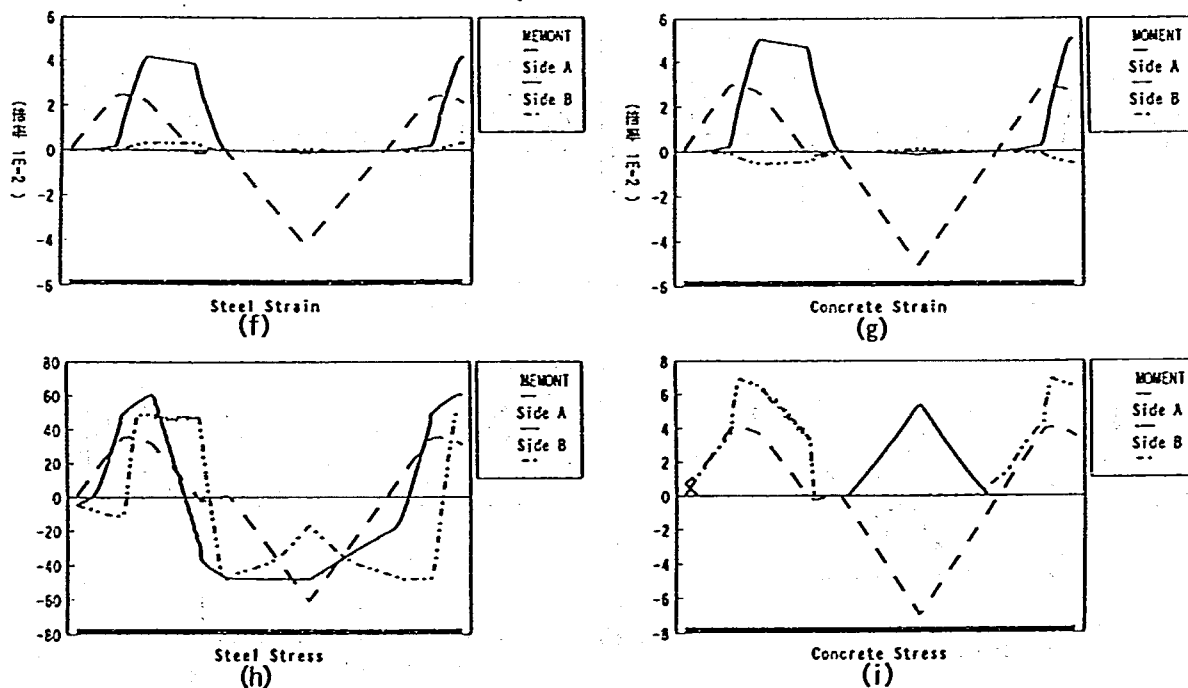
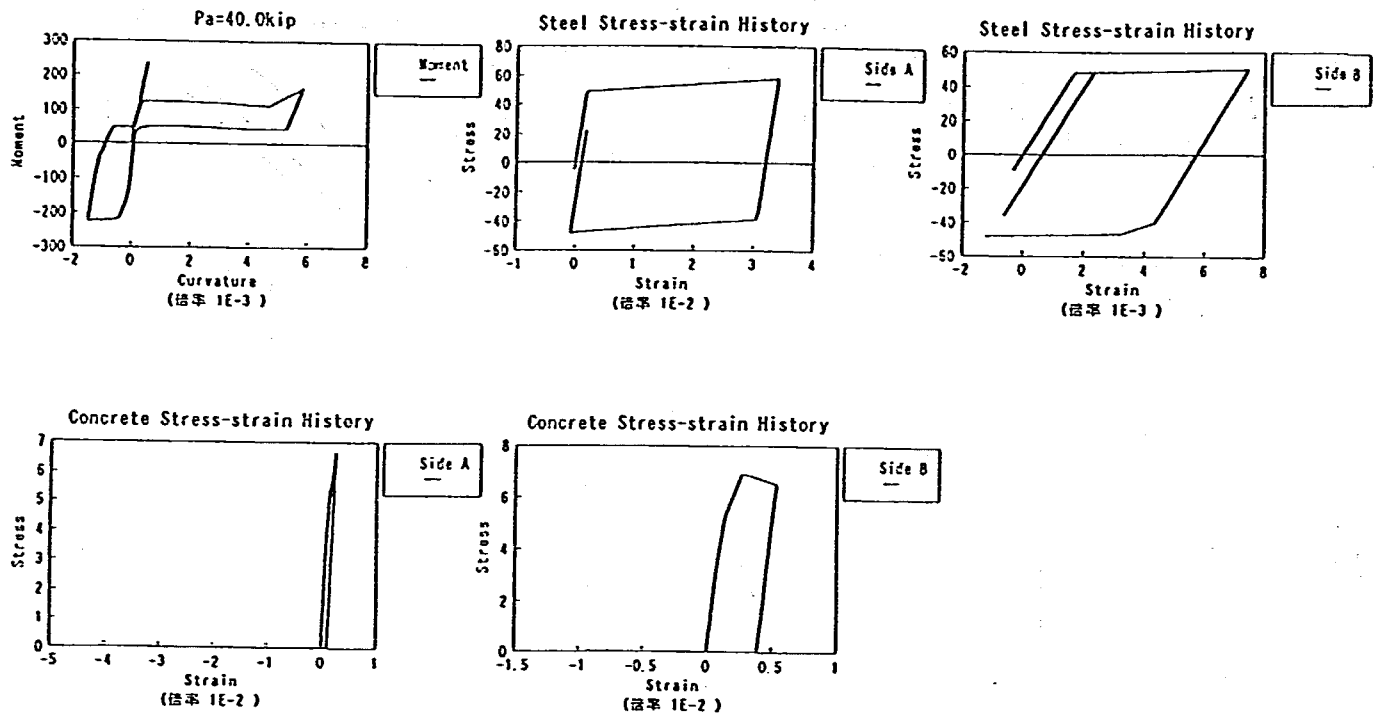


Fig. 34 Plots for varying axial force with $P_a=40.0\text{kip}$ and period ratio $T_a/T_b=1.0$

$$T_a/T_b=0.75$$



$$T_a/T_b=0.75$$

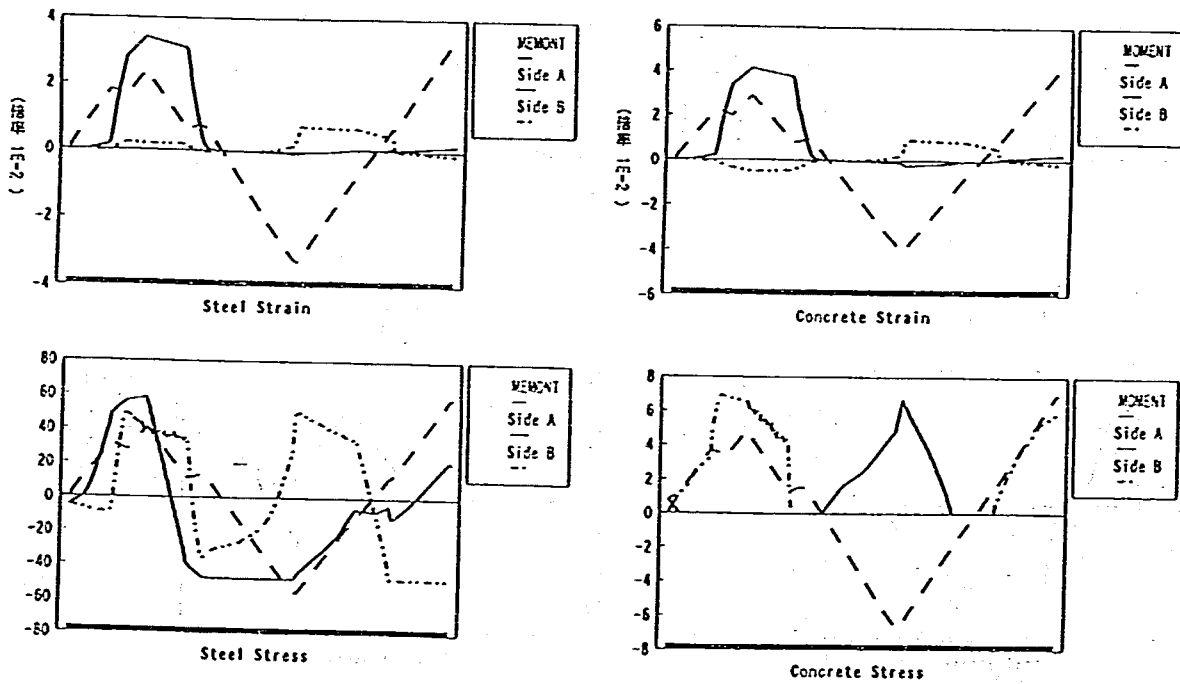


Fig. 35 Plots for varying axial force with $P_a=40.0\text{kip}$ and period ratio $T_a/T_b=0.75$

$$T_a/T_b=0.5$$

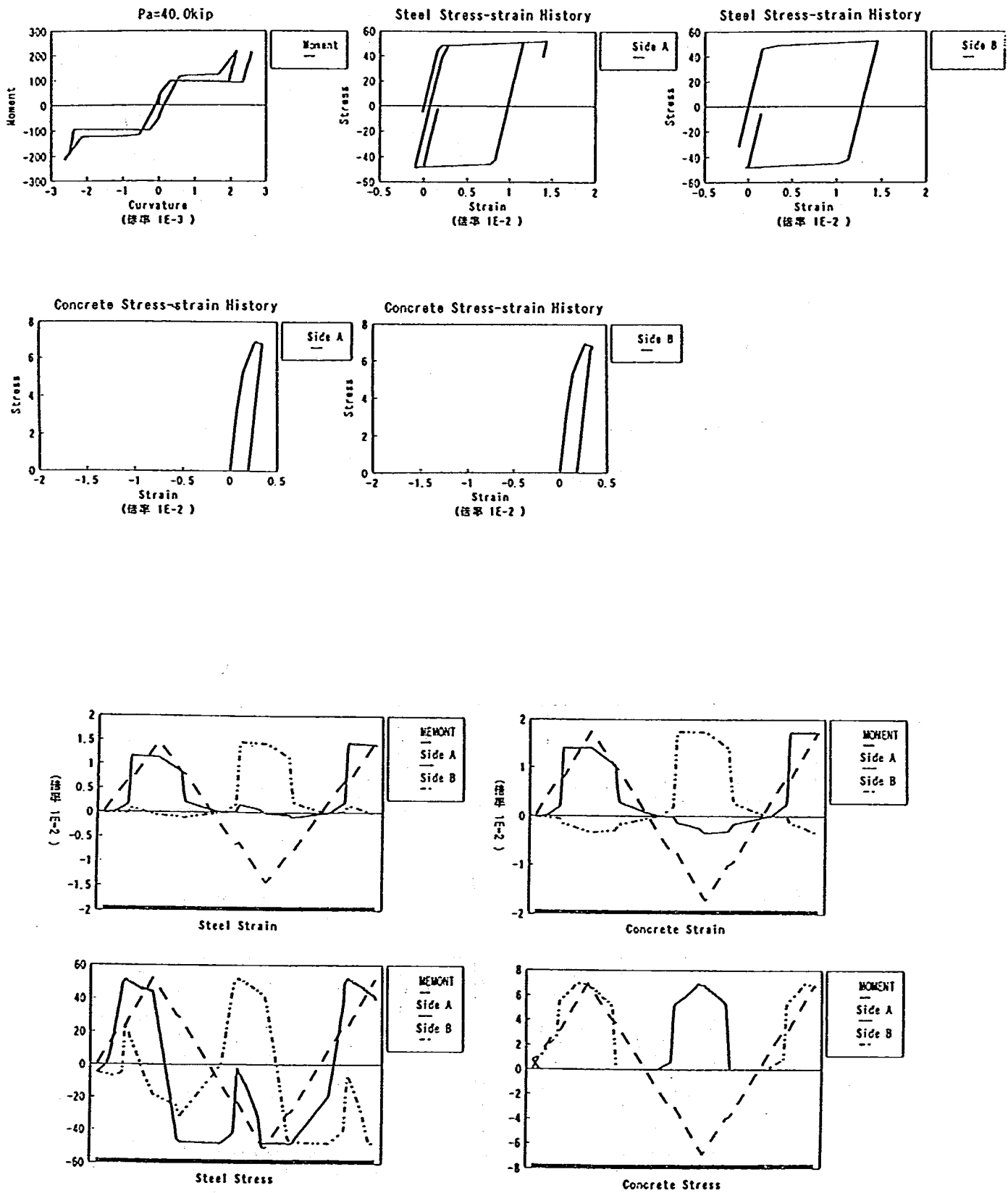


Fig. 36 Plots for varying axial force with $P_a=40.0 \text{ kip}$ and period ratio $T_a/T_b=0.5$

$$T_a/T_b=0.25$$

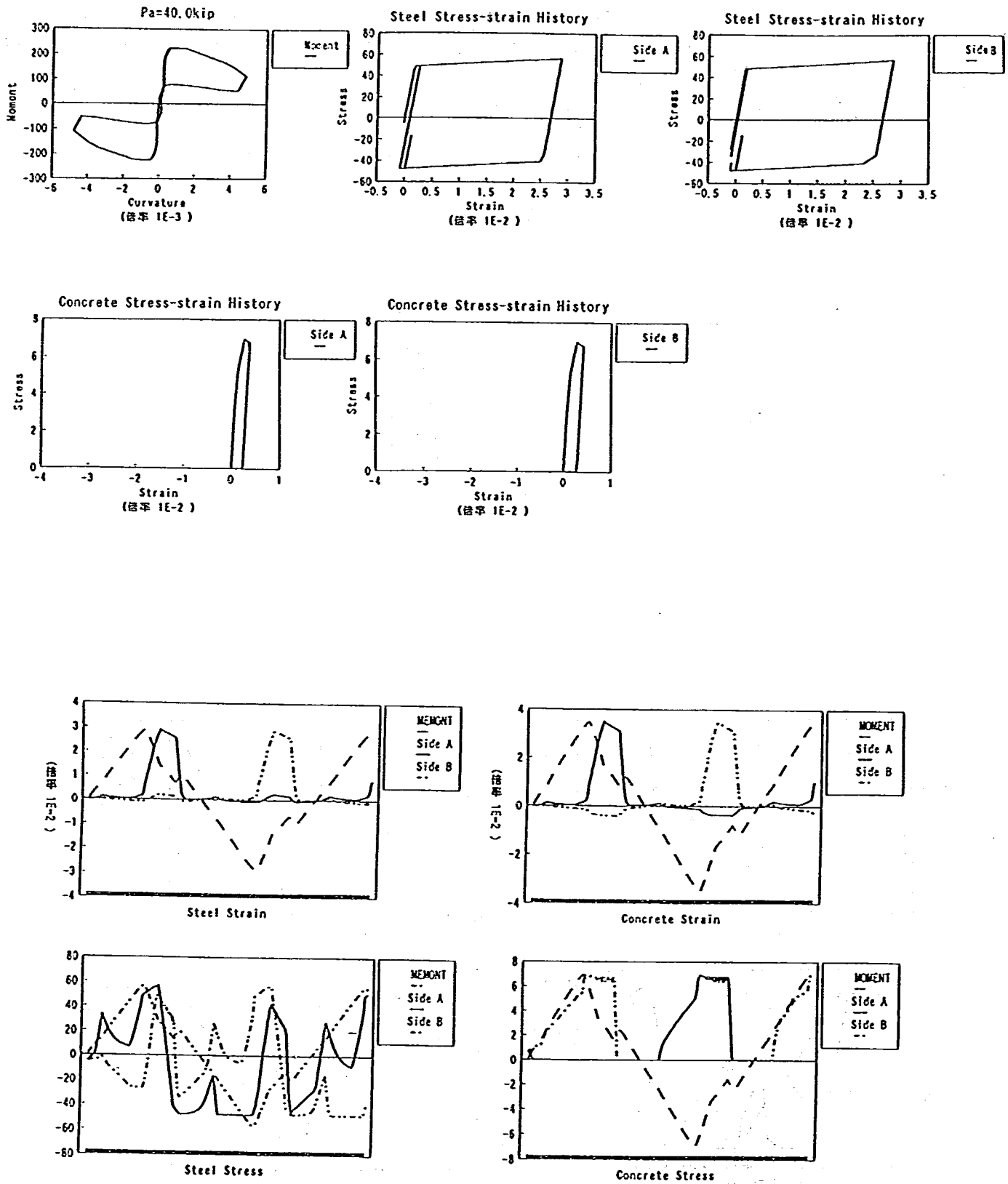


Fig. 37 Plots for varying axial force with $P_a=40.0\text{kip}$ and period ratio $T_a/T_b=0.25$

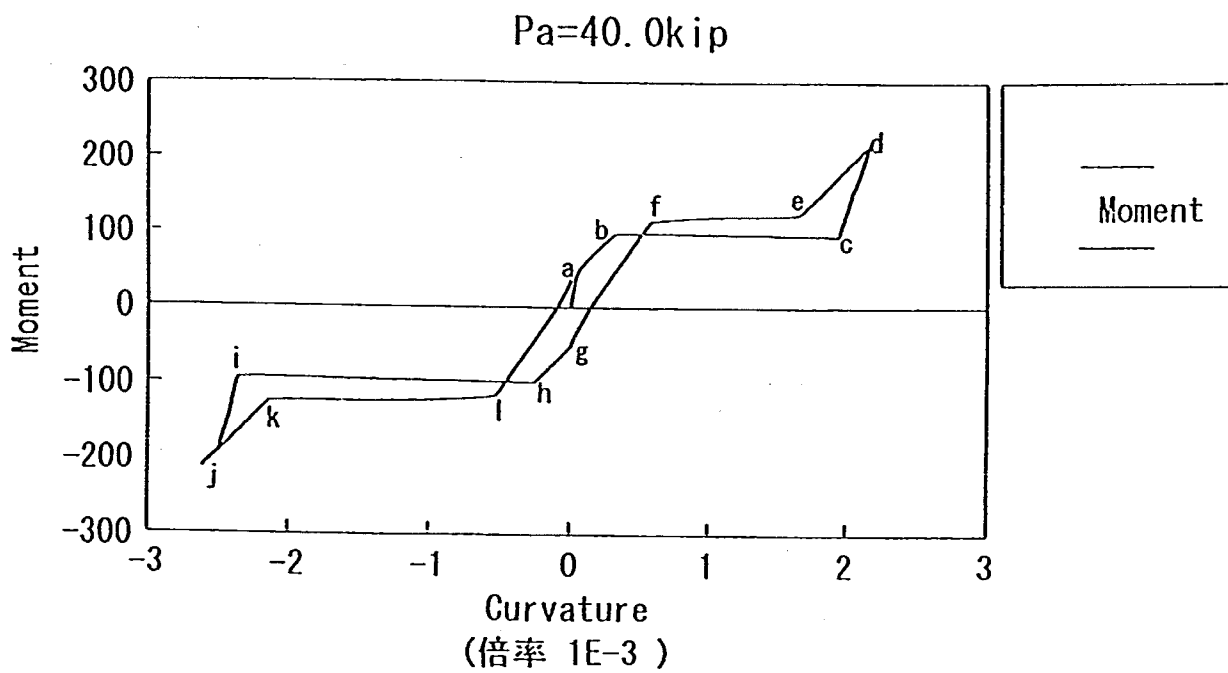


Fig. 38 Moment curvature history for $P_a = 40.0 \text{ kip}$ and period ratio $T_a/T_b = 0.5$

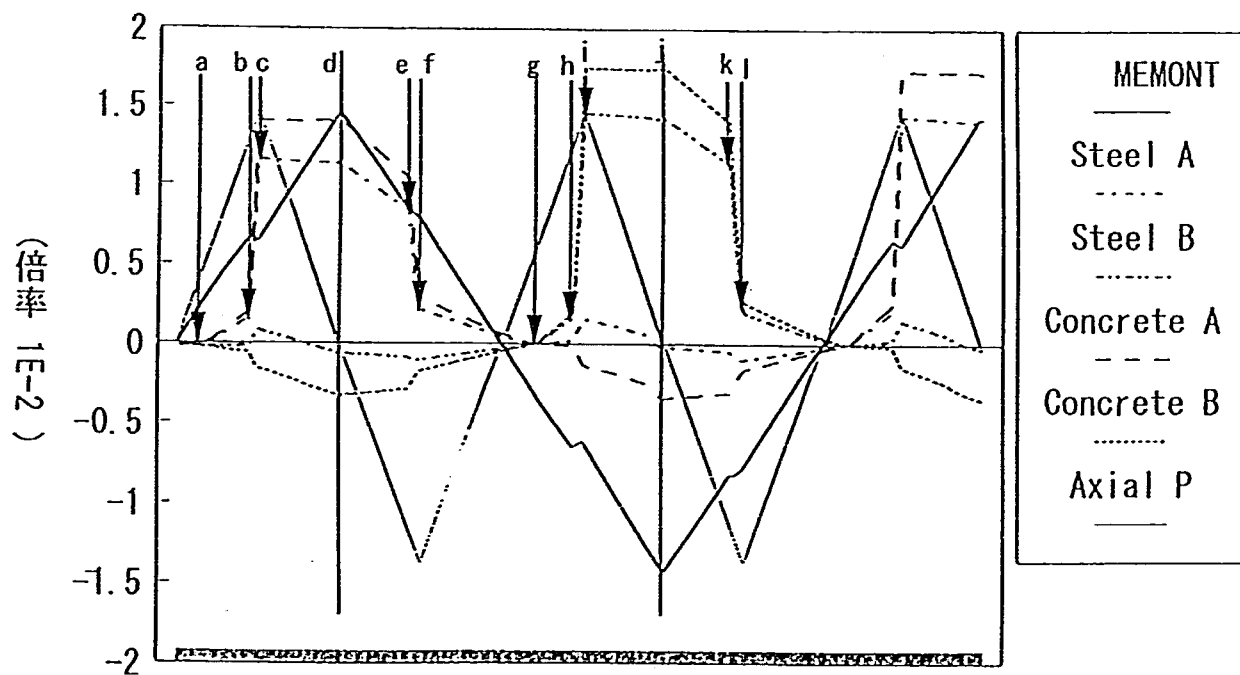


Fig. 39 Histories of steel and concrete strains with histories of moment and axial force P

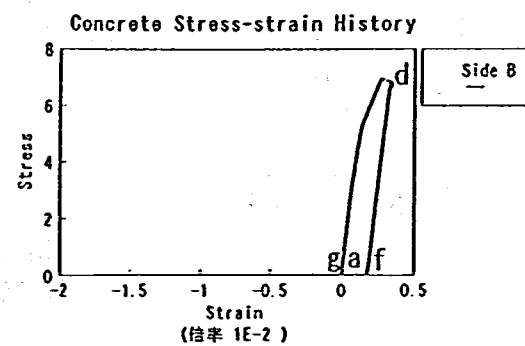
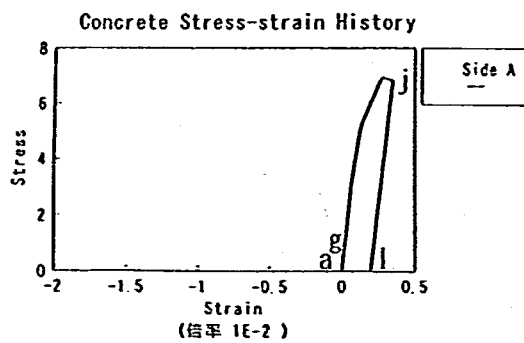
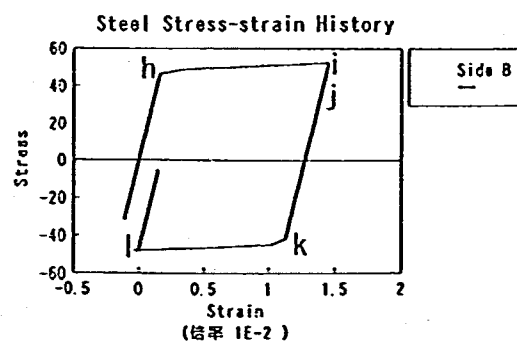
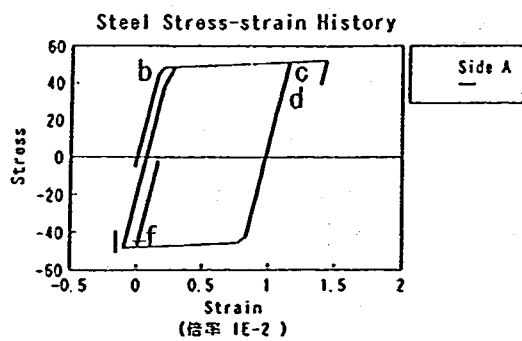


Fig. 40 Stress-strain histories of steel and concrete for $T_a/T_b=0.5$ and $P_a=40.0\text{kip}$

$T_a/T_b=1.0$ Phase angle=90

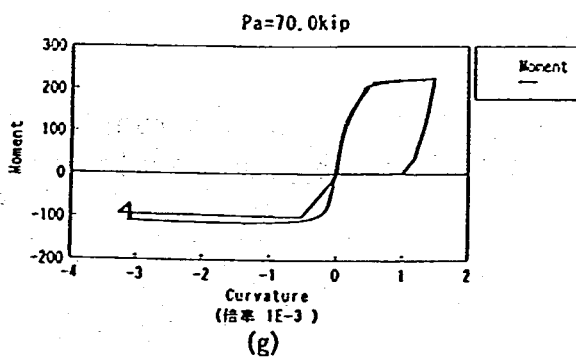
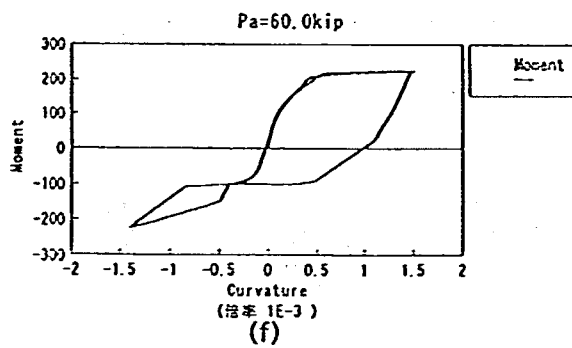
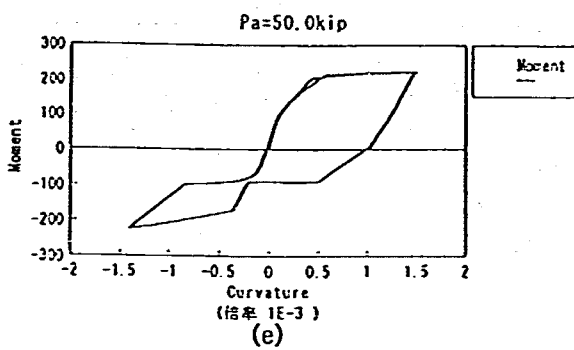
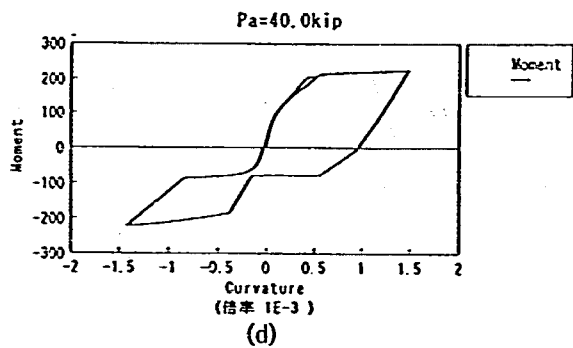
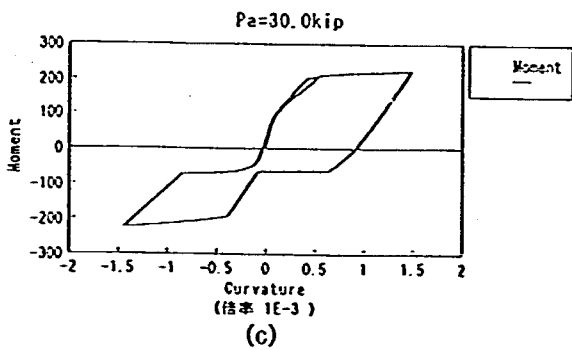
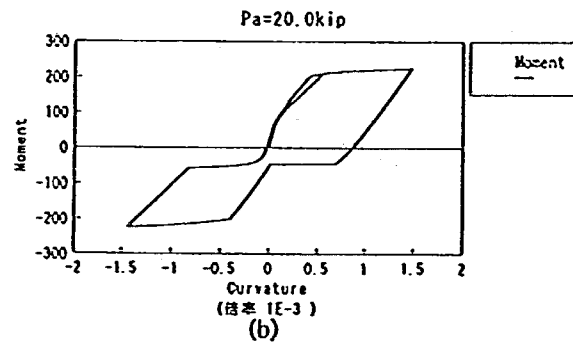
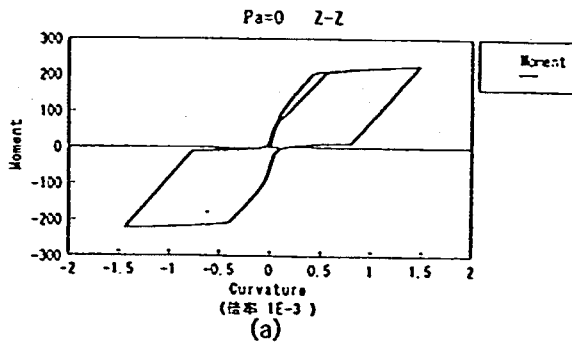


Fig. 41 Hysteresis curves
when $T_a/T_b=1.0$ phase angle=90

$T_a/T_b=0.75$ Phase angle=90

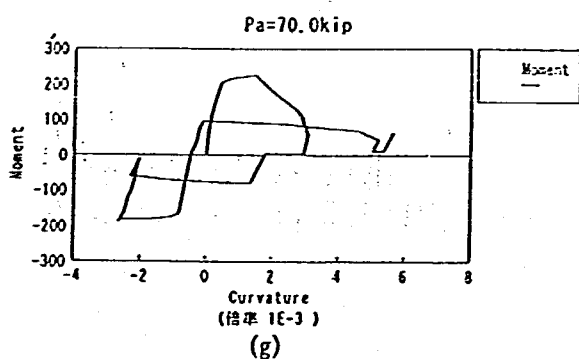
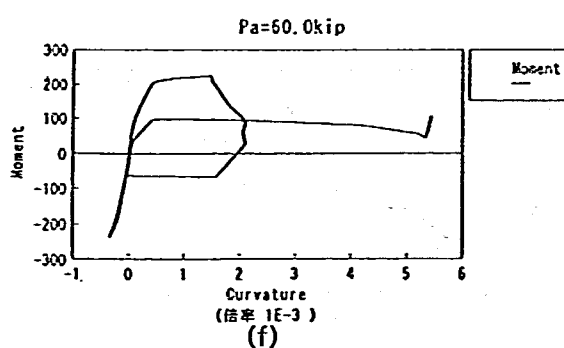
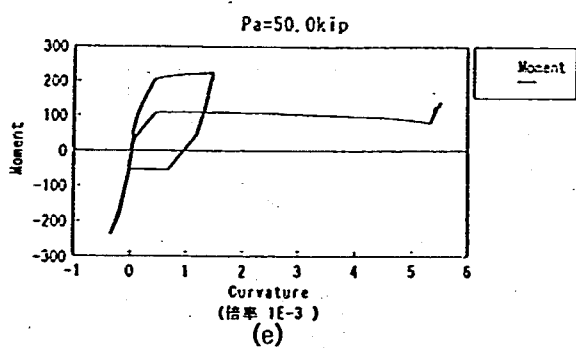
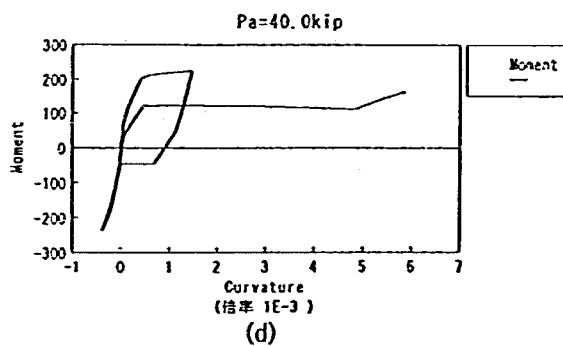
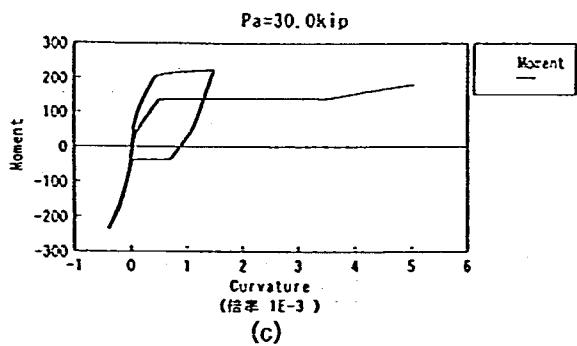
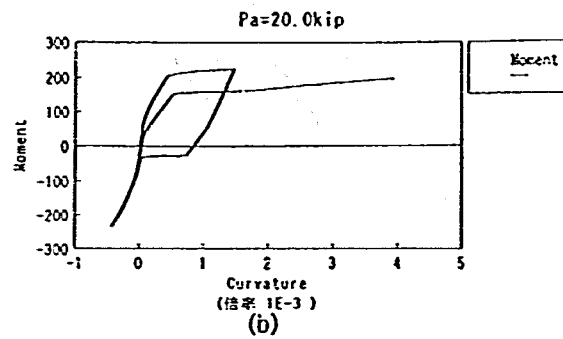
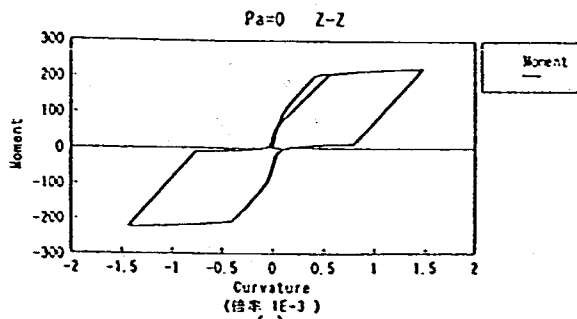


Fig. 42 Hysteresis curves when $T_a/T_b=0.75$ phase angle=90

$T_a/T_b=0.5$ Phase angle=90

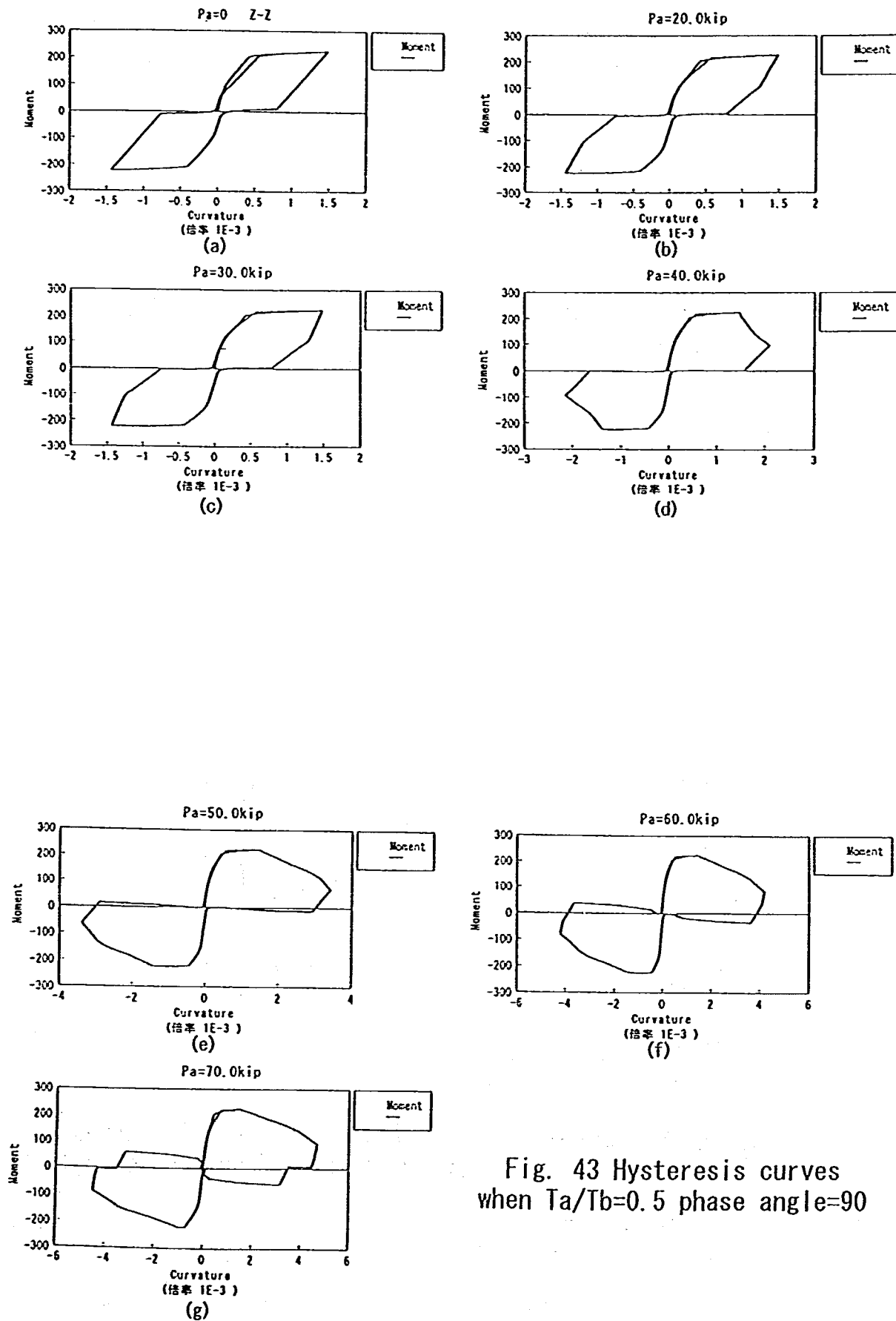


Fig. 43 Hysteresis curves when $T_a/T_b=0.5$ phase angle=90

$T_a/T_b=0.25$ Phase angle =90

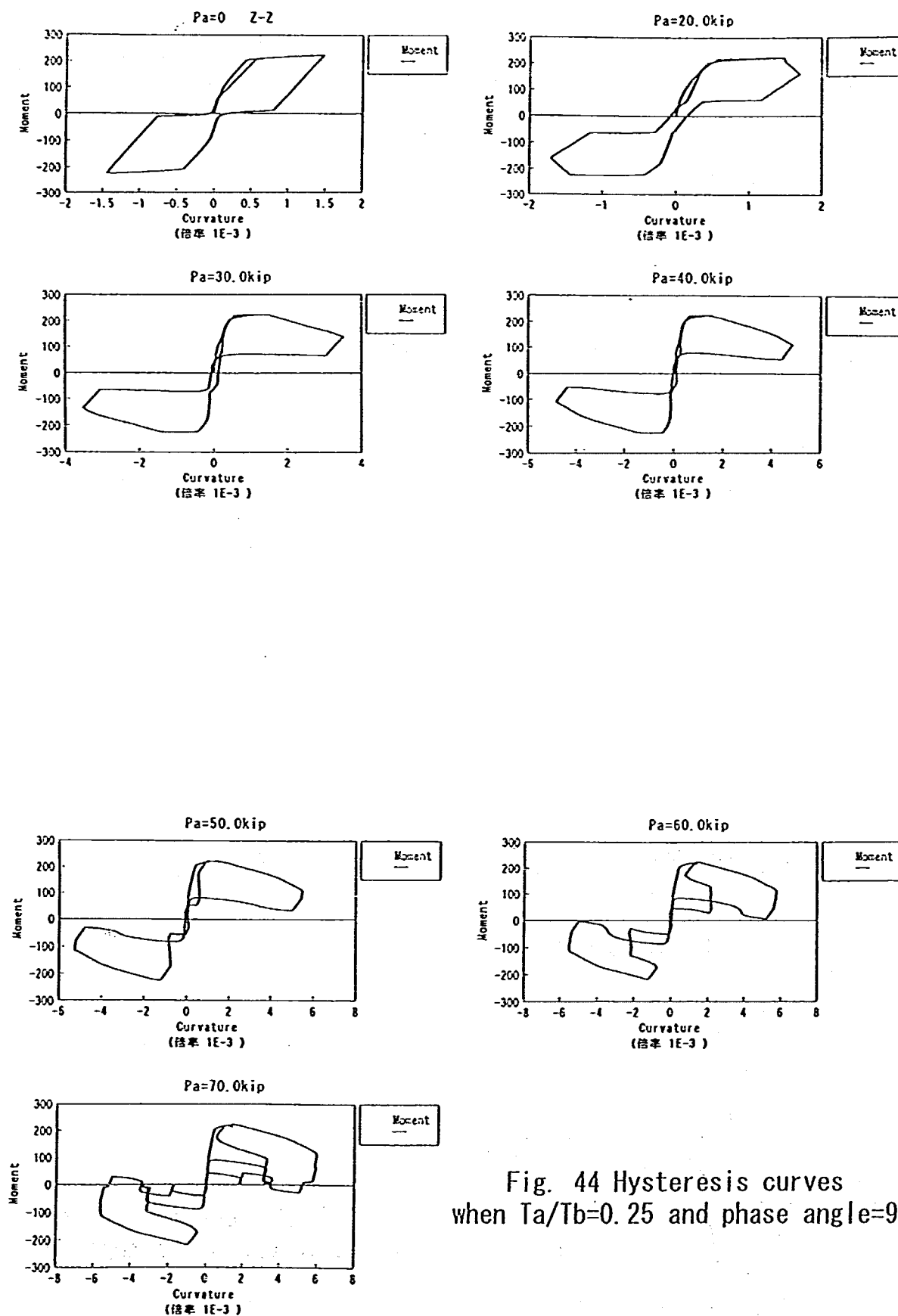


Fig. 44 Hysteresis curves when $T_a/T_b=0.25$ and phase angle=90

$T_a/T_b=1.0$ Phase angle=45

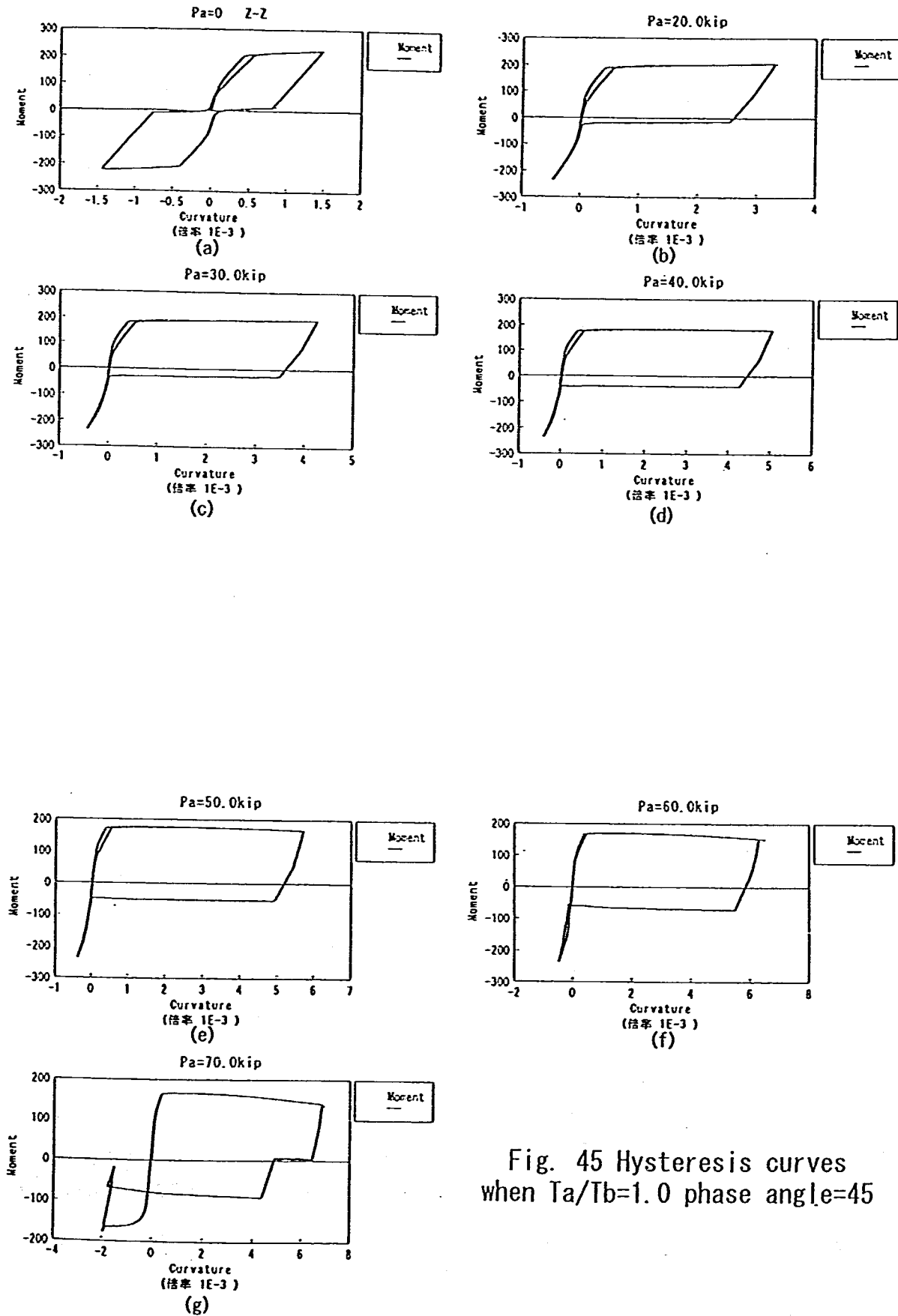


Fig. 45 Hysteresis curves when $T_a/T_b=1.0$ phase angle=45

$T_a/T_b=0.75$ Phase angle=45

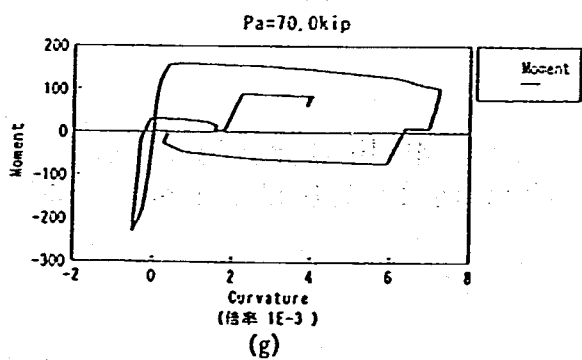
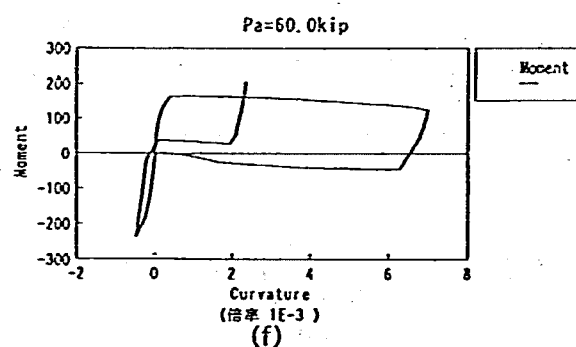
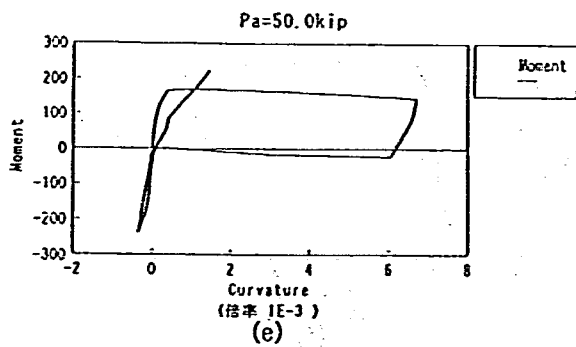
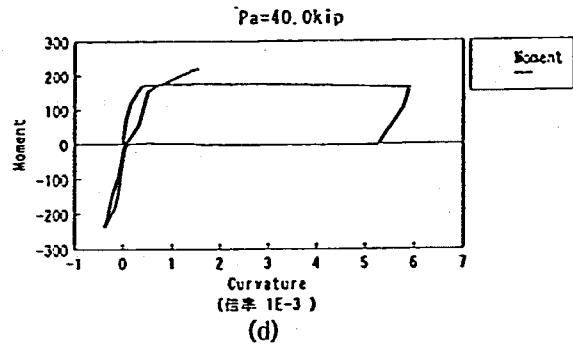
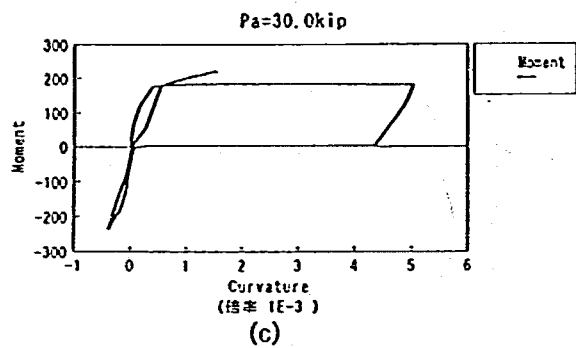
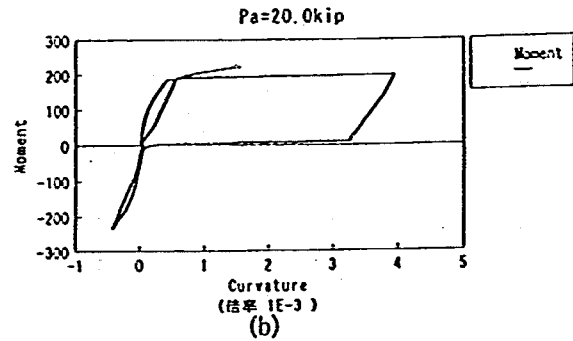
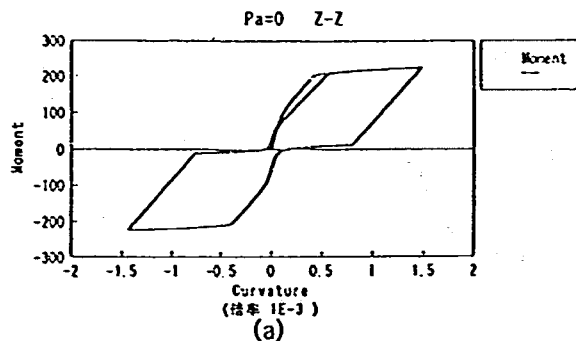


Fig. 46 Hysteresis curves when $T_a/T_b=0.75$ phase angle=45

$T_a/T_b=0.5$ Phase angle=45

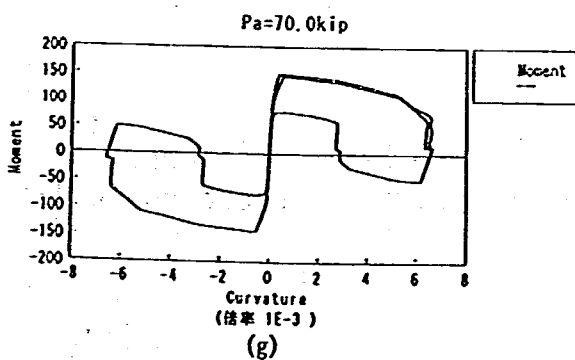
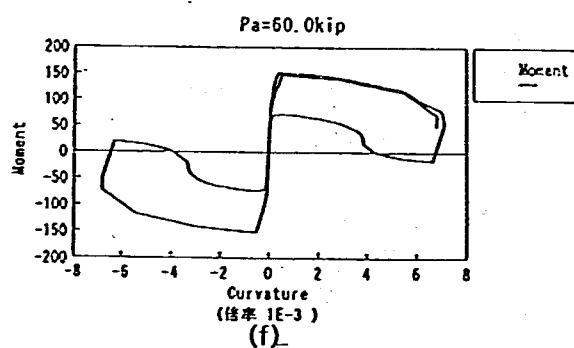
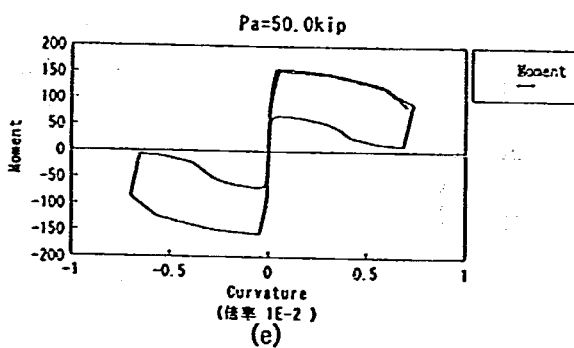
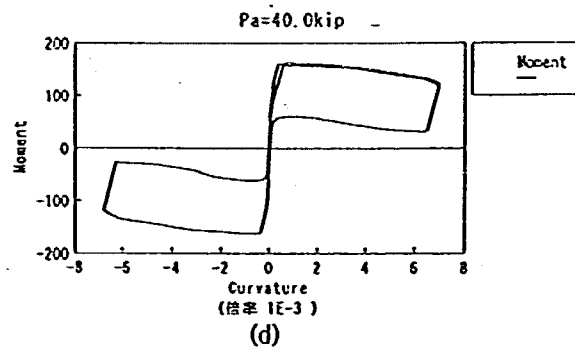
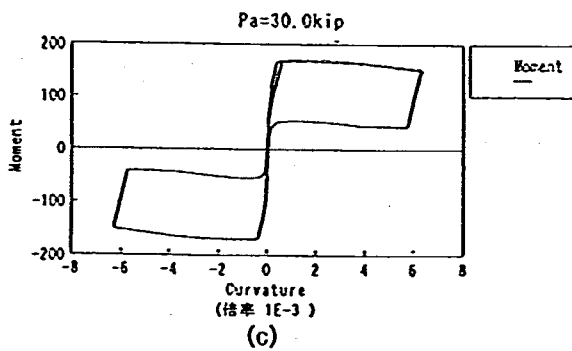
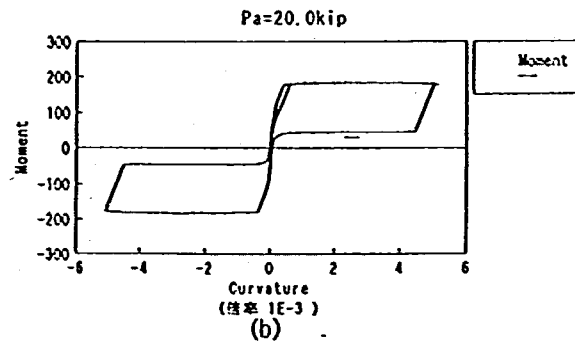
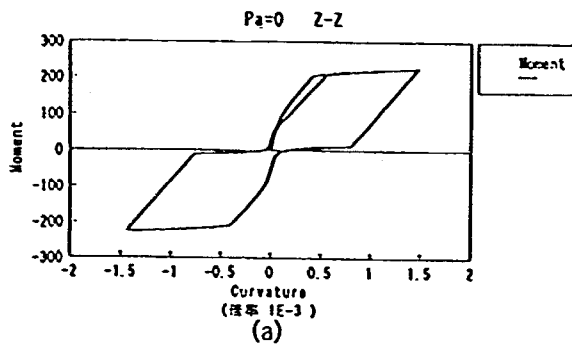


Fig. 47 Hysteresis curves
when $T_a/T_b=0.5$ phase angle=45

$T_a/T_b=0.25$ Phase angle=45

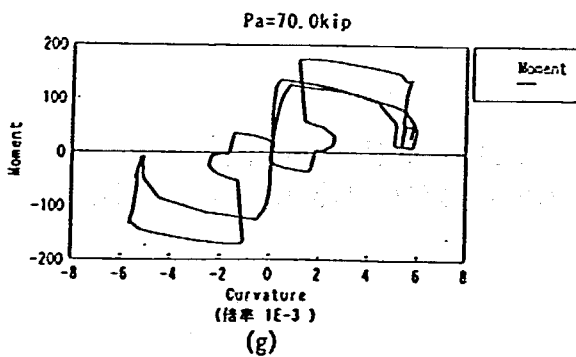
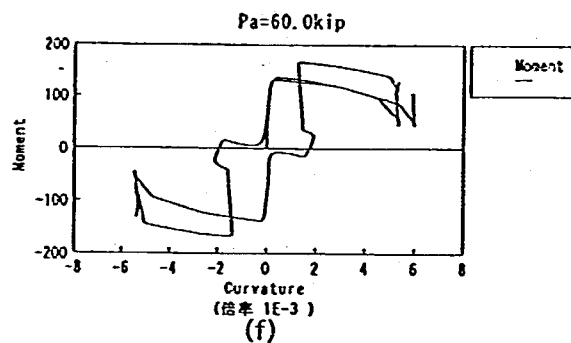
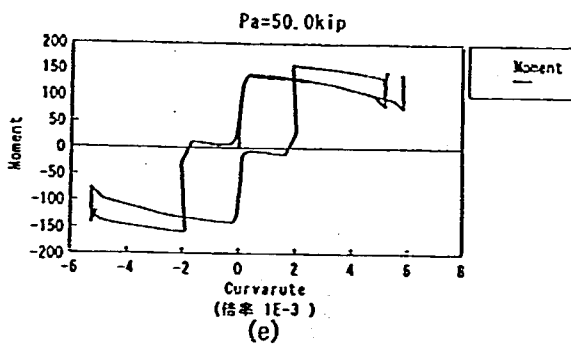
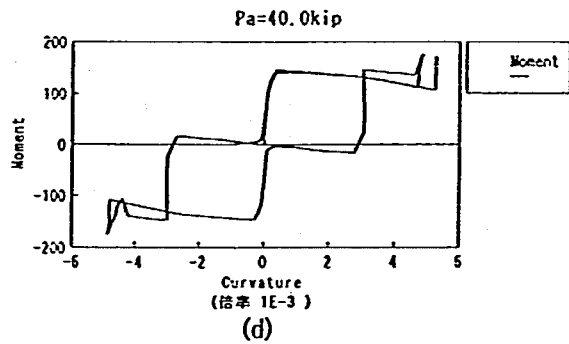
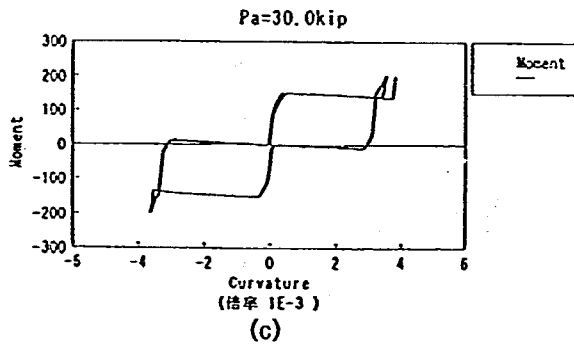
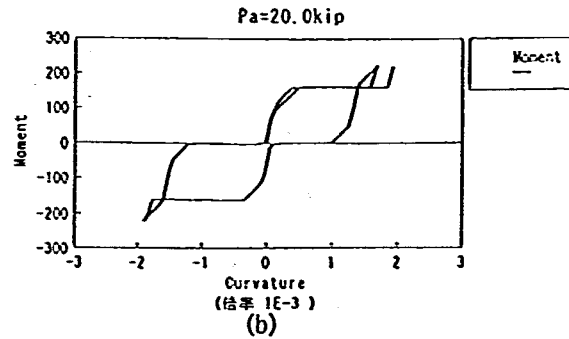
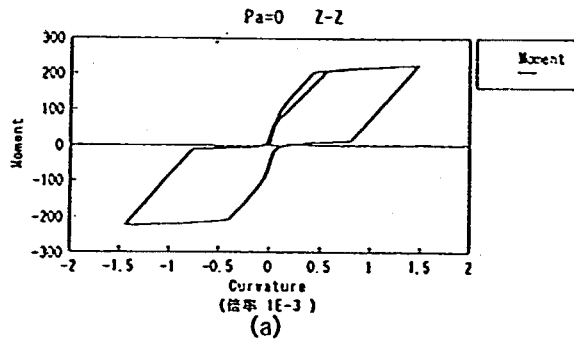


Fig. 48 Hysteresis curves when $T_a/T_b=0.25$ phase angle=45

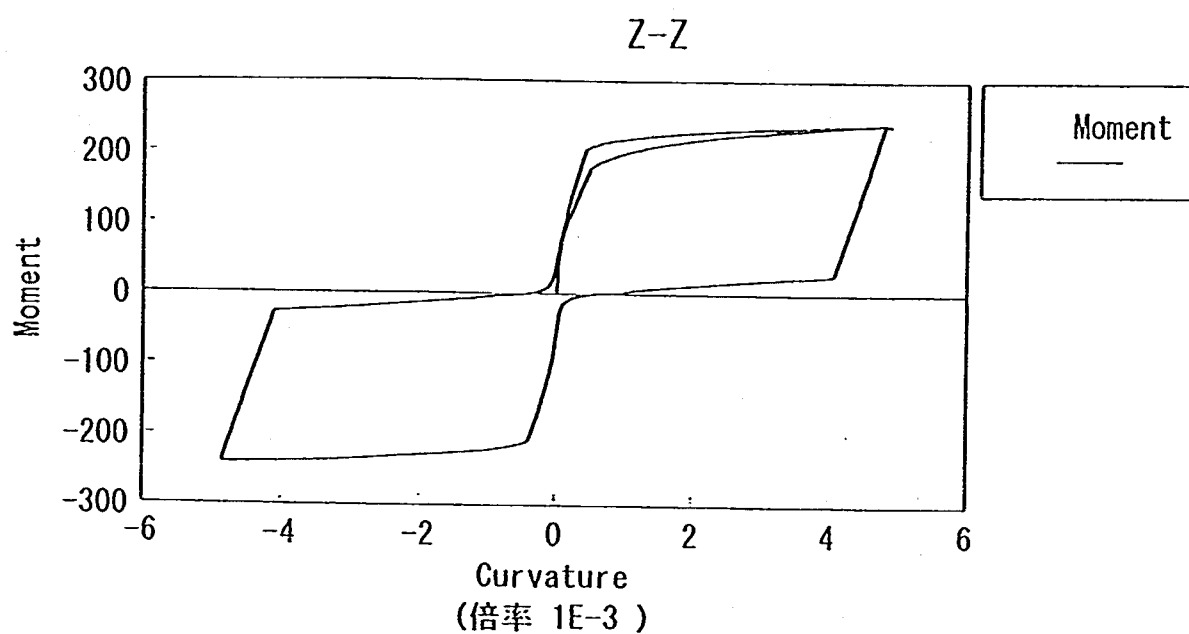


Fig. 49 Hysteresis loop under uniaxial bending along Z-axis

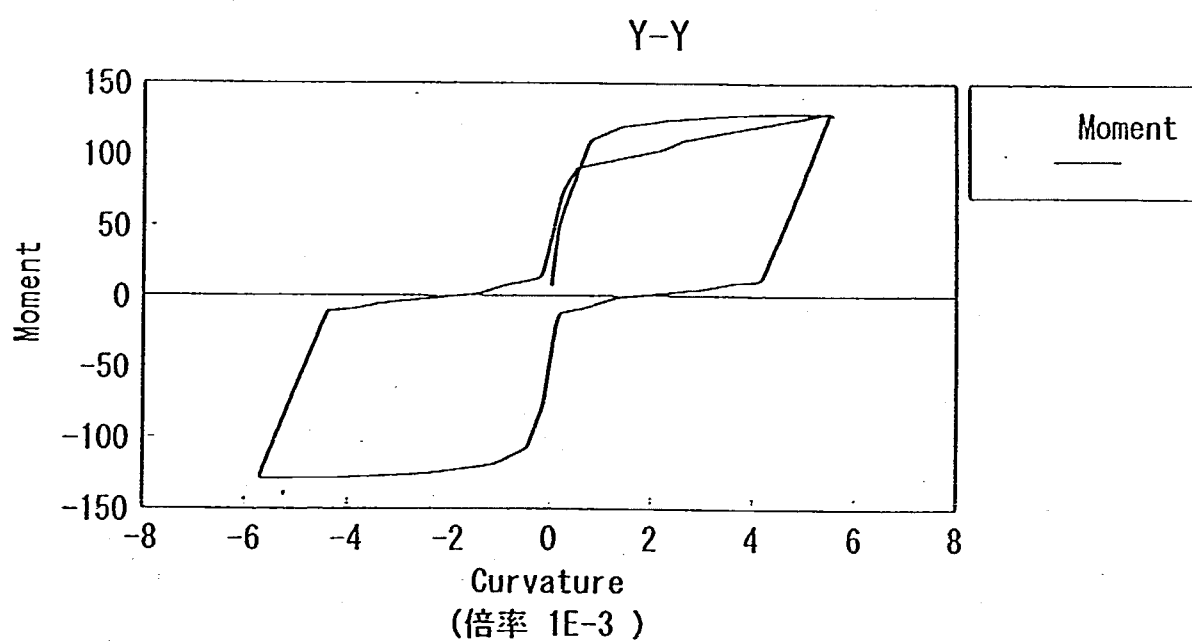


Fig. 50 Hysteresis loop under uniaxial bending along Y-axis

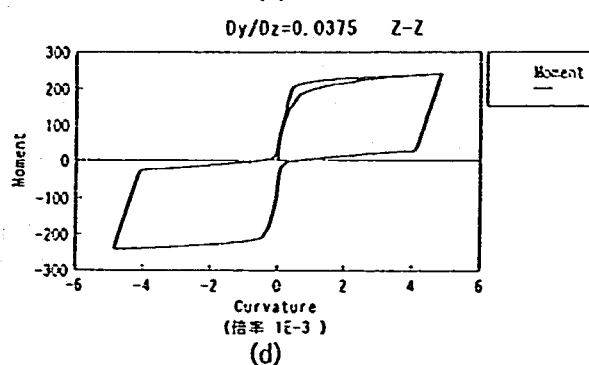
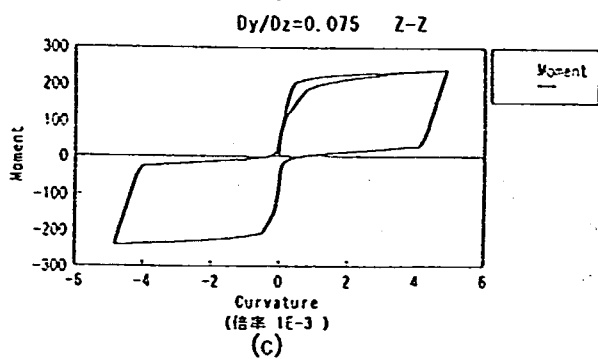
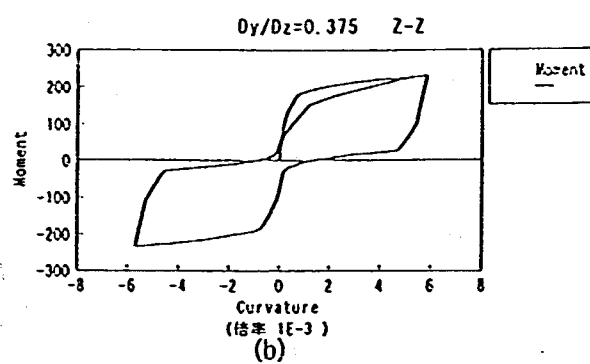
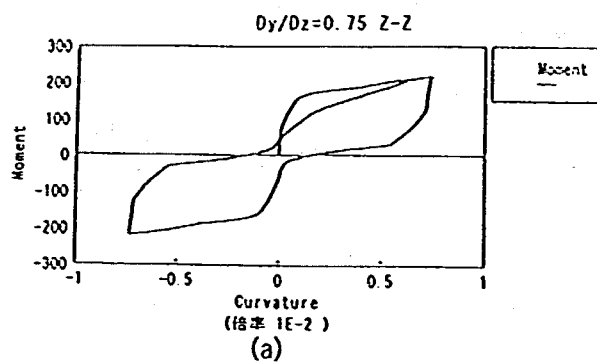


Fig. 51 Hysteresis loops in Z direction for $Ty/Tz=1.0$

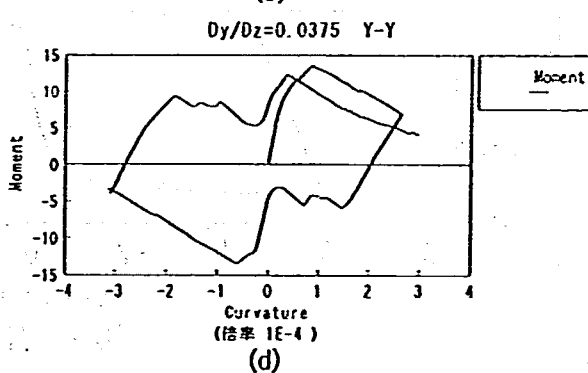
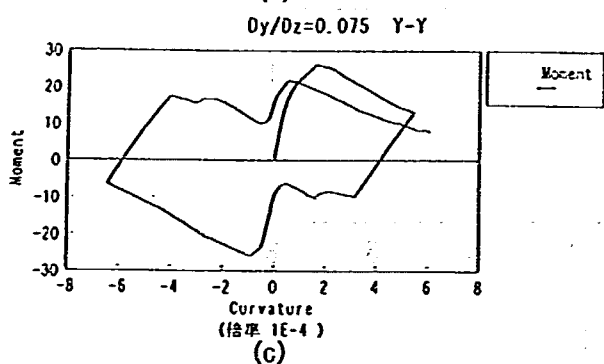
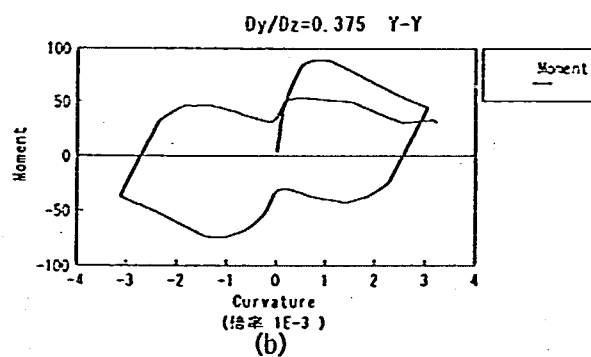
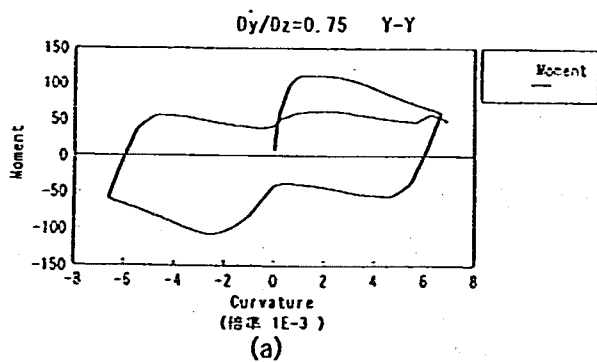


Fig. 52 Hysteresis loops in Y direction for $Ty/Tz=1.0$

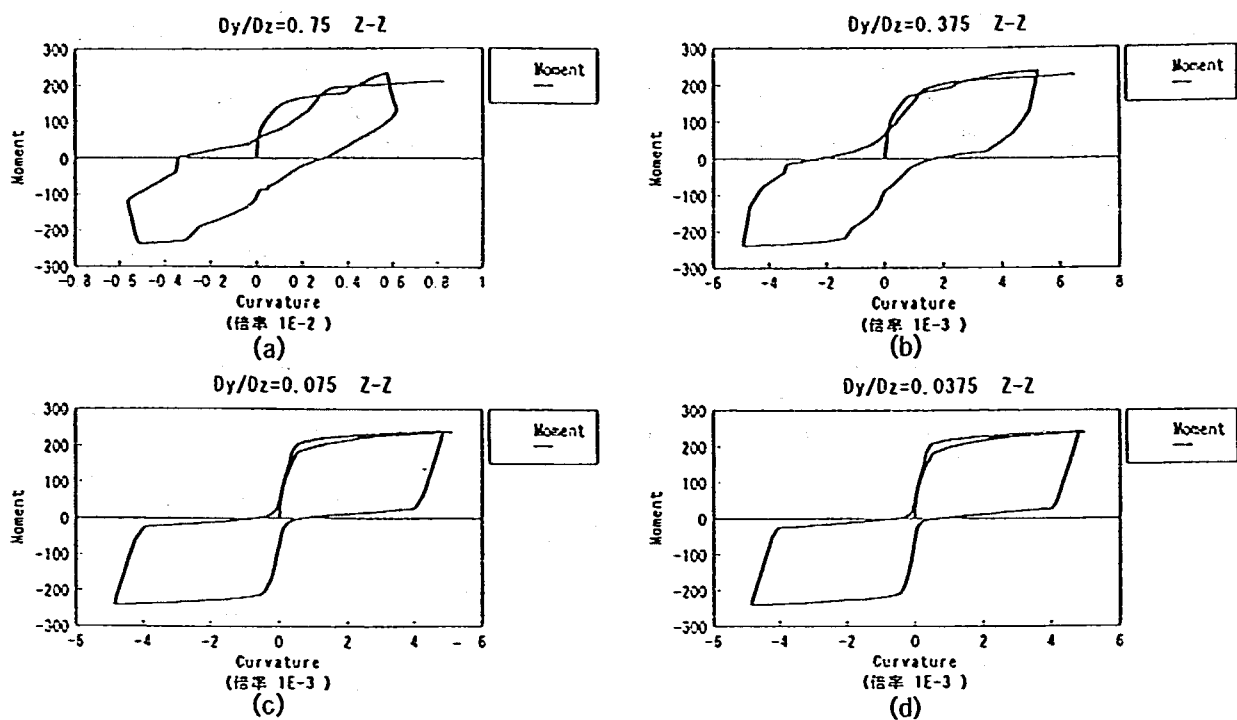


Fig. 53 Hysteresis loops in Z direction for $T_y/T_z=0.75$

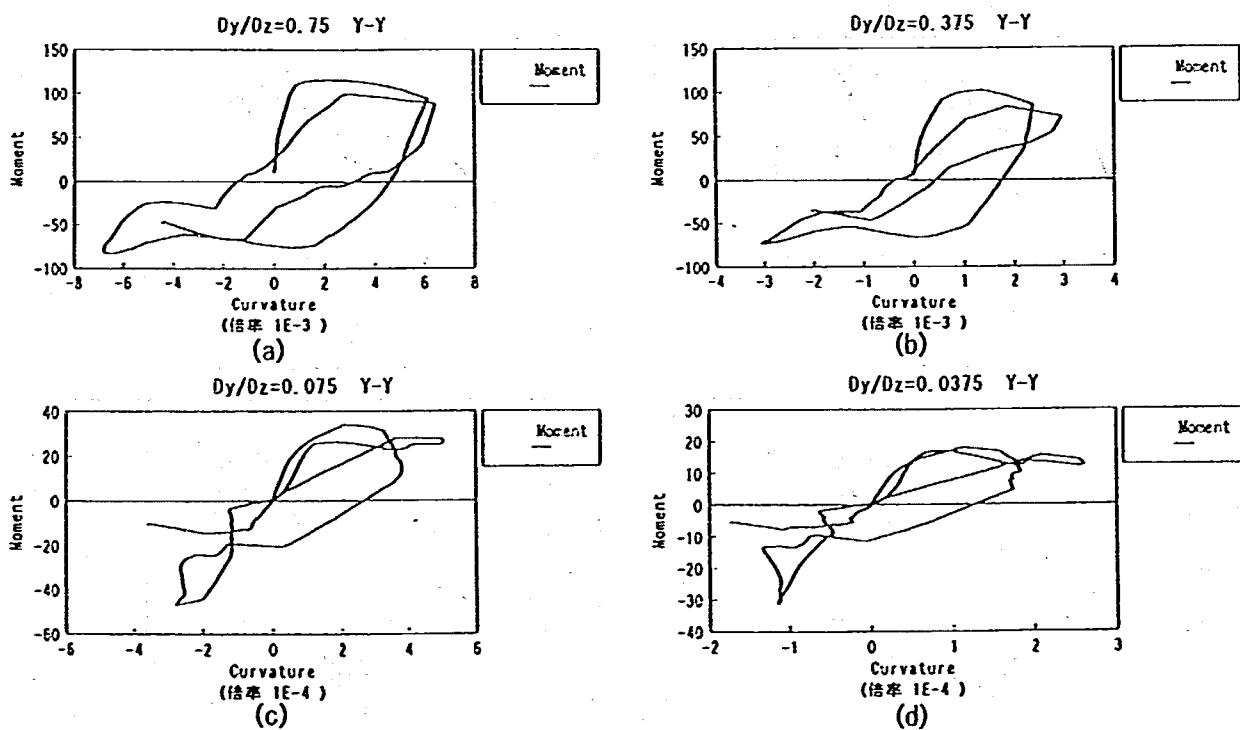


Fig. 54 Hysteresis loops in Y direction for $T_y/T_z=0.75$

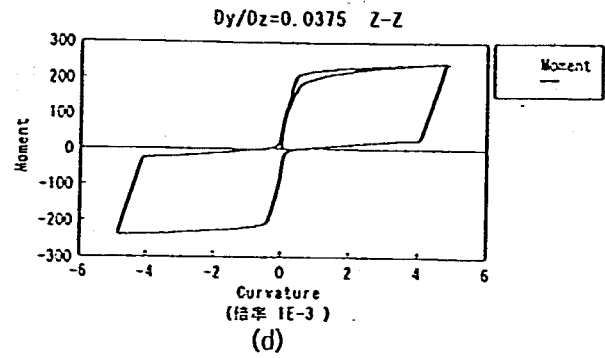
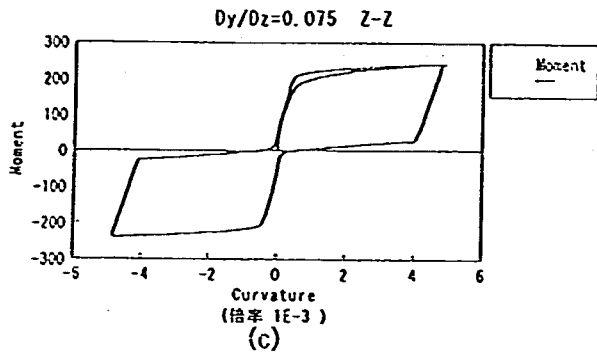
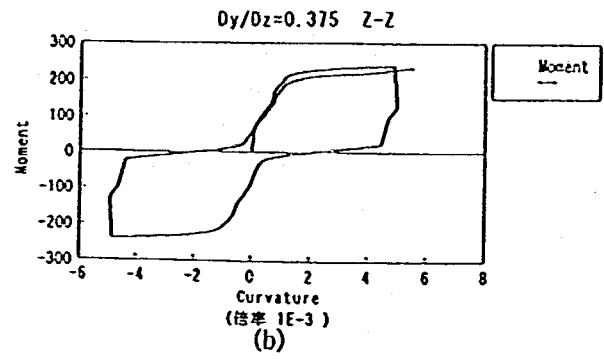
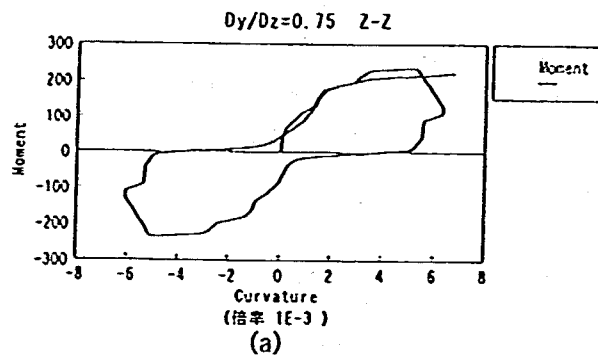


Fig. 55 Hysteresis loops in Z direction for $T_y/T_z=0.5$

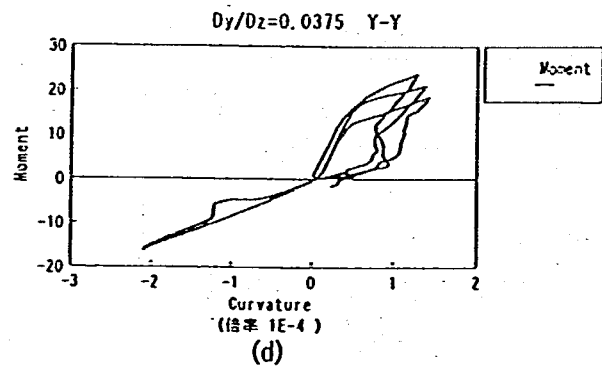
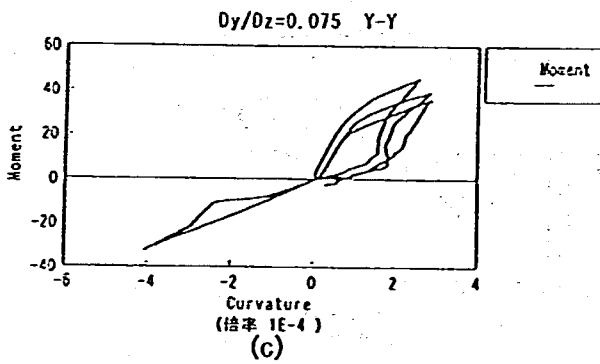
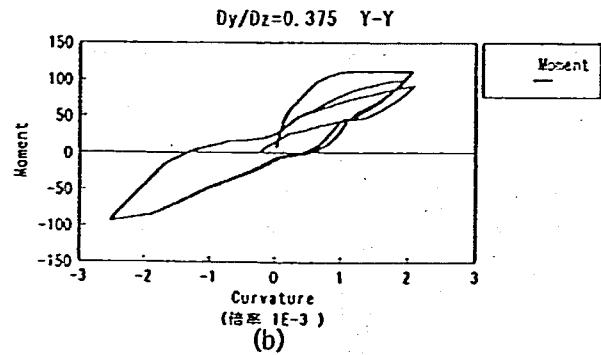
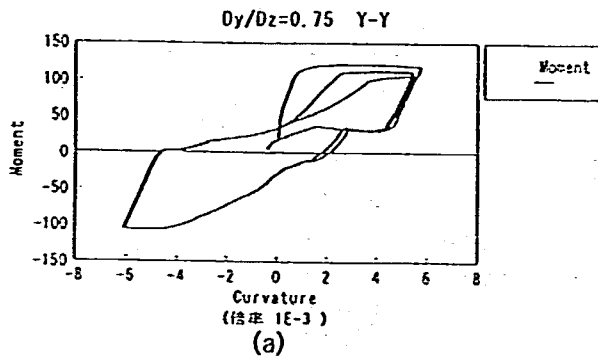


Fig. 56 Hysteresis loops in Y direction for $T_y/T_z=0.5$

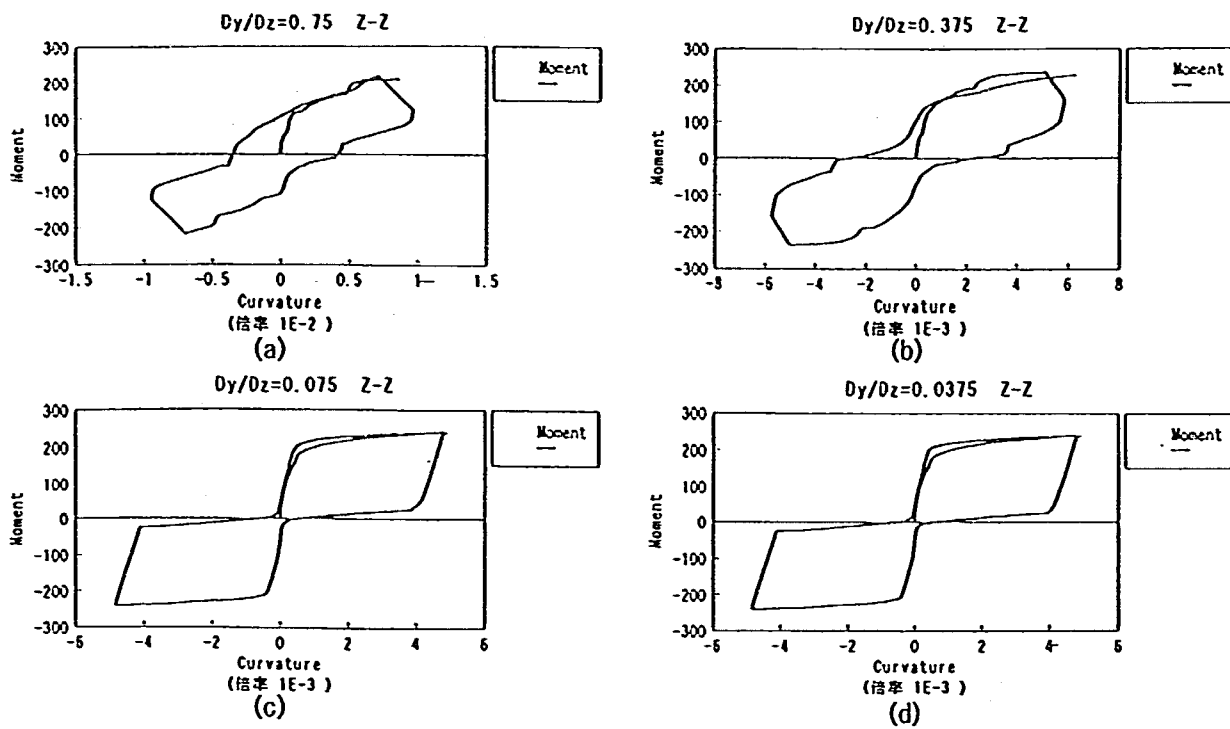


Fig. 57 Hysteresis loops in Z direction for $T_y/T_z=0.25$

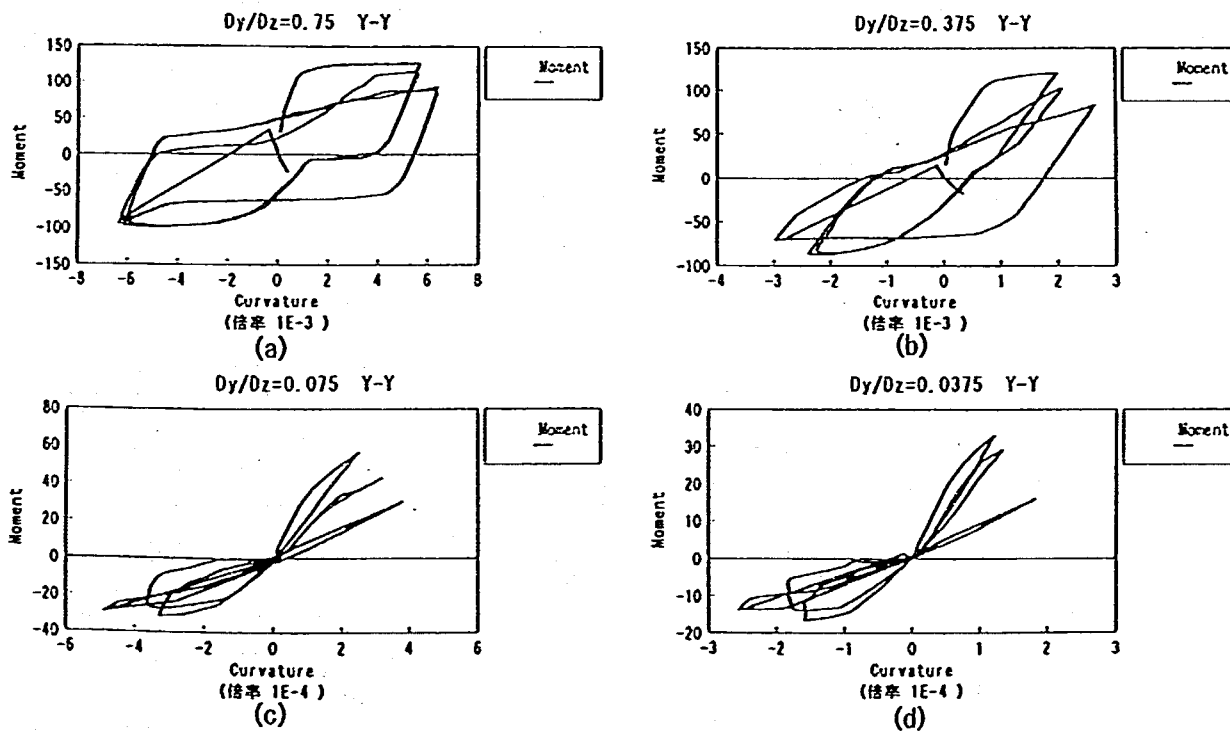


Fig. 58 Hysteresis loops in Y direction for $T_y/T_z=0.25$

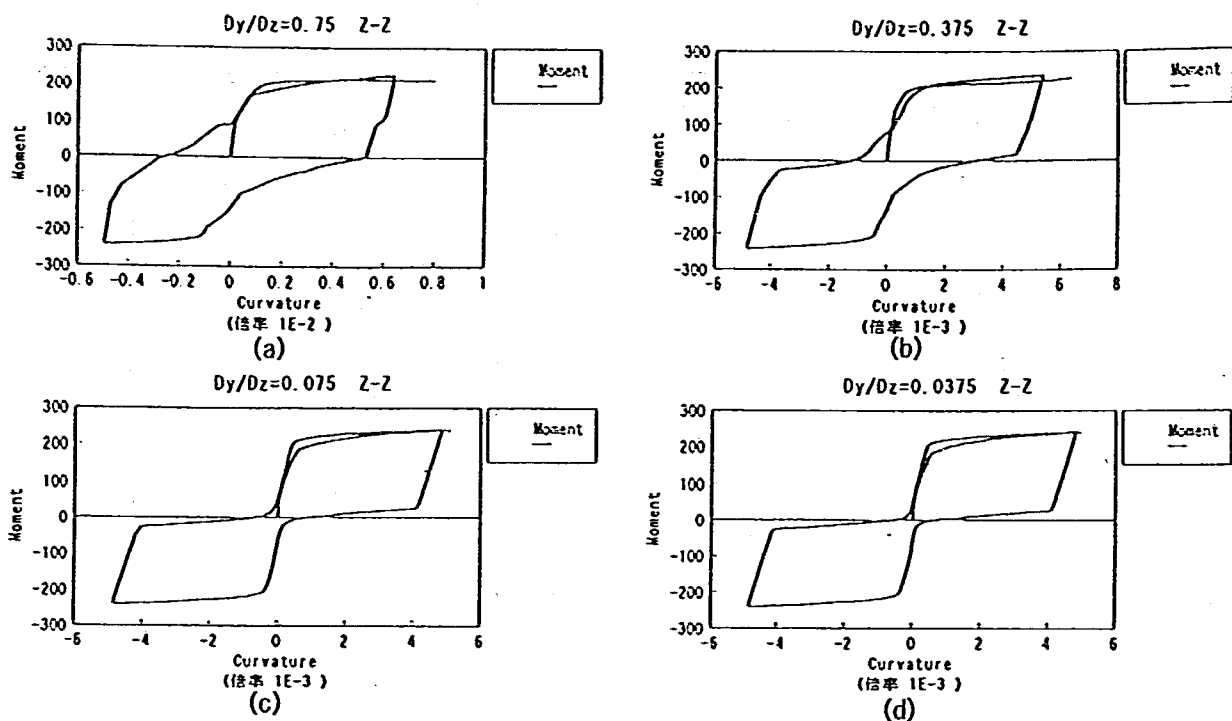


Fig. 59 Hysteresis loops in Z direction for $T_y/T_z=1.5$

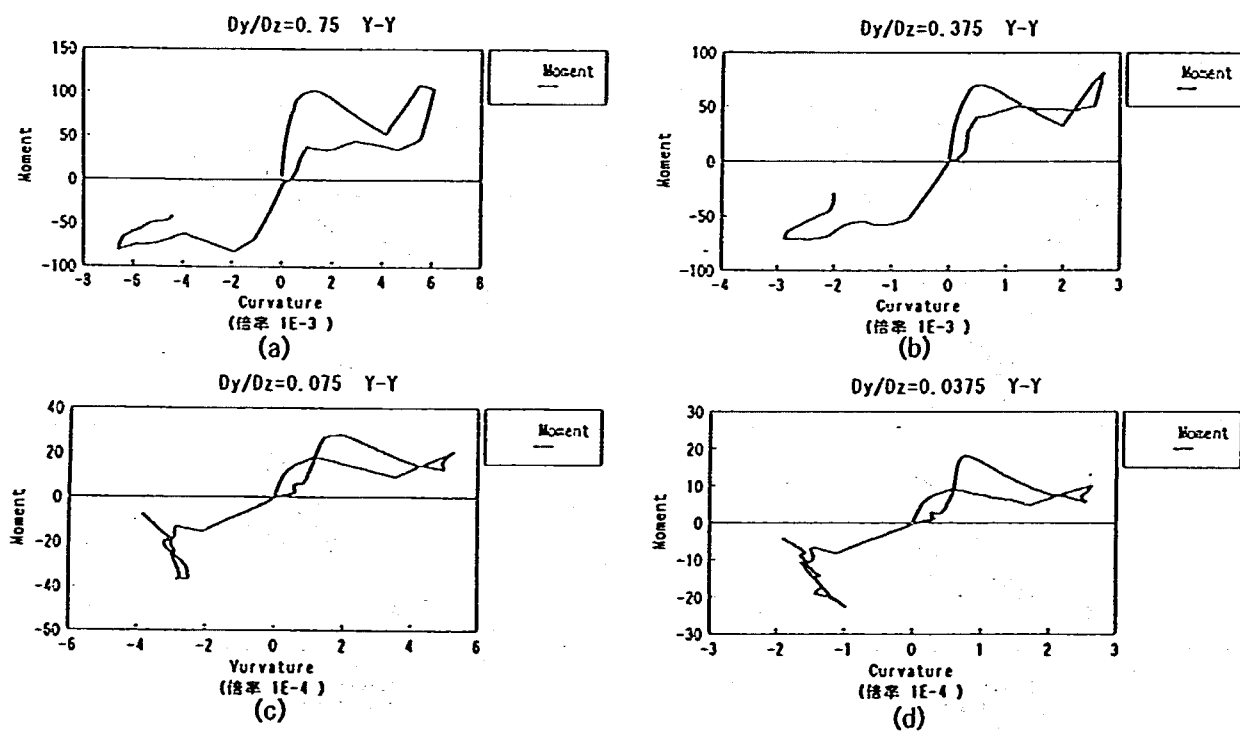


Fig. 60 Hysteresis loops in Y direction for $T_y/T_z=1.5$

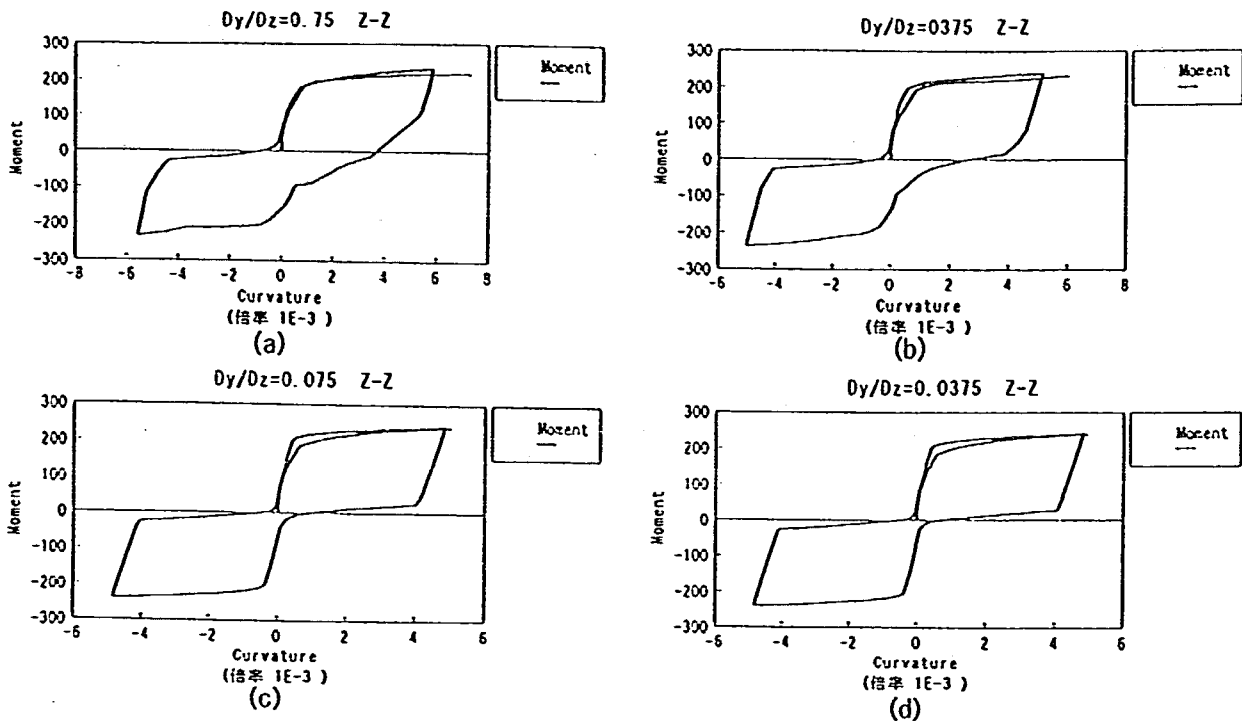


Fig. 61 Hysteresis loops in Z direction for $T_y/T_z=2.0$

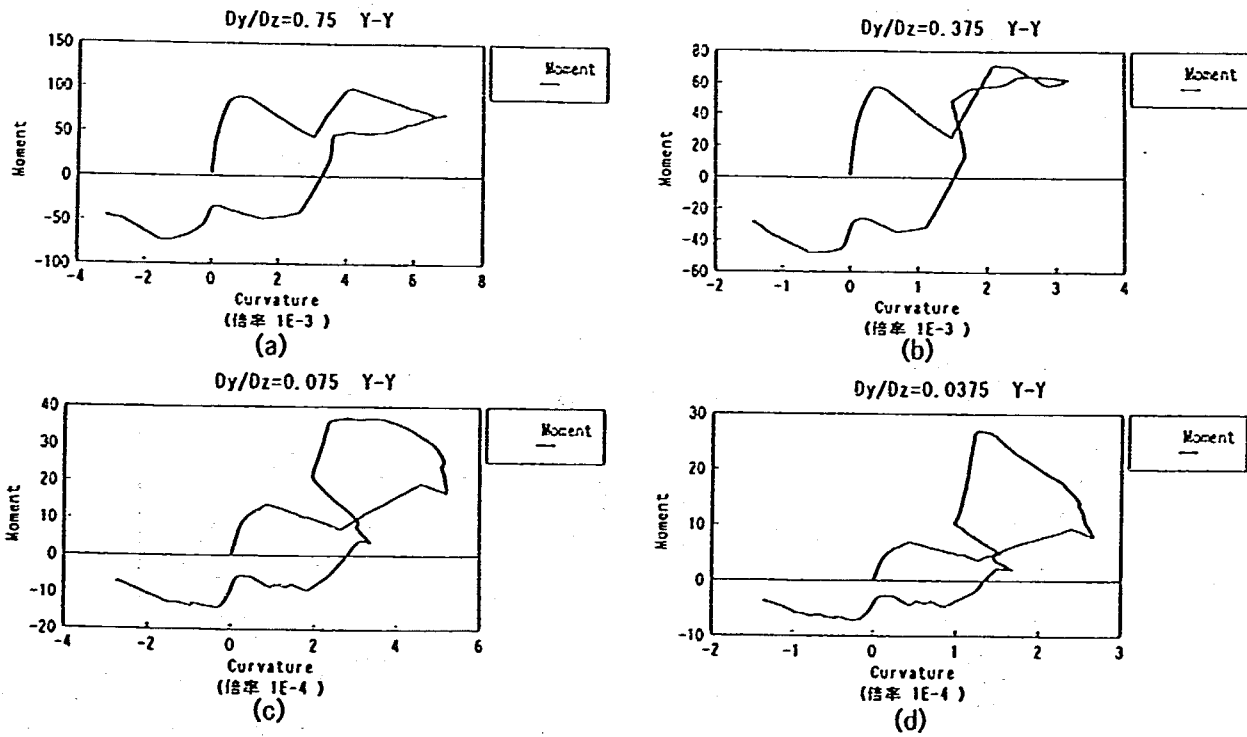


Fig. 62 Hysteresis loops in Y direction for $T_y/T_z=2.0$

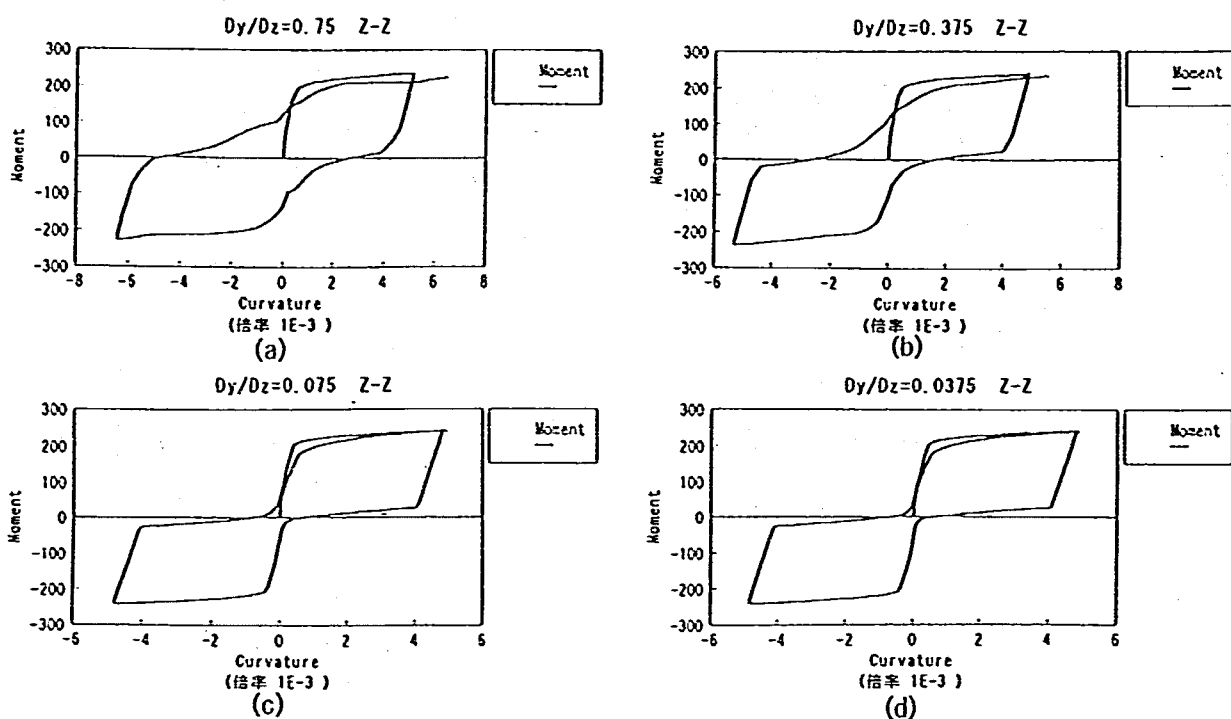


Fig. 63 Hysteresis loops in Z direction for $T_y/T_z=4.0$

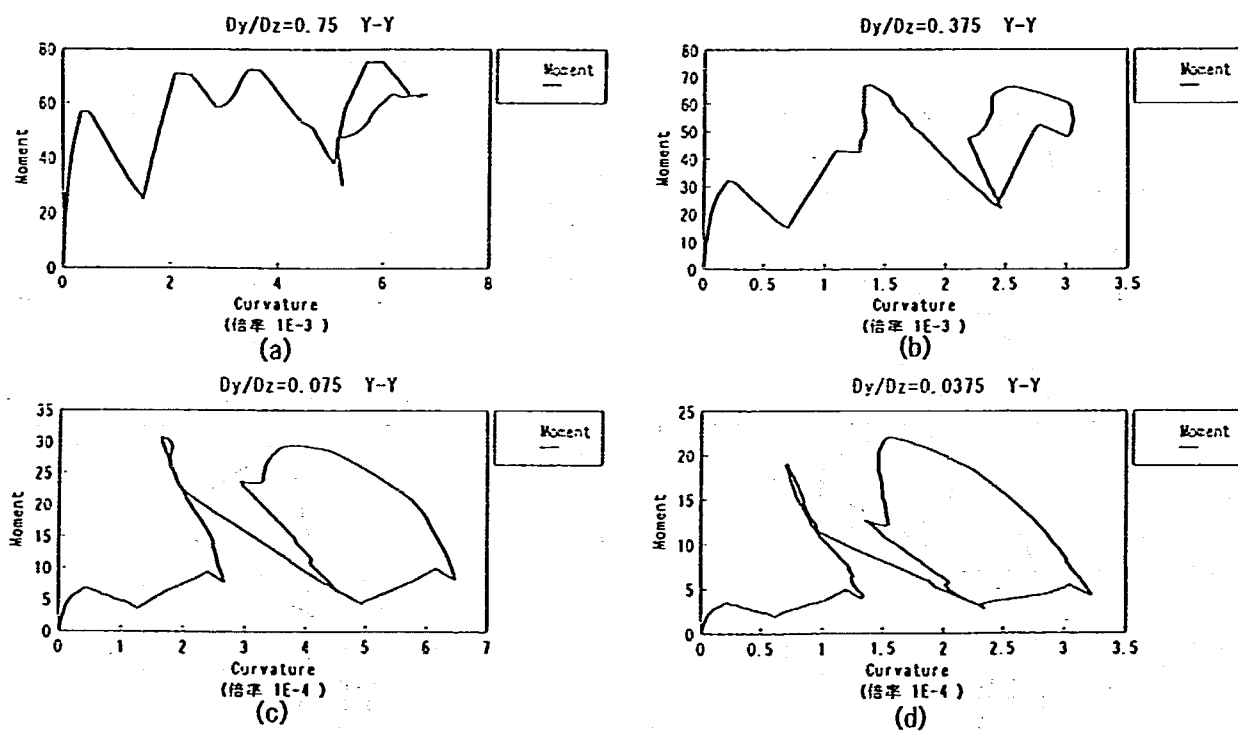


Fig. 64 Hysteresis loops in Y direction for $T_y/T_z=4.0$

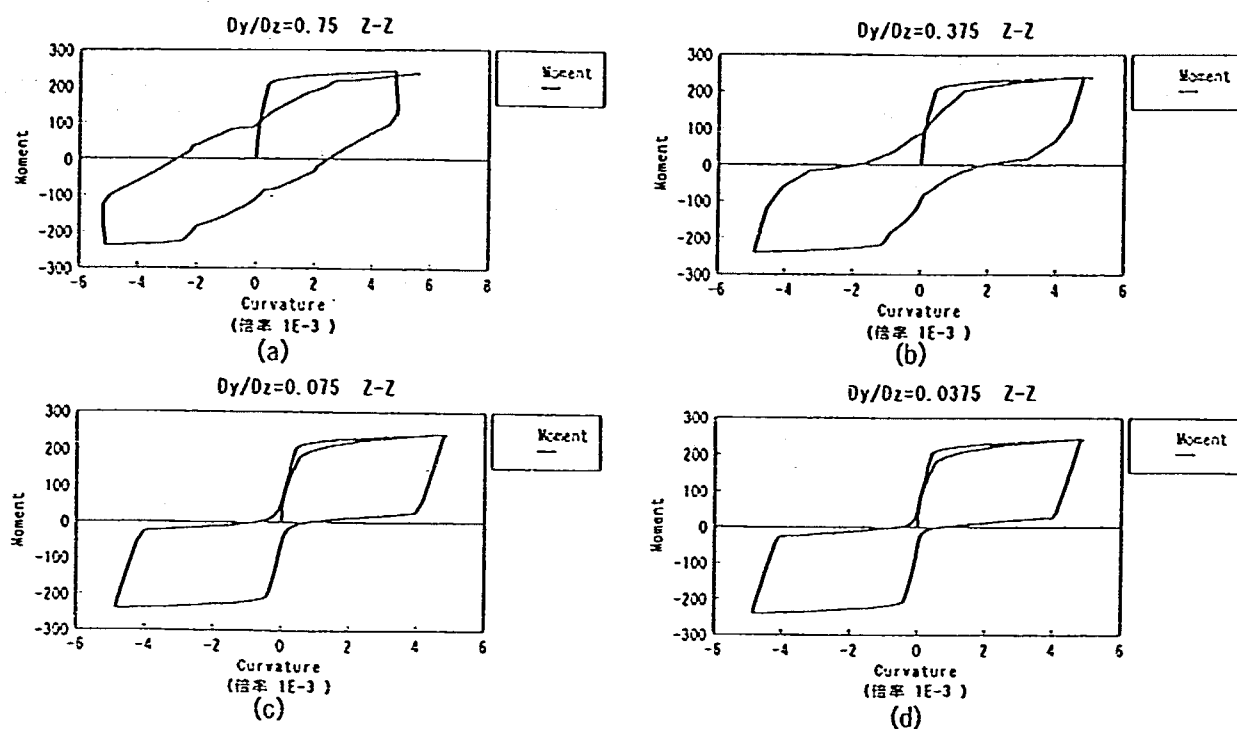


Fig. 65 Hysteresis loops in Z direction for $T_y/T_z=1.0$ and phase angle=90

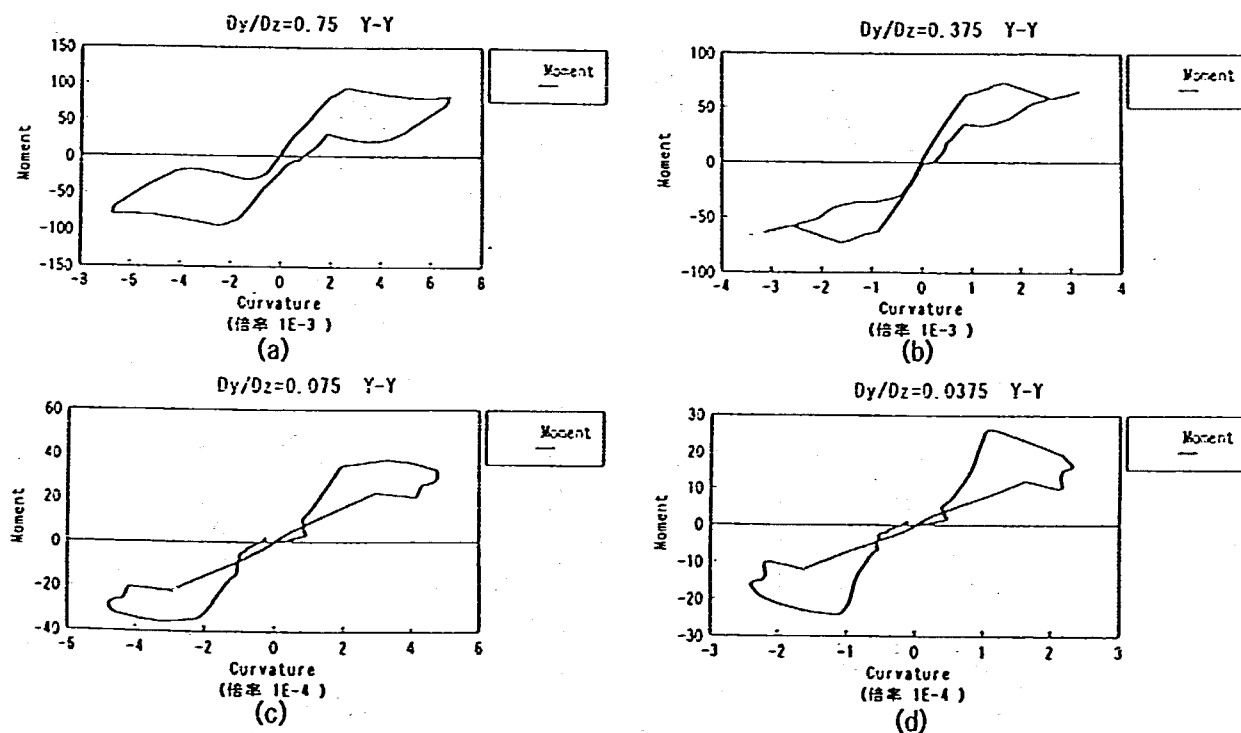


Fig. 66 Hysteresis loops in Y direction for $T_y/T_z=1.0$ and phase angle=90

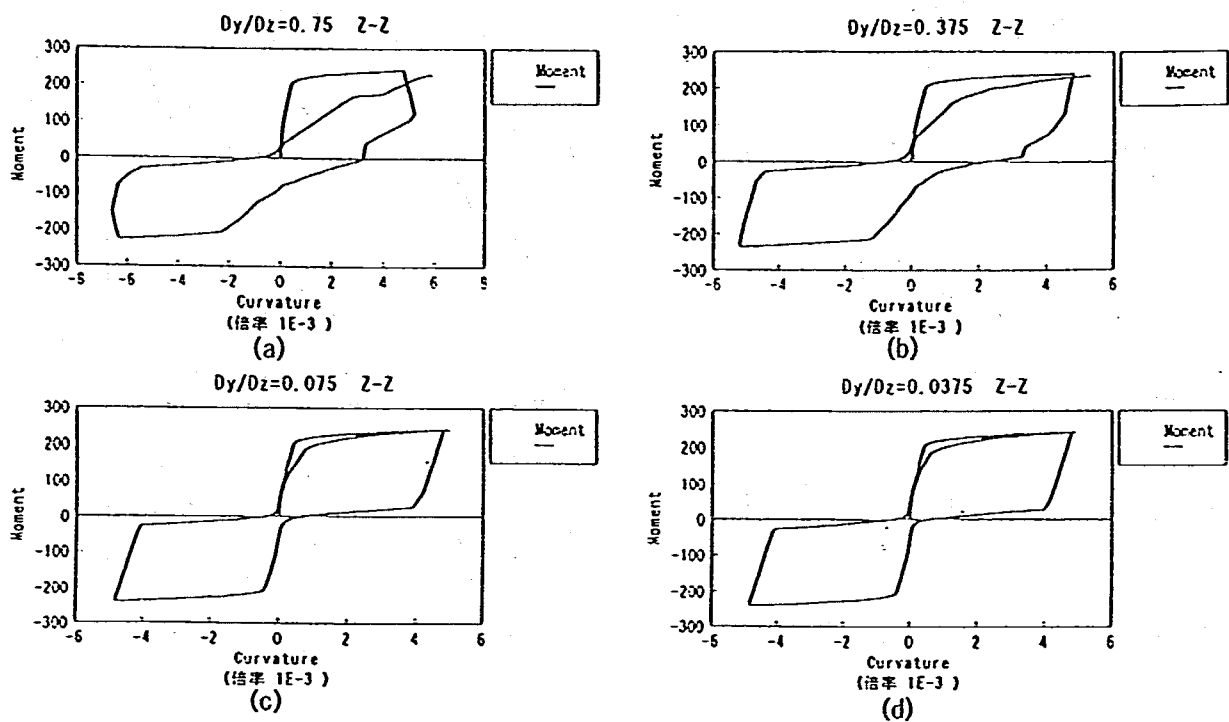


Fig. 67 Hysteresis loops in Z direction for $T_y/T_z=0.75$ and phase angle $=90$

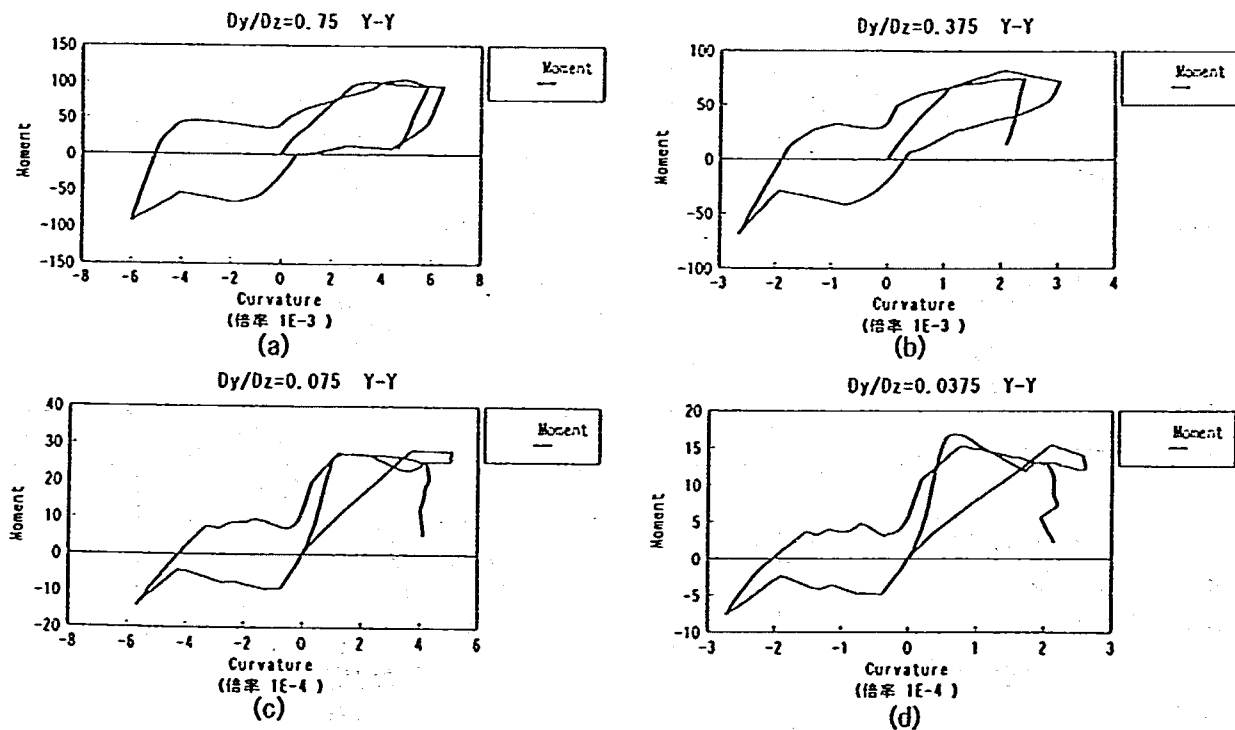


Fig. 68 Hysteresis loops in Y direction for $T_y/T_z=0.75$ and phase angle $=90$

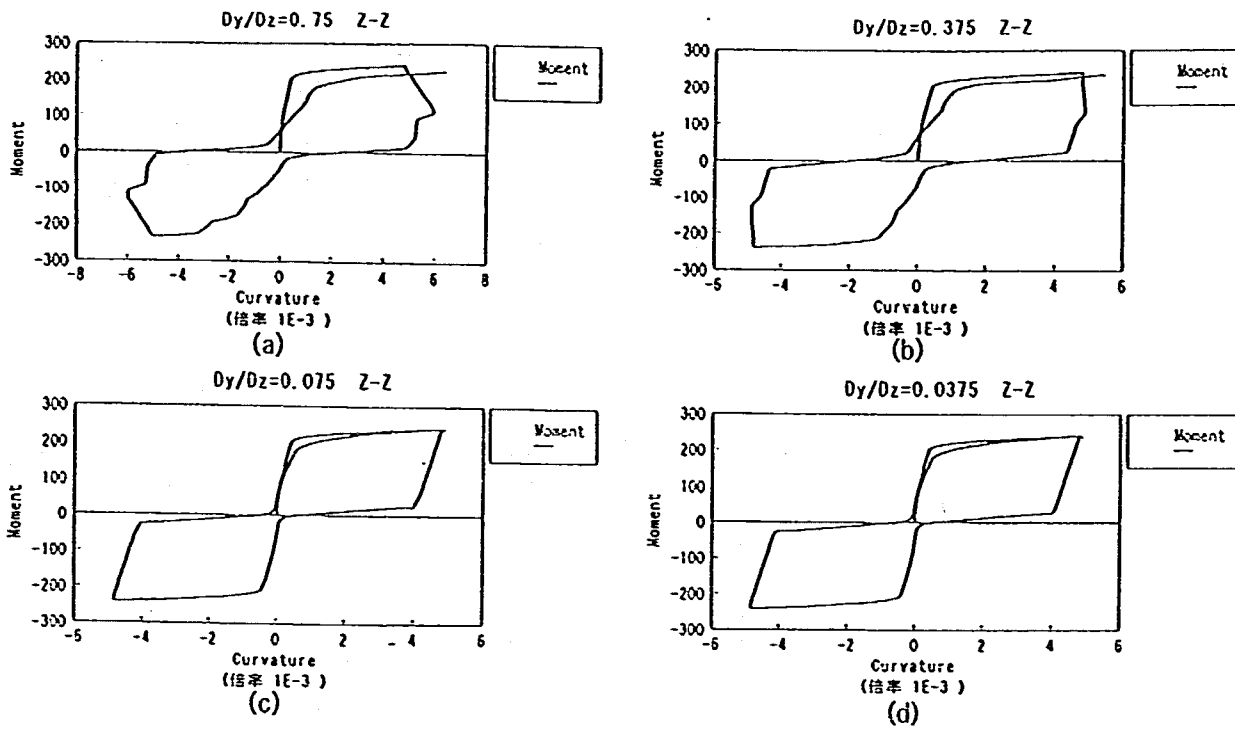


Fig. 69 Hysteresis loops in Z direction for $T_y/T_z=0.5$ and phase angle $=90$

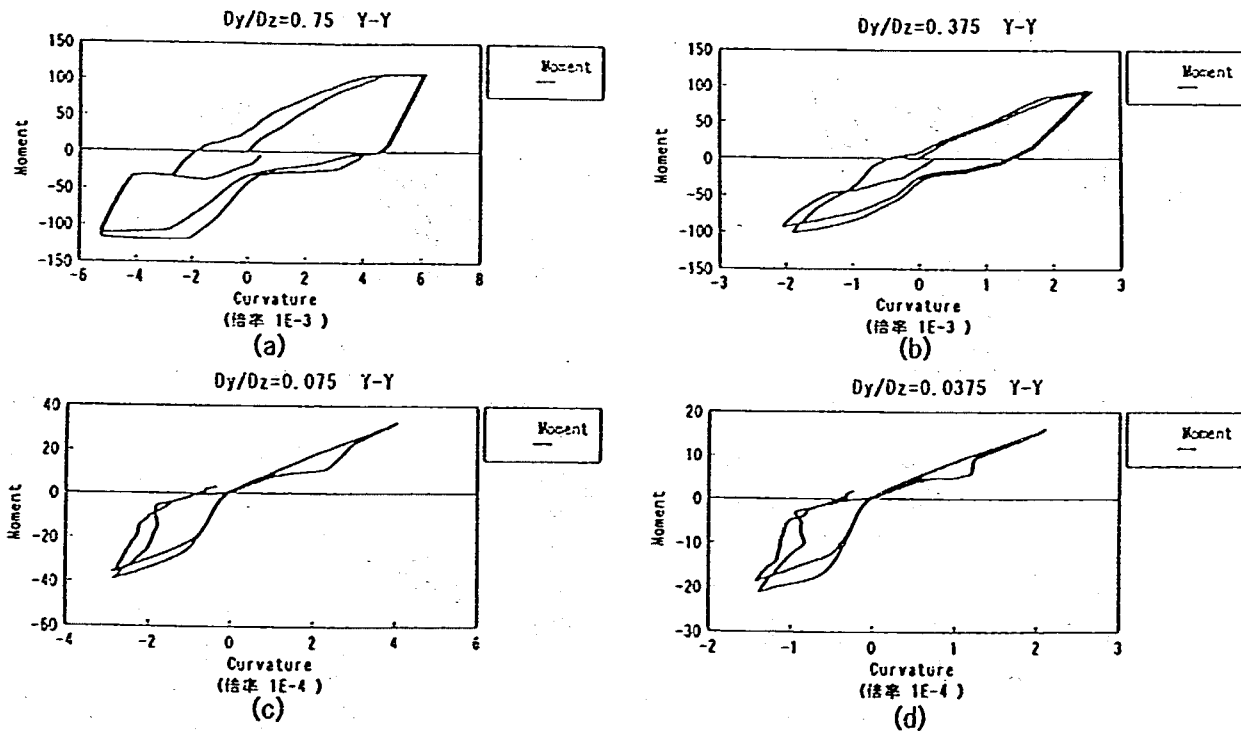


Fig. 70 Hysteresis loops in Y direction for $T_y/T_z=0.5$ and phase angle $=90$

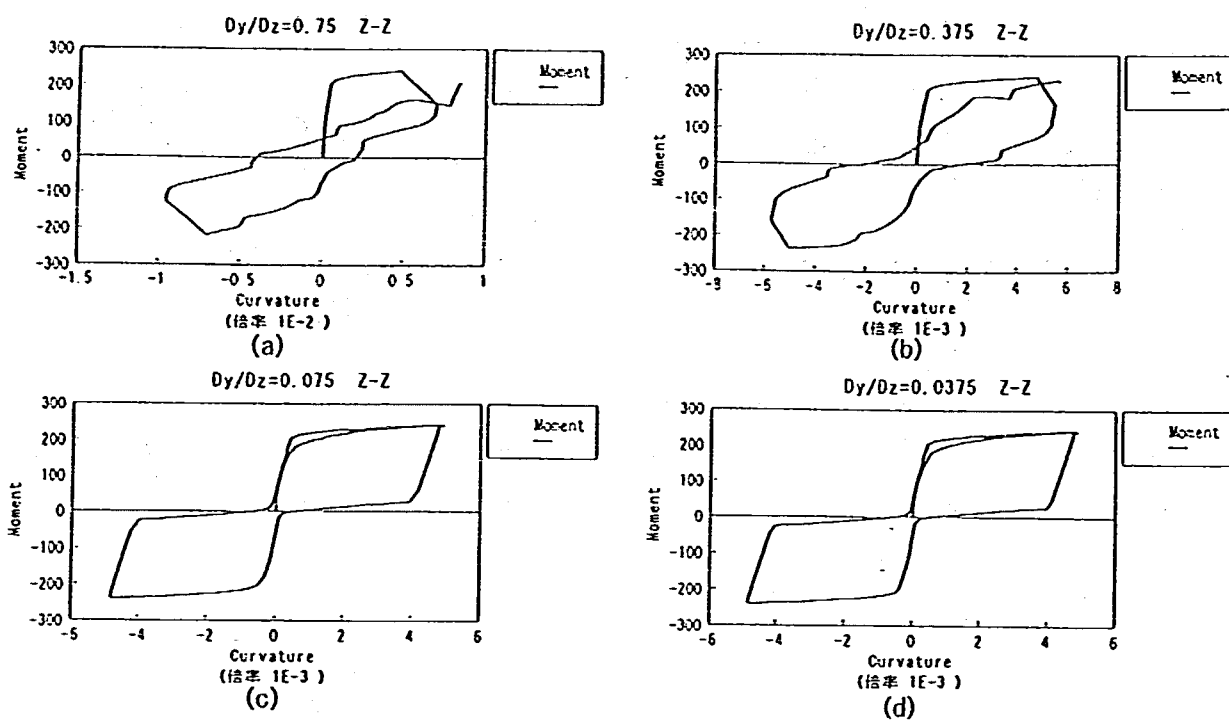


Fig. 71 Hysteresis loops in Z direction for $T_y/T_z=0.25$ and phase angle $=90^\circ$

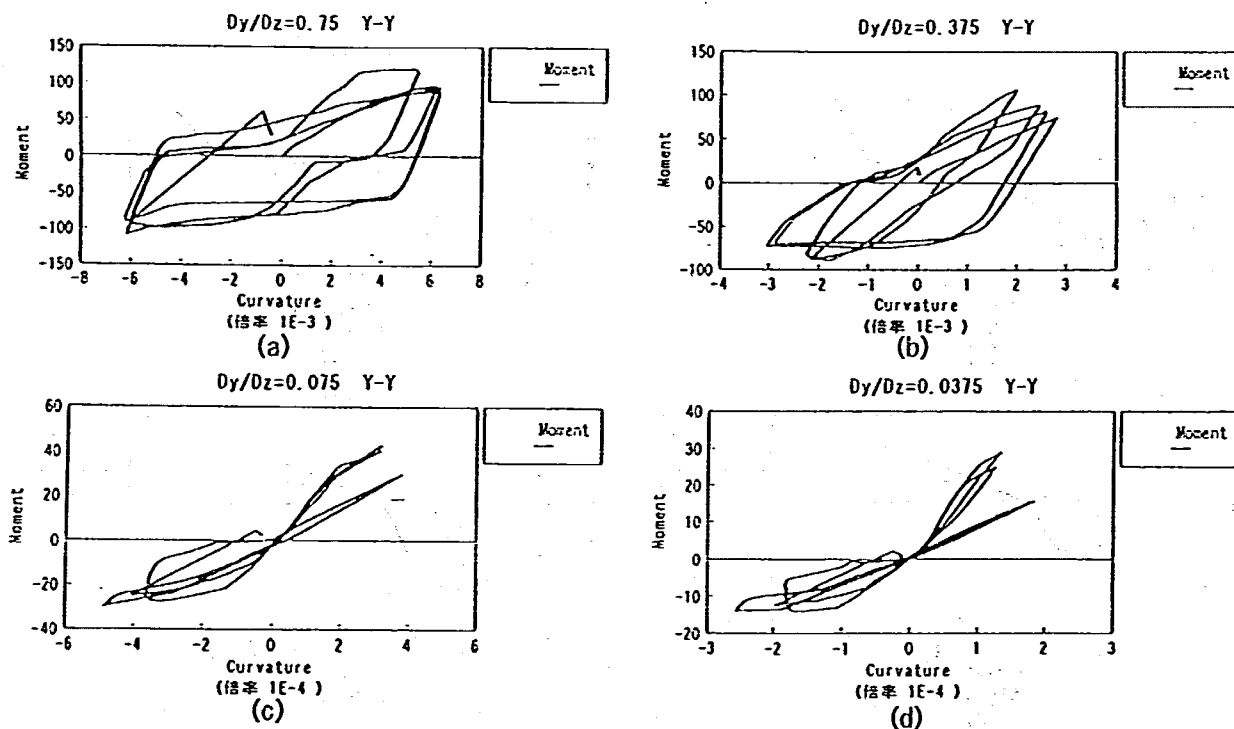


Fig. 72 Hysteresis loops in Y direction for $T_y/T_z=0.25$ and phase angle $=90^\circ$

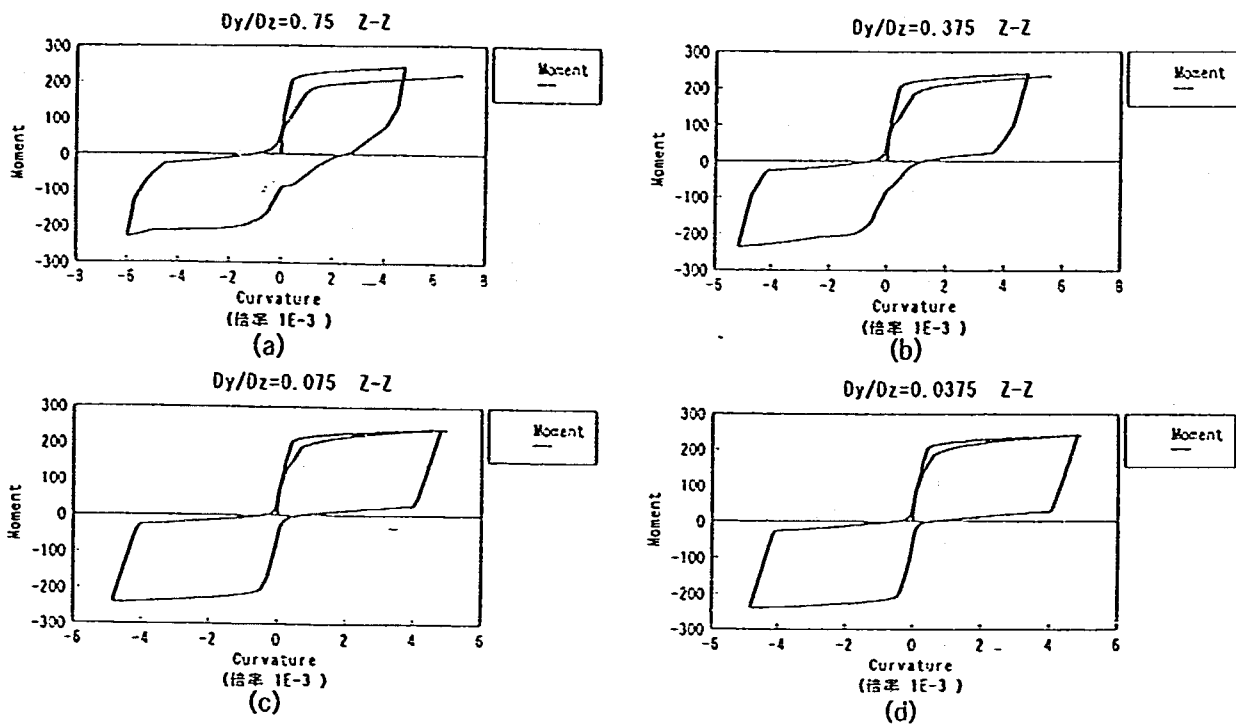


Fig. 73 Hysteresis loops in Z direction for $T_y/T_z=1.5$ and phase angle $=90^\circ$

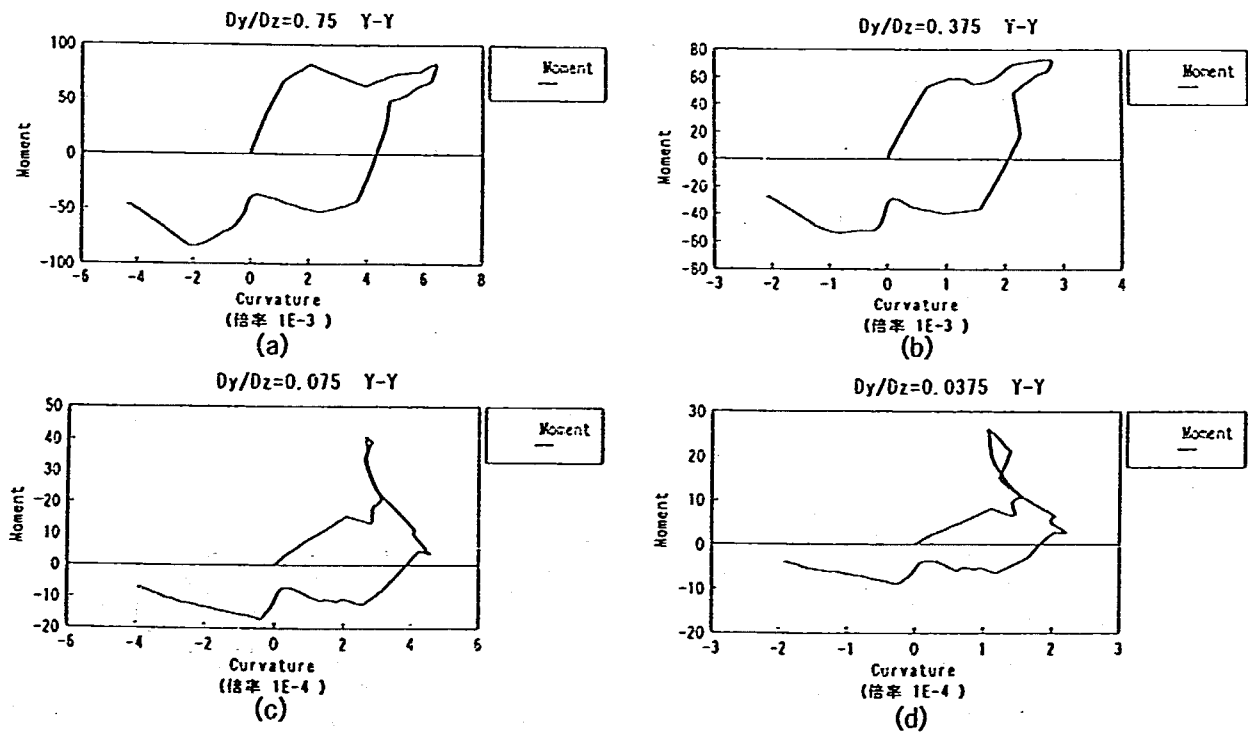


Fig. 74 Hysteresis loops in Y direction for $T_y/T_z=1.5$ and phase angle $=90^\circ$

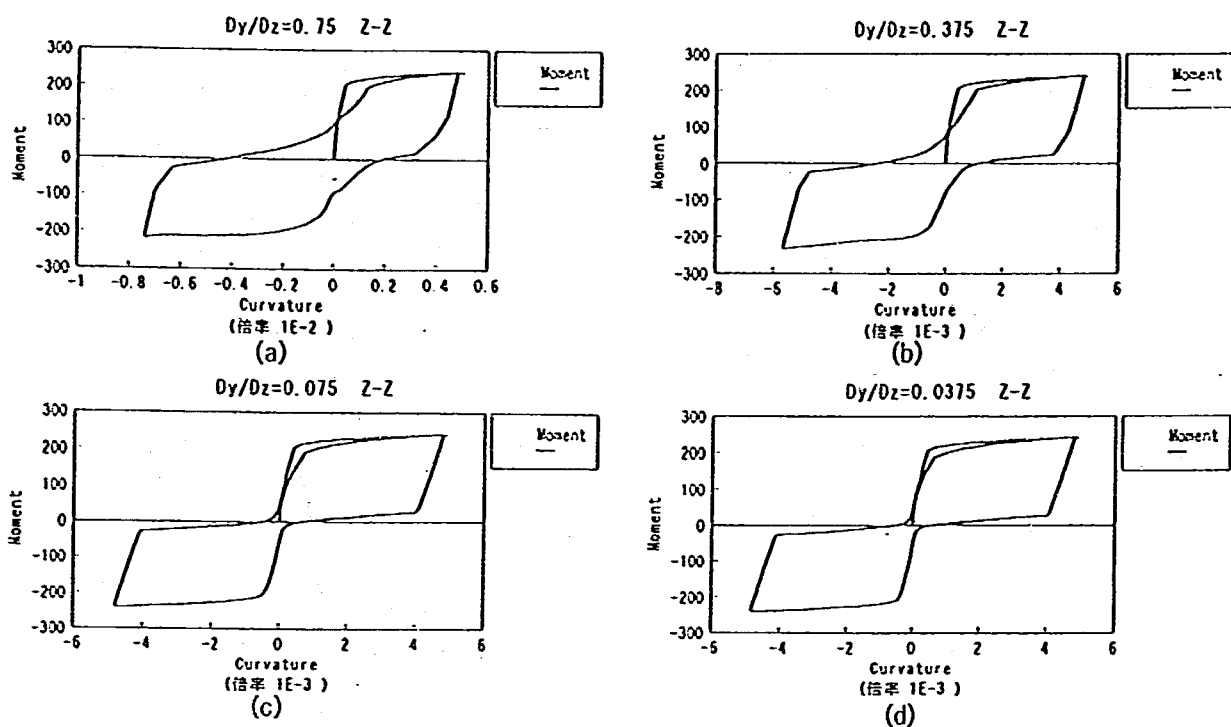


Fig. 75 Hysteresis loops in Z direction for $T_y/T_z=2.0$ and phase angle $=90^\circ$

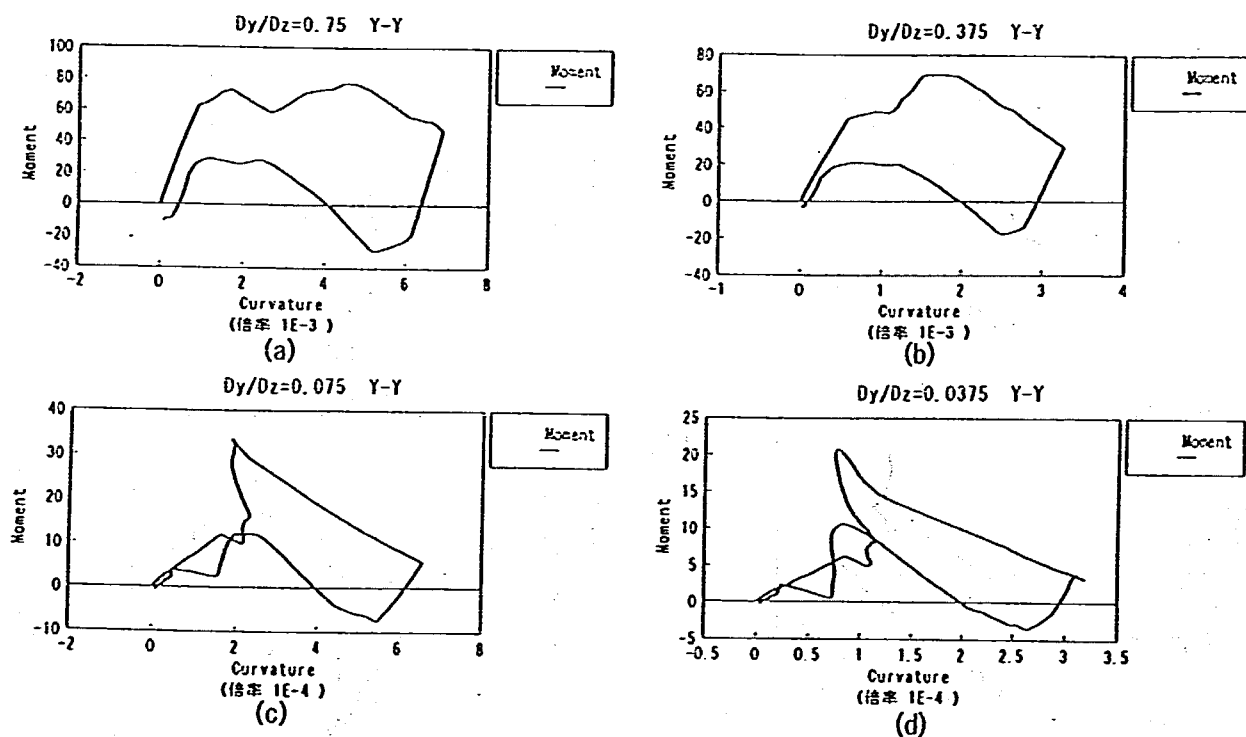


Fig. 76 Hysteresis loops in Y direction for $T_y/T_z=2.0$ and phase angle $=90^\circ$

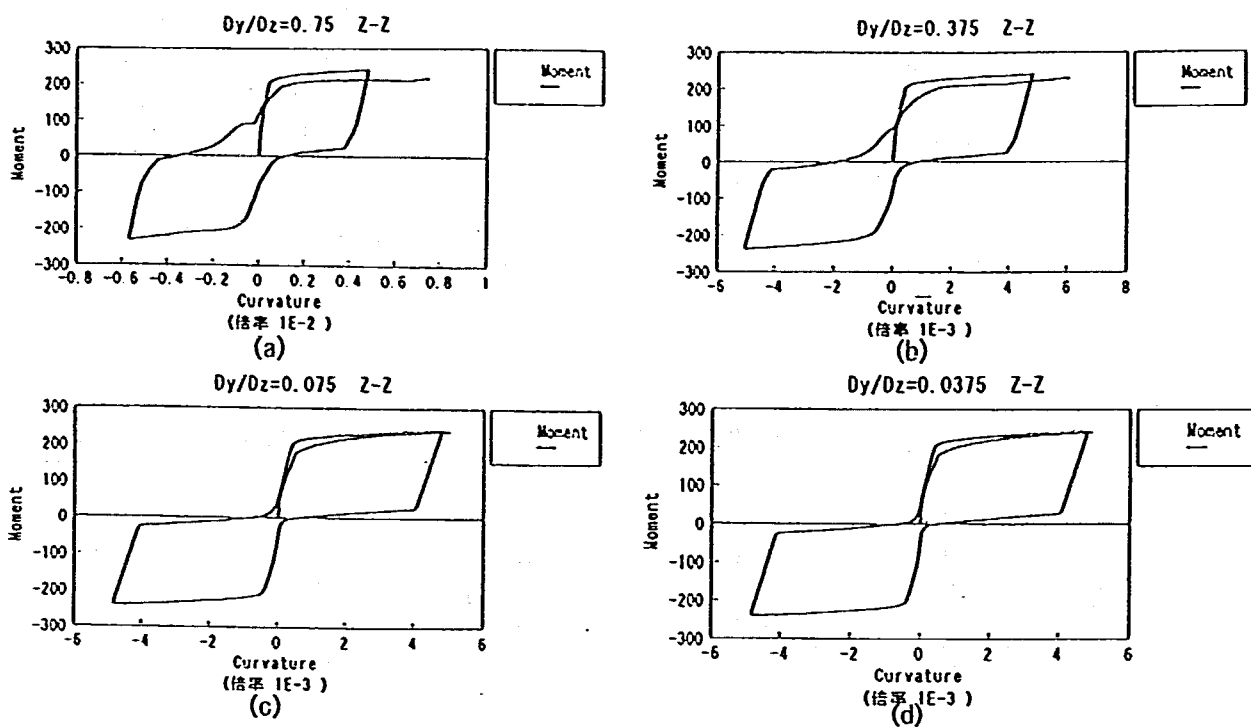


Fig. 77 Hysteresis loops in Z direction for $T_y/T_z=4.0$ and phase angle=90

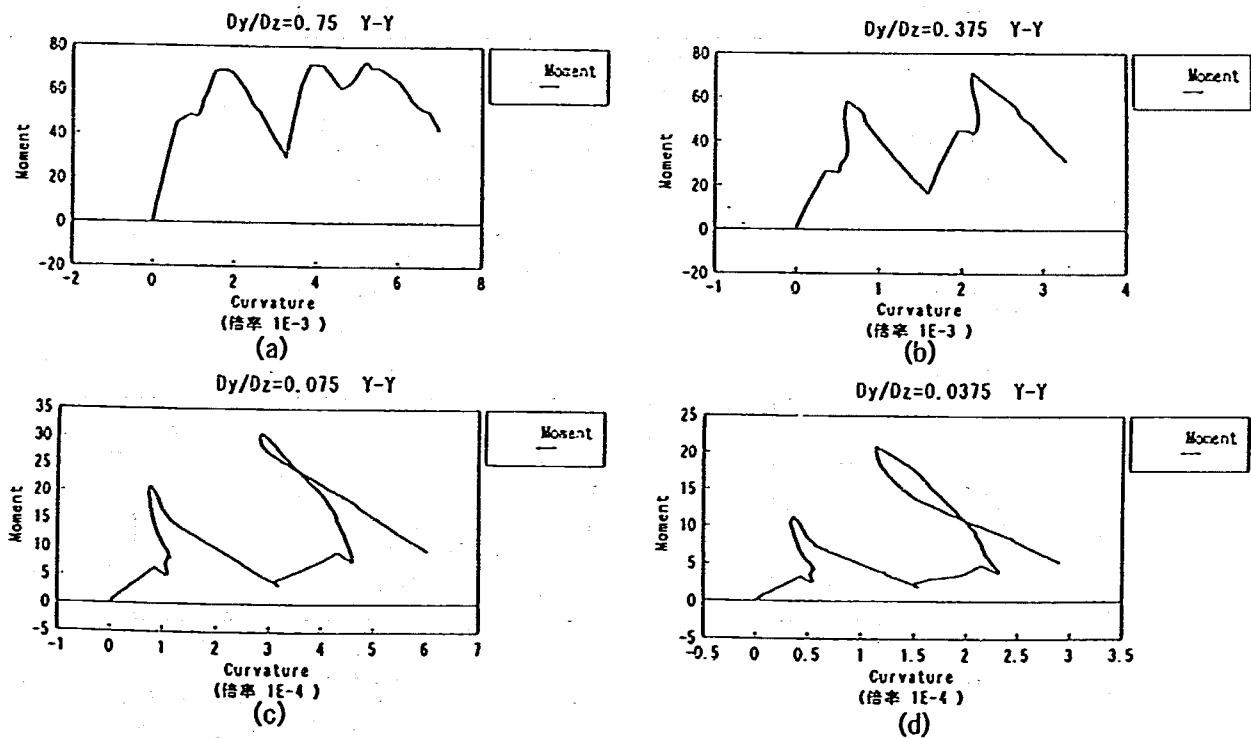


Fig. 78 Hysteresis loops in Y direction for $T_y/T_z=4.0$ and phase angle=90

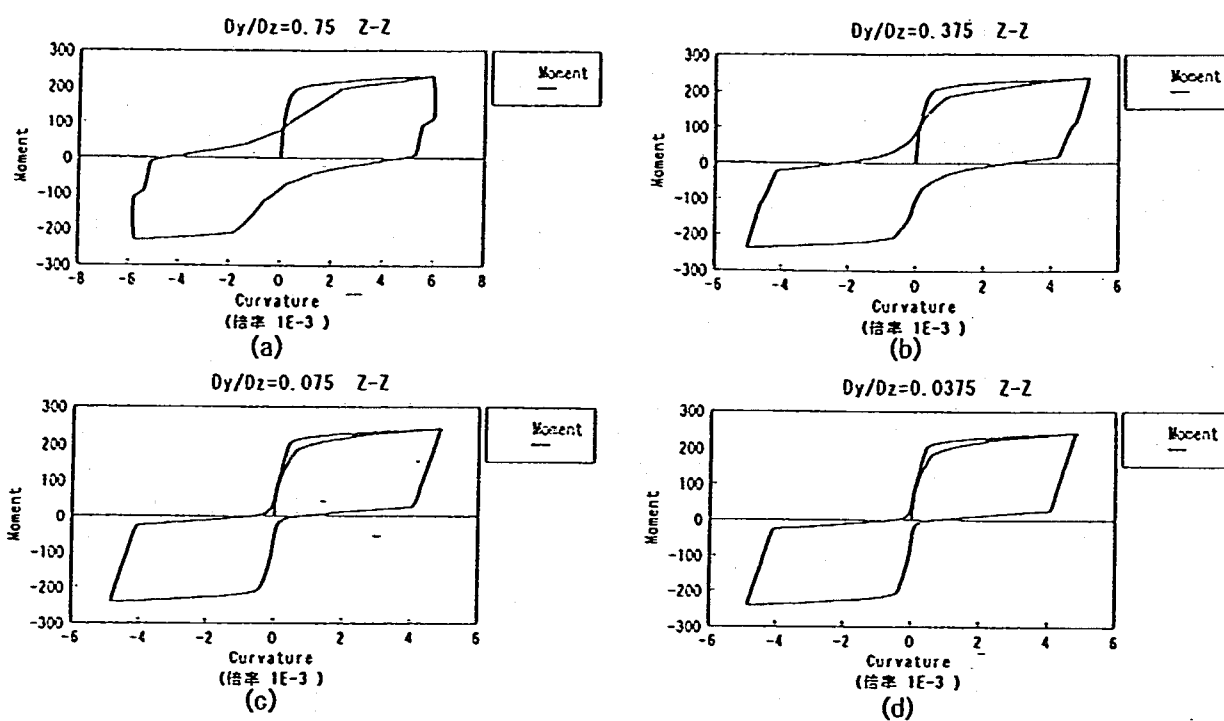


Fig. 79 Hysteresis loops in Z direction for $T_y/T_z=1.0$ and phase angle $=45^\circ$

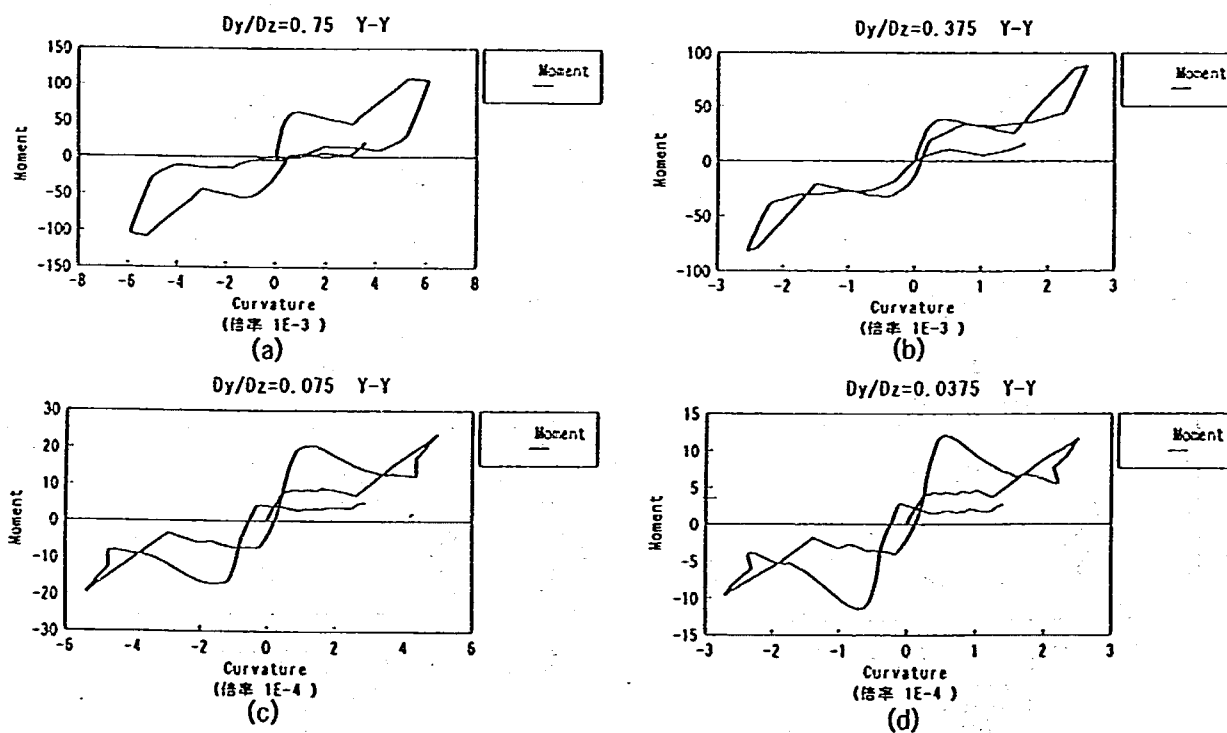


Fig. 80 Hysteresis loops in Y direction for $T_y/T_z=1.0$ and phase angle $=45^\circ$

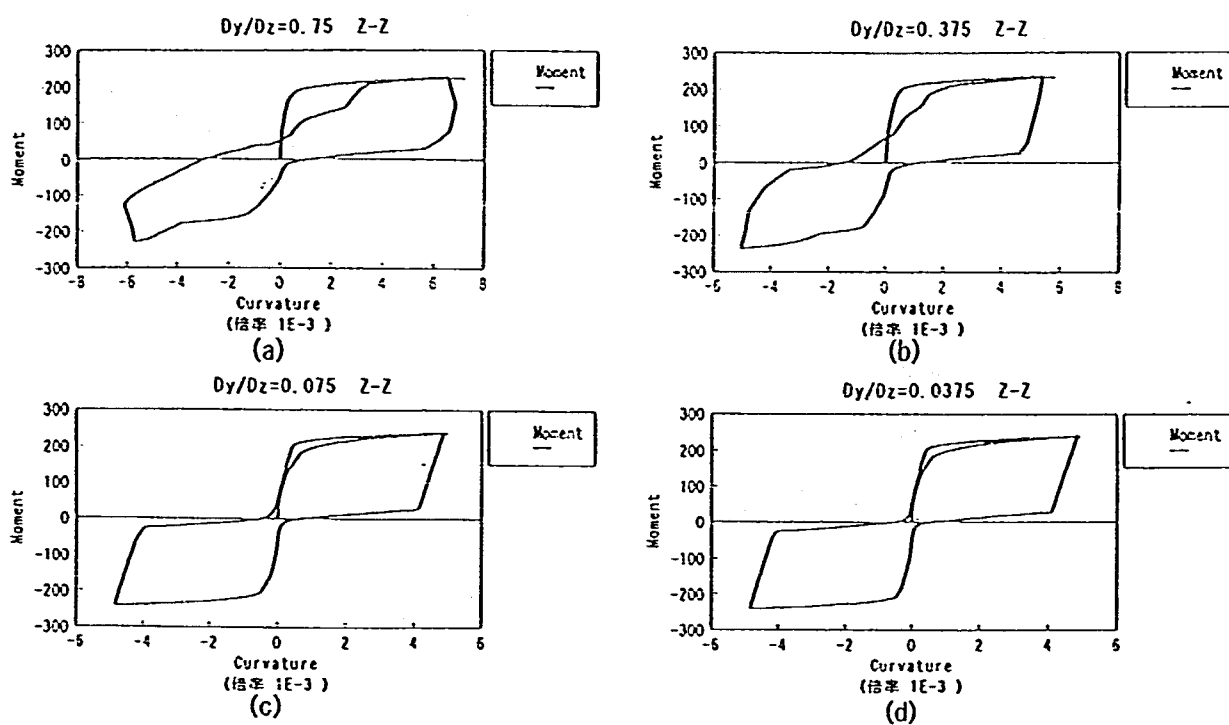


Fig. 81 Hysteresis loops in Z direction for $T_y/T_z=0.75$ and phase angle $=45^\circ$

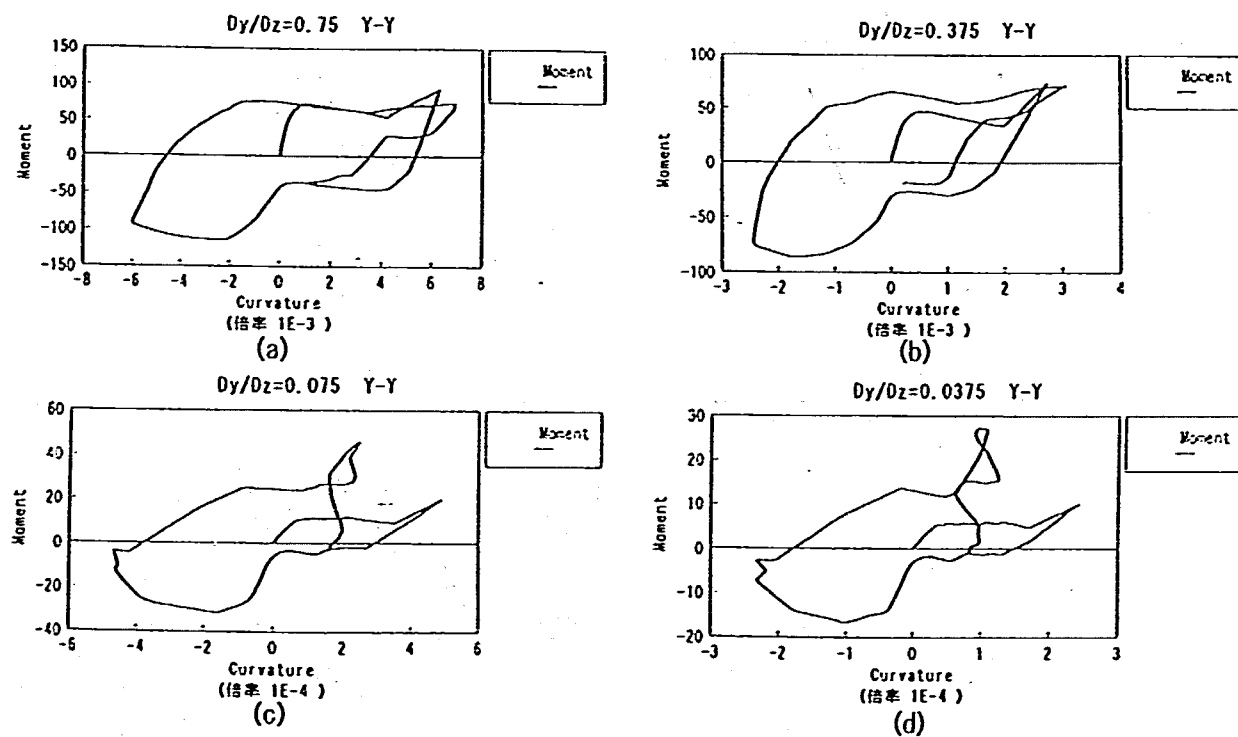


Fig. 82 Hysteresis loops in Y direction for $T_y/T_z=0.75$ and phase angle $=45^\circ$

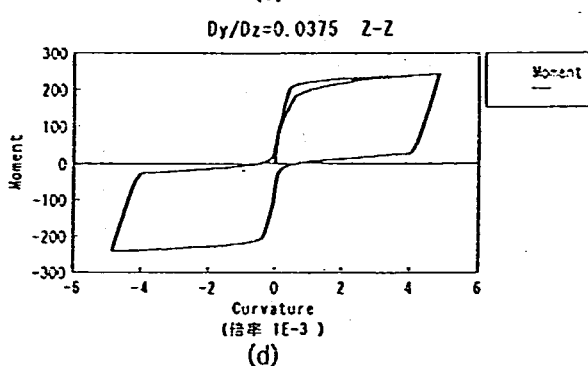
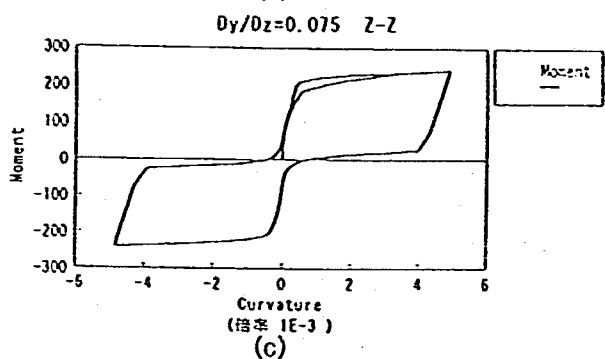
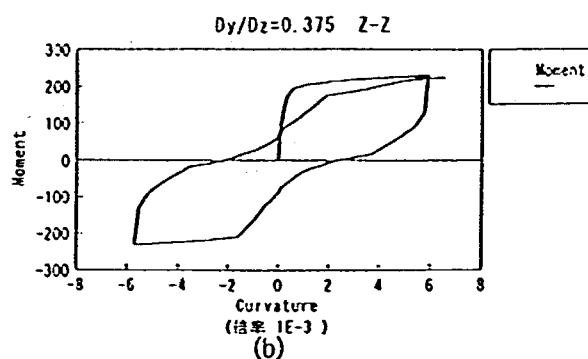
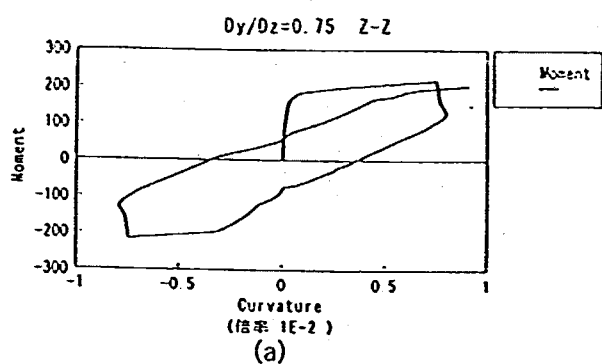


Fig. 83 Hysteresis loops in Z direction for $T_y/T_z=0.5$ and phase angle=45

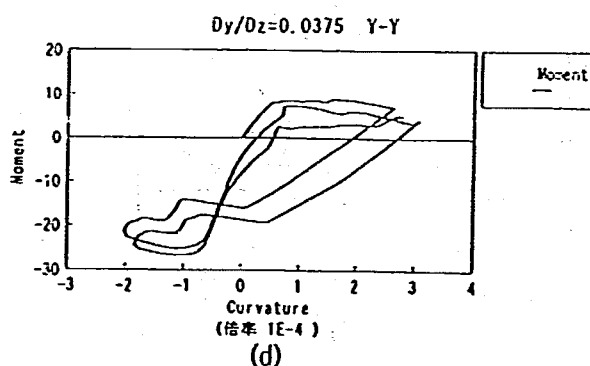
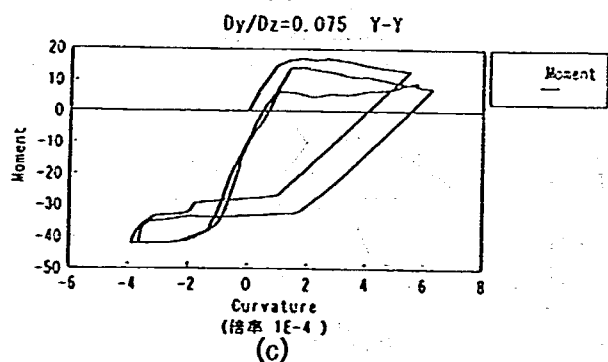
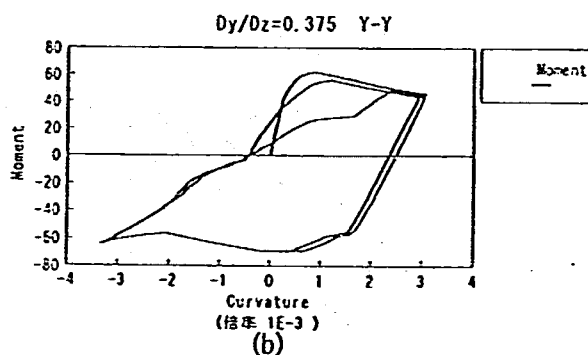
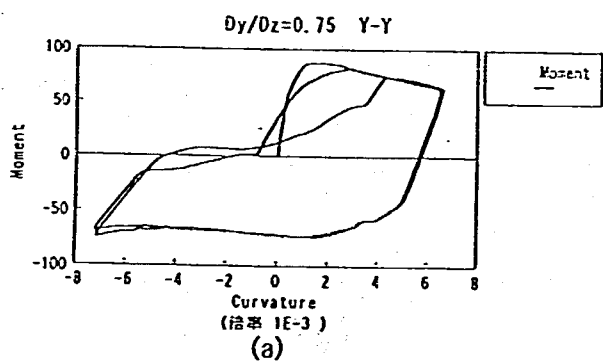


Fig. 84 Hysteresis loops in Y direction for $T_y/T_z=0.5$ and phase angle=45

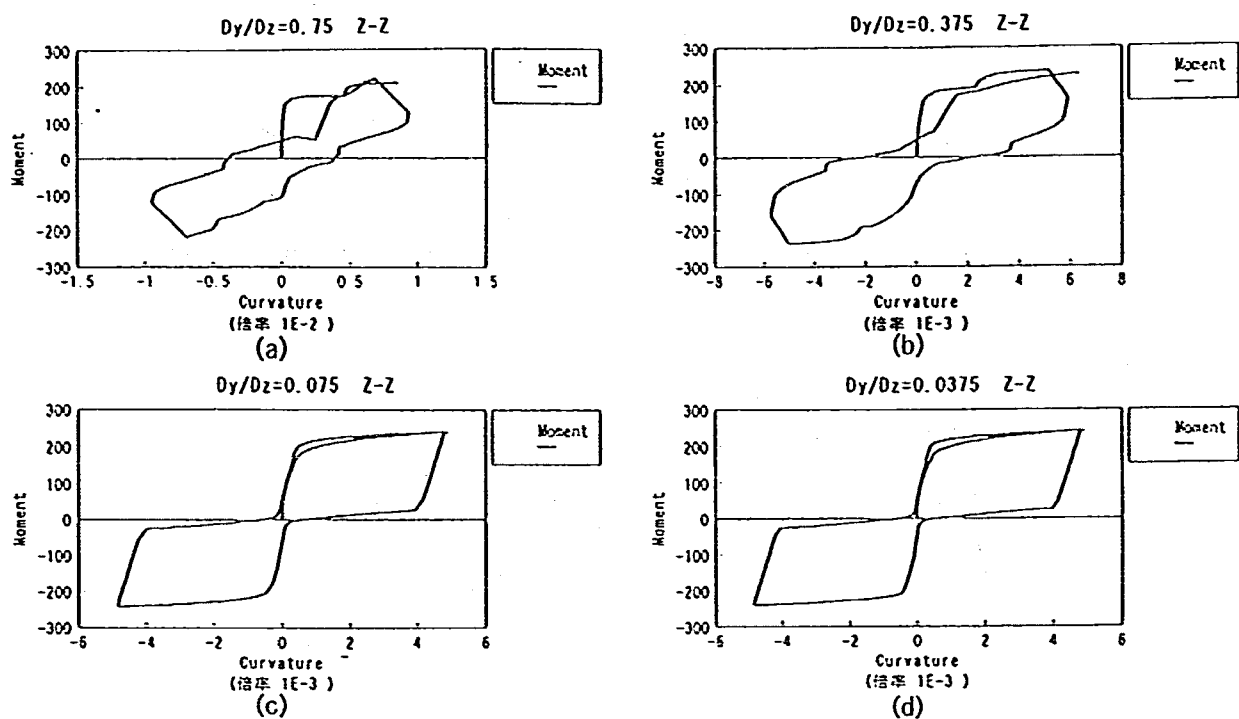


Fig. 85 Hysteresis loops in Z direction for $T_y/T_z=0.25$ and phase angle $=45^\circ$

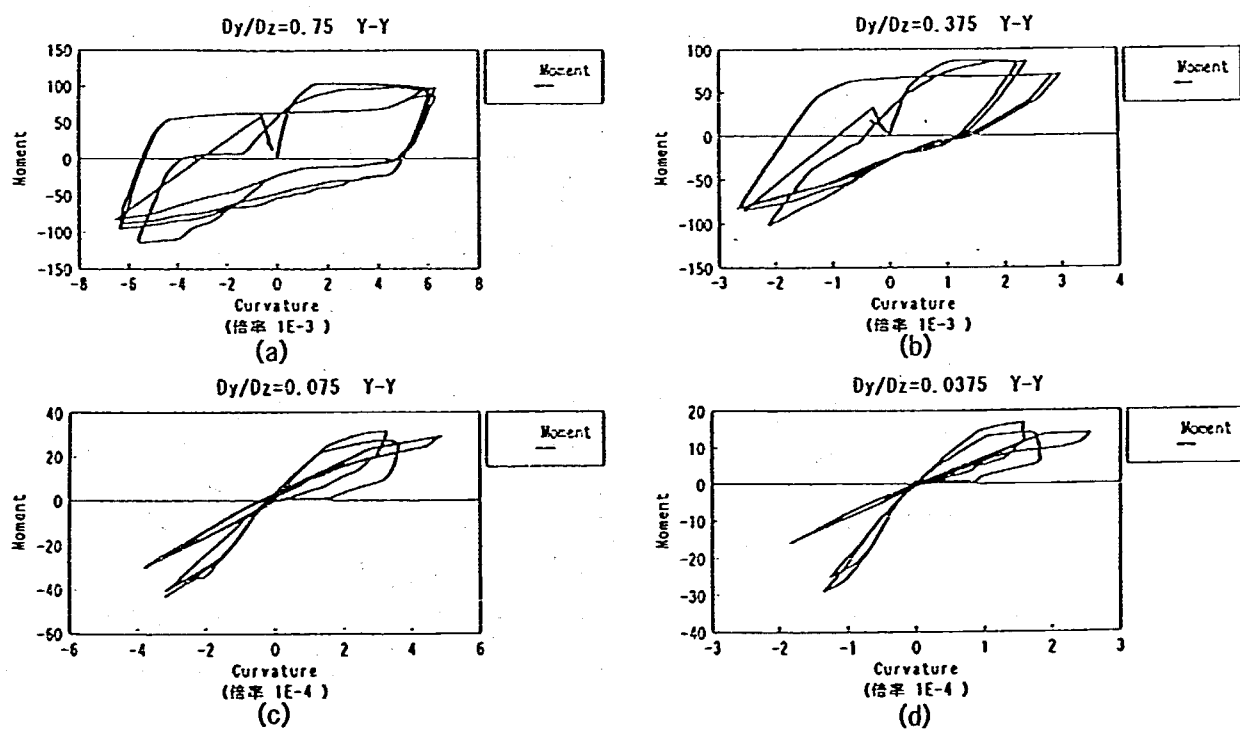


Fig. 86 Hysteresis loops in Y direction for $T_y/T_z=0.25$ and phase angle $=45^\circ$

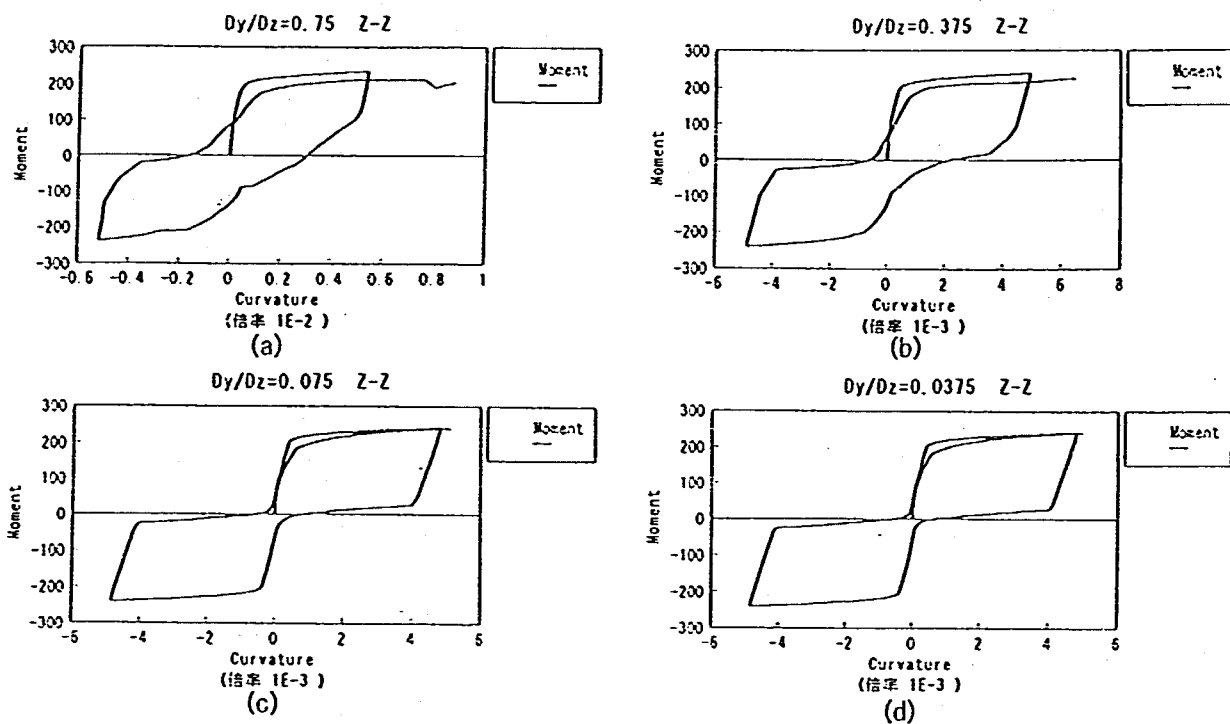


Fig. 87 Hysteresis loops in Z direction for $T_y/T_z=0.25$ and phase angle $=45^\circ$

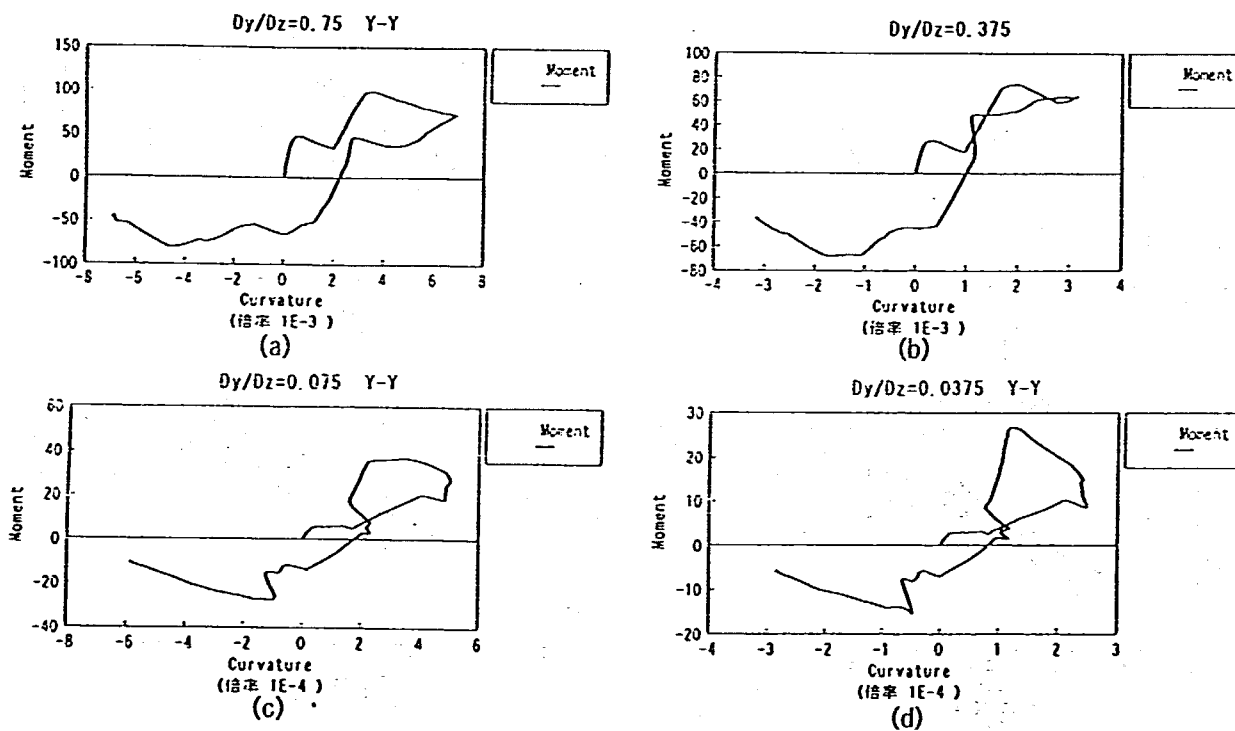


Fig. 88 Hysteresis loops in Y direction for $T_y/T_z=1.5$ and phase angle $=45^\circ$

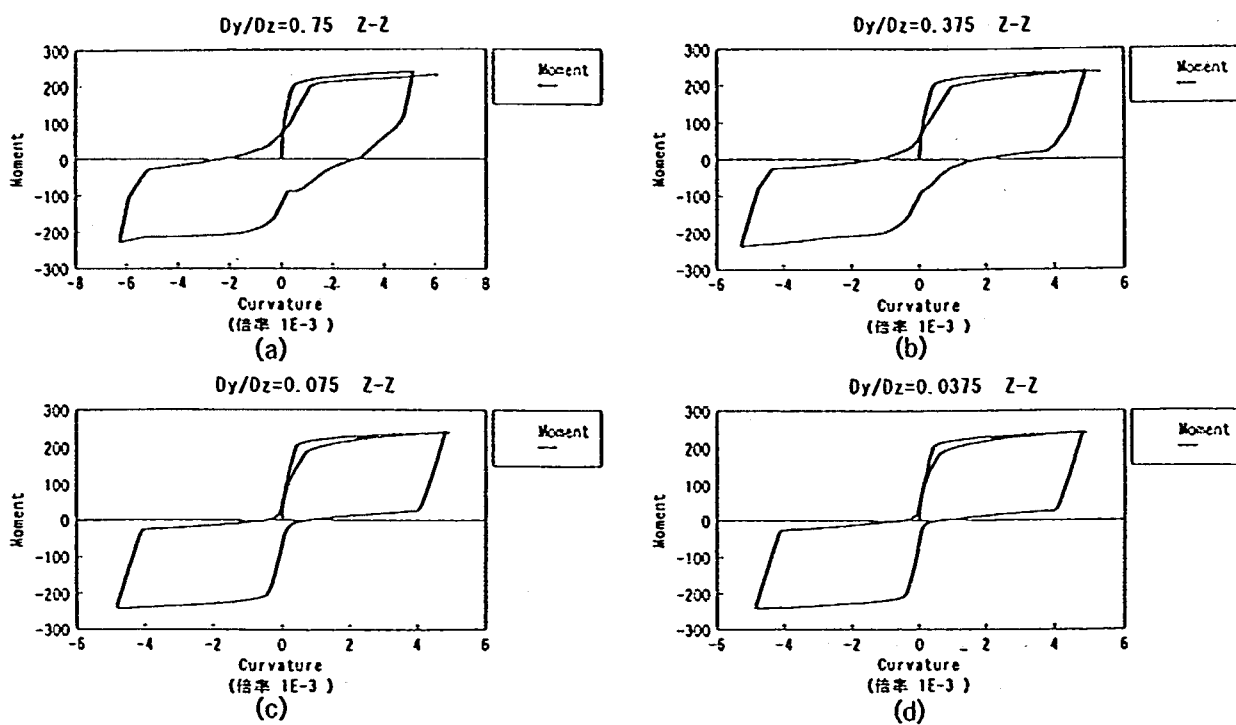


Fig. 89 Hysteresis loops in Z direction for $T_y/T_z=2.0$ and phase angle=45

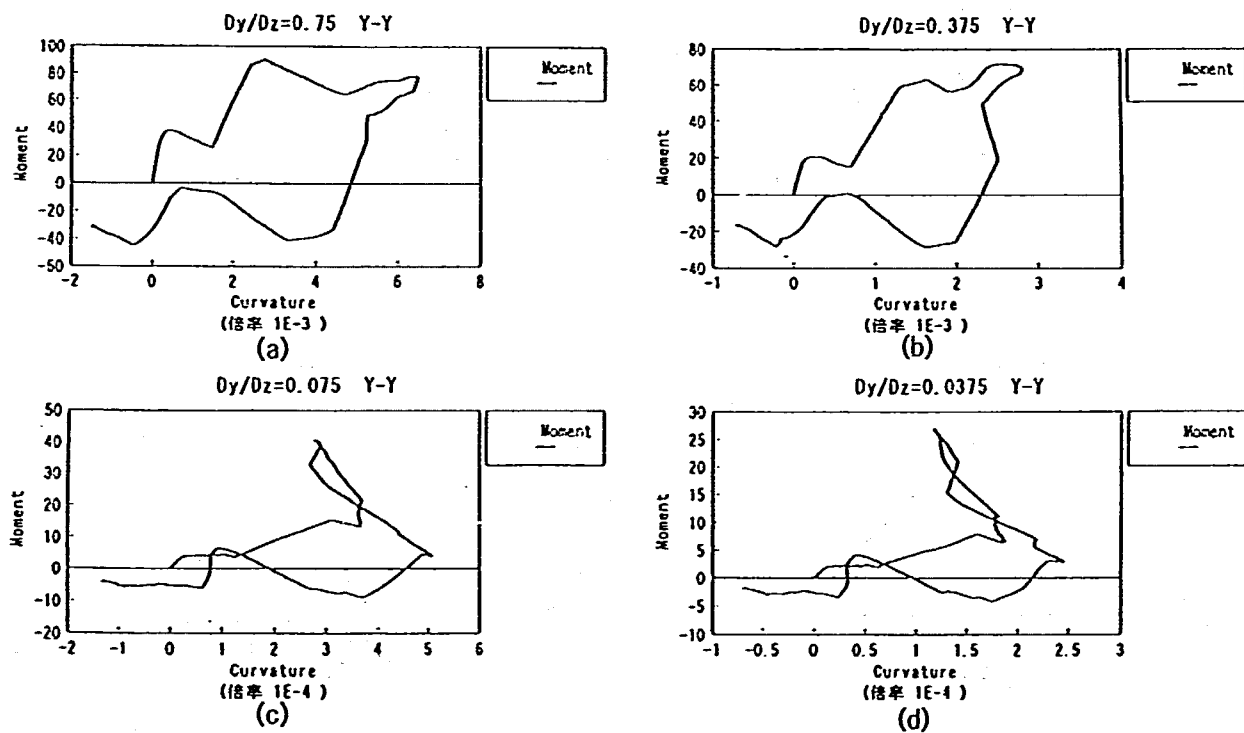


Fig. 90 Hysteresis loops in Y direction for $T_y/T_z=2.0$ and phase angle=45

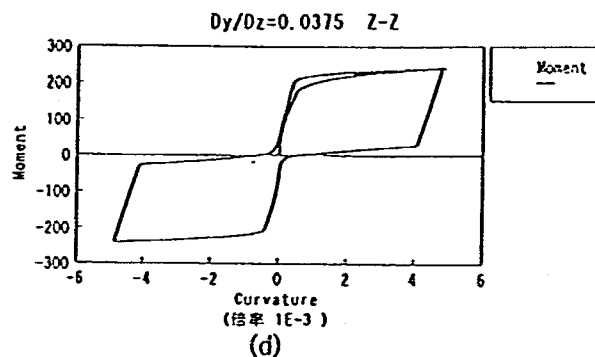
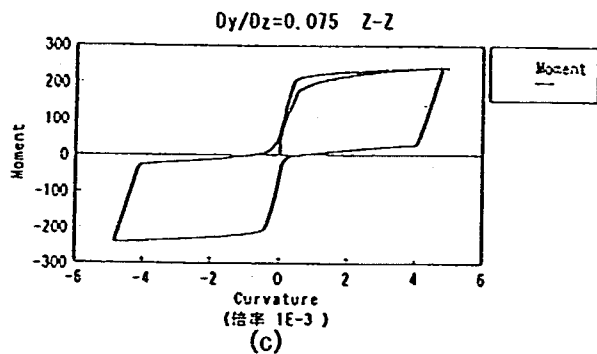
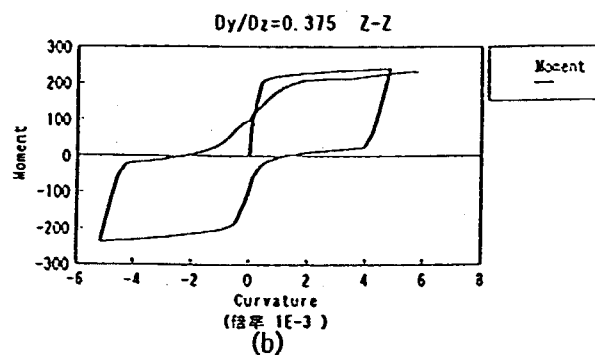
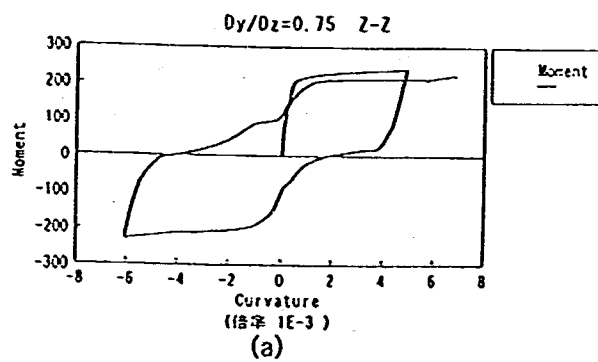


Fig. 91 Hysteresis loops in Z direction for $T_y/T_z=4.0$ and phase angle=45

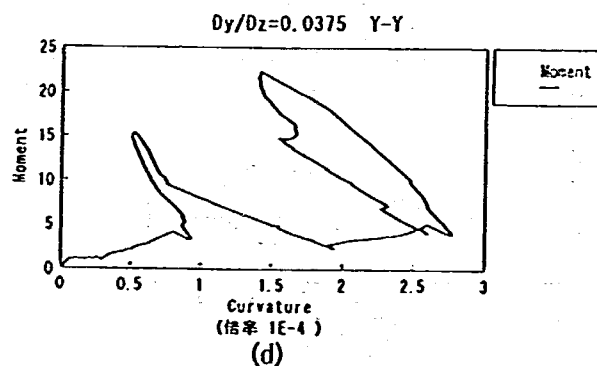
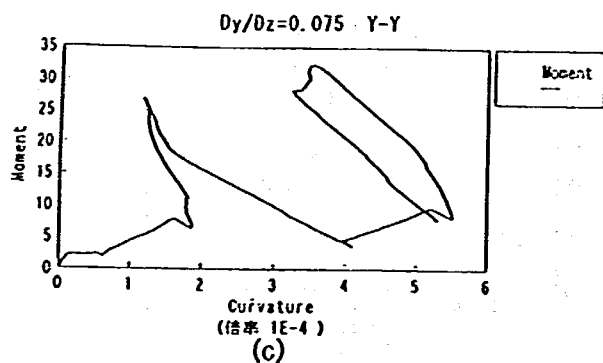
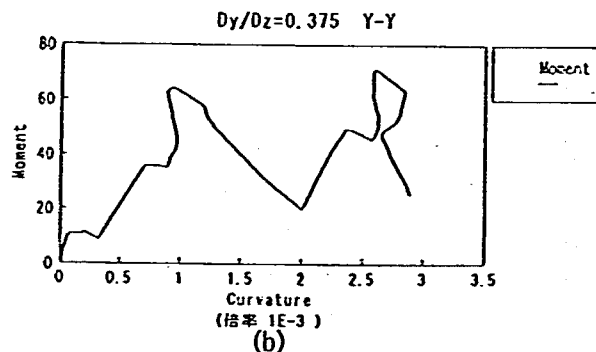
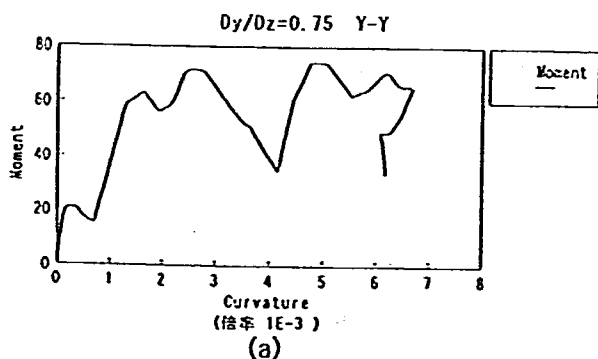


Fig. 92 Hysteresis loops in Y direction for $T_y/T_z=4.0$ and phase angle=45

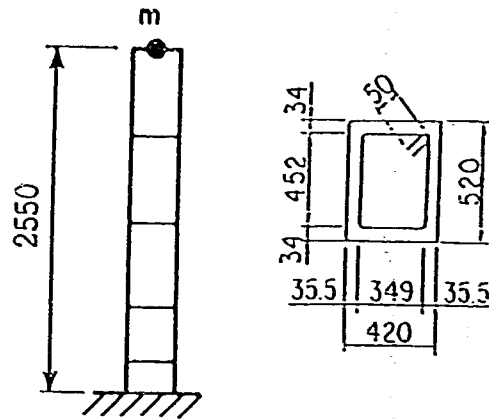


Fig. 2.5.1 Model for RC Column

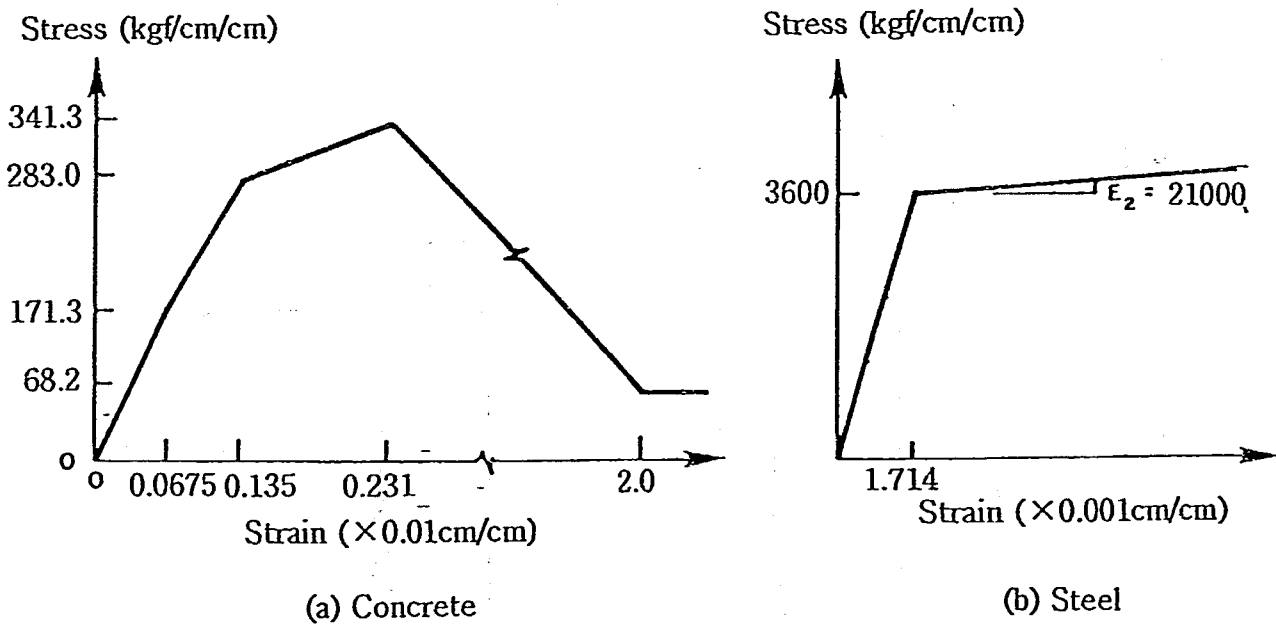


Fig. 2.5.2 Idealized Material Properties of Model RC Column

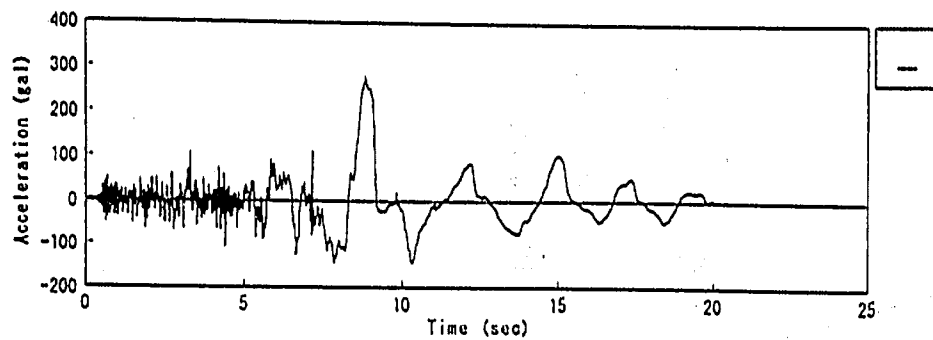


Fig. 2.5.3 input acceleration in X direction

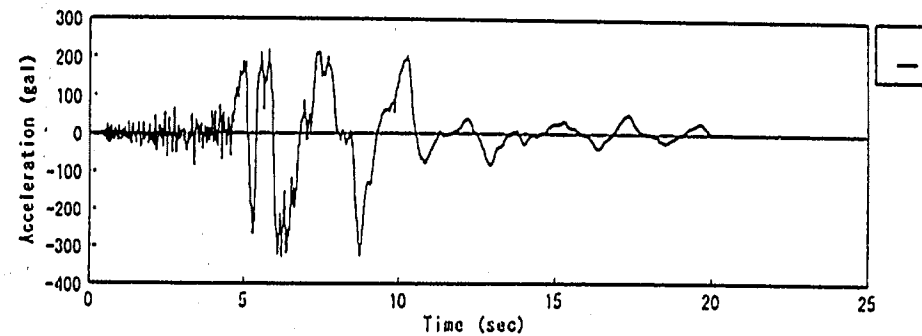


Fig. 2.5.4 input acceleration in Y direction

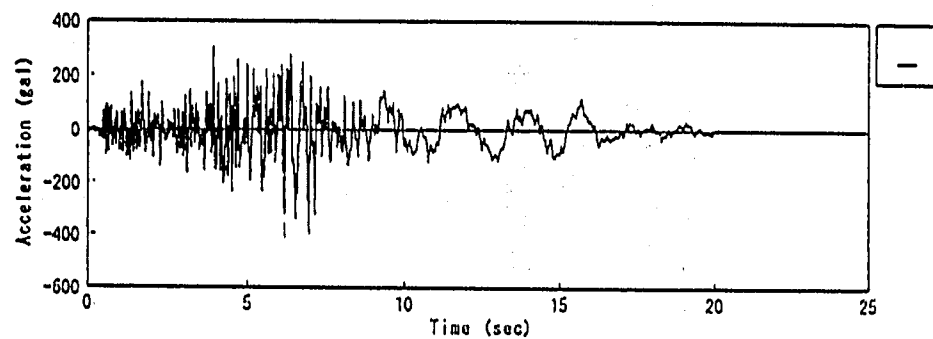


Fig. 2.5.5 input acceleration in Z direction

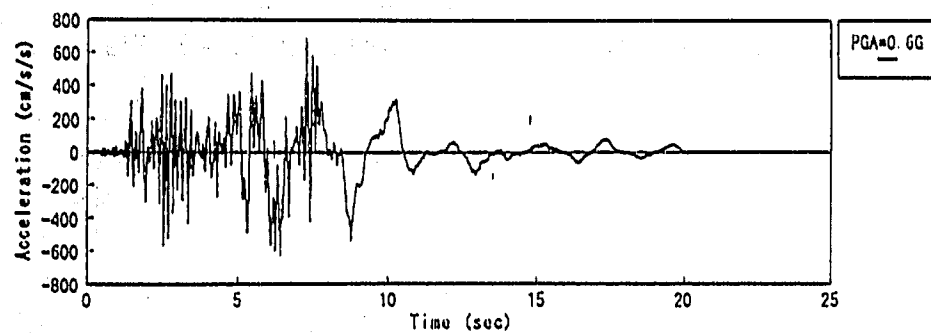


Fig. 2.5.6 Acceleration history at top in X direction with horizontal excitation only

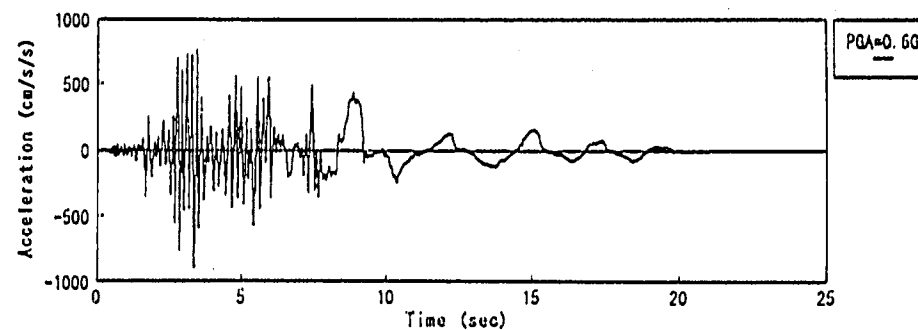


Fig. 2.5.7 Acceleration history at top in Y direction with horizontal excitation only

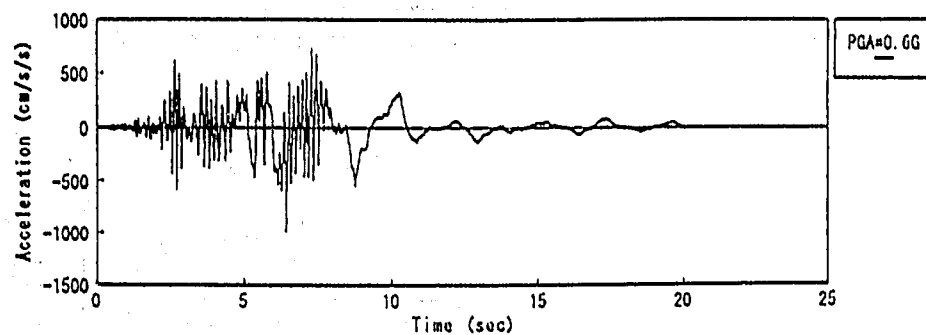


Fig. 2.5.8 Acceleration history at top in X direction with both horizontal and vertical excitations

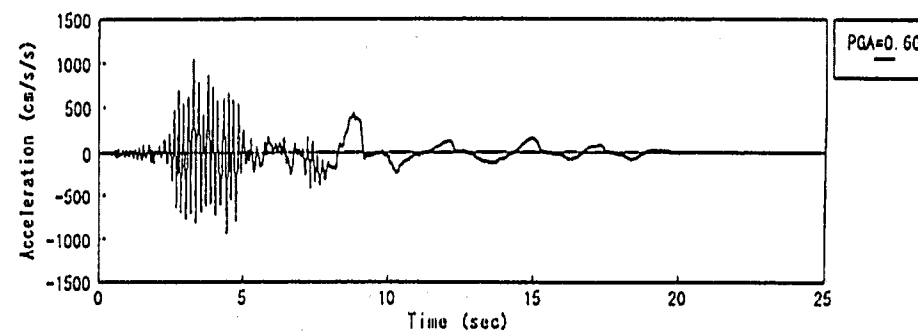


Fig. 2.5.9 Acceleration history at top in Y direction with both horizontal and vertical excitations

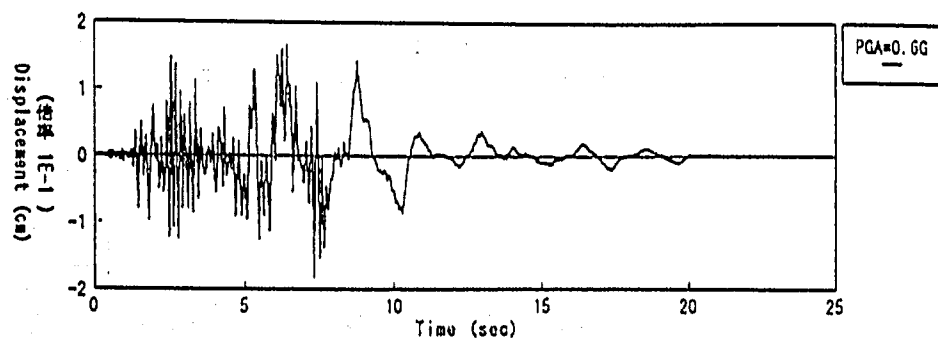


Fig. 2.5.10 Displacement history at top in X direction with horizontal excitation only

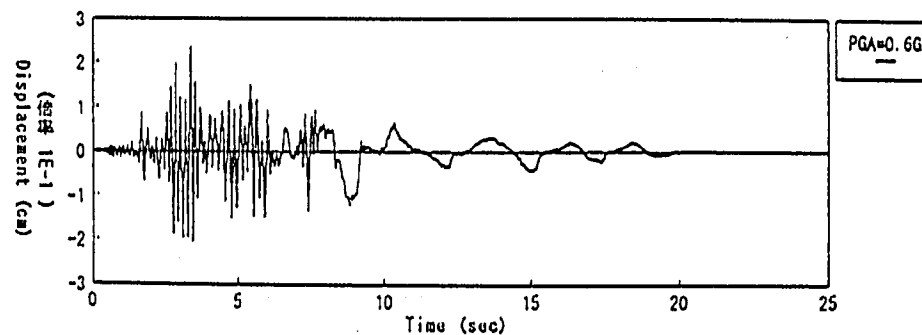


Fig. 2.5.11 Displacement history at top in Y direction with horizontal excitation only

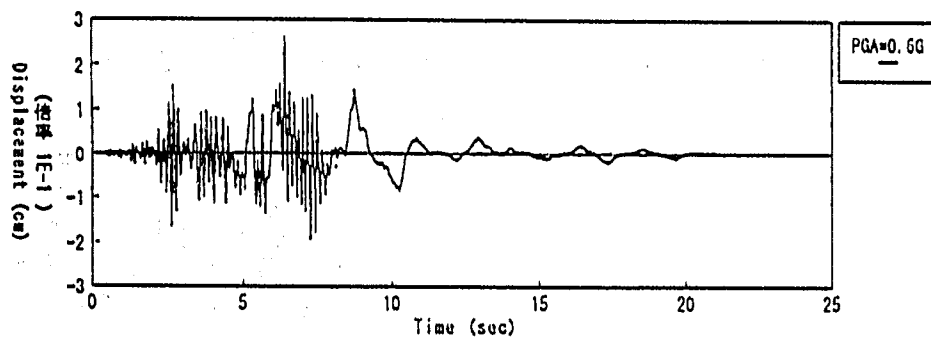


Fig. 2.5.12 Displacement history at top in X direction with horizontal and vertical excitations

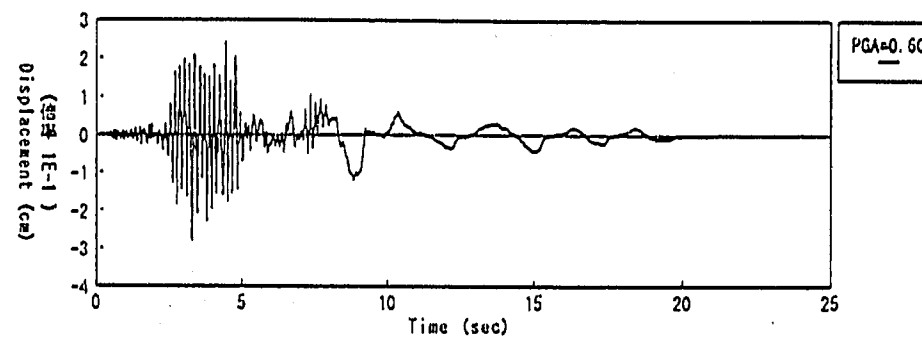


Fig. 2.5.13 Displacement history at top in Y direction with horizontal and vertical excitations

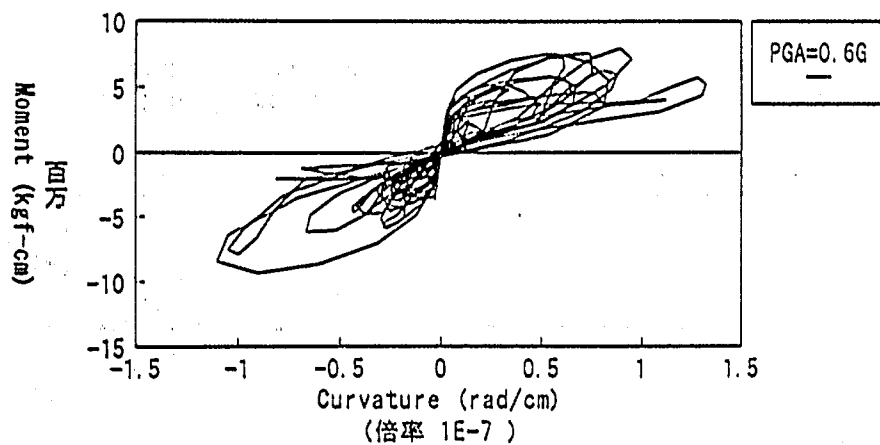


Fig. 2.5.14 Moment-curvature relation at base in X direction with horizontal excitation only

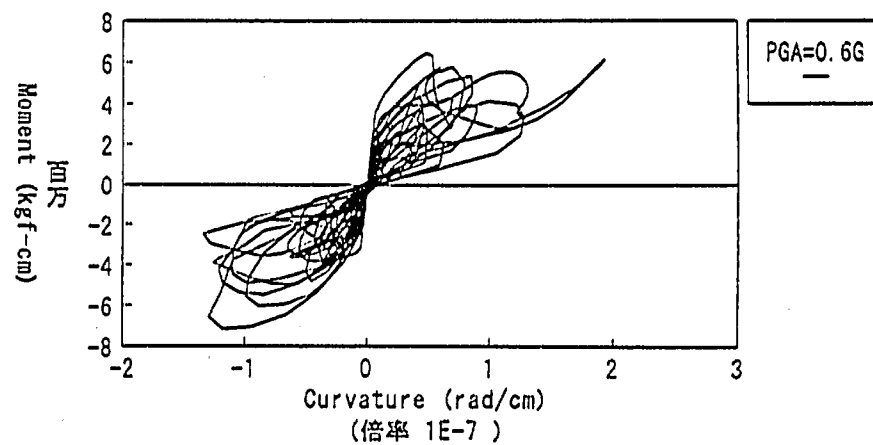


Fig. 2.5.15 Moment-curvature relation at base in Y direction with horizontal excitation only

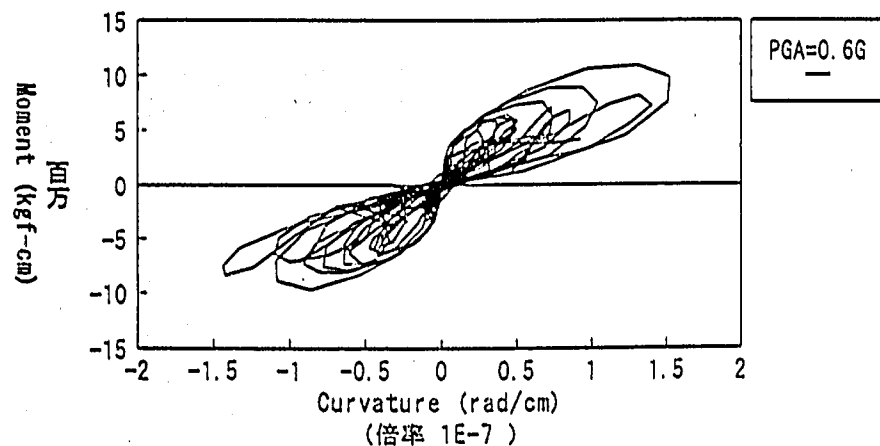


Fig. 2.5.16 Moment-curvature relation at base in X direction with both horizontal and vertical excitations

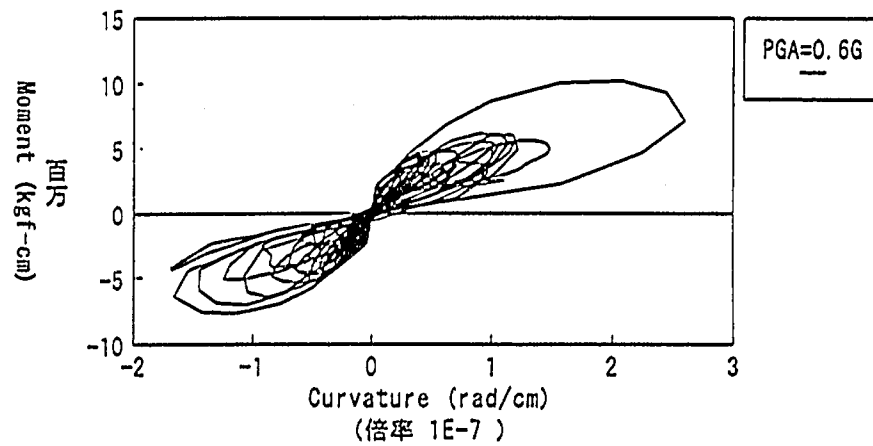


Fig. 2.5.17 Moment-curvature relation at base in Y direction with both horizontal and vertical excitations

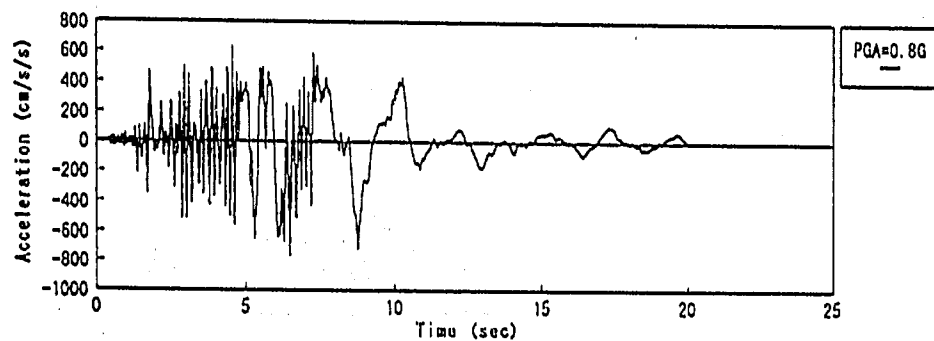


Fig. 2.5.18 Acceleration history at top in X direction with horizontal excitation only

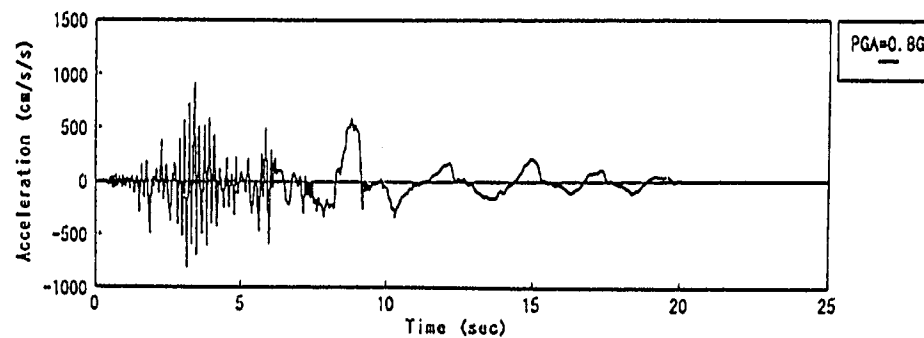


Fig. 2.5.19 Acceleration history at top in Y direction with horizontal excitation only

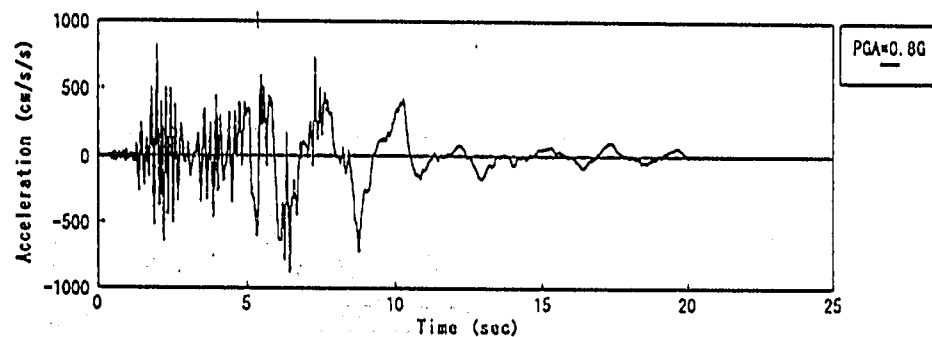


Fig. 2.5.20 Acceleration history at top in X direction with both horizontal and vertical excitations

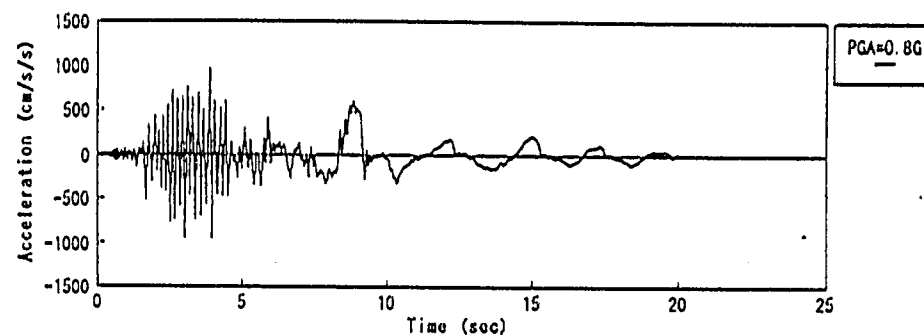


Fig. 2.5.21 Acceleration history at top in Y direction with both horizontal and vertical excitations

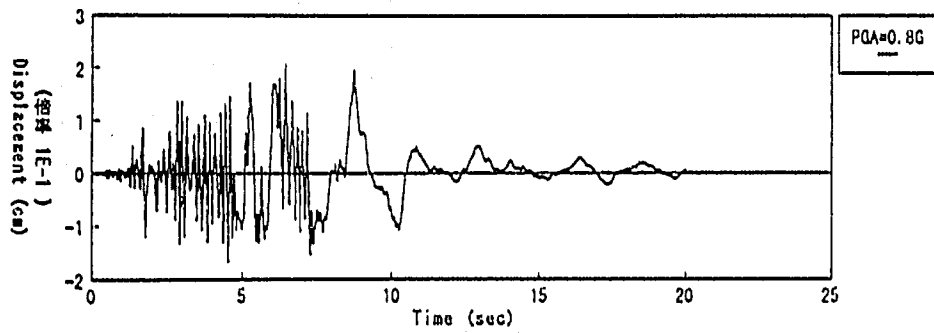


Fig. 2.5.22 Displacement history at top in X direction with horizontal excitation only

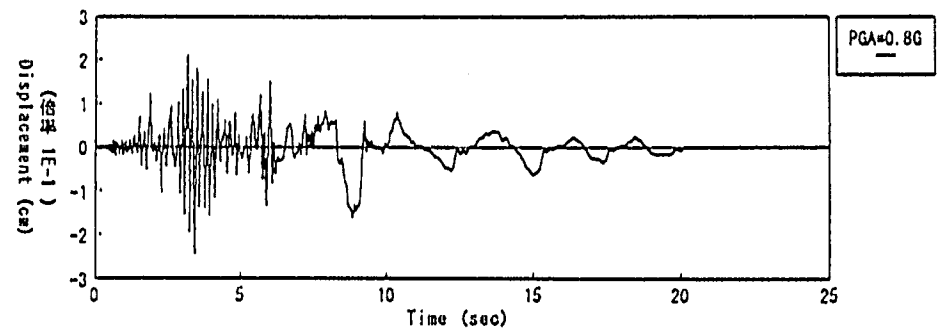


Fig. 2.5.23 Displacement history at top in Y direction with horizontal excitation only

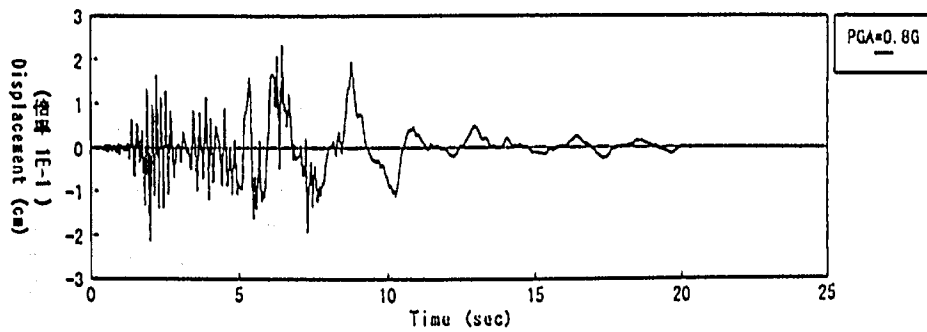


Fig. 2.5.24 Displacement history at top in X direction with both horizontal and vertical excitations

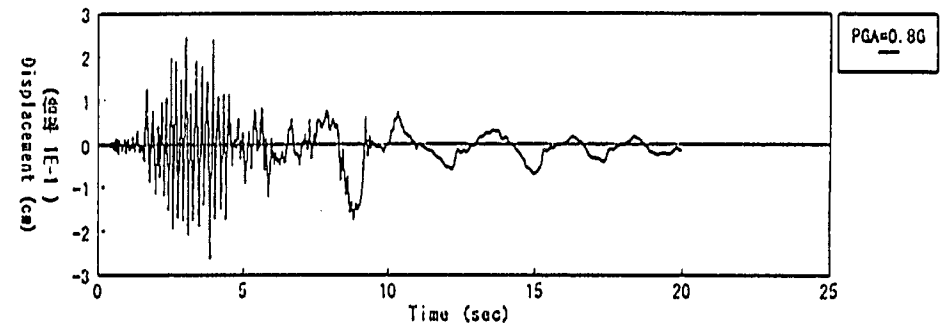


Fig. 2.5.25 Displacement history at top in Y direction with both horizontal and vertical excitations

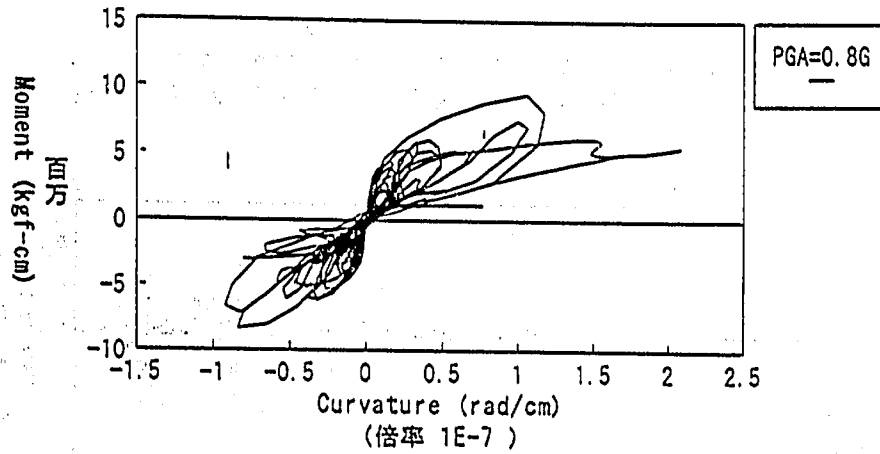


Fig. 2.5.26 Moment-curvature relation at base in X direction with horizontal excitation only

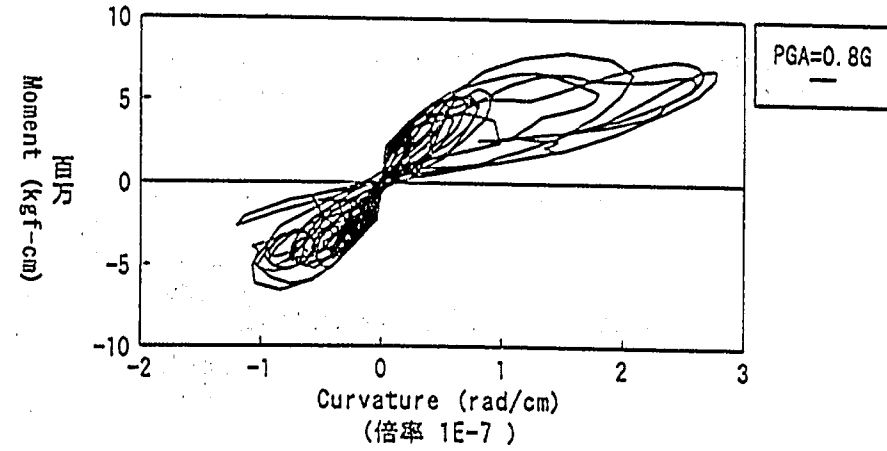


Fig. 2.5.27 Moment-curvature relation at base in Y direction with horizontal excitation only

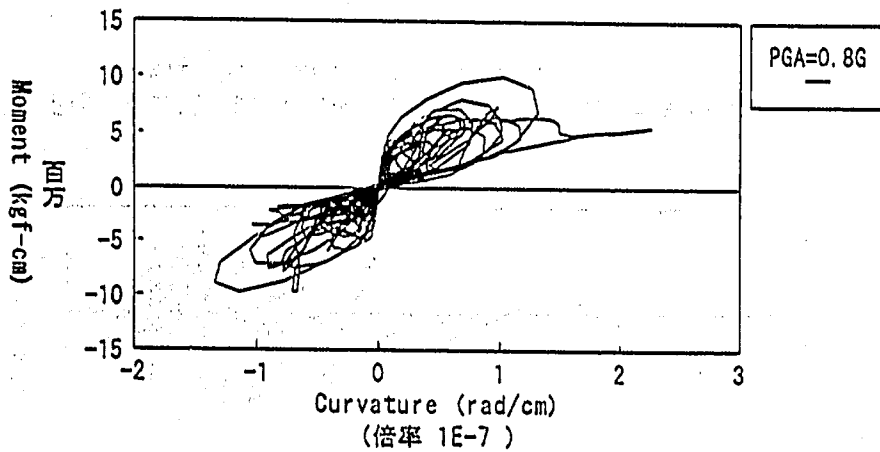


Fig. 2.5.28 Moment-curvature relation at base in X direction with both horizontal and vertical excitations

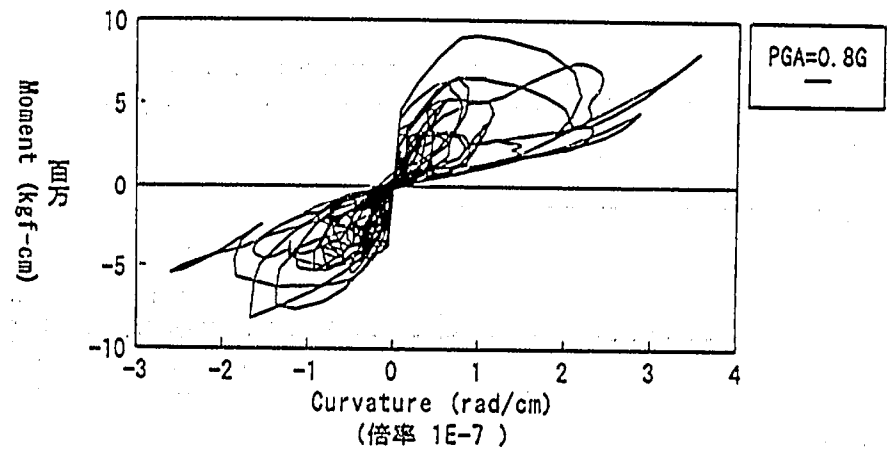


Fig. 2.5.29 Moment-curvature relation at base in Y direction with both horizontal and vertical excitations