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OPTIMAL DESIGN OF FRICTION-BRACED FRAMES UNDER SEISMIC LOADING

by

M. A. AUSTIN
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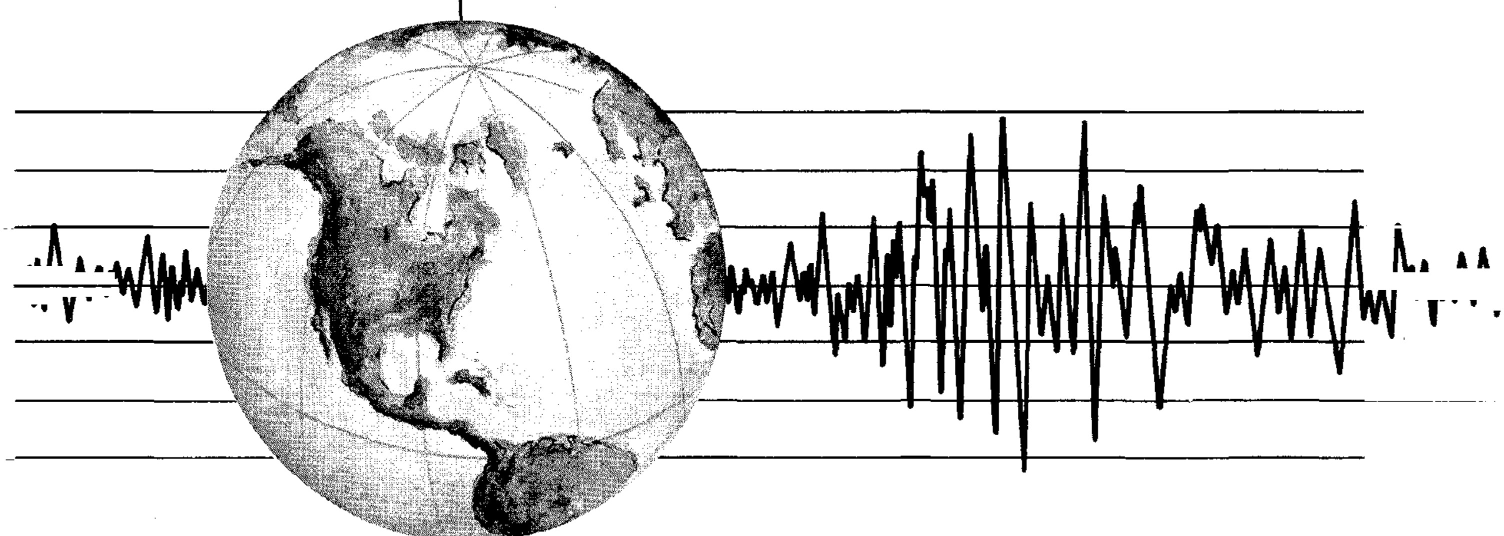
Report to the National Science Foundation



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The frame's performance is assessed on the basis of its response to three different loadings; gravity loads only, gravity loads plus moderate earthquake and finally gravity loads combined with a rare severe earthquake ground motion.

A preliminary analysis was implemented to determine the performance constraints likely to become active during the optimization process. The friction-bracing was then removed and the resulting moment-resistant frame was resimulated for the same ground motion input. Approximate bounds on the performance constraints to be expected during the optimization stage were therefore obtained. A sensitivity analysis for frame response was implemented for perturbed ground motion and modelling parameters.

Recommendations are given to enhance both DELIGHT.STRUCT and the design process utilized. This report concludes with a summary of the potential advantages of incorporating friction-bracing into steel frames.

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Report No. UCB/EERC-83/10
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University of California
Berkeley, California

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I. ABSTRACT.

This report describes the results of a multi-objective investigation. Several optimal earthquake-resistant designs for a ten story, single bay, friction-braced steel frame excited by a single scaled ground motion are calculated. As this was only the second major problem area to be examined with the CAD environment DELIGHT.STRUCT, a further assessment of its performance was also required.

A review of current earthquake-resistant design philosophy is presented. The frame's performance is assessed on the basis of its response to three different loadings. These are gravity loads only, gravity loads plus moderate earthquake and finally gravity loads combined with a rare severe earthquake ground motion.

A preliminary analysis was first implemented to determine the performance constraints likely to become active during the optimization process. The friction-bracing was then removed and the resulting moment-resistant frame was resimulated for the same ground motion input. Approximate bounds on the performance constraints to be expected during the optimization stage were therefore obtained. The optimization problem is formulated for the aforementioned frame. The method of feasible directions is then employed to solve the constrained optimization problem for various objective functions. These include minimum volume, minimum dissipated energy and minimum sum of story drifts squared. A sensitivity analysis for frame response was implemented for perturbed ground motion and modelling parameters. An alternative ground motion scaling procedure is presented.

Recommendations are given to enhance both DELIGHT.STRUCT and the design process utilized. This report concludes with a summary of the potential advantages of incorporating friction-bracing into steel frames.

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III. TABLE OF CONTENTS.

1. INTRODUCTION.

This report continues the work initiated by Balling et al.[2] and Bhatti[3] on the optimal design of planar multistory steel frames subject to earthquake excitation. In Balling et al.[2], results of an optimal design for a four story,three bay moment-resistant frame are presented. It is reported[8] that structural systems of this type have excellent energy dissipation capacity but tend to be relatively flexible and may become uneconomical if a high lateral load resistance is required. Concentrically braced frames conversely offer considerable stiffness and strength, but their ability to dissipate energy is poor because the braces tend to buckle. One solution to this shortcoming as suggested by Pall and Marsh[5], is to provide sliding friction devices in the bracing system. During an earthquake large quantities of energy are dissipated by mechanical friction rather than by inelastic yielding of the main structural elements. It is claimed that the confinement of energy dissipation to the braces permits the remainder of the members to respond elastically, or at least delays the onset of inelastic deformations. Consequently, the structural performance in a severe earthquake is significantly enhanced.

This study was initiated with the following objectives:

1. To verify the aforementioned statements pertaining to the structural response of a ten story, single bay friction-damped, braced frame.
2. To employ the computer-aided design environment DELIGHT.STRUCT as discussed in Balling et al.[1] to calculate several *optimal* designs corresponding to different cost functions.
3. To gain experience with the setting of optimization algorithm parameters in the DELIGHT.STRUCT system.
4. To assess the performance of DELIGHT.STRUCT as a computer-aided environment.

5. To assess the usefulness of optimization in the seismic design of friction-damped braced frames.

1.1 Report Contents.

A design philosophy review applicable to the design of multistory steel frames is presented in the remaining section of this chapter.

In the following chapters the geometry, loading, and modelling assumptions for the design problem studied, the Workman frame[4] are discussed in detail. A discussion of desirable inelastic frame hinging mechanisms is presented. The optimization process is then discussed. The method of feasible directions is briefly reviewed. This is followed by a discussion of the constraint functions applicable to each of the three loading categories considered. A summary of the cost function options available in DELIGHT.STRUCT concludes Chapter Three. Chapter Four presents the results of a preliminary frame analysis. Both the braced Workman frame and an equivalent moment-resistant frame are simulated and constraints plotted. A hierarchy of important constraints is presented together with a discussion of the influence of bracing on the structural response. On the basis of these observations, an equivalent optimization problem is formulated.

Results of the minimum volume, minimum dissipated energy and final designs are presented in Chapter Five. A structural response sensitivity analysis is carried out in Chapter Six for perturbed spectral intensity, percentage of Rayleigh damping, and response to other records of equal spectral intensity. A report summary and conclusions is given in Chapter Seven.

1.2 Design Philosophy Review.

Present day seismic design codes utilize a static lateral loading as an approximation to a design earthquake excitation. It is distributed in a manner to follow closely that of the fundamental mode of vibration[7] and has a total magnitude equal to the structure's weight times a seismic coefficient. In the Uniform Building Code, UBC, this coefficient is composed of factors dependent on region seismicity, the structure's importance, fundamental period of vibration, material type and expected soil-structure interaction[11]. For a large class of structures and sites this approximation gives dynamic forces and member forces of the correct order. Any errors associated with its application have been regarded as small when compared to the spatial and temporal nature of future ground motions.

The code requires that service loads be factored to ensure an adequate, if not conservative, margin of protection against unsatisfactory performance in the event of a maximum credible earthquake actually occurring. Although it is implied that the structure will respond elastically to the design loadings, inelastic deformations are in fact relied on to absorb the additional demands. Redistribution of the elastic bending moment diagram is therefore permitted to find an upper-bound [lowest ultimate load] collapse mechanism. The designer will typically return to check serviceability requirements only after preliminary member sizes have been allocated on the basis of *strength*.

This simplified loading has previously been employed as the starting point for seismic design through practical necessity. More sophisticated approaches were not considered as computational aids were not readily available to support their development. As this restriction no longer applies, it is possible to critically assess the present code procedure while simultaneously being able to consider an alternative approach. The principal deficiencies in using a static lateral loading for design are:

1. Simultaneous hinge formation is assumed. This generally will not occur in a dynamic structural response and the selection of a collapse mechanism will depend on the order of plastic hinge formation.
2. The interaction of column axial loads and moments is commonly ignored. While this may be of small consequence for low-rise frames, neglecting this effect in the lower stories of taller structures may lead to an unconservative design.
3. P-delta effects due to geometry changes are commonly ignored.

4. This approach fails to define a level of protection against either structural failure, or expected earthquake loading[7].

The alternative approach is more general. It recognizes that a balance between a structure's strength, stiffness and ability to dissipate energy is required for an effective design. In this respect, the structure's performance under *working stress, damageability*, and *ultimate strength* limit states is considered. In summary:

1. The structure should respond to minor tremors with no damage.
2. The structure should respond elastically to moderate earthquakes that can be expected to occur several times during its lifetime. After each event the maximum acceptable levels of damage are limited nonstructural and minimal structural damage.
3. In the event of the maximum credible earthquake, extensive structural damage, possibly beyond that of repair, is accepted. Collapse should nevertheless be prevented.

A rational design procedure would make provision for all limit states to be simultaneously taken into account. In this respect, DELIGHT.STRUCT makes no a priori judgement as to the limit state that is likely to control the design. All three load conditions are considered with equal importance. Each of the load cases is first *simulated* to find the *active constraints* within each group. The information gathered at this stage is only then utilized to *calculate* a refined design.

DELIGHT.STRUCT therefore makes a selective application of the above-mentioned design philosophy. It customizes the design process to each type of structure considered.

2. DESCRIPTION OF THE DESIGN PROBLEM.

A ten story, single bay friction-braced steel frame as discussed by Workman[4] is utilized as the basis for the *example* frame in this study. The *example* frame, however, has all ten floors braced and the concentric bracing system utilized by Workman[4] was replaced by a friction slipping system as discussed by Pall and Marsh[5].

2.1 Geometry and Gravity Loading

Figure(1a) shows the frame dimensions, starting beam and column moments of inertia and friction brace cross-sectional areas. The frame geometry was fixed throughout the optimization process and shearing deformation, out-of-plane deformation and end eccentricities were not considered in order to simplify the analysis.

Each of the girders was loaded with a uniform loading of 0.3 kips/in, to account for the weight of structural and non-structural components. It was assumed that live load would constitute 0.3333 of the total dead plus live loading.

2.2 Modelling

A good structural model should be capable of accurately reproducing the response of the *prototype structure,* without needless complication. The accuracy of results will in general be no better than that of the model's representation of the structure. One should consequently be aware of its formulation so that possible shortcomings may be examined and assessed as to their significance.

2.2.1 Beams and Columns

The beams and columns were modelled using the lumped-plasticity, parallel-component elements as shown in Figure 8 of Balling et al.[2]. The moment of inertia of each section was considered to be the primary unknown and empirical relations as introduced by Walker[6] for wide flange steel sections were employed to estimate member properties of secondary importance. Each member is therefore represented by a single design variable.

The range of permissible section moments of inertia, coefficients for the empirical relations and damping properties are summarized in Appendix 1. A strain hardening ratio of 0.05 was assumed for the hysteresis loops. The yield interaction relation adopted was as shown in Figure 9 of Balling et al.[2] and the parameters Yp and Ym were set at 1 and 0.15 respectively. The reader should also see Balling et al.[2], Section 2.2.3 for additional discussion of the element modelling.

2.2.2 Friction Braces.

The concentric bracing system utilized by the Workman frame is capable of supplying high stiffness and strength resistance against wind and moderate earthquake loadings. However, during a severe earthquake in which brace buckling is likely to occur, this system is characterized by pinched hysteresis loops, stiffness degradation and a large reduction in the capacity of the braces to dissipate energy[12].

A suggestion made by Pall and Marsh[5] was adopted for the investigation to mitigate this shortcoming. It is assumed that sliding friction devices capable of providing limited compression and tension resistance before sliding are installed in the bracing system. They are tuned so that story drift under moderate earthquakes is controlled without slippage. The constant force brace resistance is nevertheless set to ensure that brace slippage, with a redistribution of moments and forces throughout the frame occurs before the braces can buckle. Furthermore, experiments indicate that these devices are capable of being repeatedly cycled through rectangular hysteresis loops without strength deterioration[5]. During a severe earthquake, the bracing system acts to brake the motion, dissipate large quantities of energy and delay the onset of inelastic deformations in the beam and column elements.

For the purposes of this study, brace compression resistances were set to zero. The allowable tensile friction force was taken to be the product of section area times the brace yield stress. All constraint parameters relating to the brace buckling were removed and parameters set so that optimum cross section areas would not be influenced by the brace's inability to dissipate energy or provide a specified level of ductility. A strain hardening ratio = 0.005 for the braces was assumed.

2.2.3 Damping.

The Rayleigh damping model adopted by Balling has been used for this study. An explanation of its form and setup is located in Balling et al.[2], section 2.2.2. A damping ratio of 2% critical is applicable to frames of this type for low amplitude motions. These might typically be due to frequent wind loadings where it is assumed that all stresses remain within working stress load limits. For larger amplitude motions in which yielding at the joints may occur, the damping may increase to 5-7%[18]. Although a precise value is difficult to specify, it is acknowledged that extra dissipation can be expected in frames with bolted connections where joint slippage is a distinct possibility. The influence of non-structural components should also be recognized as their rubbing acts to increase the damping. Their presence is automatically accounted for in the mass matrix. A realistic stiffness representation is however difficult to formulate as the difference in stiffness degradation rates of main-frame and non-structural components would require unnecessarily complicated modelling. For this reason, the latter effect is ignored and the bare steel frame stiffness is used in this study. Furthermore, it is noted that for multi-degree of freedom structures the effect of inelastic deformation is to reduce the systems apparent vibrational frequency indicating in part a loss of stiffness[18]. An increase in the participation factor of the fundamental mode may also be expected[9]. The Rayleigh damping model will therefore be an approximation to the structure's damping. If the adopted damping value is nevertheless too large, the decreased response will lead to a non-conservative design. 5% damping in each of the lower two modes was consequently chosen as being most realistic in modelling the frame's response while not leading to a non-conservative design.

The program *Feap[16]* was used to find the frame's first two natural frequencies and associated mode shapes. These are shown in Figure(2).

2.3 Earthquake loading

A scaled version of the N40W component of a record measured on October 15, 1979 at the El Centro Community Hospital on Keystone Road was chosen as the single record for design optimization in this study. It corresponds to the E6 record as discussed in Balling et al.[2], section 2.1.3, and has a peak ground acceleration of *0.437g*. The spectral intensity over the period range 0.1 ~ 1.0 seconds is 24.4in. It was selected only because it was found to cause the most damaging structural response in a preliminary analysis of that study. The worthiness of this decision is examined in Chapters Six and Seven.

2.4 Desirable Inelastic Frame Response.

The combined action of gravity loads plus static lateral earthquake loading on the Workman frame may lead to two general mechanisms of collapse. They will either be *column sway* or *beam sway*. Examples of each are shown in Figure(3).

The former mechanism requires a single story to supply a high level of curvature ductility. If the columns are also required to carry a high axial load, the critical section's ability to supply the required plastic rotations is diminished. This could result in a catastrophic soft story collapse. For example, the lower floors of a tall frame will be particularly susceptible to this mode of failure should the *column sway* mechanism be permitted to form.

A hierarchy of probable beam and column strengths can however be imparted to the structure to encourage the commencement of inelastic beam deformations before incipient yielding occurs in the columns. A progressive beam side sway collapse has a lower demand of ductility on the girders and will be less serious in terms of the structure's overall safety.

The static lateral load bending moment diagram will be significantly modified in practice due to the effect of higher modes. Even though relative beam/column strengths may be specified to discourage a *column side-sway* mechanism, this objective can only be achieved in a probabilistic sense. In fact, the total preclusion of column hinging[1] would lead to an overly conservative design. A designer should therefore compromise by detailing for limited inelastic

[1] Except at the frame base.

deformation in the columns while also proportioning members to encourage the beam sway mechanism.

Dynamic response analyses will typically indicate a modified combination of the two mechanisms in which the total number of hinges forming is less than that required for the *beam-sway* mechanism.

3. THE OPTIMIZATION PROCESS.

In this chapter the seismic design problem is examined in the format of a constrained optimization problem.

3.1 The Method of Feasible Directions.

DELIGHT.STRUCT employs the method of feasible directions to solve a constrained optimization problem. The general method is outlined in Rao[14] and Nye[20]. A detailed discussion of the method used by the program is in Austin[17]. The key points in summary are:

1. The frame design process is transformed to that of finding a design vector representing the unknown member section size properties.
2. A series of constraints corresponding to each of the limit states in the design philosophy is formulated.
3. If the design vector X satisfies all the constraints, it is termed feasible. Otherwise, a constraint violation occurs and the design vector is said to lie in the infeasible domain.
4. The feasible direction method is a two step process. A direction vector is first calculated. The length of this vector is then adjusted to attain either one of the following two objectives. If the design vector is infeasible, the objective is to minimize the maximum constraint violation. The cost function is unimportant in this case. If the design vector is however feasible, the new position is chosen so as to minimize the cost function along the direction vector while simultaneously remaining feasible.

3.2 Constraint Functions.

Conventional, functional and *box* constraints are utilized to define the boundary between the feasible and infeasible domains. The former are scalar valued functions of the design vector having the form:

$$PSI(X) = \left[\frac{g(X)}{g(\,allowable\,)} \right] - 1$$

Functional constraints are represented as similar scalar functions maximized over time or some other independent parameter. They have the form[1]:

$$PSI(X,t) = \left[\frac{g(X,t)}{g(allowable)} - 1 \right]_{max\ over\ time} \quad where \ \left[T_{min.} \leqslant t \leqslant T_{max.} \right]$$

The functional and conventional constraints in DELIGHT.STRUCT are evaluated in the fortran routines *fconve.f* and *ffunct.f,* respectively. *Box* constraints limit the range of permissible section sizes[2]. Potential scaling problems are mitigated by mapping all N section size and dummy story drift elements in the design vector on to the range:

$$\left[-1 \leqslant X_i \leqslant 1 \right] \quad where \quad i=1,N$$

All members were constrained in the present design problem even though the columns were not subject to design. The total number of conventional constraints was 191 and the number of functional constraints totalled 111. A further subdivision of constraints is made for the frame response under gravity loads only, combined gravity and moderate earthquake only, and combined gravity and severe earthquake. The constraints applicable to each loading condition are now outlined.

3.2.1 Constraints under Gravity Loading.

The following conventional constraints apply to the beams, columns and braces under gravity loading only:

[column axial force] < Colax × Column axial force or Euler buckling load.

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1 The measured structural response ranged from $T_{min.} = 0\ sec.$ to $T_{max.} = 11\ sec.$ in this study.

2 See Appendix 1 for the range of section sizes.

$$[\textit{ column end moment }] < \textit{Colgra} \times \textit{Column yield moment.}$$

$$[\textit{ girder end moment }] < \textit{Girgra} \times \textit{Girder yield moment.}$$

$$[\textit{ girder midspan deflection under live load }] < \textit{Girdef} \times \textit{Girder span.}$$

The interactive constants set were Colax = 0.5, Colgra = 0.6,Girgra = 0.6 and Girdef = 0.00417.

3.2.2 Constraints under Combined Gravity and Moderate Earthquake.

The guidelines of the accepted design philosophy specify that the frame should respond elastically to moderate earthquakes. Non-structural damage should be limited and only minimal superficial structural damage is permissible. As the former will be related to interstory drifts and floor accelerations, the following two functional constraints are enforced:

$$[\textit{ absolute floor acceleration }]_{\textit{max over time}} < \textit{Accel} \times \textit{acceleration of gravity.}$$

$$[\textit{ story drift }]_{\textit{max over time}} < \textit{Drift.}$$

The interactive parameters *Drift* and *Accel* were set to 0.005 and 0.5, respectively. The constraint level for story drift is as recommended by the Uniform Building Code[11]. In addition, the following constraints were included to ensure an elastic frame response:

$$[\textit{ column end moments }]_{\textit{max over time}} < \textit{Colyld} \times \textit{column yield moments.}$$

$$[\textit{ girder end moments }]_{\textit{max over time}} < \textit{Giryld} \times \textit{girder yield moments.}$$

$$[\textit{ brace force }]_{\textit{max over time}} < \textit{Brayld} \times \textit{brace yield force.}$$

In this problem, *Colyld* = *Giryld* = *Brayld* = 1. We note that constraints pertaining to the brace buckling do not apply and have been removed.

3.2.3 Constraints under Combined Gravity and Severe Earthquake.

The design philosophy adopted accepts major structural damage, possibly beyond repair, resulting from a severe earthquake. Collapse is nevertheless prohibited. Large displacements at the top of the frame are used as an approximate measure of the possibility of collapse. Consequently, *Sway* defined as the maximum relative horizontal displacement at the top of the frame divided by the frame height is limited as follows:

$$[\,\textit{frame sway}\,]_{\textit{max over time}} < \textit{Sway}.$$

This parameter was set to 0.015. It is recognized in Balling et al.[2] that structural damage will be closely related to the extent of inelastic deformation. Furthermore, a single cycle at a high ductility range may cause damage equivalent to many cycles at a lower ductility range. The following constraint on inelastic energy dissipation under monotonic loading was adopted:

$$Ed < Ey \cdot [\,\mu - 1\,] \cdot [\,1 - S\,] \cdot [\,2 + S \cdot [\,\mu - 1\,]]$$

where $Ed =$ *Inelastic dissipated energy.*

$Ey =$ *Elastic strain energy at yield.*

$\mu =$ *Allowable ductility factor.*

$S =$ *Strain hardening ratio.*

The conventional constraints represented in this equation are:

Column end inelastic energy dissipation $<$ *f(Colduc)* $\times$ *yield strain energy.*

Girder end inelastic energy dissipation $<$ *f(Girduc)* $\times$ *yield strain energy.*

Brace inelastic energy dissipation $<$ *f(Braduc)* $\times$ *yield strain energy.*

The beam and column ductility factors were set to 6 and 3 respectively. The quantity of energy that may be dissipated by the sliding friction braces however exceeds that specified by this formulation. As previously noted, the braces are capable of undergoing repeated cycling through hysteresis loops with little deterioration. It is contended by Pall and Marsh[5] that large quantities of inelastic energy can be dissipated by the bracing. The remaining beams and columns are left to undergo minimal inelastic deformations. It was decided that the capacity of braces to dissipate energy should not be an active constraint throughout the design process[3]. Consequently, the parameter *Braduc* was arbitrarily set to 75. This not only ensured the brace energy constraints would not become active, but allowed the total energy dissipated by each brace to be calculated via the constraints performance information contained within the file *state*.

3.3 Cost Functions.

The present frame software allows any linear combination of the listed terms to be cost functions:

1. Volume of structural elements.
2. Moderate earthquake : Sum of squares of maximum story drifts.
3. Severe earthquake : Input energy.
4. Severe earthquake : Inelastically dissipated energy.
5. Severe earthquake : Energy dissipated by the columns.

A positive coefficient is specified if one wishes to minimize a term. Conversely, a term is maximized by specifying a negative coefficient. If more than one term is included in the cost function, the coefficients should be weighted to reflect the relative importance of each term in the structure's overall performance. The optimal choice of weighting coefficients for such a multi-objective cost criterion is a major task in itself, reflecting trade-offs among competing structural performance attributes. Accordingly, only single term objective functions were considered in this study.

A discussion of the contribution made by each listed term in describing the structure's lifetime performance is located in Balling et al.[2], Section 2.4.2.

[3] This is equivalent to saying that the braces ability to dissipate energy during the design process was effectively unconstrained.

4. PRELIMINARY ANALYSIS.

A preliminary analysis was implemented to establish the influence of the friction bracing on frame response. This was achieved by first calculating the constraints for the Workman friction-braced frame. The friction braces were then removed and the constraints recalculated for the moment-resistant frame. Thus, the difference in constraint percentages for each response approximately represents the range in constraint violations expected throughout the various designs obtained. Results, as shown for each case in Figures(5) to (12), will explained in the sequel.

4.1 Influence of Friction Braces on Response.

The program *Feap[16]* was employed to find the fundamental and second mode shapes and periods for both frames. The natural periods are [0.78,0.26] and [2.31,0.81] seconds for the friction-braced and moment-resistant frames, respectively. Modal shapes are shown in Figure(2).

The gravity load-case constraint percentages are the same for both frames, as shown in Figures (5) to (8). In general, they are much less than 100% and are therefore unlikely to control the design by becoming active during the optimization iterations.

The moderate earthquake absolute floor accelerations are graphed in Figure(10). The peak value in the lower floors of both frames approaches the peak ground acceleration[1]. A whipping effect due to the influence of higher modes in the more flexible moment-resistant frame leads to upper floor absolute accelerations approximately twice the peak ground acceleration.

The frame sway and story drifts [see Figure(9)] for the moment-resistant frame are unacceptably high[2]. The addition of friction bracing increases the frame's elastic stiffness. In

[1] 26% constraint is equivalent to a peak ground acceleration of 0.13g.

[2] The allowable frame *Sway* was set to 0.015. The moment resistant frame constraint was 100% after 6.34

general, the fourier amplitude of earthquake ground motion about the braced-frame fundamental period will be larger than for the moment-resistant frames fundamental period. Increased base forces imparted to the braced structure would be expected to amplify story drifts. However, the enhanced structural stiffness more than offsets this change. Resultant inter-story drifts are reduced accordingly. Figure(12) shows that the moderate earthquake girder end moments are reduced to an acceptable level when friction-bracing is added to the frame.

In the severe earthquake, brace slippage is accompanied by a redistribution of forces throughout the frame. Figures(4) and (11) show that inelastic girder deformation is confined to the lowest seven floors of the friction-braced frame. A small amount of inelastic deformation also takes place on one side of the moment-resistant frame eighth floor. The columns remain elastic in both frames.

Plots of frame earthquake input energy, dissipated energy and work done by loads are shown in Figures (13) to (18)[3]. Input energy is the inner product of the shear force at the frame base moving through the ground displacement. As previously mentioned, the average base shear of the Workman frame is greater than for the moment-resistant frame. Although a 50% reduction in total girder energy dissipation is achieved by the addition of bracing, this is at the expense of the braced frame having to dissipate a total of fifteen times more energy than the moment-resistant frame. Figure(16) shows that most of the energy dissipated by the girders is in two pulses located in the 2.5 ~ 4.0 second range of the response. The frames apparent frequency of vibration will be significantly reduced during these intervals. However, since the intervals of inelastic deformation are small for the moment-resistant frame, this effect on the overall frame response may be minor. The cumulative distribution of energy dissipation for the friction-braced frame is shown in Figure(13). It shows energy being dissipated in high frequency ripples that occur at a near uniform rate over the entire frame response. This suggests that although the fundamental period will be reduced due to inelastic deformations, the friction-brace deformations themselves are controlled by the higher modes. The work done by applied loads is shown in Figures(14) and (17) for the Workman and moment-resistant frames

seconds into the response. The maximum braced-frame sway constraint is 69% after 6.2 seconds.

[3] The differential energy formulation is outlined in Balling et al.[2], Section 2.4.1.

respectively. This quantity is calculated as the opposite of the sum of the remaining terms in the energy balance equation presented in Balling et al.[2], Section 2.4.1.

4.2 Constraint Hierarchy.

A hierarchy of braced-frame constraint percentages was formed. The criterion used was to subjectively decide whether or not a constraint would be likely to become active in the direction vector calculation. In summary:

Inactive Constraints.

Column end bending moment : gravity loading.

Girder end moment : gravity loading.

Girder midspan deflection : gravity loading.

Absolute floor acceleration : moderate earthquake.

Total frame sway : severe earthquake.

Active Constraints.

Girder end moment : moderate earthquake.

Story drift : moderate earthquake.

Girder end energy dissipation : severe earthquake.

Frame Sway : severe earthquake.

Graphs of girder end moment, story drifts, absolute floor accelerations and girder end energy dissipation are presented for the optimal designs in Figures(27)-(30). Variations in the so-called *inactive* constraints are not presented herein.

4.3 Formulation of the Optimization Problem.

The frame elements are each modelled by a single unknown section property parameter. The capabilities of DELIGHT.STRUCT allow a member to be subject to design or to be fixed at its initial size. Elements may also be constrained in groups to take equal sizes. In practice a balance in objectives is required. Repetition of equally sized elements implies economic construction. The computational work needed at the optimization stage is approximately proportional to the number of design variables so grouping reduces the required calculation. On the other hand, section sizes chosen within each group will be bounded by the most critical constraint within the group. Hence, grouping should retain flexibility in the optimal design while simultaneously keeping the problem practical in terms of element repetition and required calculation.

The preliminary analysis indicates that the starting design is feasible. Inelastic girder deformations were confined to stories one through seven and the columns remain elastic throughout the response for both frames. On the basis of these observations, restrictions on computer funding, and comments presented in Sections 1.2 and 2.4, it was decided to initially fix the columns at their starting section inertias and design only the beams and braces. The beams were subjectively divided into three groups and the braces into five as shown in Figure(1c).

The dimension of the design vector is 18. The first eight elements correspond to unknown section parameters. The remaining elements are dummy parameters utilized when story drifts are included in the objective function.

5. OPTIMAL DESIGNS.

Since the initial design was feasible, the remainder of this study examined the sensitivity of designs to the choice of objective functions. The sensitivity of of response to modelling parameters was also investigated. An additional objective was to find acceptable bounds on some of the optimization algorithm parameters.

The cost of materials in construction is roughly proportional to member volume. Although this is commonly of lesser importance than other factors when looking at the structure's lifetime cost, minimum volume has traditionally been employed as the cost function to minimize. It was therefore chosen as the starting objective function for this investigation. Five iterations were completed with this cost function. It was then decided to minimize total frame dissipated energy for two more iterations. Finally, after the designer intervened to modify the brace areas, three more iterations were completed with minimum story drift as the cost function.

The results of each main section are now discussed in detail.

5.1 Minimum Volume Design.

Utilizing minimum volume as a design objective reflects a typical design philosophy. Although volume is correlated to material cost, a modest material saving may be of lesser importance than other possible objective functions when considering the structure's expected lifetime performance and cost. Nevertheless, minimum volume was used as the starting point for the study in-as-much as it may reflect the minimum initial cost of the structure.

The total starting volume of design elements (girders + braces) was 53420 in^3. As the braces constituted 15.7% of this quantity, the main influence of this function would be to decrease girder volume. Five iterations of optimization were calculated. The major section changes occurred during the first two iterations, as shown in Figures(19) and (20). The remaining three iterations were carried out to verify equilibrium. The results in summary are[1]:

[1] See Figure(1b) for an element key.

Lower girder	element 21	2.7% section inertia decrease.
Middle girders	elements 22-25	17.2% " " "
Upper girders	elements 26-30	30.5% " " "
Lower braces	elements 31-34	11.2% volume decrease.
Middle braces	elements 35-38	12.9% " "
Upper braces	elements 39-50	13.9% " "

The final design element volume is 47170 in^3. This represents an overall brace/girder volume decrease of 11.7%. The ratio of brace to total volume remained constant at 15.4%. The reduction in middle and upper girder section inertias is significant in terms of reduced material cost. However, the 12.6% brace volume decrease should be gauged against the change in other objective functions.

The decrease in brace sizes is predominantly responsible for the sum of story drifts squared increasing from 0.0001012 to 0.0001178. It will also be influenced to a lesser degree by the decreased rotational joint stiffness in the upper floors resulting from the reduced girder inertias.

The total frame dissipated energy decreased from 20300 to 18190 kip-in [see Figures(25) and (26)]. As more than 95% of this quantity is via mechanical brace friction, the reduction is associated with the decrease in brace cross-sectional area. The distribution of energy dissipated by the bracing is shown in Figure(26). The major change between the starting and minimum volume designs is an approximate 10% reduction of energy dissipation in the lowest five floors. The quantity of energy dissipated in the upper floors remained constant even though a reduction in brace area occurred. A corresponding constraint functions increase for girder end energy dissipation in the upper girders is shown in Figure(30). The energy dissipated by the lower girders remained unchanged.

A total of 90 simulations was necessary to complete the 5 iterations. Each iteration required an average of 558 time steps. It is noted that this average will represent the split between the times a full 1100 time-step constraint calculation is needed and the times a when partial derivatives of the constraints with respect to the design vector are required. In the latter case, the simulation would only be completed up to the time of maximum response, ie, approximately the 400th time step. An average of 0.96 stiffness reformulations per time step was needed. Of the 418617 seconds (4.8 days) of real time taken by the VAX 11/780 to complete the five iterations, only 76108 seconds was actually spent within the routines. As expected, the largest time user was the ANSR[21] structural analysis subroutine *analys.*

During the iteration process the design vector moved against the sixth floor girder end moment constraint under moderate earthquake loading. In fact, the second, third, fourth, sixth, and seventh floors right and left-hand girder moment constraints are in excess of 90% for this design. An examination of Figures(25) and (27) to (30) shows that this set of constraints is most critical in terms of limiting the design. Furthermore, the sixth floor girder is the lowest in the upper group. If the girders had been subjectively grouped in a less restrictive manner, even a smaller reduced volume would have been obtained.

5.2 Minimum Dissipated Energy Design.

The optimization parameter *Deltax* in DELIGHT.STRUCT is the design vector element perturbation made when computing the jacobian matrix at the direction vector stage of application of the feasible directions algorithm. The default was set at 0.0002. If the expected change in cost function or maximum constraint violation is small, the roundoff error Order(*Deltax*) in the derivative calculation may be comparable to the slope of the derivatives themselves. In such cases, constraint derivatives of incorrect sign can occur, leading to an increase in constraint violation or cost function along the direction vector. The feasible directions method fails unless the derivatives can be estimated more accurately. This requires *Deltax* to be reset at a smaller value. As a guideline, it should be made as small as possible without entering the realm of numerical roundoff.

Throughout the minimum volume design *Deltax* was set to 0.0001. It was decided to decrease *Deltax* to 0.00001.

In the event of a severe earthquake the structure will be required to supply sufficient strength and ductility to withstand the excitation without frame instability and collapse. Given that this constraint is met, the designer still has freedom to choose how the frame responds. This choice in general will be influenced by the functional planning requirements and the acceptable levels of damage after the event. In this respect, the following energy designs are conceivable.

1. The functional requirements may require that the beams and columns remain elastic. Dissipated energy would be confined to the friction-bracing[2]. An appropriate cost function would be to find either the minimum structural volume or minimum energy dissipated by the friction-bracing. The latter cost function can be interpreted as minimizing the possibility of the friction-brace mechanism malfunctioning after incipient slippage[3].

2. If inelastic deformation throughout the frame is permitted, the possibility of excessive localized yeilding in the frame is reduced. The extent of inelastic deformations in the beams and columns will be approximately proportional to the total structural damage. This concept of structural damage does not apply to the bracing. Consequently, a feasible energy design would be to allow limited beam and column inelastic deformation and require the friction-bracing to dissipate the remainder of the energy. For the deterministic earthquake input a minimium dissipated energy design with a constraint on maximum permissible volume appears reasonable.

3. Another permissible energy functional is to minimize the integral of kinetic energy for the frame response. This cost function would reflect the structure's ability to dissipate internal energy at approximately the same rate as that of the work done on it by external loads.

Results from the preliminary analysis indicate in excess of 95% of the total frame energy dissipated is via the friction-bracing. In addition, the maximum energy constraint function for

[2] This occurs in Chapter Six for the response of the final design to ground motions E3 and E4. See Figure(42).

[3] An elastic frame response may not always be possible due to constraints on maximum allowable element sizes.

the girders was approximately 70%. It was decided to follow the guidelines of the second energy design[4]. Since the total energy dissipated in the minimum volume design was less than the Workman frame, the former was taken as the starting design. The allowable girder ductility was set to 6. A maximum allowable volume increase of ten percent was implemented as a constraint.

Figures(21) and (22) show design vector values for the two iterations completed. A mere 0.6% decrease in total frame dissipated energy was obtained. This was accompanied by redistribution of girder inertias and brace cross-section areas at the first iteration. Negligible changes were recorded at the second iteration. The results in summary are:

Lower girder	element 21	unchanged
Middle girders	elements 22-25	5.2% section inertia decrease.
Upper girders	elements 26-30	1% " " "
Lower braces	elements 31-34	1.3% volume decrease
Lower braces	elements 35-38	3.1% " "
Middle braces	elements 39-42	5.2% volume increase
Upper braces	elements 43-50	1.0% " "

The overall design element volume changed from 47170 to 46170 in^3. This represents a 2.1% total volume decrease. It is noted that the brace volume remained constant at 15.5% of the total design volume. An average of 31 simulations per iteration was required. Each simulation needed an average of 742 time steps. The average number of stiffness formulations per time step was 0.93.

The most active constraints were the moderate earthquake girder end moments. The changes in all critical constraint violations from the minimum volume design were negligible. Consequently, the contours of Figures(25) to (30) for the minimum volume design apply to

[4] The first energy design could have easily been implemented by setting μ to 1 and requiring the design to move towards the new feasible domain.

this section as well.

It is noted that although a reduction in dissipated energy has been achieved, it is unlikely that the value obtained represents a global minimum. If the member sizes were all significantly increased, an almost elastic response, having close to zero dissipated frame energy is conceivable. However, an increase in the frames natural periods occurs with a decrease in member sizes in addition to that caused by the inelastic deformation. If the fourier amplitude of ground motion locally decreases with increasing period, the frame will be subject to reduced base forces and thus a reduced response. Figure(46) indicates that for the E6 record this is the case for natural periods greater than 1.5 seconds.

5.3 Minimum Story Drifts : Final Design.

The study was continued by changing the cost function to minimize the sum of the squares of story drifts. This quantity is related to the expected non-structural damage resulting from one or more moderate earthquakes likely to occur throughout the frames lifetime.

An examination of the minmum volume, dissipated energy and starting Workman frame designs showed the latter to have the lowest sum of story drifts squared [see Figure(27) and Table(1)]. It has the highest stiffness and consequently is closest to modelling the ideal minimum story drifts: rigid body motion[5].

The Workman frame was nevertheless not chosen as the starting frame, because in practice we are not trying to get three completely independent designs, rather a single design that incorporates a balance of desirable features from various design objectives (cost functions). A modified form of the starting Workman frame was chosen accordingly as the beginning design. The girder sizes from the minimum dissipated energy design together with the starting Workman frame brace areas were combined to define a new frame. A 10% maximum allowable volume increase over the minimum dissipated energy design was also imposed ; ie Volmax = 49100 in^3. The starting volume with the increased brace sizes was 48930 in^3. Thus, the objective of this section was to obtain a *final design* rather than a minimum story drifts design *per*

[5] This condition will only become possible when member sizes are unconstrained.

se.

Figures(23) and (24) show that all girder inertias and brace areas remained essentially unchanged for the two iterations completed. The braces constituted 17.1% of the total design element volume. A small volume decrease from 48930 to 48890 in^3 occurred. This result is opposite to what one initially expects. However, the frames response to the moderate earthquake is elastic. Figure(46) shows that the response spectra for the E6 record about the 0.6 ~ 0.75 second period range locally decreases. The sum of story drifts squared is therefore reduced for the same reasons as outlined in Section 5.2.

The value of this quantity in the final design is 0.0001071. This compares to 0.0001012 for the starting Workman frame, 0.0001178 for the minimum volume design and 0.0001181 for the minimum dissipated energy design[6]. The story drifts of the final design are between the starting and minimum volume designs. In all cases, the constraint does not exceed 90% of the 0.005 allowable value. This is plotted in Figure(27). The peak absolute floor accelerations are shown in Figure(28). This quantity is insensitive to the various designs considered and takes a mean value close to the peak moderate earthquake ground acceleration. It is noted that a small decrease in upper frame floor accelerations results from the decreased girder inertias.

The brace energy dissipated by the final design is distributed in a manner similar to the starting Workman frame.

The active constraints for this design are the dummy story drift parameters.

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[6] See Table(1) for a complete summary.

6. SENSITIVITY ANALYSIS.

The purpose of this section is to address the issue of frame damage sensitivity to modelling parameters in the optimization-based design process.

In general the usefulness of structural response calculations will be no better than the accuracy with which the model describes the real structure's behavior. Cost function quantities such as story drifts, input energy etc, are intended to measure the degree of damage imparted to the structure. These values will be related to structural response. If the model response is however sensitive to both the choice of modelling parameters and those describing the earthquake record input, the subsequent design refinements may not be reliable.

The most sensitive parameter influencing the structural response is the percentage of critical damping assumed. As discussed in Section 2.2.3, structural damping is dependent on the amplitude of vibration and can range from 2 - 7%. The Rayleigh damping matrix in DELIGHT.STRUCT is based on the starting frame member sizes and held constant throughout the optimization process. It is acknowledged that this approach will only be an approximation and therefore should be considered in a sensitivity analysis.

The earthquake record employed for the design process was subjectively selected from a group of El Centro records scaled to have equal spectral intensity. The purpose of scaling the records is to approximately equalize the group in terms of ability to impart damage to the structure. The response sensitivity to record input was divided into two sections. Spectral intensity of the E6 record was first perturbed to observe the constraint sensitivities. The remaining records from the group were then simulated to look at the variation of constraints and the effectiveness of the scaling procedure adopted by Balling et al. [2].

The details of each analysis are now discussed. This is followed by a general discussion of results and implications to be drawn from the analysis.

6.1 Perturbed Damping

The final design was simulated with 4, 5 and 6% Rayleigh damping. Graphs of constraint functions are shown in Figures(31) to (34).

6.2 Perturbed Spectral Intensity.

The ground motion used for the designs was a scaled version of the 1979 El Centro earthquake. It corresponds to the E6 record employed in Balling et al.[2]. The peak ground acceleration and spectral intensity of the scaled record was $0.437g$ and $24.5in$ respectively.

A perturbation analysis was carried out by modifying the scaling factors to give records of spectral intensities $0.9 \times 24.5in$ and $1.1 \times 24.5in$. Frame simulations were then completed to evaluate the constraints. They are plotted in Figures(35) to (40).

6.3 Response to Other Scaled Records.

The response of the final frame design was simulated using four other El Centro records scaled to equal spectral intensity. By examining the order of constraint variation it was hoped to assess the present scaling procedure for its ability to produce records leading to structural responses causing equal damage.

The records selected correspond to E2 through to E5 of Balling et al.[2]. In his study, the worst 10 seconds of each record was first isolated. The records were then scaled so that each would have equal severe and moderate spectral intensities over the period range 0.1 to 1 seconds while simultaneously ensuring a peak ground acceleration of $0.5g$ in the group. A summary of the scaling factors, peak ground accelerations etc; is given in Balling et al.[2], Figure(3).

The response constraints are shown in Figures(41) to (45). Several numerical problems were encountered at this stage with DELIGHT.STRUCT. They are outlined here:

Although the scaled E1 record was simulated the results are not included in this section. The reason is that the maximum lower frame moderate earthquake absolute floor accelerations obtained were of the order $0.4g$ and occurred during the first 0.5 seconds of the response. Accelerations of this order exceed the peak ground acceleration and are obviously in error. It is the authors' opinion that the error is associated with the Newmark integration method. The version adopted assumed constant acceleration across a time step. The method requires that the initial input be smooth, or an over-shoot problem will occur. An examination of the first 25 entries of this record shows wild acceleration fluctuations. The results were thus disregarded.

The same numerical problem also extends to the E2 record. We note that the 1-2, 3-4, story constraints are 25 and 27% respectively. The remaining constraints are significantly lower. The peak response for these two floors occurs after 0.6 seconds. The remaining peak floor accelerations occur after two seconds in the response. Once again, these two results cannot be considered to be reliable.

6.4 Discussion

The following points are noted from the analysis:

1. A summary of story drifts and dissipated frame energies for the optimal designs and the perturbed parameter frame simulations is given in Table(1).
2. The record chosen for the optimization process has the maximum final design response constraints.
3. The constraint deviations due to the 4 and 6% simulations are approximately equal to those of the *0.9* and *1.1* factored spectral intensity structural responses. Furthermore, the relative constraint sensitivities within each group are similar to the constraint group changes noted through the design process. For example, the floor acceleration constraints were insensitive to the various designs considered. A small variation in constraints with perturbed damping was similarily observed.
4. The girders and columns remain elastic for the E3 and E4 records. The bracing energy dissipation demand is approximately half that required for the E2 and E6 responses, as shown in Figure(45). Table(1) also indicates that the dissipated energy and sum of story drifts squared is significantly less for these two responses. It is therefore concluded that

the records utilized do not impart equal damage to the frame.

It is the authors' opinion that this discrepancy is due to the scaling procedure employed to equalize the record spectral intensities. Figure(46) shows that although the records have equivalent intensities over the period range 0.1 ~ 1.0 seconds, a wide range of values occurs above the 1 second period range. It is noted that the structure's first two natural periods fall within the scaled range. However, the effect of inelastic action will be to decrease the apparent frequency of frame vibration. It is therefore intuitively reasonable to expect the structure's response to be dependent on the total period range of spectral velocities. An alternative scaling scheme is outlined in Appendix 2.

5. In retrospect, the selection of the worst segment of ground motion acceleration to economize on computer time is not a reliable procedure, unless smoothing of initial conditions is also utilized. In view of the difficulty encountered in scaling, it appears that at least when making design sensitivity studies and comparing design objectives, use of a family of simulated ground motions would provide a more reliable loading. In this respect, see [15].

7. SUMMARY AND CONCLUSIONS.

A report summary and conclusions are now presented, along with a subjective assessment of DELIGHT.STRUCT based on experience gained from the design problem studied. The reliability of the results is discussed and suggestions for further work are outlined. Finally, a summary of findings relating to the design of friction braced frames is given.

7.1 Assessment of DELIGHT.STRUCT.

DELIGHT.STRUCT is presently located on a VAX 11/780 virtual memory machine. The following points summarize the authors' opinions of the performance of this environment.

1. Although the present documentation does describe DELIGHT.STRUCT, its complexity is a little overbearing for the uninitiated *user* who just wants to use the program once to solve a problem. The writing of a cookbook type guide that exemplifies the program's key features and works through an example problem in a step-by-step manner could help to mitigate this problem. In this respect, see [17].

2. As demonstrated by the minimum volume design, the program can run for extended periods (4.8 days) before completing a task. A program of this size and complexity ideally requires a machine to itself rather than a time sharing environment. It is recalled that the majority of this period was taken up with the swapping of data in the machine. Furthermore, the number of design variables was 8, distributed among 50 elements. A problem of this size represents the lower end of any realistic design problem. Yet, the capabilities of the VAX 11/780 were considerably taxed. It is nevertheless acknowledged that enhanced computing facilities will become available in the near future. Consequently, it should only be a matter of time before the solution of large scale problems will be possible in a more reasonable time interval.

3. The Method of Feasible Directions does not guarantee a global minimum within the feasible domain. In fact, convergence to a local minima will generally occur. When story drifts are being minimized, the numerical method may appear incapable of increasing brace areas to achieve this objective. In a similar manner, when dissipated energy is being minimized, an increase in member sizes could lead to an elastic response. This would give zero dissipated energy. The *designer* should be prepared to employ engineering judgement to override the algorithm and reposition the design vector so that the required convergence can occur.

4. It is the authors' opinion that only in rare cases should minimum story-drifts be used as a single objective function. The resulting *rigid body* design will clearly be uneconomical. However, it could be used in conjunction with other cost functions. For example, if it is minimized together with volume, the former quantity can reflect the cost due to expected structural damage and the latter the increased cost due to additional construction material required. Cost function coefficients need to be chosen to reflect the relative importance of each term. The selection of these weighting factors is presently subjective and represents a completely new research problem in itself.

5. The present ANSR model assumes Rayleigh damping. Section 2.2.3 discusses the criteria for its choice. It is noted, however, that the damping matrix remains constant throughout the optimization procedure. Yet, the sensitivity analysis indicates that damping is an important modelling parameter and the expected response will be sensitive to minor perturbations in the Rayleigh damping assigned. Consequently, the *user* must have a good design to start with and regard the optimization as *possible refinement.* An optimal design having major differences in section size throughout the frame from the starting design should not be accepted if the present program structure has been employed.

7.2 Reliability of Results.

The optimization calculations in this report are based on the performance of the Workman Frame excited by a single scaled ground motion at a specified site. In summary :

1. The basis of earthquake record selection for this study was that it caused the most damaging response from a group simulated in a preliminary frame study by Balling et al.[2]. No attempt was made in this investigation to find a more damaging record a priori.

2. The minimum dissipated energy and story drifts designs represent local minima for each objective function within the feasible domain. The E6 plot in Figure(46) suggests that the fourier amplitude of ground motion locally decreases with increasing period over the 0.6 ~ 0.75 and 1.5 ~ 3 second ranges. The former range is important for the story drifts design and the latter for the energy design. Acceptance of a design influenced by the frequency content of a single record may be unwise since not all the remaining records follow this trend. It should be verified that its frequency content distribution is representative of an ensemble of ground motions to be expected at a given site.

3. The results of the sensitivity analysis indicate that the same record causes the maximum overall final frame design response from the group. However, it has been shown that the scaling procedure adopted is defective in its attempt to impart approximately uniform damage to the structure. The revised scaling procedure in Appendix 2 appears to give more uniform results in so far as the coefficient of variation of frame dissipated energies is reduced from 0.246 to 0.017. The scaling of earthquake records to a peak ground acceleration would seem inappropriate for the design of structures having a long fundamental period[19].

4. The two major areas of uncertainty in the optimization procedure are the loading input and the structural modelling. Parameter perturbations in both areas resulted in constraint variations larger than those due to the change in designs produced by the optimization. The final design is thus sensitive to the setting of these parameters.

5. Since the starting design was based on code recommendations, the final design's conservatism might be gauged by comparing the ground motion input to the earthquake records that influence the code recommendations. If the input is similar, one could be led to accept the final design in the knowledge that the code requirements have been met. However, the purpose of the code is to ensure most structures perform in a manner that provides the general public with the minimum acceptable requirements of protection against

unsatisfactory performance. The credibility of this implicit *level of protection* is largely based on historical precedence. If a designer wishes to provide an enhanced *level of protection,* it is difficult to quantify improvements when there is no clear definition of what constitutes a minimum value nor how it may have changed with time.

A probabilistic approach to design would appear to be a more rational manner in which to explicitly describe the level of protection supplied. It should quantitatively take into account uncertainty in the choice of modelling parameters and incorporate estimates of scatter for the inherent randomness of ground motions expected at a site. Error bounds on the subsequent frame response should also be defined. In addition, a design should be qualified with a complete description of loading inputs, modelling constraints and performance criteria. Since the present environment falls short of meeting these requirements, the question of *results reliability* remains unresolved. The final design *may* or *may not* be unduly conservative. This is not seen as a major shortcoming of the investigation, rather identification of a problem in the hope that work will eventually be done to reduce this deficiency.

7.3 Suggestions for Continued Work.

Although further problem areas could be investigated with the present structure of DELIGHT.STRUCT, eg : eccentric braced frames, the quality of results will contain the same input and modelling uncertainties as presently contained in this report. It is not the authors' intention to discourage investigation into new problem areas, rather to make the *designer* aware of the problems and limitations inherent in the present environment.

In summary, some of the required modifications to be considered in future software development are:

1. Since the distribution of frequency content in a ground motion influences the frame response, the use of a single scaled record for both moderate and severe earthquake loadings may not lead to a conservative result. In this respect, it may be worthwhile using different records for each load case. Their choice would be influenced by the natural

periods of the structure in both the elastic and inelastic ranges. As an example, the frame response due to a moderate ground motion for a structure with a long fundamental period would probably be more critical for a large magnitude event at a large epicentral distance than for a small earthquake occurring nearby. The spatial and temporal nature of these events could be incorporated into a probabilistic approach to design.

2. The ground motion input must be more comprehensively defined in the future. Work needs to be done to find descriptive parameters capable of distinguishing differences between records that significantly influence structural response. This is the most important area to be examined as the quantification of gains in optimization are conditional on a prescribed ground motion input. The present use of scaled earthquake records could be replaced by ARMA models[15] for generation of ground motion records.

3. Under the present program structure the grouping of elements is set at the *finput*[1] stage and held constant throughout the design procedure. As the calculation proceeds, the *designer* has no means of regrouping the elements without returning to the *finput* stage. By including a regrouping capability, the *designer* could initiate a calculation with only a few unknowns and successively reorganize and subdivide groups as constraints moved against constraint boundaries. Graphical display of constraint violations at the end of each iteration would aid the *designer* in formulating a grouping strategy.

   Another useful option capability would be to allow elements to be restrained not so they take equal values, but rather that their ratio of sizes remains constant. This would have been useful for the Workman frame design. The initial column sizes could have been included as a single parameter and an overall size reduction calculated. With the present program structure the columns have to be considered as a minimum of five groups.

4. The damping matrix should be updated at the end of an iteration if a major section size change has occurred. Further work on the choice of damping parameters should also be implemented.

[1] Frame input.

5. Only single term cost functions were utilized in this study. However, as previously mentioned, story drifts could be combined with volume in a cost function. The relative weighting of coefficients in the objective function is presently subjective. Guidelines as to an appropriate setting should be established.

6. The *designer's* ability to interprete algorithm performance could be increased by incorporating into DELIGHT.STRUCT both the *gradient clock* and *performance comb,* as discussed by Nye[20]. The former aid is used to look at the angles between the search direction and the remaining active constraints. The *performance comb* gives a graphical display of *good* and *bad* constraint performance[2].

7.4 Friction-Braced Frames.

Notwithstanding the various limitations associated with this study, the following conclusions pertaining to the design of friction-braced frames are given:

1. The starting Workman frame was designed by code recommendations. A 30% reduction in section moment of inertia for the upper five frame girders was permitted with minimum volume as the objective. If at a later date the ground motion input employed is found to lead to a conservative level of protection against expected lifetime structural damage, this saving can be regarded as significant.

2. Friction bracing acts to decrease the damage imparted to the structure in the event of occurrence of a severe earthquake. Story drifts, moderate earthquake girder stresses and frame sway were all controlled to within acceptable limits by this system. The results show that girder energy dissipated after the addition of bracing is approximately half that dissipated by the moment-resistant frame. However, this is at the expense of the braced frame having to dissipate approximately 15 times more energy than the moment-resistant frame.

[2] In [20], *good* and *bad* constraint performance values are supplied by the *designer* when solving the multiobjective constrained optimization problem. A modified version of that presently used by Nye[20] could be built into DELIGHT.STRUCT.

3. Figure(26) shows the required demand on dissipated brace energy for each floor to ensure adequate frame response performance. If the bracing system malfunctions the response will approach that of the moment-resistant frame simulated at the preliminary design stage. Thus, if systems of this type are to be installed in frames, they must have reliable performance and be capable of dissipating the required quantity of energy.

8. REFERENCES.

1. R.J. Balling, K.S. Pister, and E. Polak. "DELIGHT.STRUCT: A Computer-Aided Design Environment for Structural Engineering", Report No EERC 81-19, Earthquake Engineering Research Center, Univ of Cal,Berkeley,Ca,December,1981.

2. R.J. Balling,V Ciampi,K.S. Pister, and E. Polak. Optimal Design of Seismic-Resistant planar Steel Frames, Report No EERC 81-20,Earthquake Engineering Research Center,Univ of Cal,Berkeley,Ca,December,1981.

3. M.A. Bhatti," Optimal Design of Localized Nonlinear Systems with Dual Performance Criteria under Earthquake Excitations, Report No 79-15,Earthquake Engineering Research Center,Univ of Cal,Berkeley,Ca,July,1979.

4. G.H. Workman, "The Inelastic Behavior of Multistory Braced Frame Structures Subjected to Earthquake Excitation", Research Report,University of Michigan,Ann Arbor; Sept,1969.

5. A.S. Pall,C. Marsh,"Response of Friction Damped Braced Frames", Journal of the Structural Division, ASCE, Vol(108), pp 1313-1323, No ST6, June 1982.

6. N.D. Walker, "Automated Design of Earthquake Resistant Multistory Steel Building Frames", Report No 77-12,Earthquake Engineering Research Center,Univ of Cal,Berkeley,Ca,May,1977.

7. R.W. Clough, "Deficiencies in Current Seismic Design Procedures", Civil Engineering Frontiers in Environmental Technology, A Program of Public Lectures to Commemorate the Dedication of Raymond E. Davis Hall, Dept of Civil Engineering,Univ of Cal, Berkeley, Ca,1971.

8. T. Paulay, "The Design of Reinforced Concrete Ductile Shear Walls for Earthquake Resistance." Report No 81-1. Dept of Civil Eng. Univ of Canterbury, Christchurch, New Zealand. January 1981.

9. C.W. Roeder,E.P. Popov, "Eccentrically Braced Steel Frames for Earthquakes", Journal of the Structural Division. ASCE,pp 391-412,Vol(104),No ST3,March 1978.

10. A.S. Veletsos,W.P. Vann,"Response of Ground-Excited Elastoplastic Systems." Journal of the Structural division, ASCE, pp 1257-1281, Vol (97), No ST4, April 1971.

11. Uniform Building Code, International Conference of Building Officials, Whittier, Ca, 1979 Edition.

12. V.A. Zayas,P.B. Shing,S.A. Mahin and E.P. Popov, "Inelastic Structural Modeling of Braced Offshore Platforms for Seismic Loading." Report No 81-04 Earthquake Engineering Research Center, Univ of Cal,Berkeley,Jan 1981.

13. R. Park and T. Paulay,"Reinforced Concrete Structures" John Wiley and Sons, New York, 1975, 769 pp.

14. S.S. Rao , " Optimization Theory and Applications " John Wiley and Sons, New York, 1979, 711 pp.

15. R.F. Nau,R.M. Oliver and K.S. Pister," Simulating and Analysing Artificial Non-stationary Earthquake Ground Motions." Operations Research Center, 80 - 16, Univ of Calif, June 1980.

16. O.C. Zienkiewicz, R.L. Taylor, " The Finite Element Method - Chapter 24. " 3rd edition, McGraw-Hill, New York, 1977, 787pp.

17. M.A. Austin, K.S. Pister," A Guide to the use of DELIGHT.STRUCT ". UCB/SESM Report No 83-07, Univ of Cal, Berkeley, June 1983.

18. N.M. Newmark, W.J. Hall, " Earthquake Spectra and Design. " - A Primer. Earthquake Engineering Research Institute, Berkeley, California 1982.

19. R.P. Kennedy, " Peak Acceleration as a Measure of Damage ". 4th International Seminar on Extreme-Load Design of Nuclear Power Facilities, Paris, France, August, 1981.

20. W.T. Nye," DELIGHT : An Interactive System for Optimization-Based Engineering Design." UCB/ERL M83/33 Univ of Cal, Berkeley, Ca, May, 1983.

21. D.P. Mondkar, G.H. Powell, " ANSR - I General Purpose program for Analysis of Non-linear Structural Response ", Report No. EERC 75-37, Earthquake Engineering Research Center, Univ of Ca. Berkeley, Ca, December, 1975.

APPENDIX 1.

The following listing is a copy of the file *assumptions* used by DELIGHT.STRUCT for the Workman frame. Loads have the units of *kips* and lengths are in *inches.*

```
Ratio of live uniform load to total uniform load = 0.33333
Global damping ratio = 0.05
Ratio of number of stories to fundamental period =12.77
Ratio of fundamental period to second period     = 2.95
Young's modulus for steel = 29000.
Yield stress for steel    = 36.
Strain hardening ratio for steel = 0.05

For columns:

50. < inertia  < 4500.
moment yield coordinate fraction = 1.
axial yield coordinate fraction  = 0.15
radius of gyration = 0.39 * depth ** 1.04
for inertia <= 429.
depth  = 1.47 * inertia ** 0.368
otherwise
depth  = 10.5 * inertia ** 0.0436

For girders:

80. < inertia < 2500.
steel poisson ratio = 0.3
radius of gyration  = 0.52 * depth ** 0.92
depth               = 2.66 * inertia ** 0.287

For braces:

0.5 < area < 10.
inertia = 0.169 * area ** 3.
```

APPENDIX 2.

As outlined in Section 6.3, the purpose of scaling different records at the same site to have approximately the same spectral intensity is to ensure that the damage imparted to a structure by each ground motion will be approximately of the same order. This provides a basis for examining uncertainty in the frame response due to inherent randomness of the ground motions.

Section 6.4 reviews the shortcomings of directly equating ground motion records on the basis of spectral intensity integrated over the range of elastic frame periods. An alternative approach to that adopted by Balling et al.[2] would be to first premultiply the pseudo-spectral velocities by a weighting function before equilibrating the spectral intensities. The weighting function could therefore take into account, in an empirical manner, the influence of inelastic action on the total structural response[1]. The following weighting function was used as a starting point:

$$W(\,T, T_o\,, a\,) = \left[\frac{T^2}{\sqrt{(T_o^{\,2} - T^2)^2 + a \cdot T_o^{\,2} \cdot T^2}} \right]^{1/2}$$

The rescaled spectral intensity is defined to be:

$$S_{V\text{ - }rescaled} = \int_{T\text{-}min}^{T\text{-}max} W(\,T, T_o\,, a\,) \cdot S_V(T)\, dT$$

where $T =$ *period (sec).*

$T_o =$ *fundamental period of the elastic structure (sec).*

$a =$ *constant chosen to define the weighting function shape.*

[1] Kennedy[19] uses a similar weighting function approach on spectral acceleration to equalize the damage potential of different records.

$S_t\ (T) =$ *spectral velocity (in/sec).*

The weighting function co-efficient, a, defines the shape of the curve. The shape that minimizes frame response scatter will certainly be related to the extent of inelastic deformation in the response. A good starting-value would seem difficult to choose. Nevertheless , a was set to 0.1 and this integral was evaluated for the E3, E4, E5 and E6 records[2]. The lower and upper integration limits were set at 0.1 and 3.16 seconds respectively. The former three records were rescaled to have the same intensity as the E6 record. Simulation results of the rescaled records are presented in the last part of Table(1).

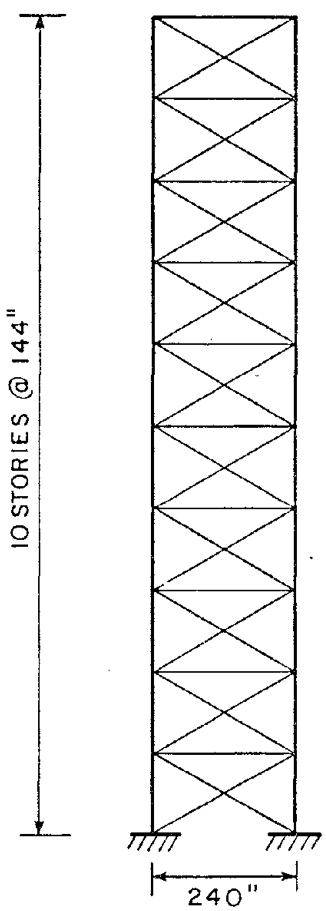
The coefficient of variation of the frame energy dissipated by these records is reduced from 0.246 to 0.017. A similar reduction in story drifts scatter is not obtained because story drifts are related to the stiffness and mass inertia forces of the elastic frame. A designer might therefore put a weighting function on spectral acceleration if the objective was to equate story drifts.

It is clear from both the work of Kennedy[19] and the preliminary results presented here that frame response scatter due to the randomness of ground motions can be reduced by a judicious scaling of earthquake records. The designer should identify the important frame response quantities and find scaling factors on this basis. Future work needs to be implemented to find good scaling functions for different quantities of frame response.

---

[2] Records E1 and E2 were not rescaled because of the reasons discussed in section 6.3.

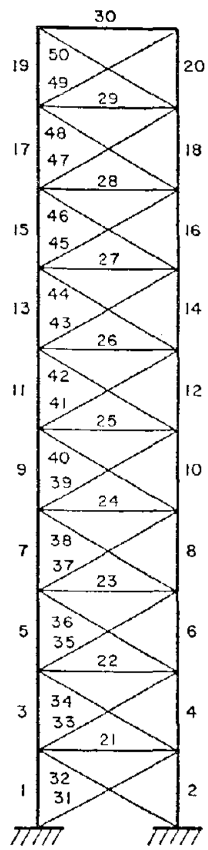
TABLE(1) : A Summary of Results.		
Design Case	Story Drifts ($\times 10^{-5}$)	Dissipated Energy (kip - in $\times 10^{4}$)
Starting Workman Frame	10.12	2.030
Moment Resistant Frame	36.80	0.131
Minimum Volume Design	11.75	1.819
Minimum Dissipated Energy	11.81	1.809
Final Design	10.71	1.975
4% damping	13.18	2.167
5% damping	10.71	1.975
6% damping	9.19	1.823
0.9 . Spectral Intensity	8.71	1.792
1.0 . Spectral Intensity	10.71	1.975
1.1 . Spectral Intensity	12.97	2.158
E2-Elcentro S90W May 1940	4.34	2.06
E3-Elcentro S00W Dec 1934	1.1	1.098
E4-Elcentro S90W Dec 1934	7.5	1.181
E5-Elcentro N50E Oct 1979	10.71	1.727
E6-Elcentro N40W Oct 1979	10.71	1.975
Rescaled E3	4.13	2.043
Rescaled E4	2.46	1.999
Rescaled E5	14.84	2.011



BEAM INERTIA	COLUMN INERTIA	BRACE ÅREA
800		
	339	2.88
800		
	542	2.88
800		
	542	2.88
800		
	851	2.88
800		
	851	2.88
984		
	1165	2.88
984		
	1165	2.94
984		
	1373	2.94
984		
	1373	3.38
1287		
	1500	3.38

UNITS INERTIA IN^4
AREA IN^2

FIG (1a) : Workman Frame ; Starting Beam and Column Inertias ; Brace cross-section areas.



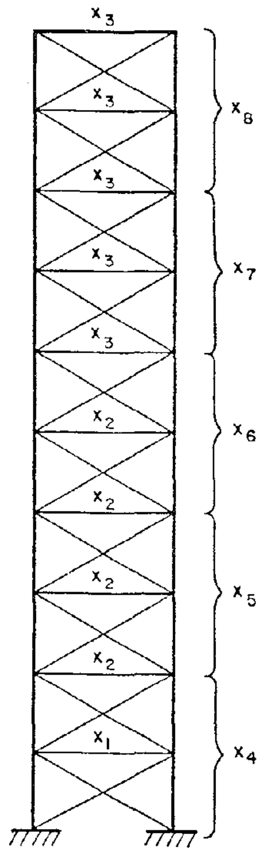


FIG (1b) : Element numbering by 'Finput' for Workman Frame.

FIG (1c) : Element grouping for Optimal Designs.

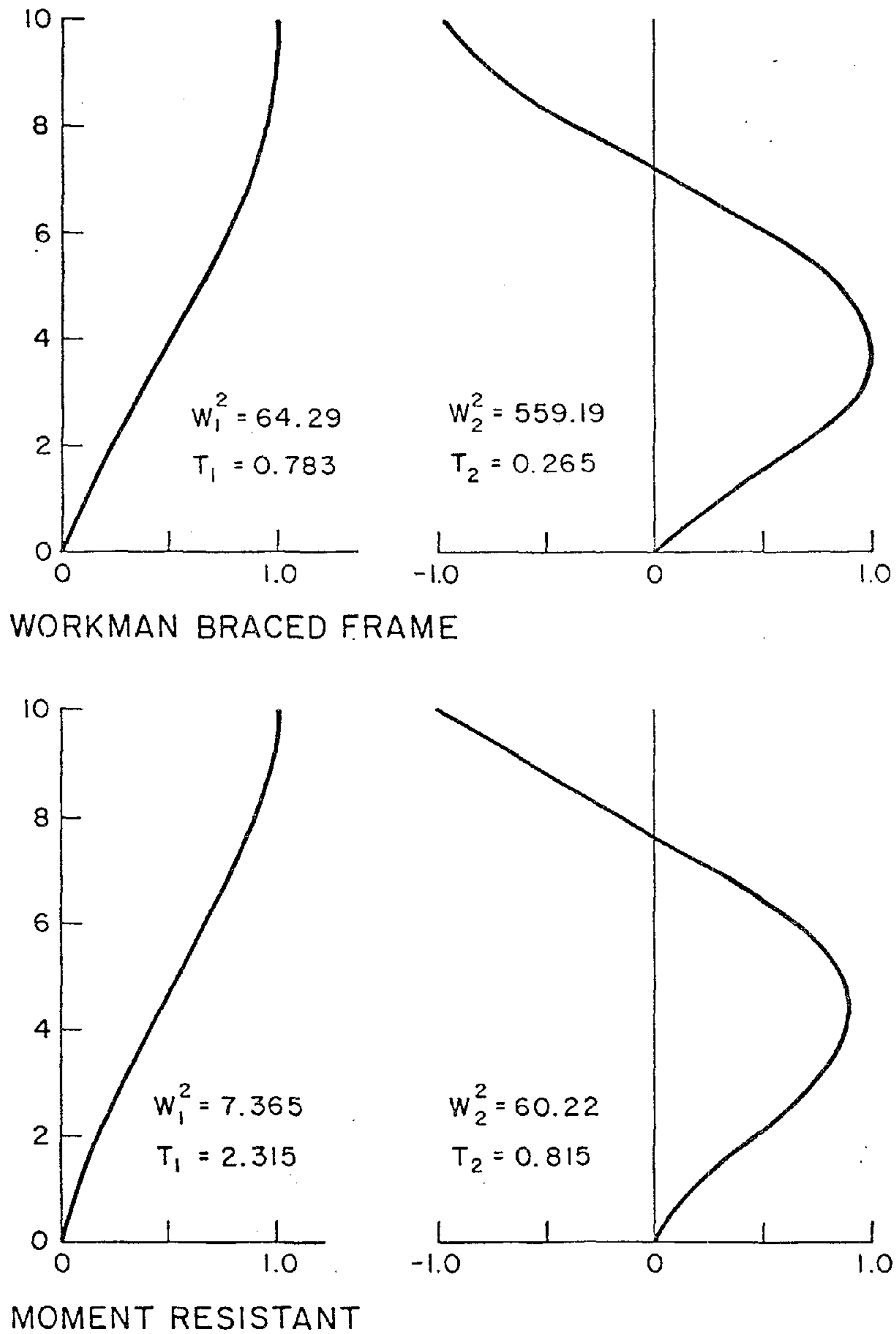


FIG (2) : Mode shapes and Natural periods for the Workman and Moment-resistant Frames.

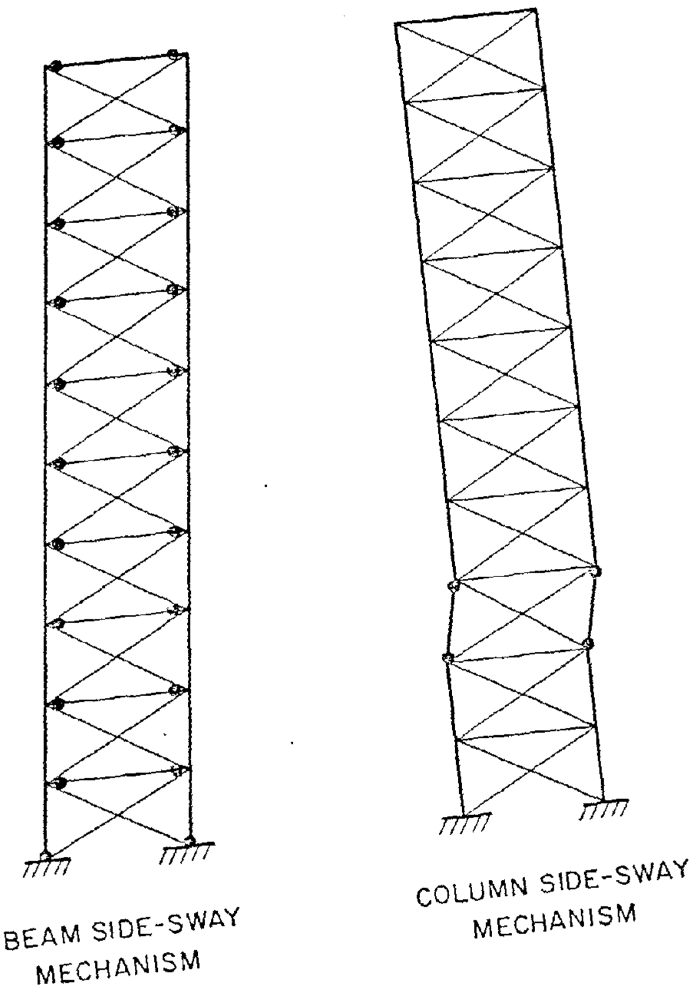


FIG (3) : Frame collapse mechanisms.

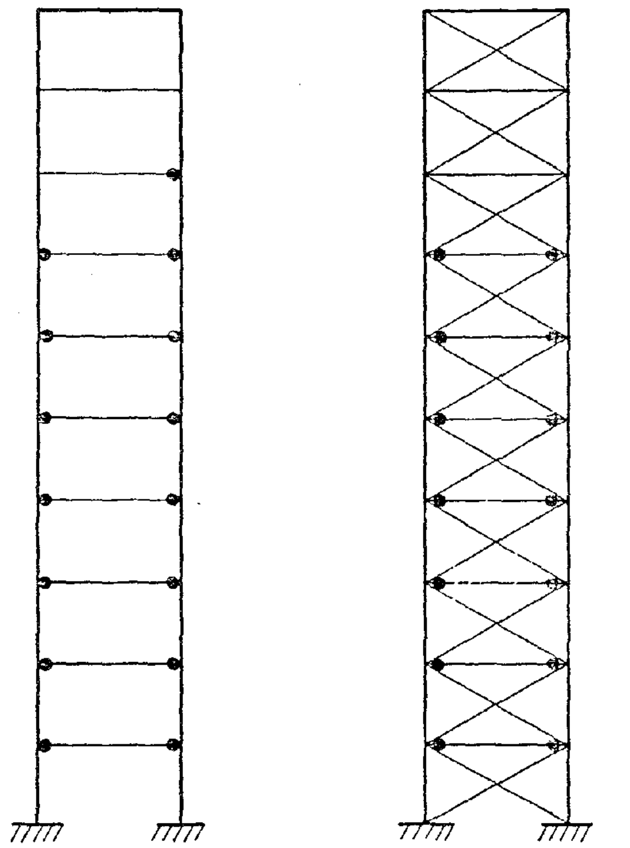
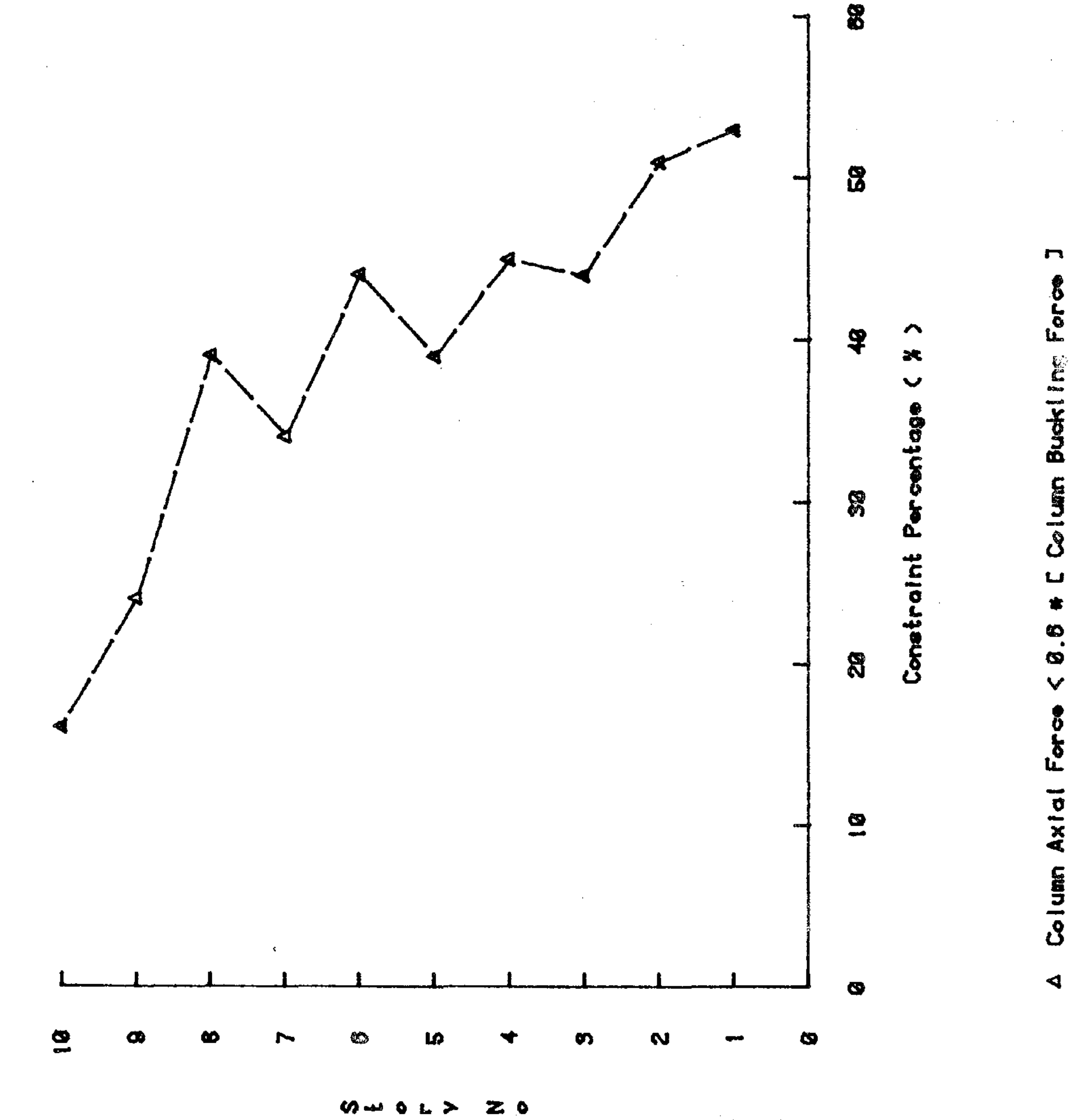
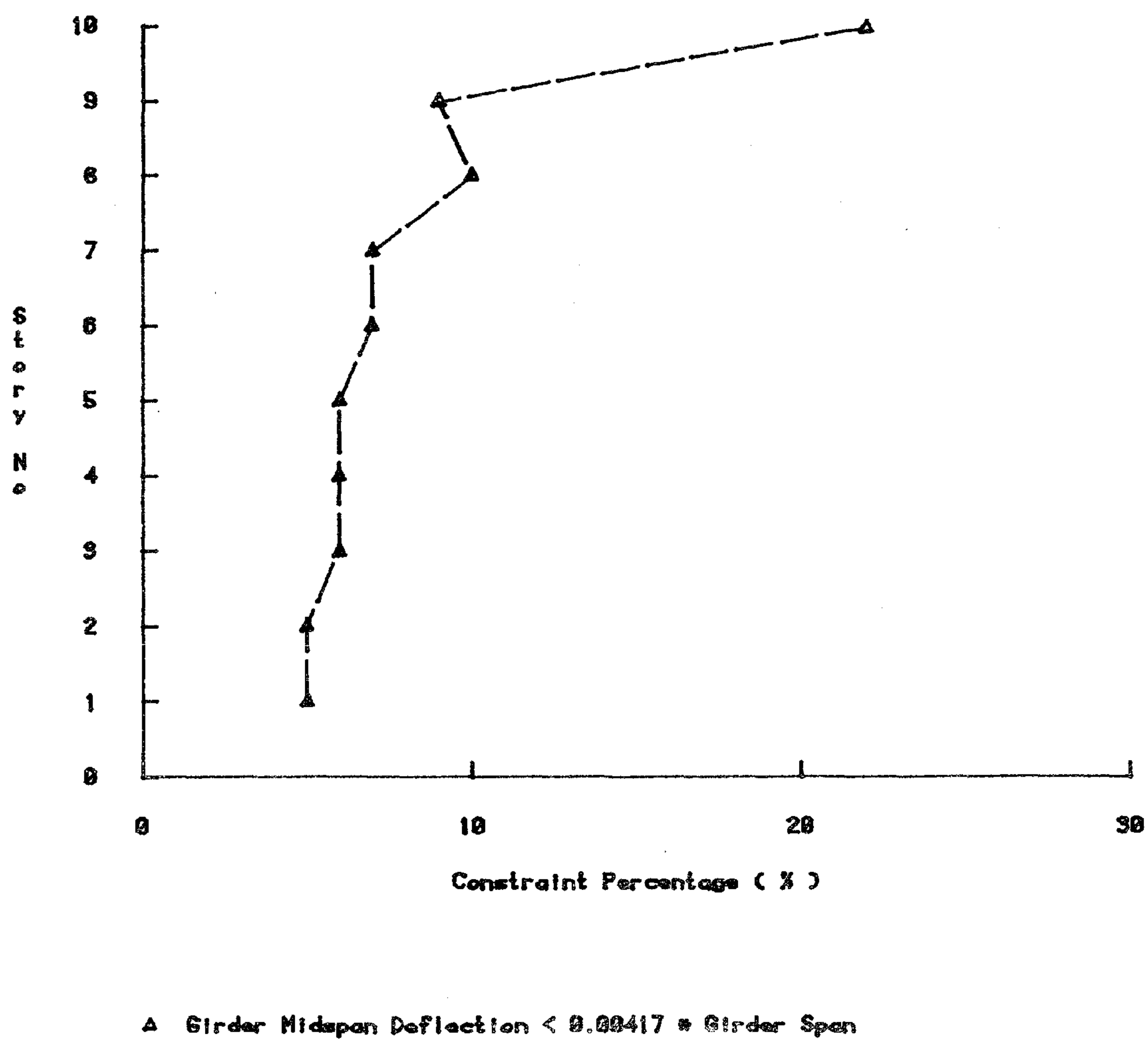


FIG (4) : Inelastic yielding elements for Moment-resistant and Friction-braced Frames.

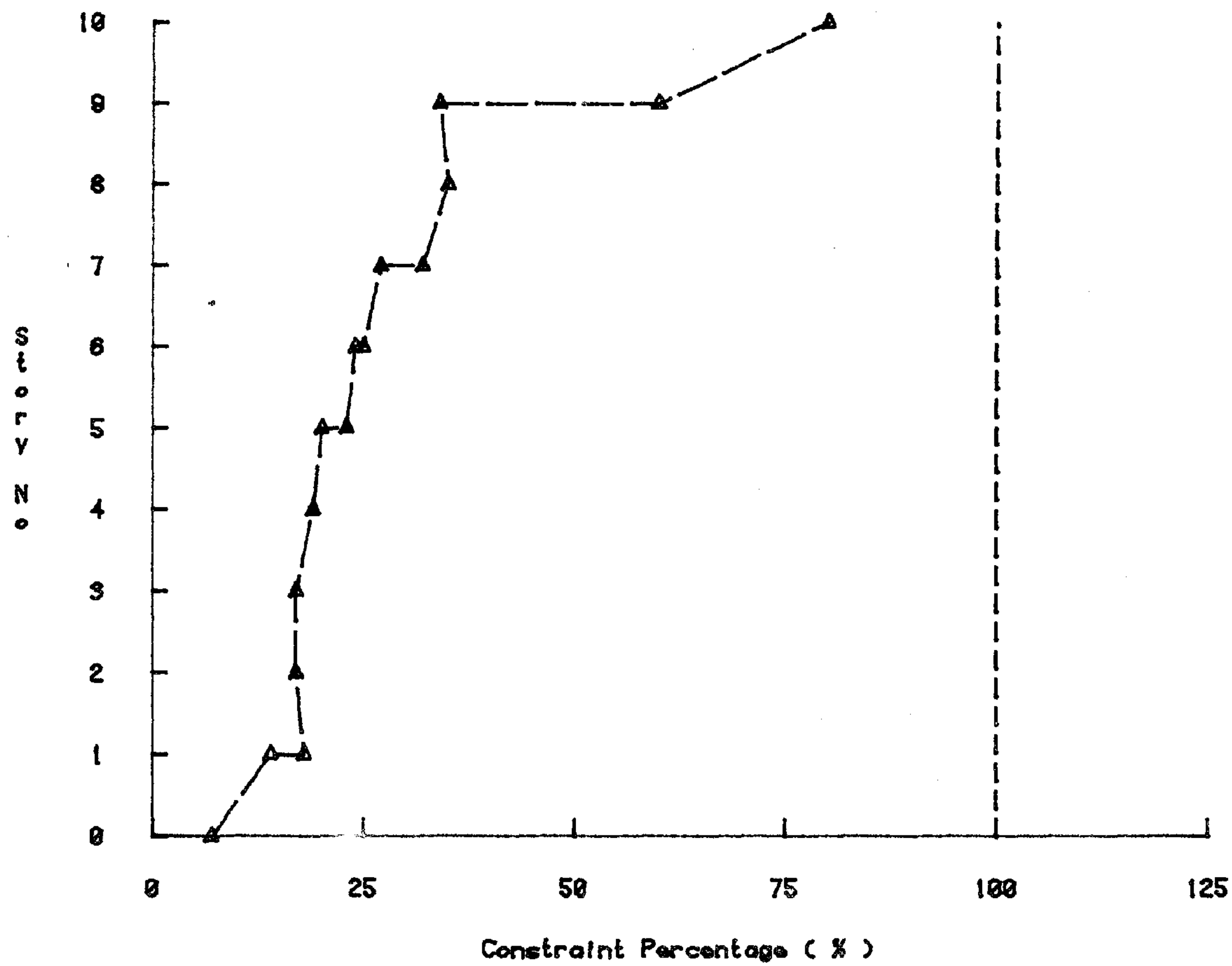
FIG(5) :Story No. vs Column Axial Force.



FIG(6) :Story No. vs Midspan Girder Deflection.

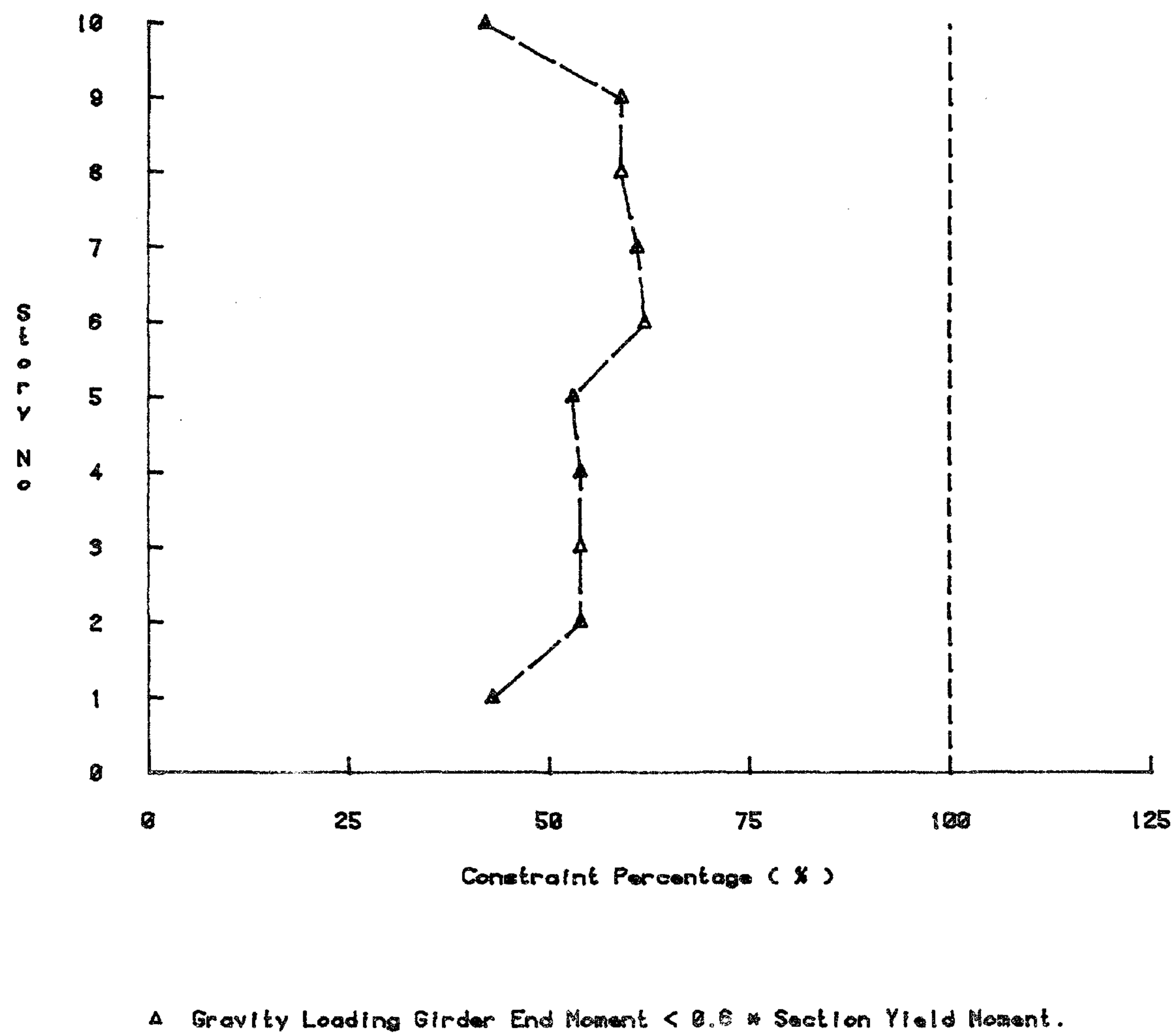


FIG(7) :Story No. vs Gravity Loading Column End Moment.

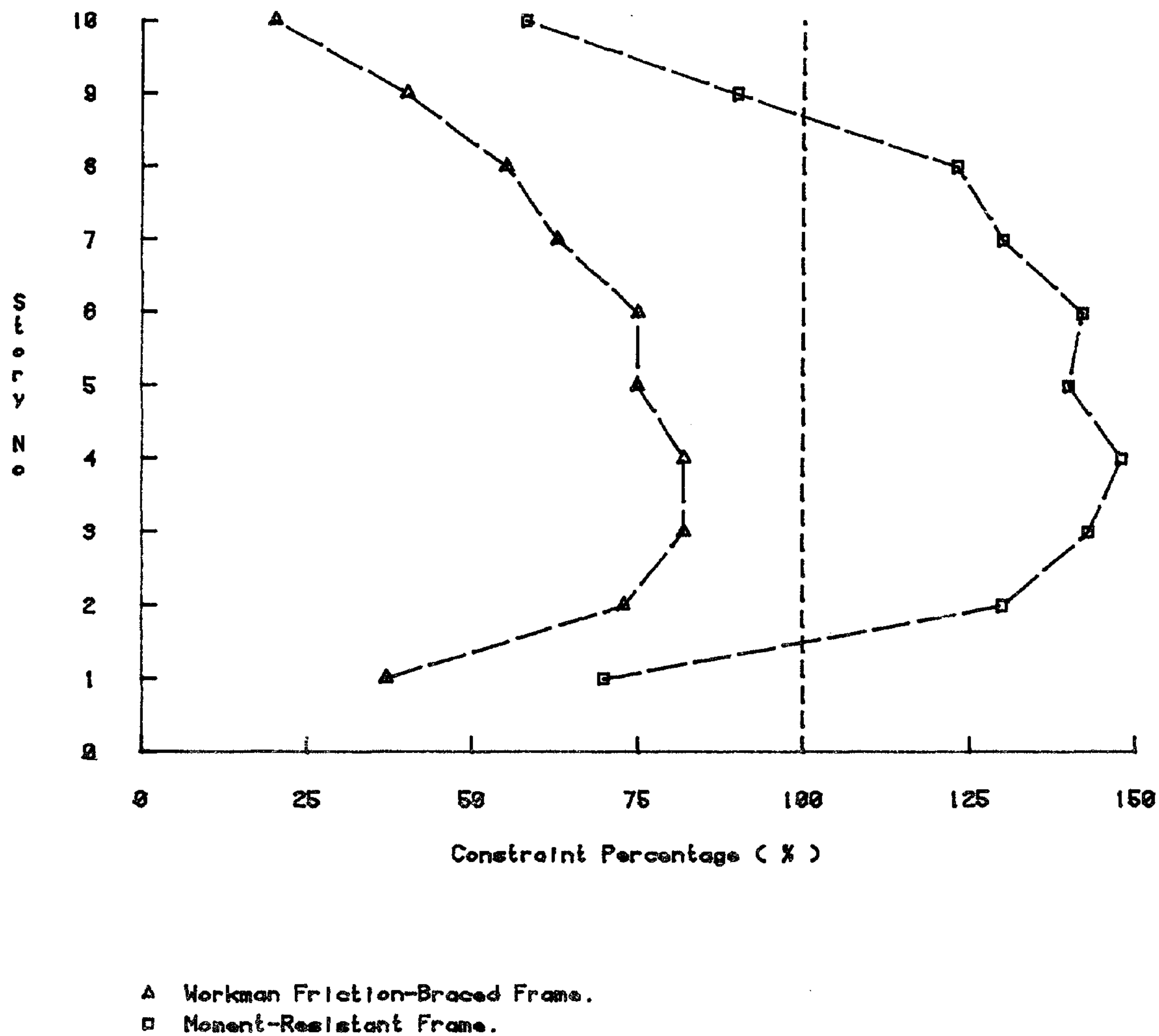


Δ Gravity Loading Column End Moment < 0.8 * Section Yield Moment.

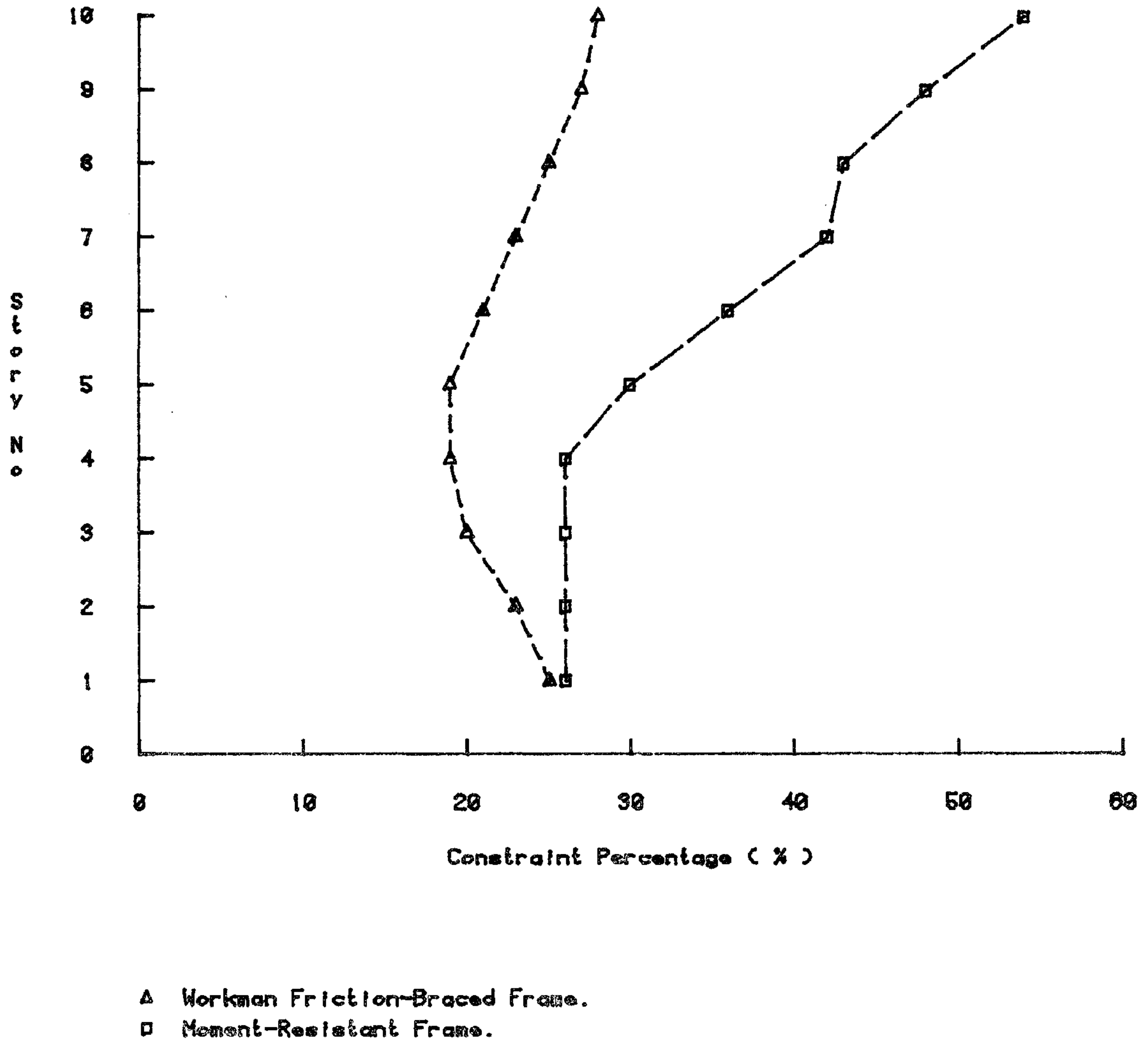
FIG(8) :Story No. vs Gravity Loading Girder End Moment.



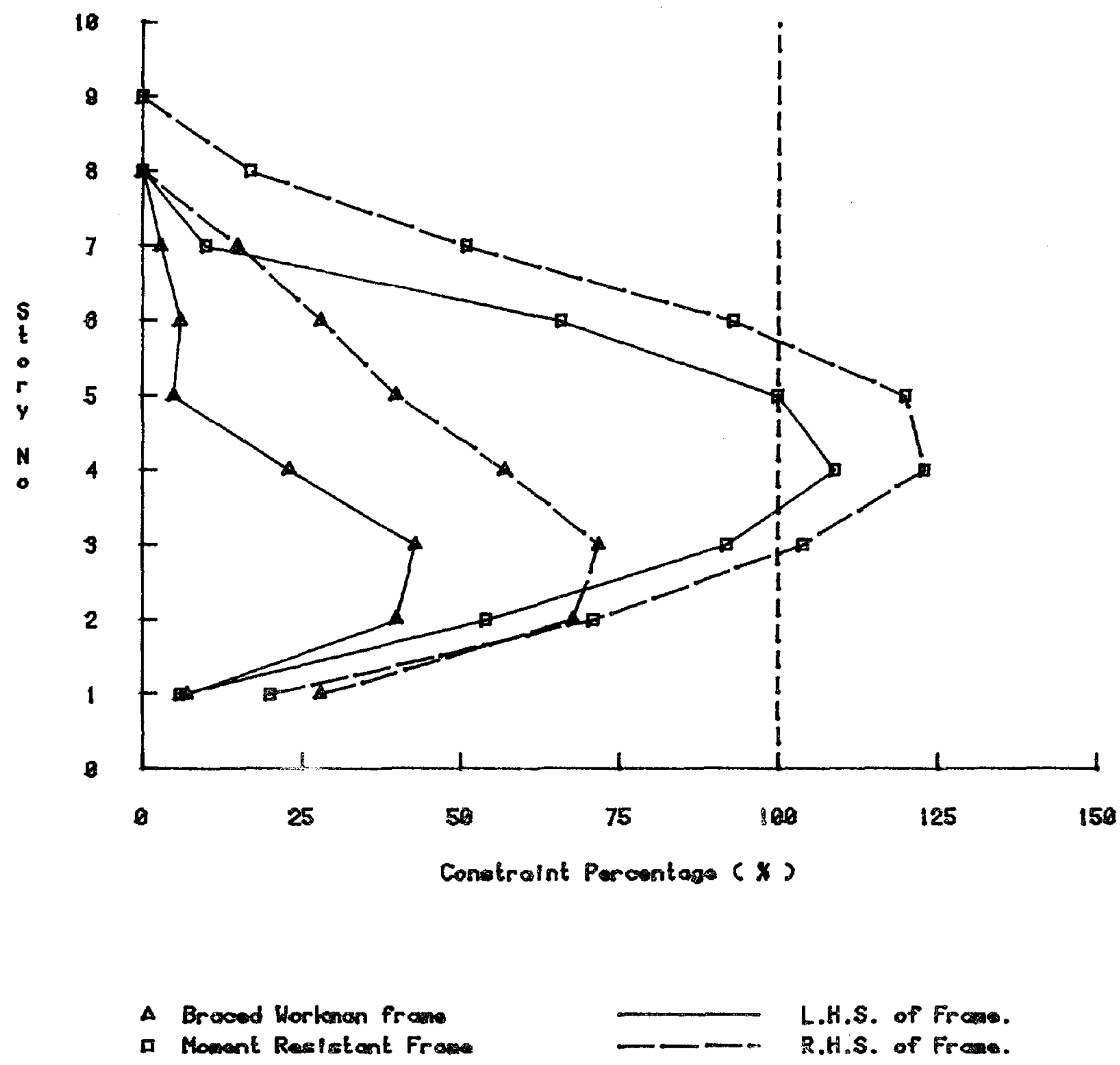
FIG(9) :Story No. vs Story Drift, [Drift] = 0.005



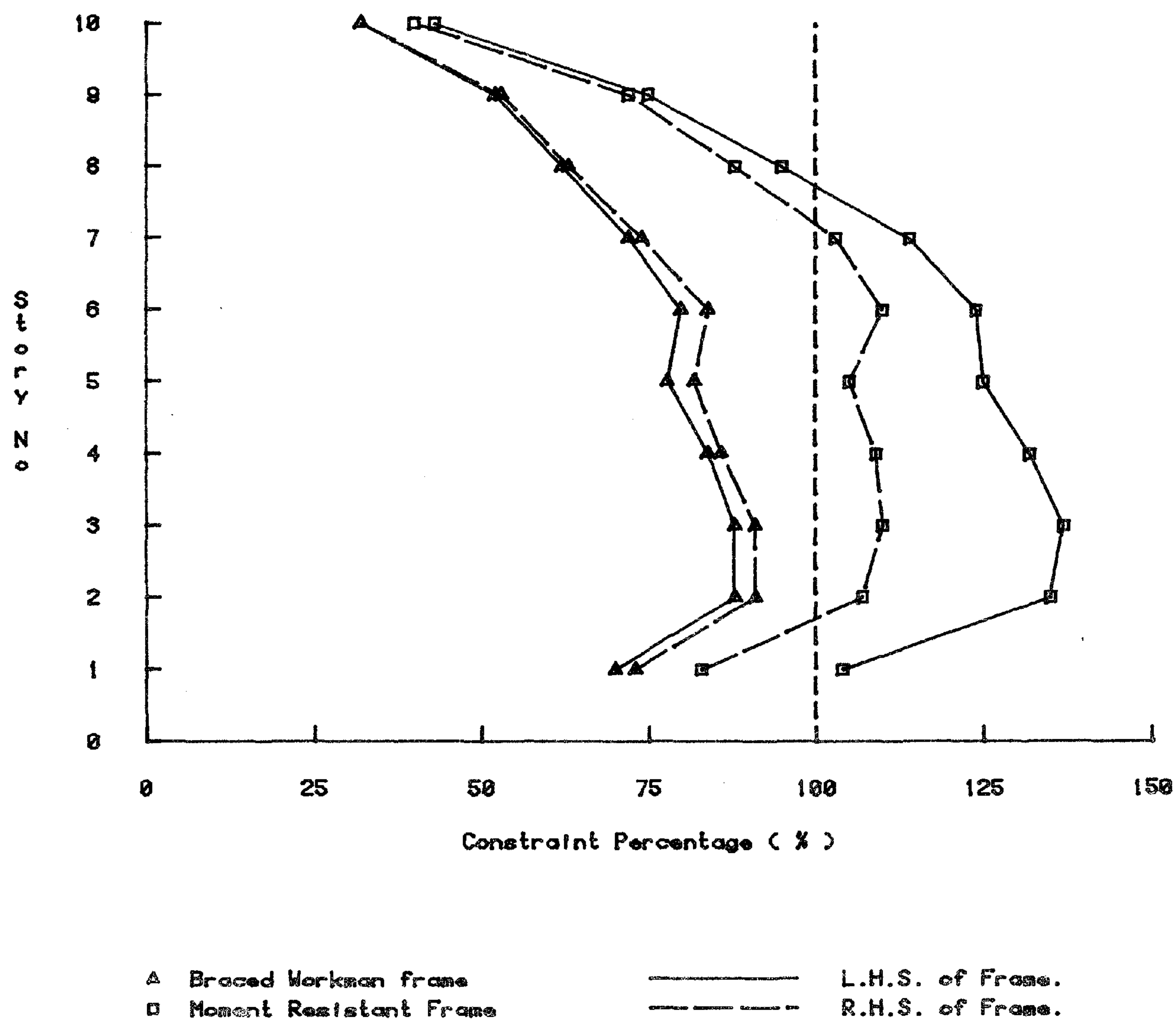
FIG(10) :Story No. vs Floor Acceleration, [Accel] = 0.5



FIG(11) : Story No. vs Girder End Energy.



FIG(12) : Story No. vs Girder End Moment.



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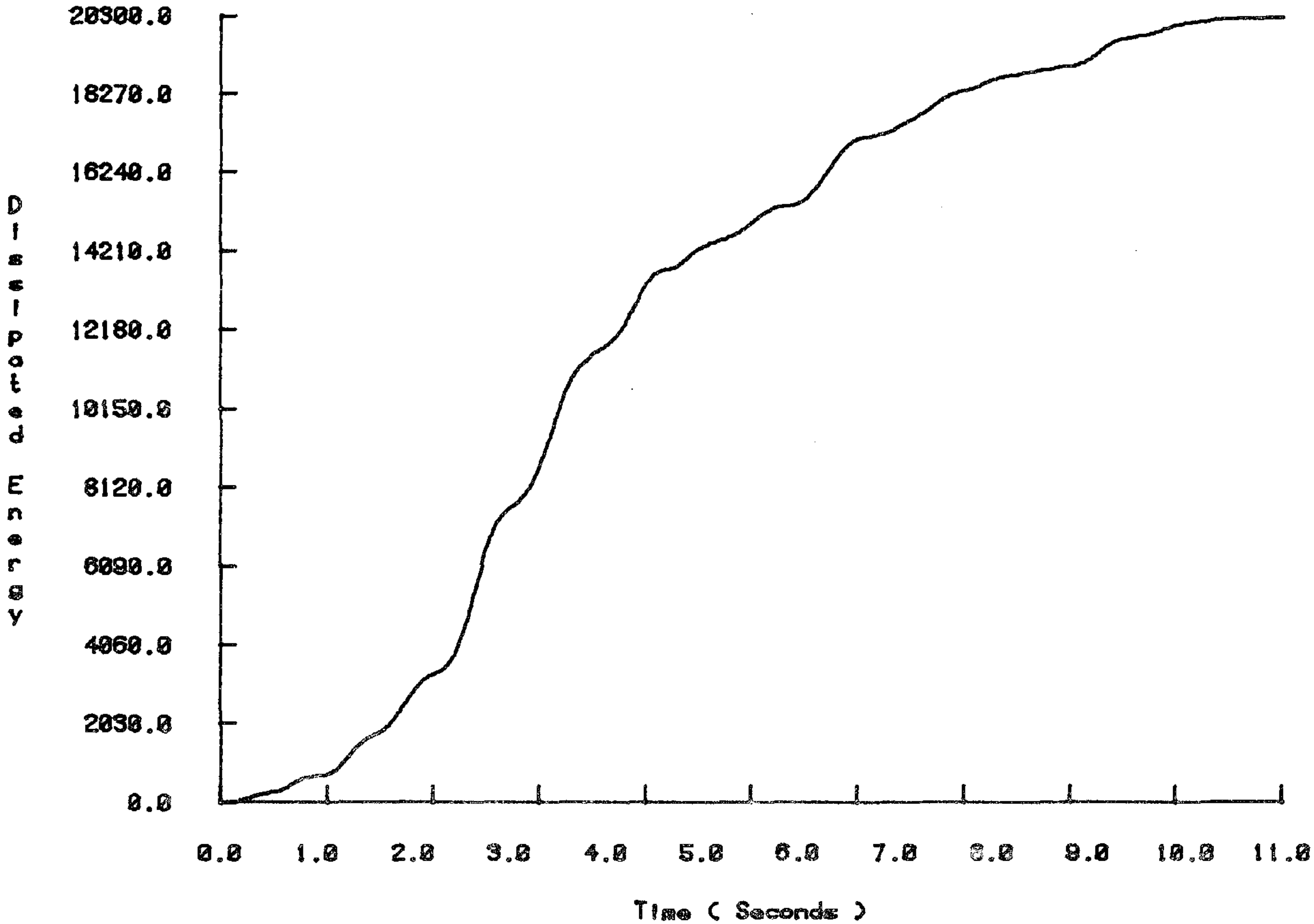
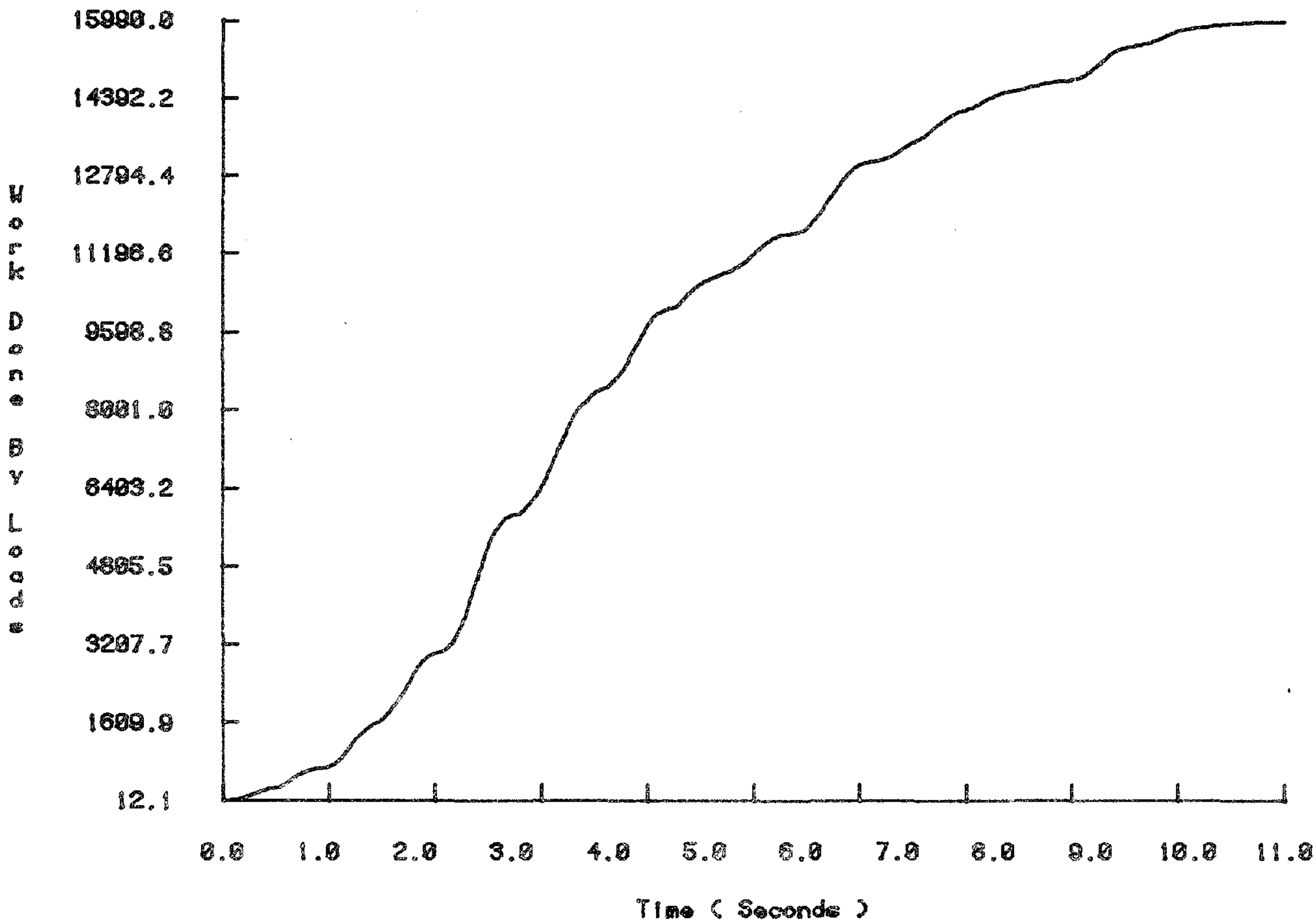
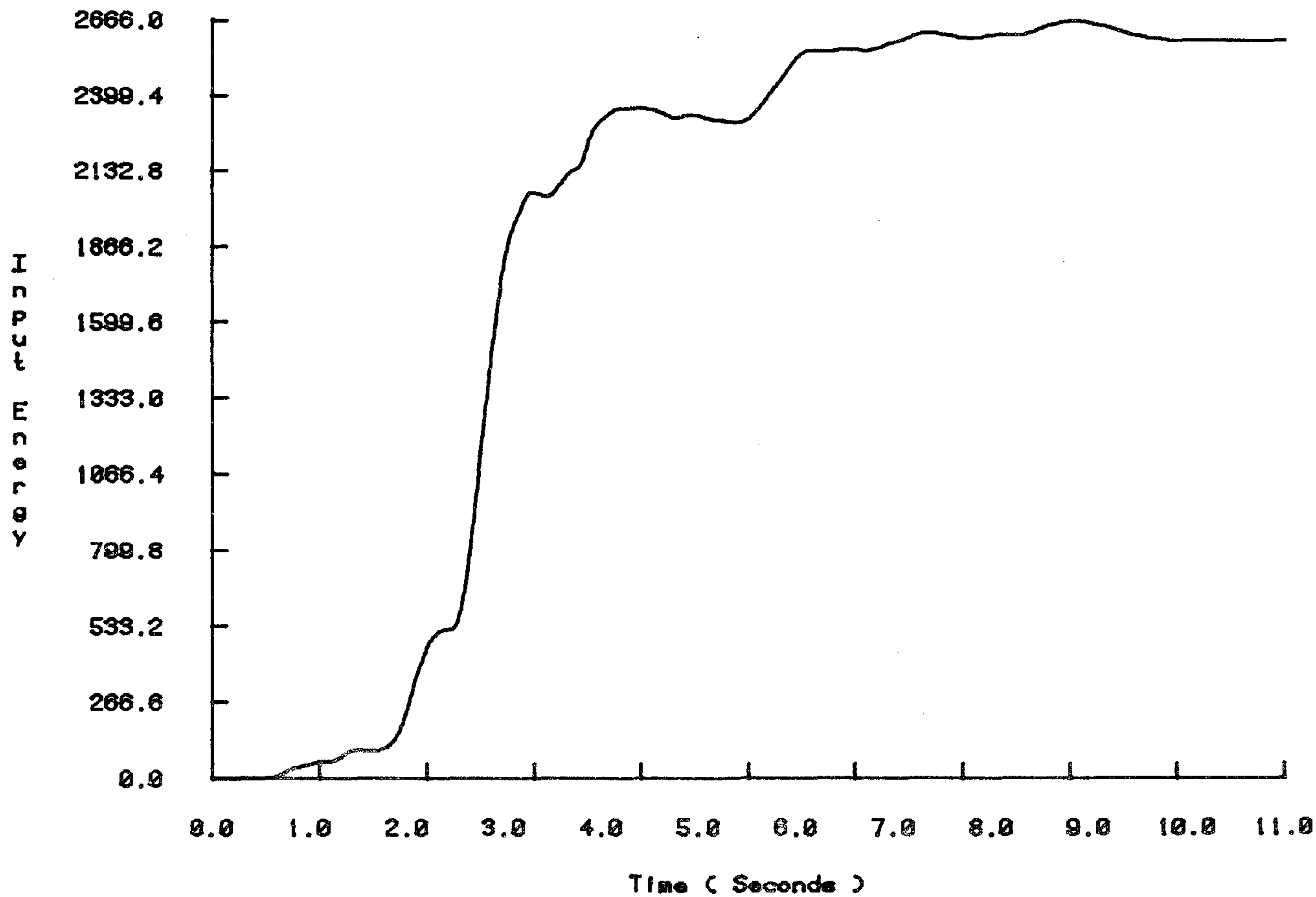
FIG(13) : Dissipated Energy (kip-in) vs Time (sec) for Workman Frame



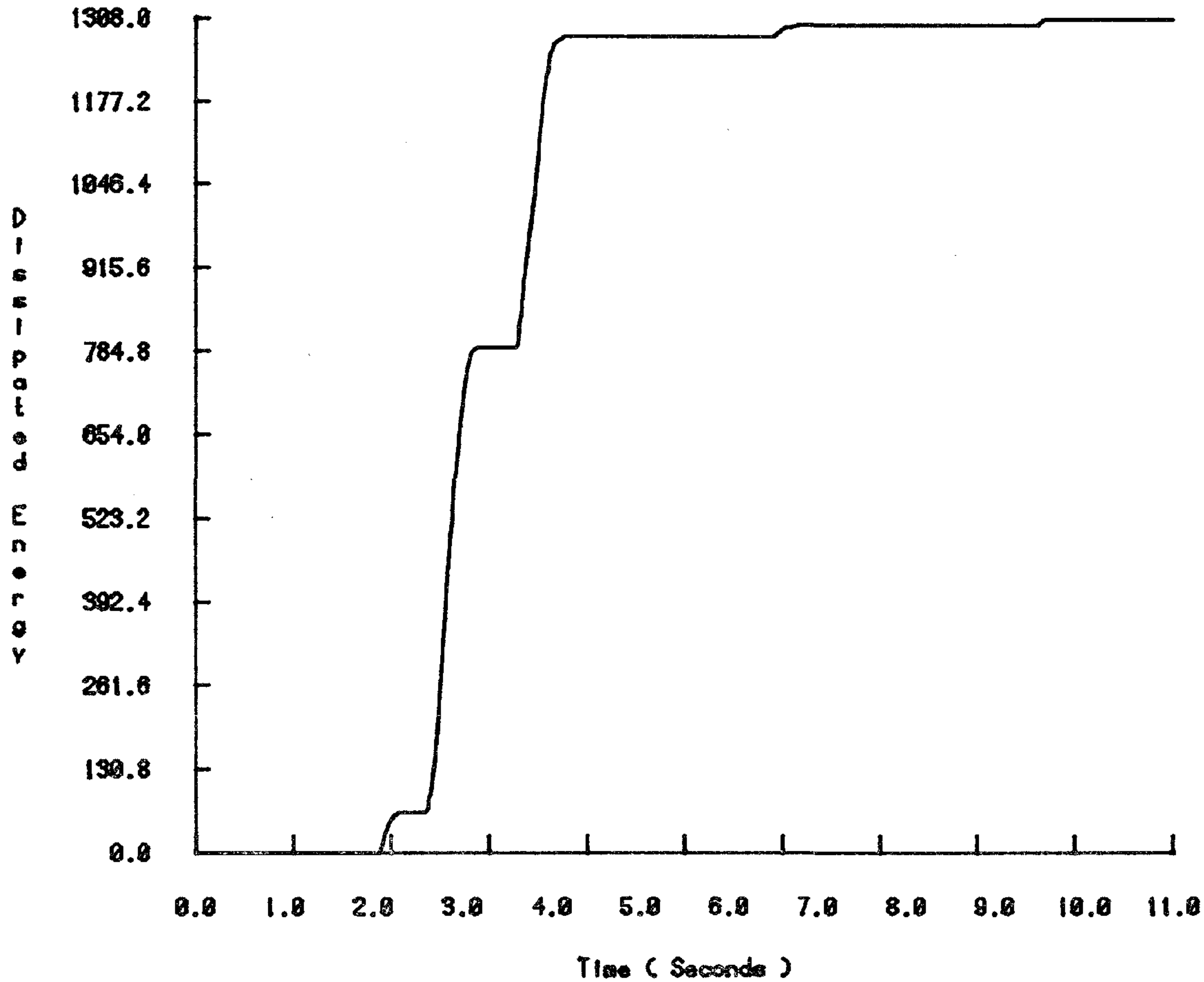
FIG(14) : Work Done By Loads (kip-in) vs Time (sec) for Workman Frame



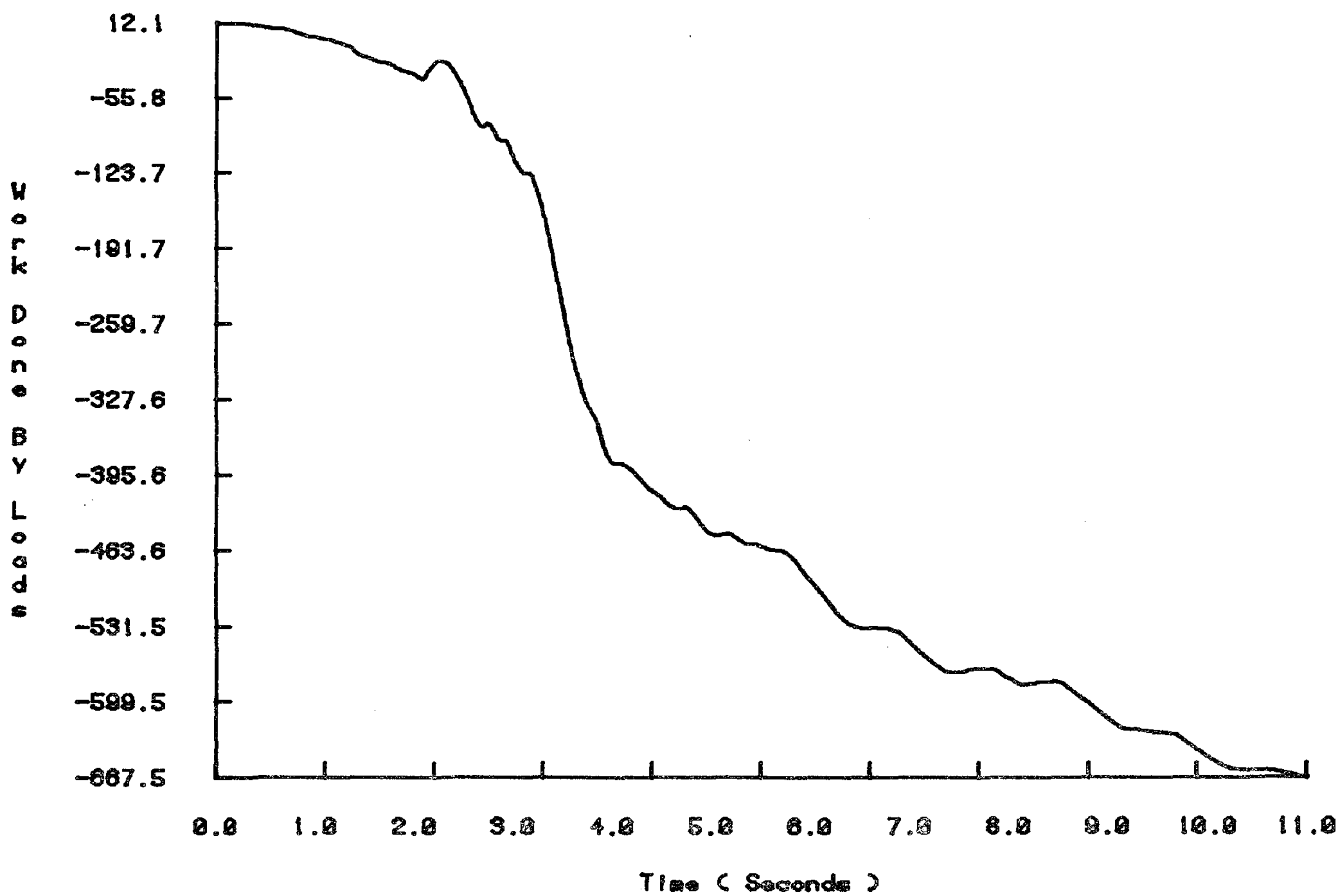
FIG(15) : Input Energy (kip-in) vs Time (sec) for Workman Frame



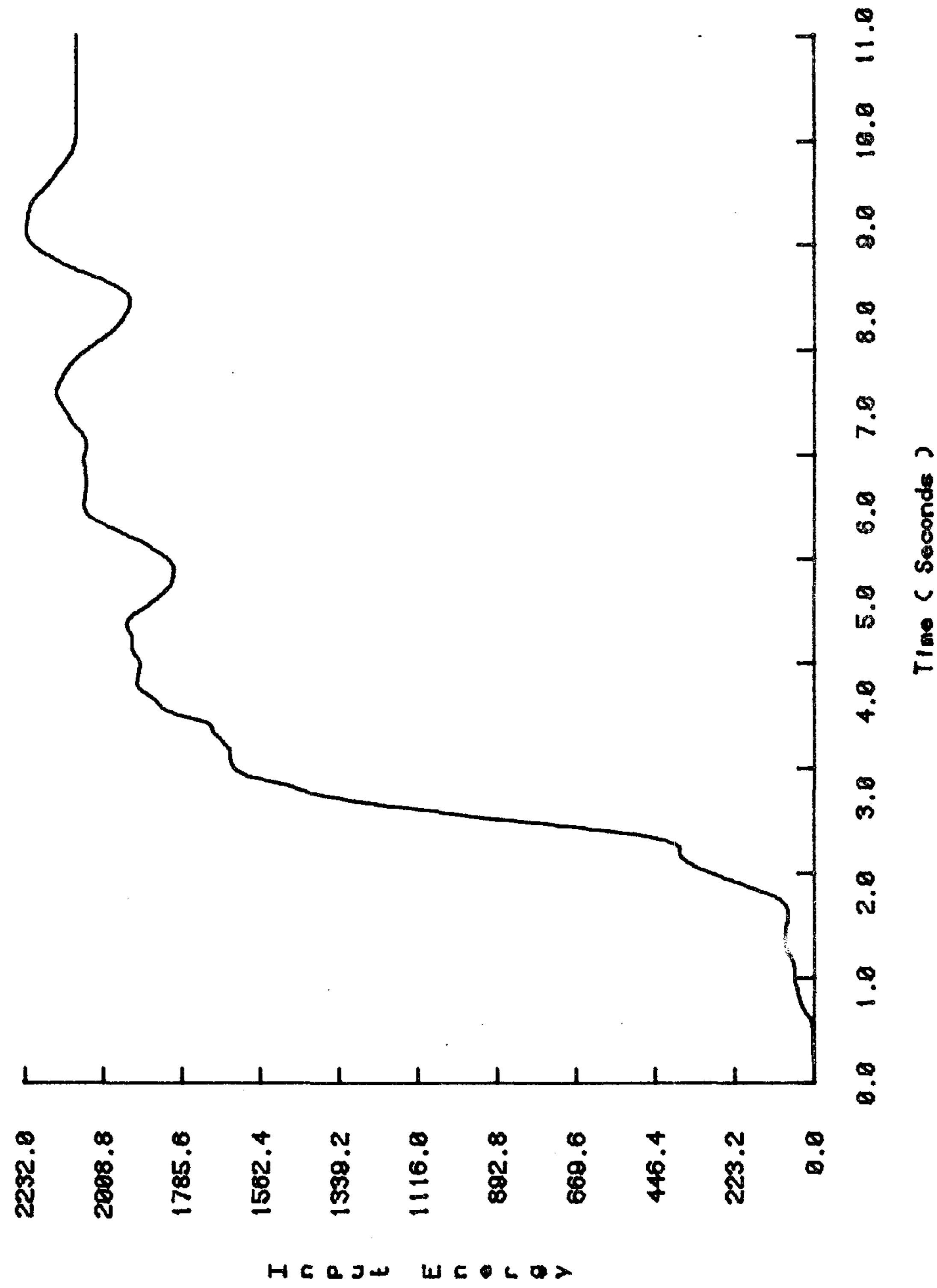
FIG(16) : Dissipated Energy (kip-in) vs Time (sec) for M.R.F.



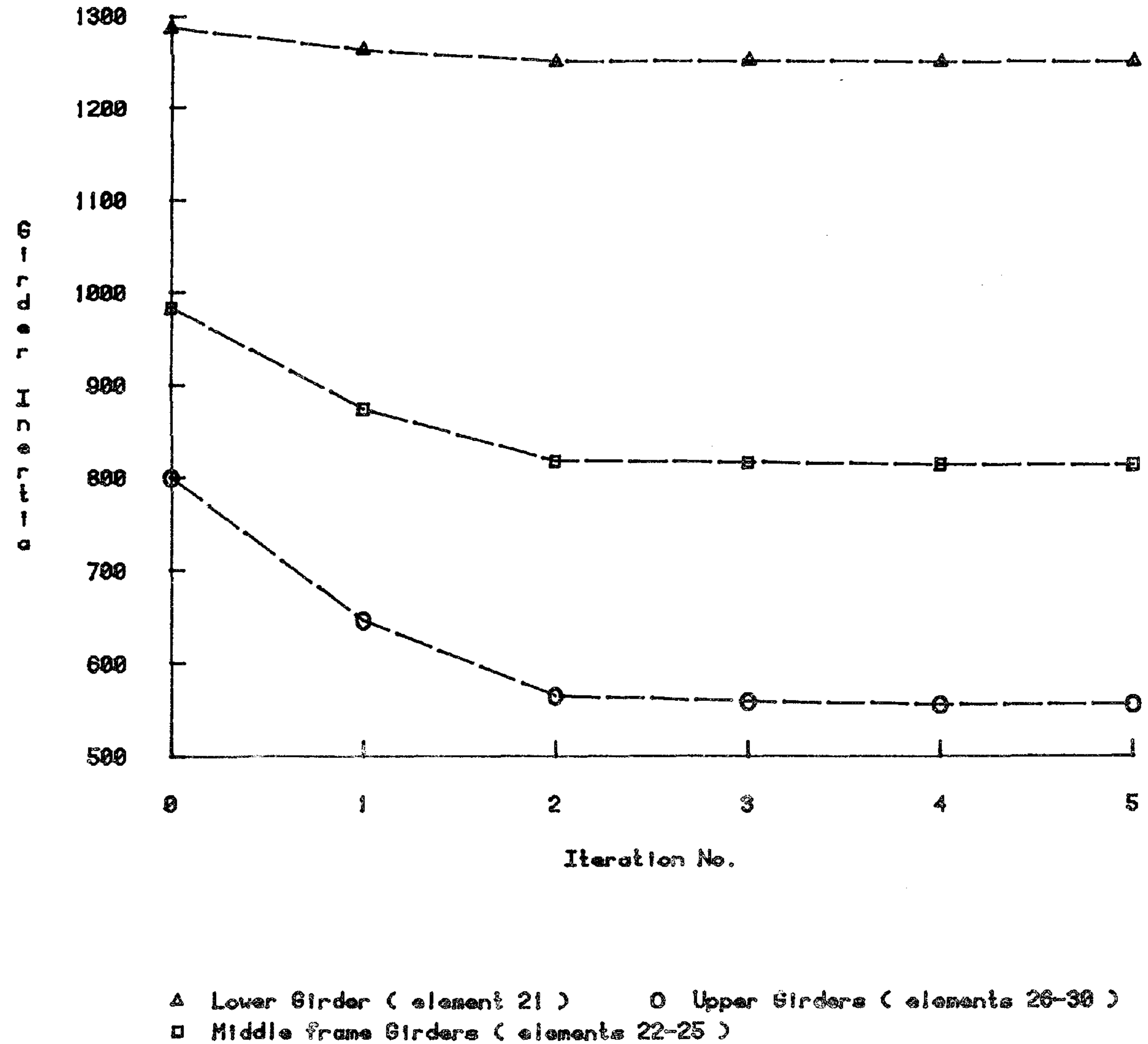
FIG(17) : Work Done By Loads (kip-in) vs Time (sec) for M.R.F.



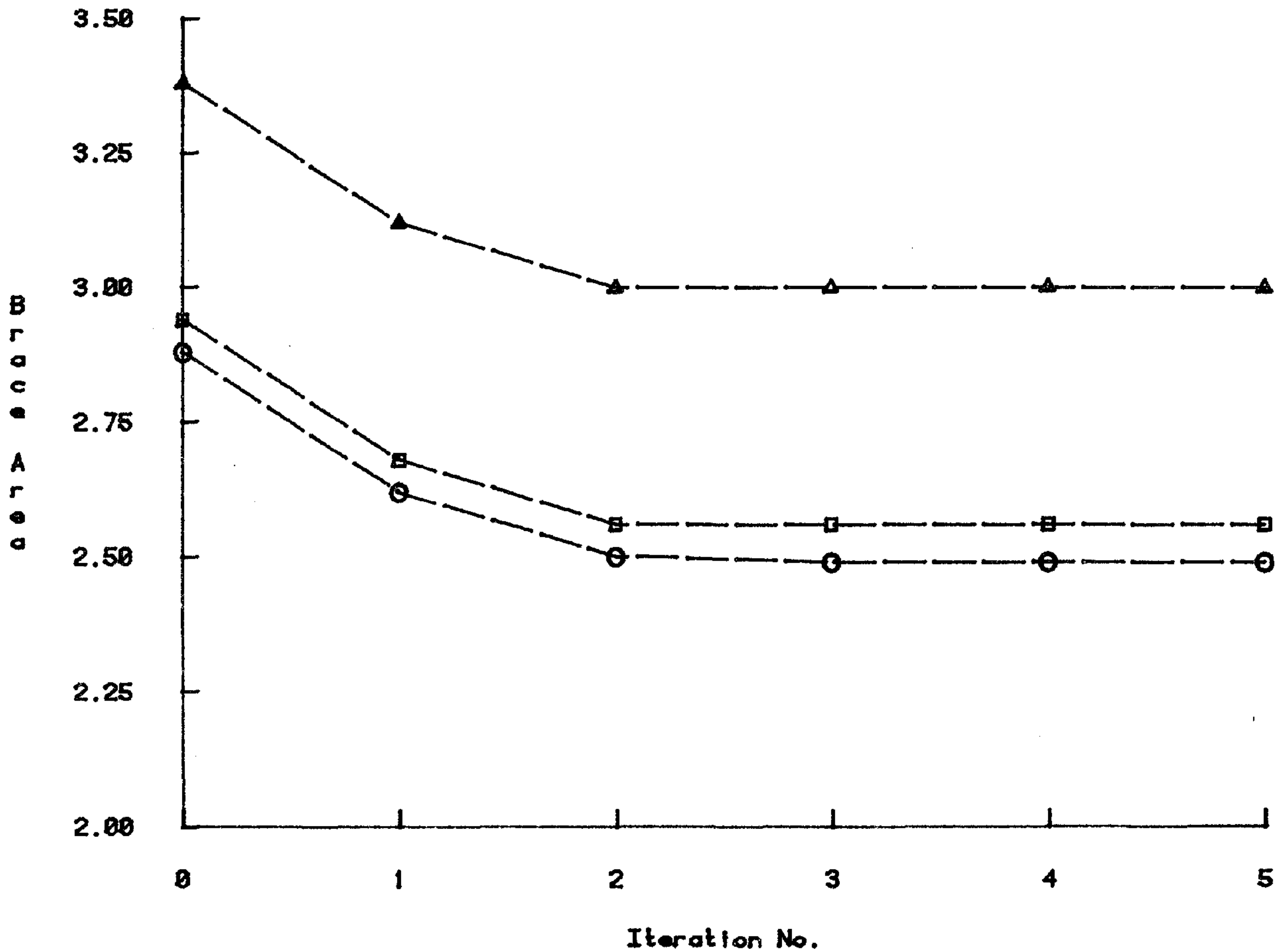
FIG(18) : Input Energy (kip-in) vs Time (sec) for M.R.F.



FIG(19) : Minimum Volume Design.

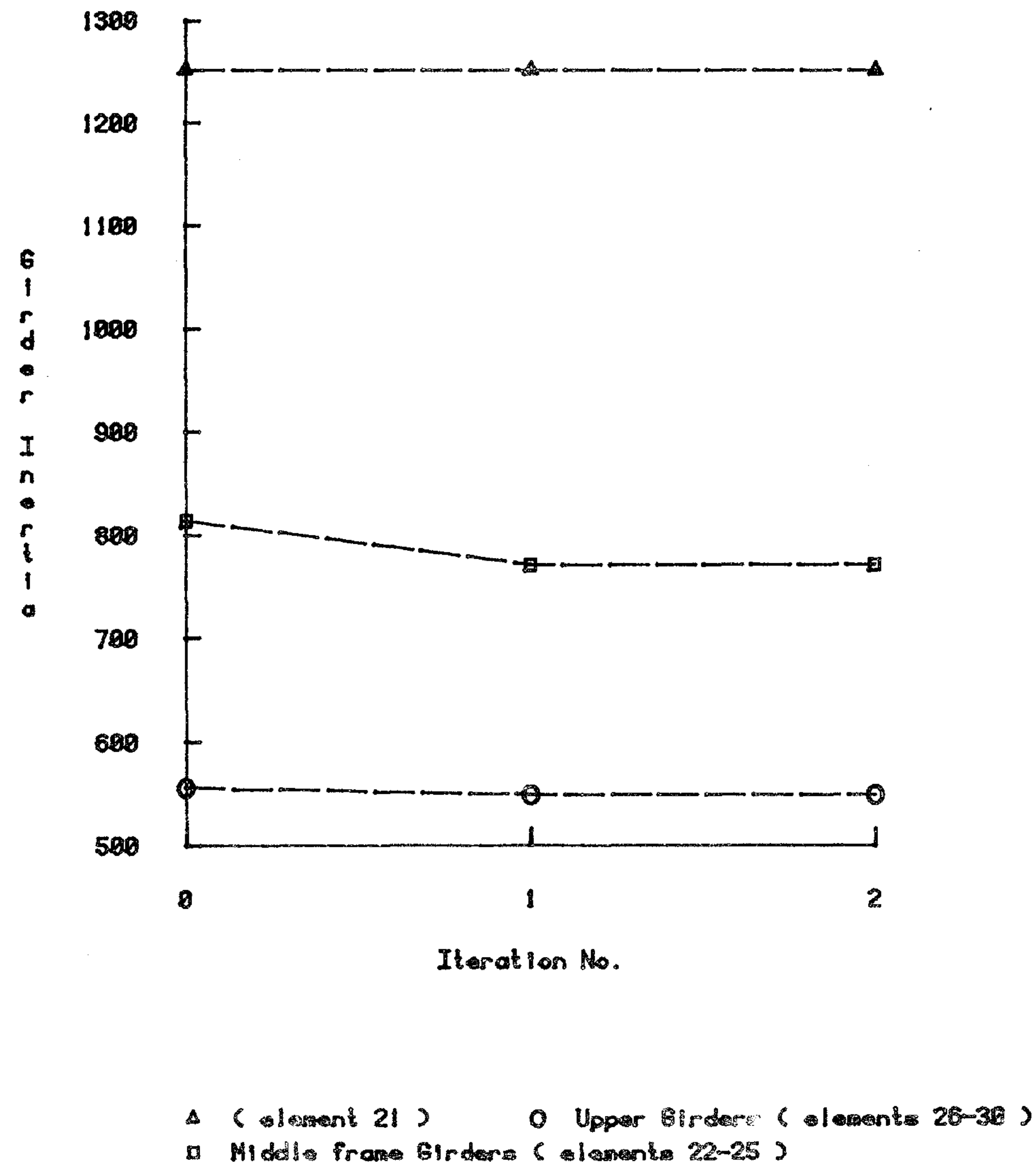


FIG(20) : Minimum Volume Design.

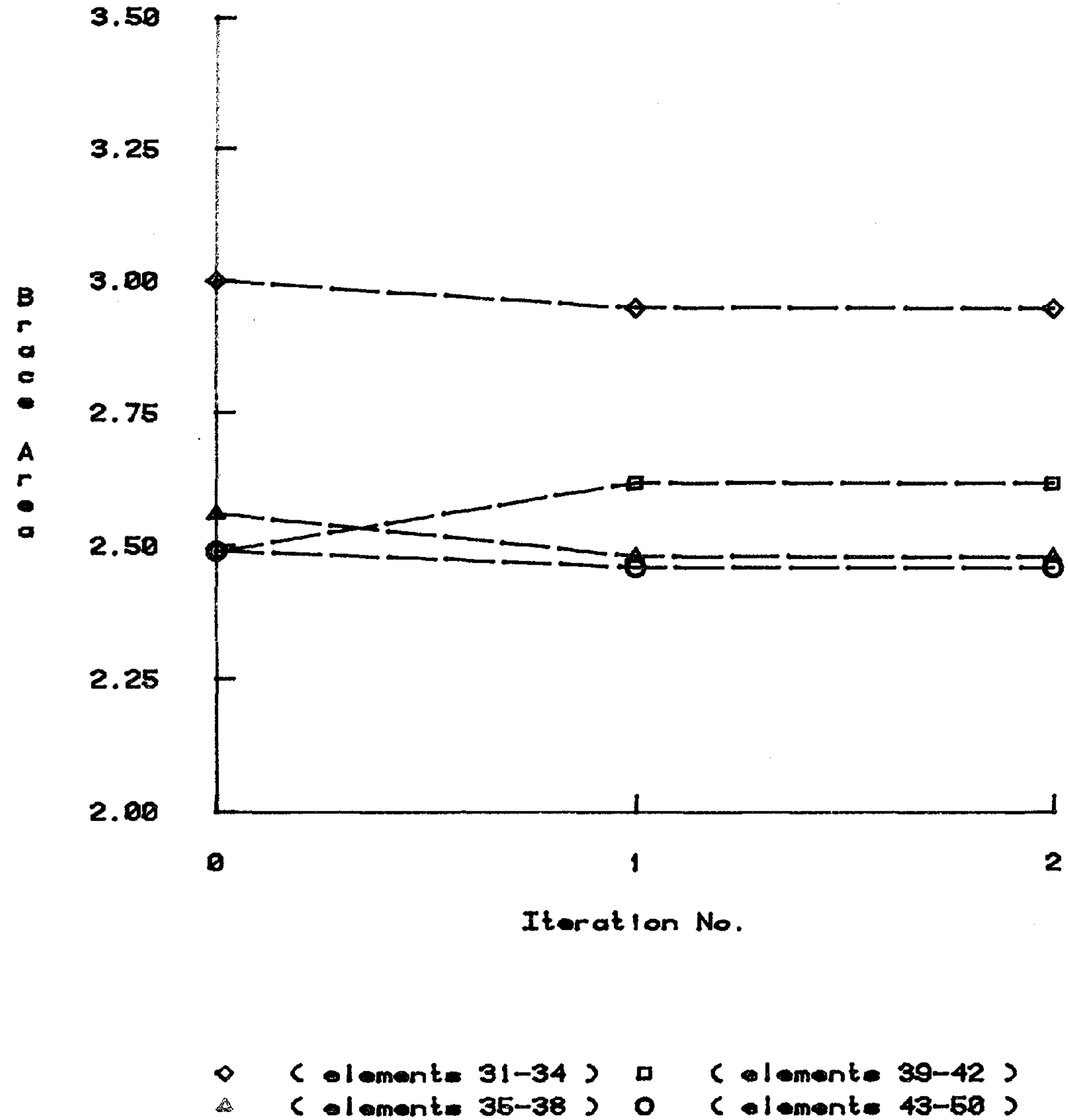


△ Lower Braces (elements 31-34) O Upper Braces (elements 39-50)
□ Middle Braces (elements 35-38)

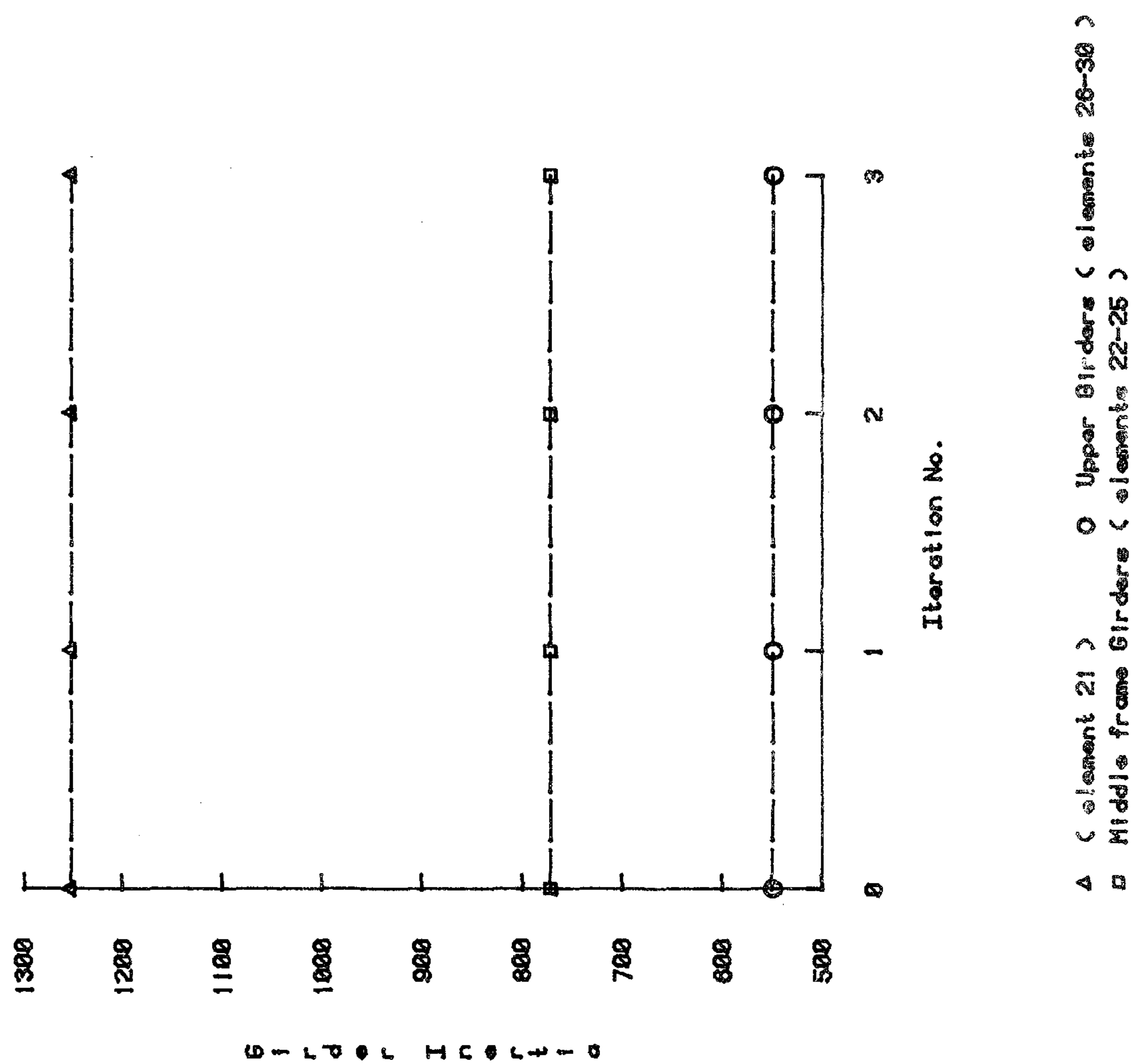
FIG(21) : Minimum Dissipated Energy Design.



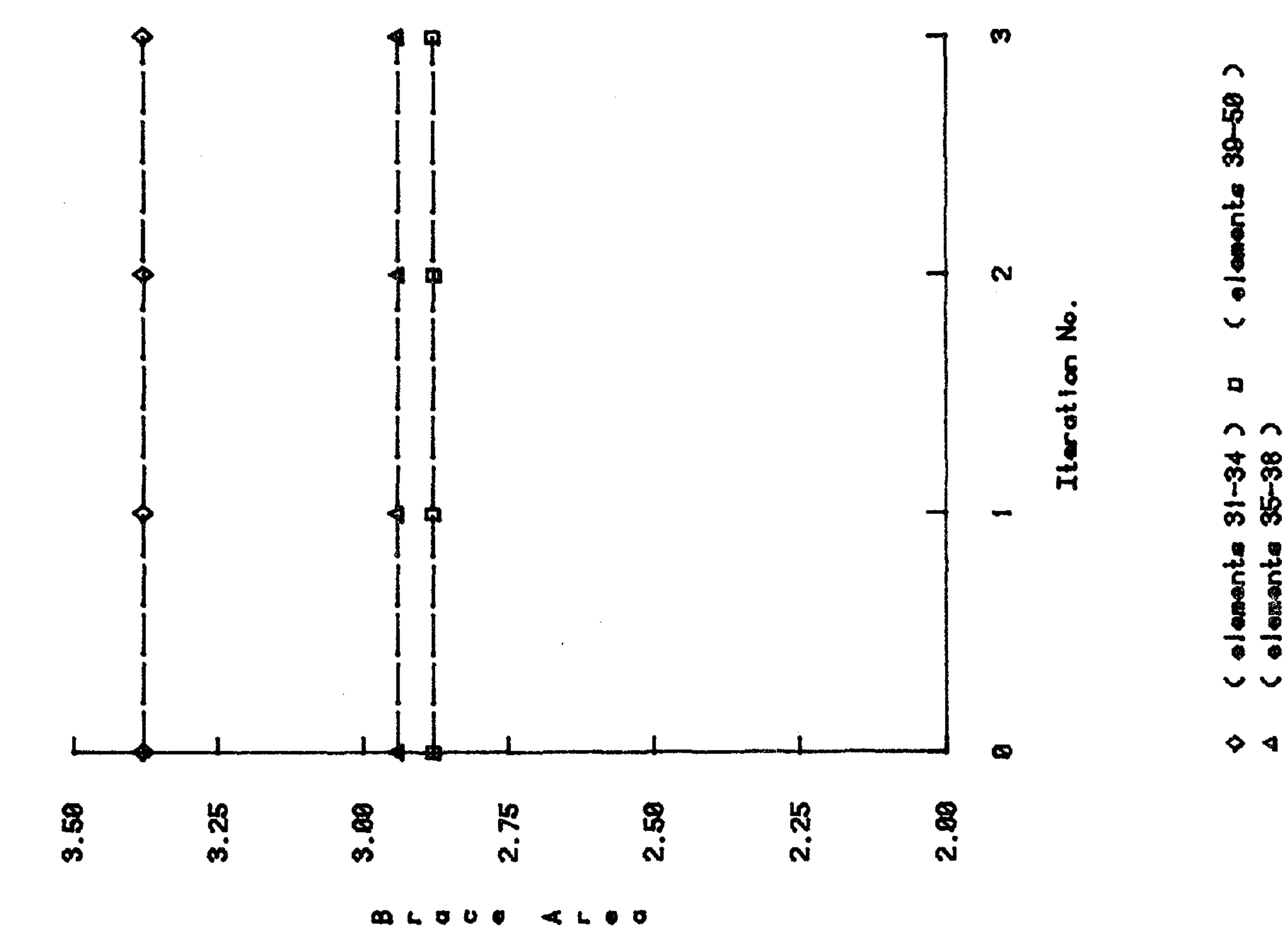
FIG(22) : Minimum Dissipated Energy Design.



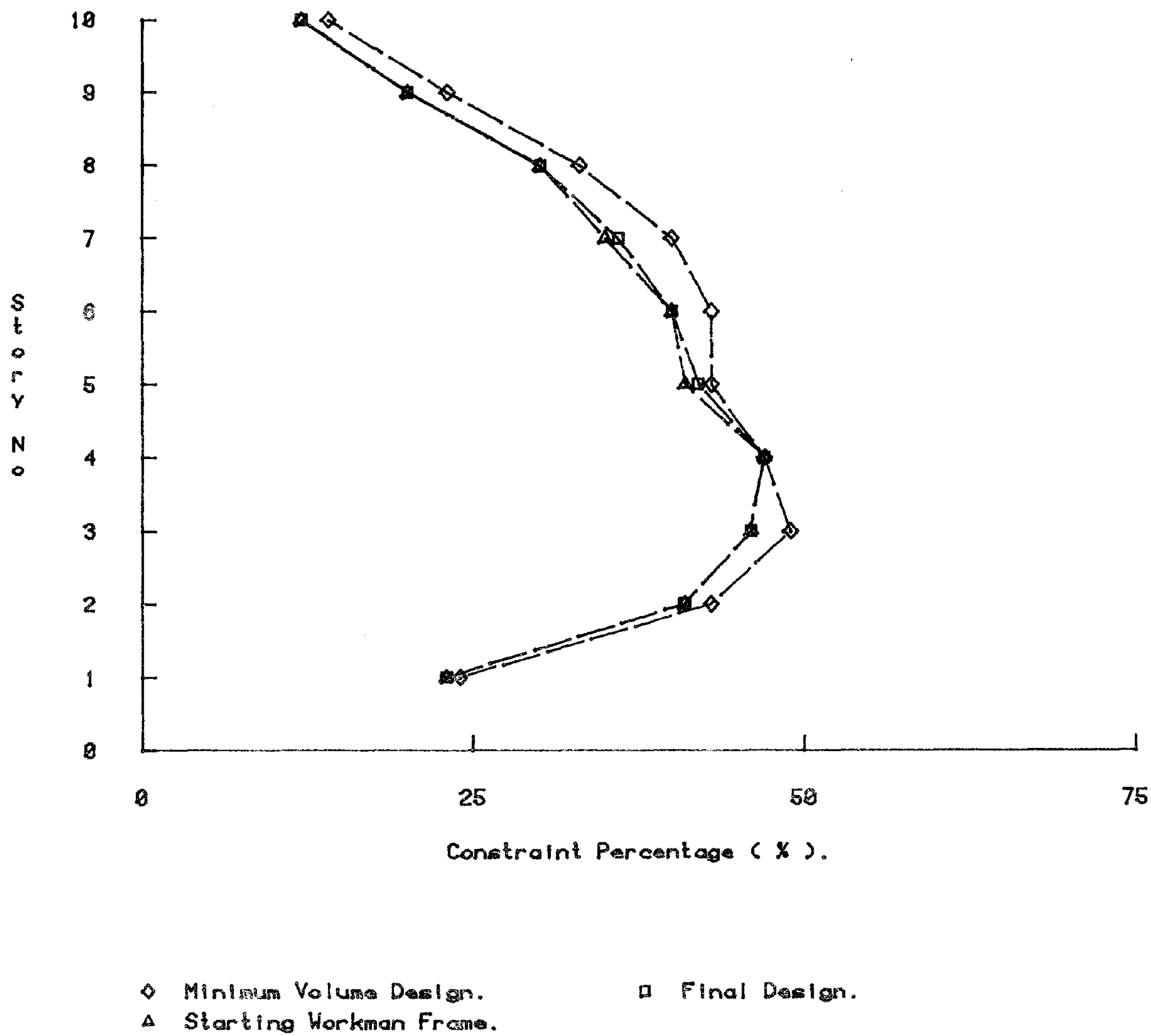
FIG(23) : Final Design.



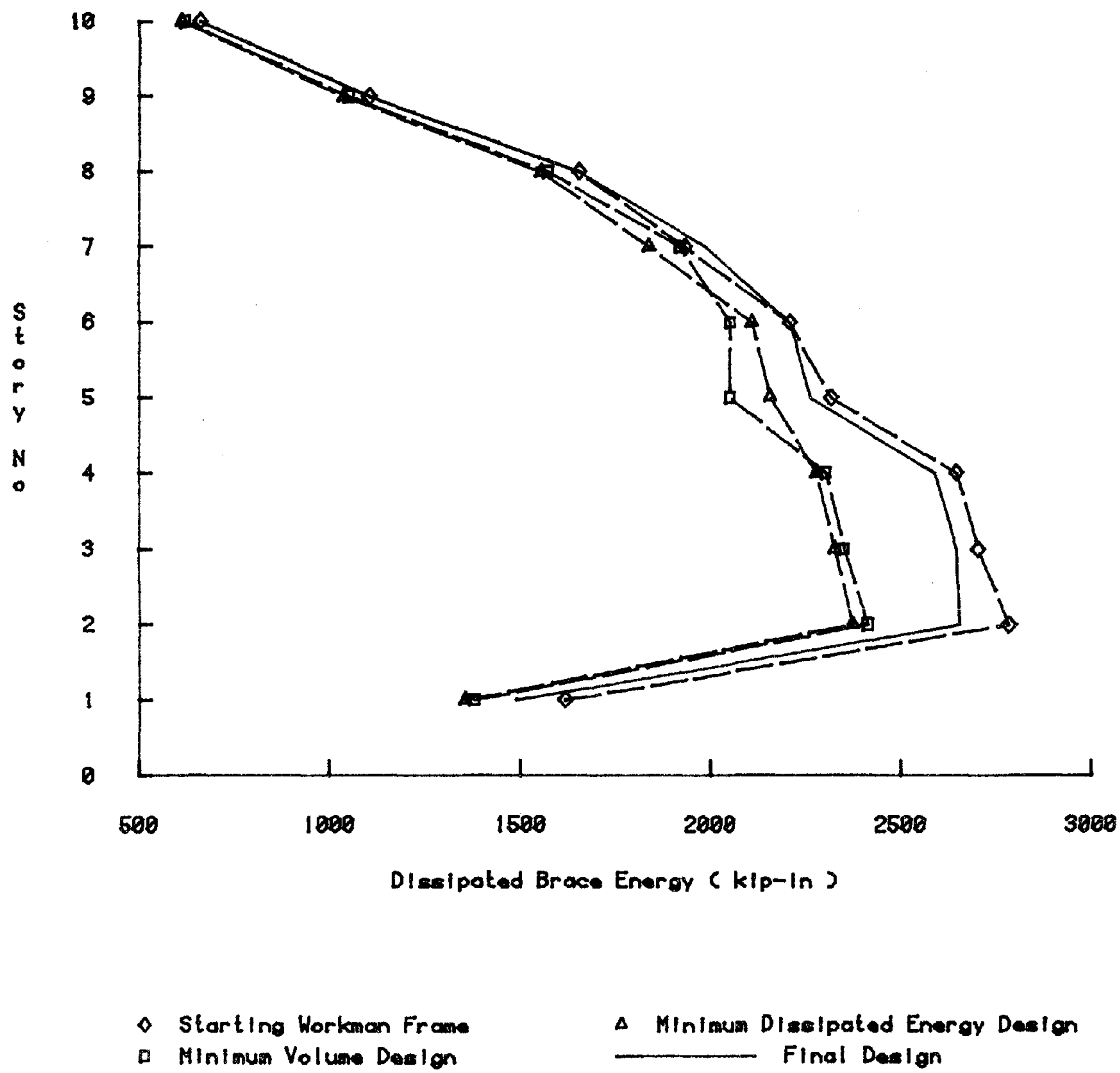
FIG(24) :Final Design.



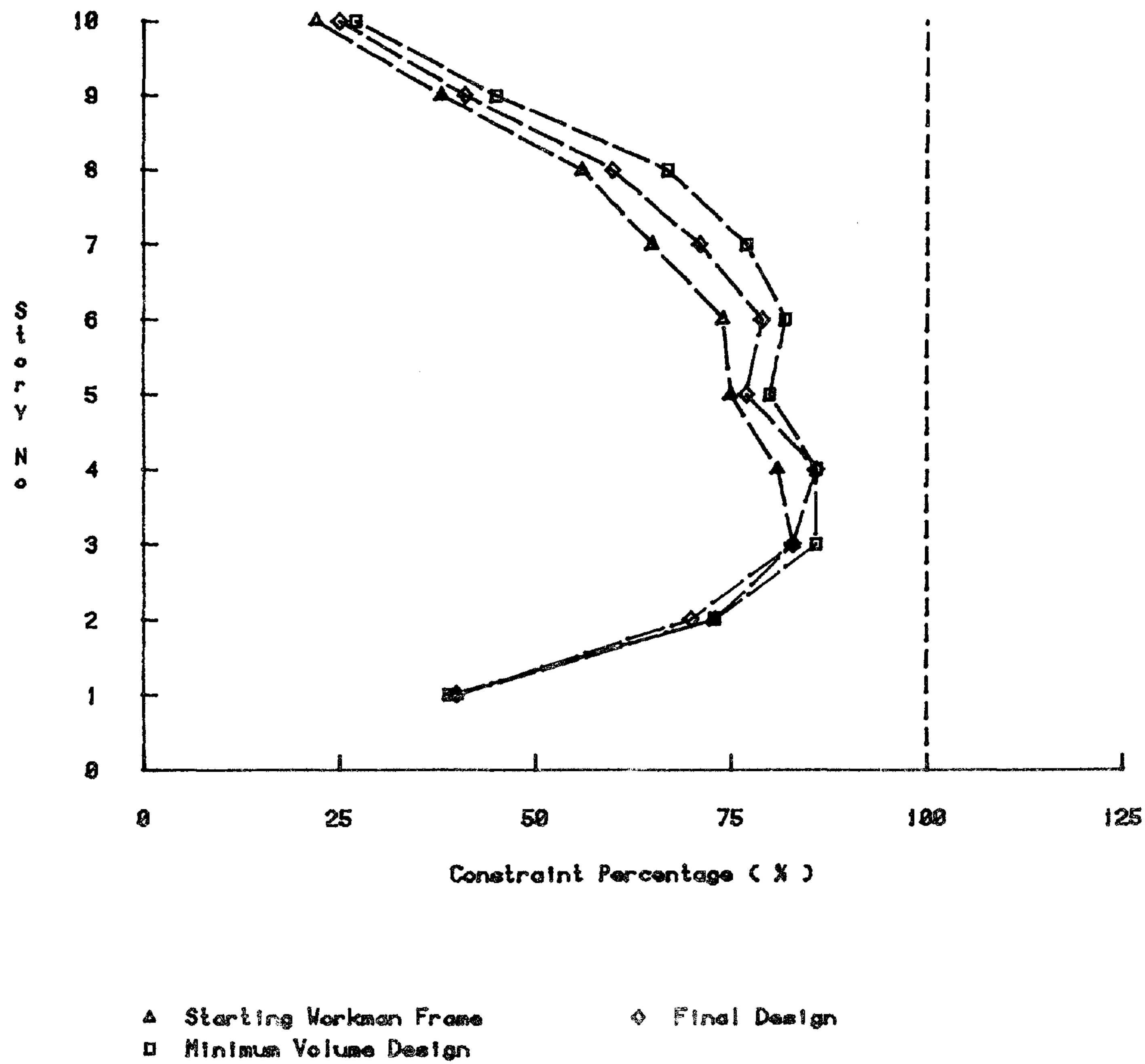
FIG(25) : Story No. vs Dissipated Brace Energy Constraint.



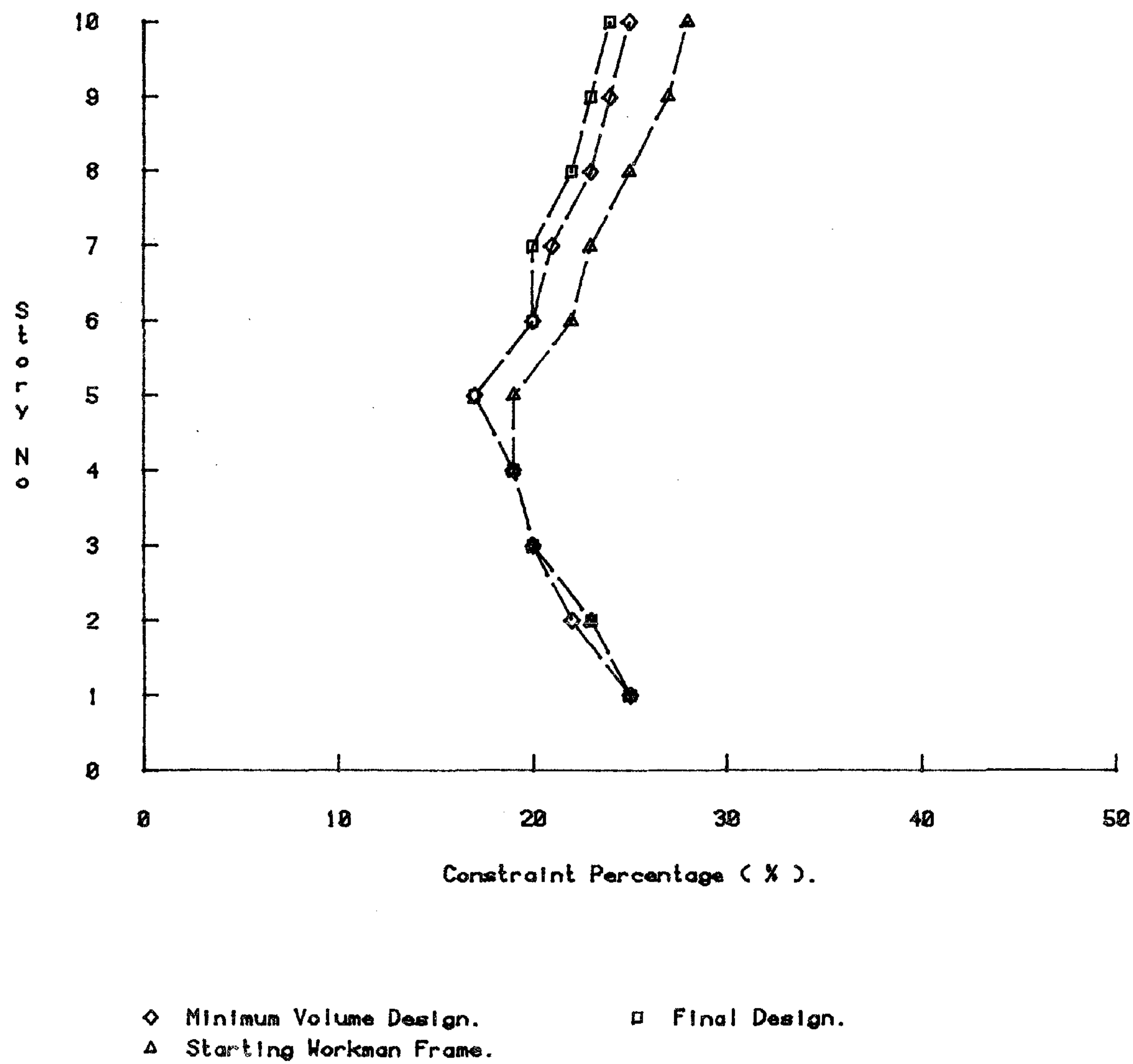
FIG(26) : Story No vs Dissipated Brace Energy (kip-in)



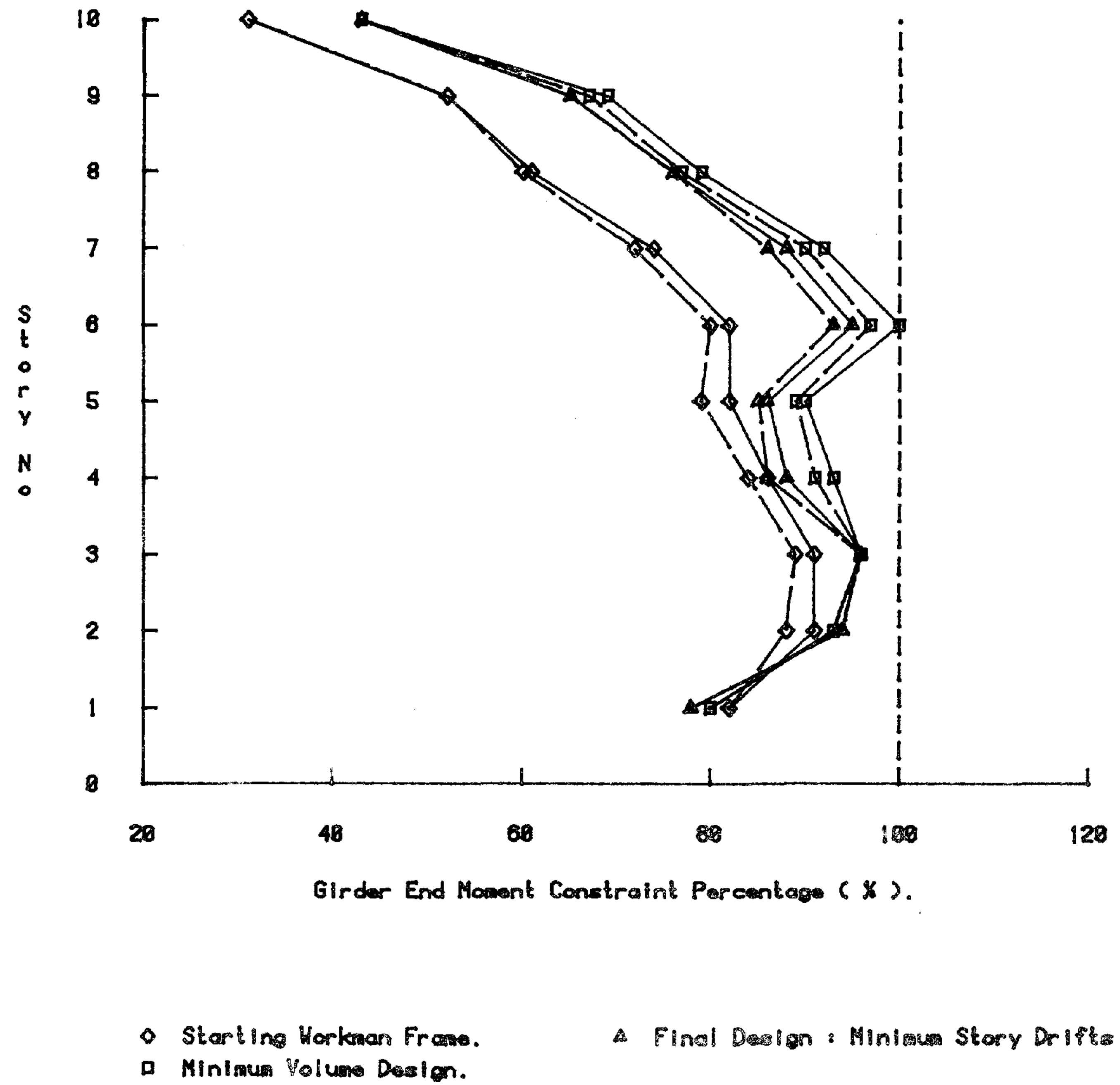
FIG(27) : Story No. vs Story Drift.(Drift = 0.005)



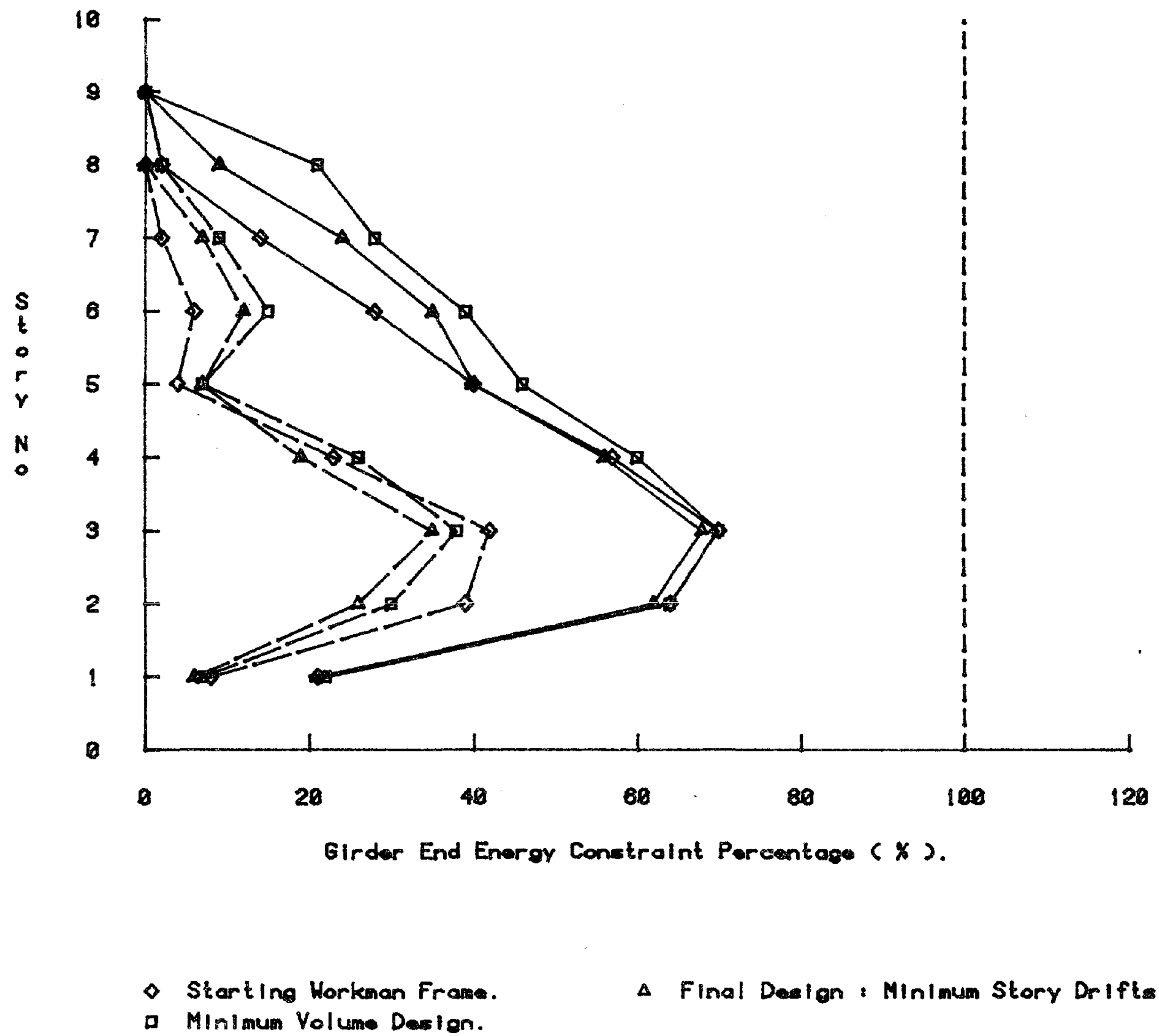
FIG(28) : Story No. vs Floor Acceleration [Accel = 0.5].



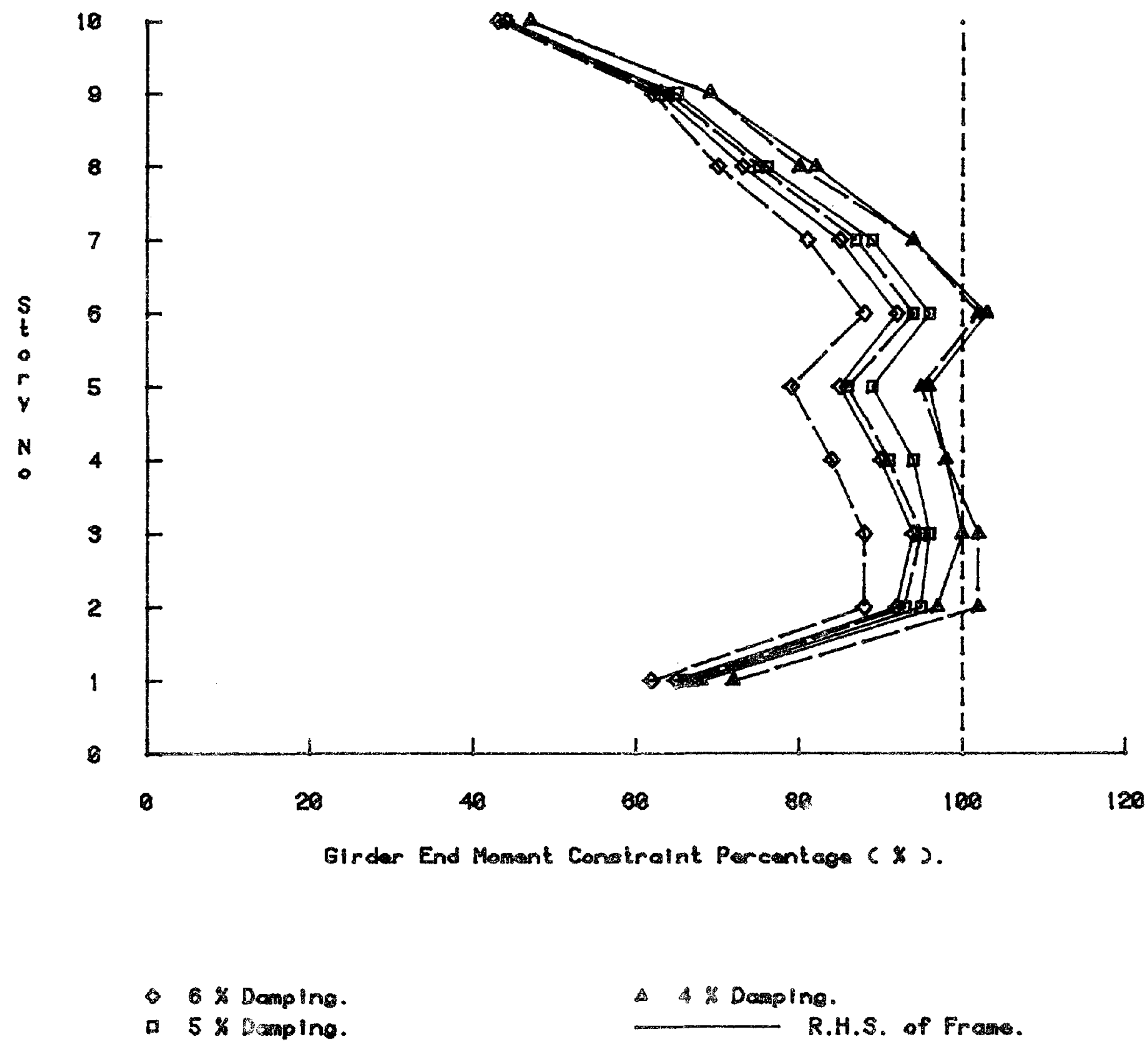
FIG(29) : Story No vs Moderate Eq. Girder End Moment.



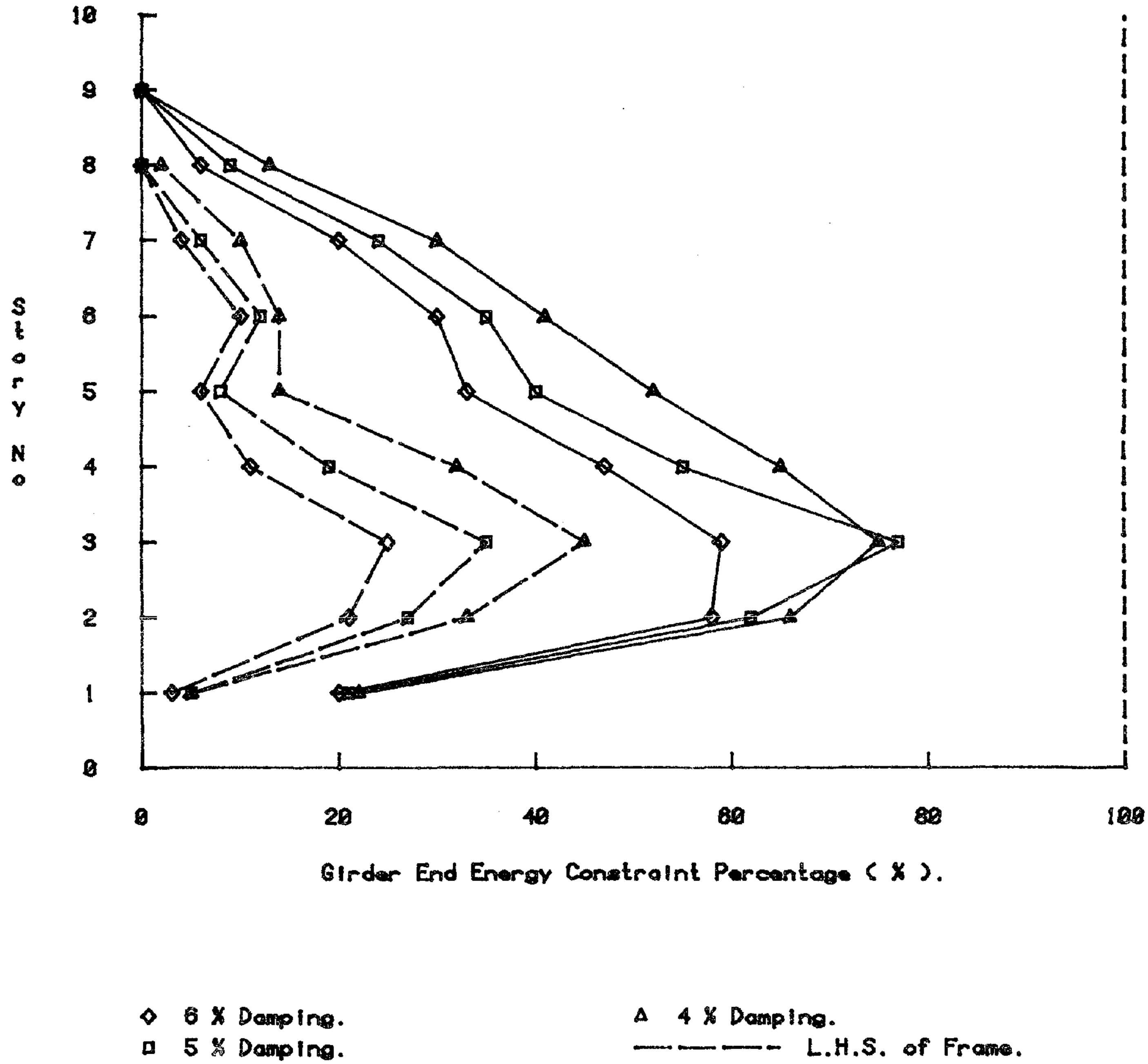
FIG(30) : Story No vs Dissipated Girder End Energy.



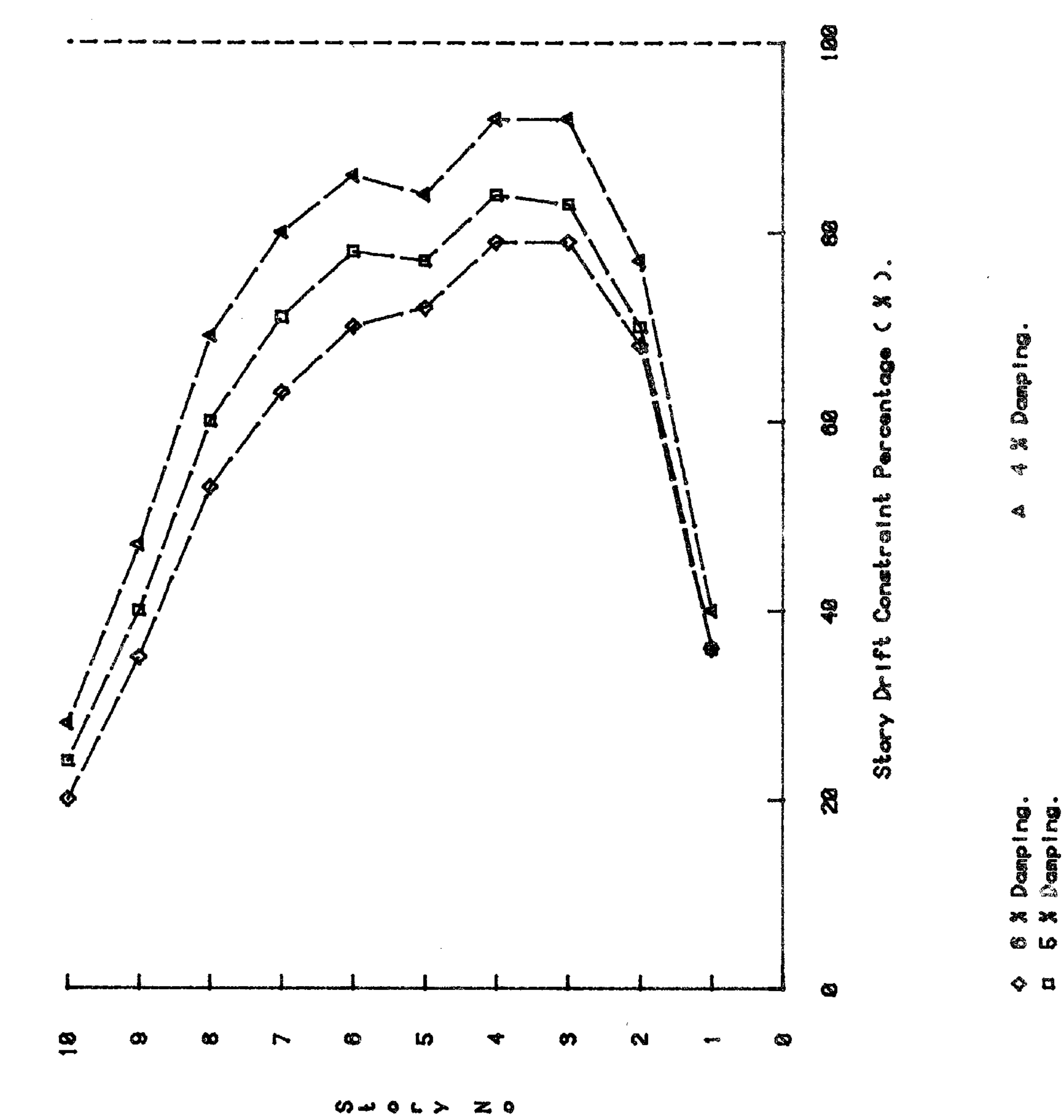
FIG(31) : Story No vs Girder End Moment.



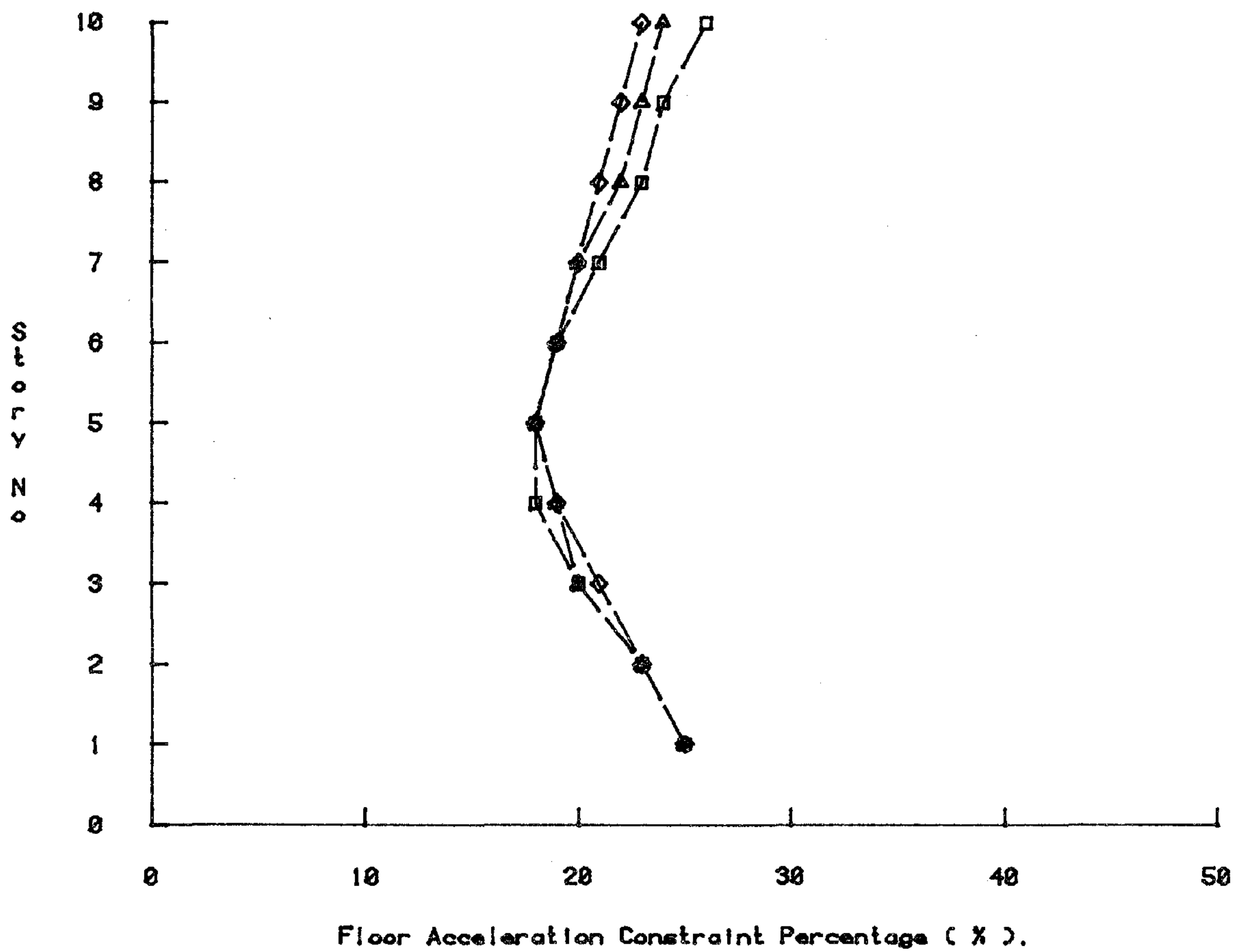
FIG(32) : Story No vs Girder End Energy.



FIG(33) : Story No vs Story Drift [Drift = 0.005]

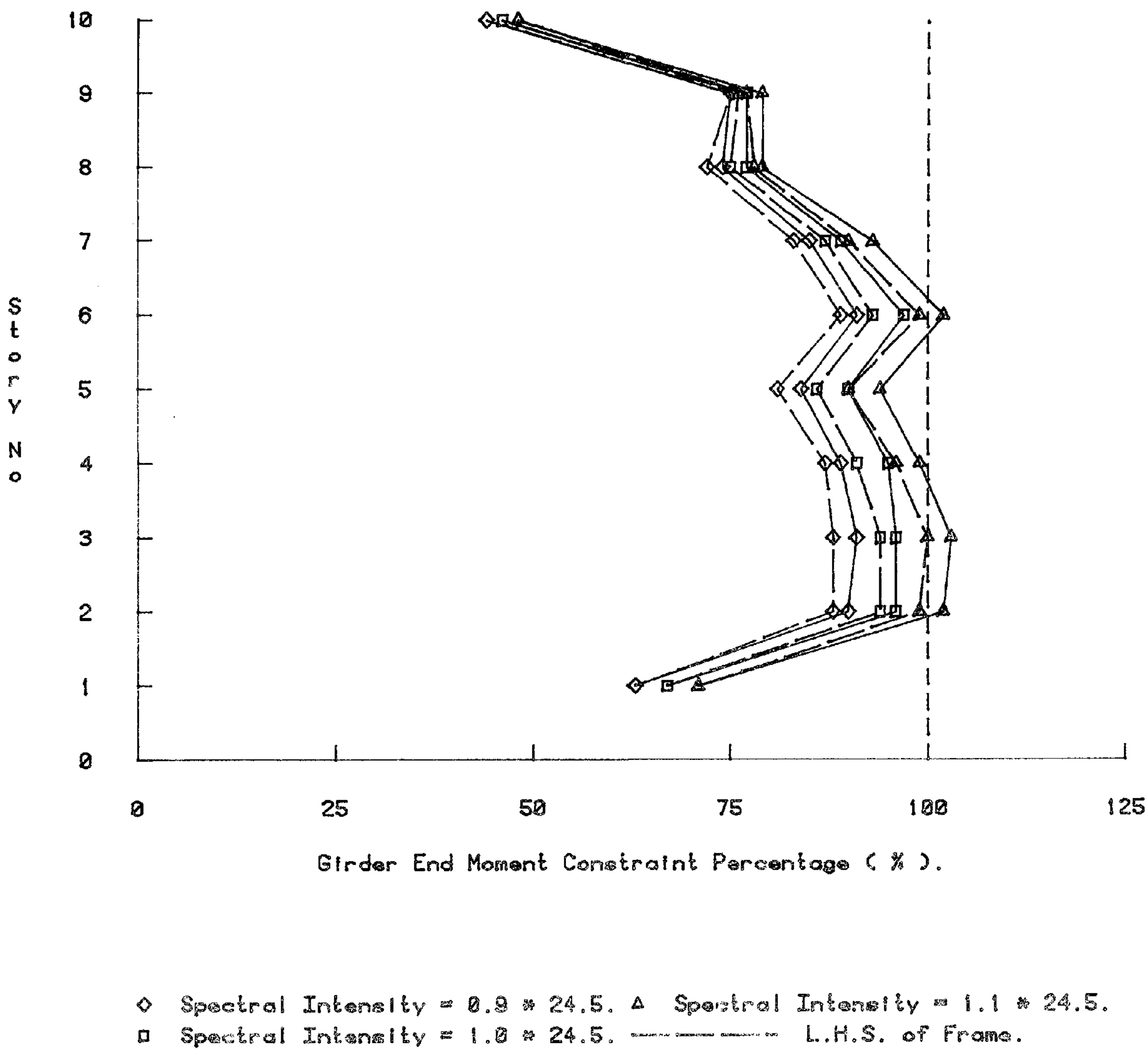


FIG(34) : Final Design : Perturbed Damping. [Accel = 0.5].

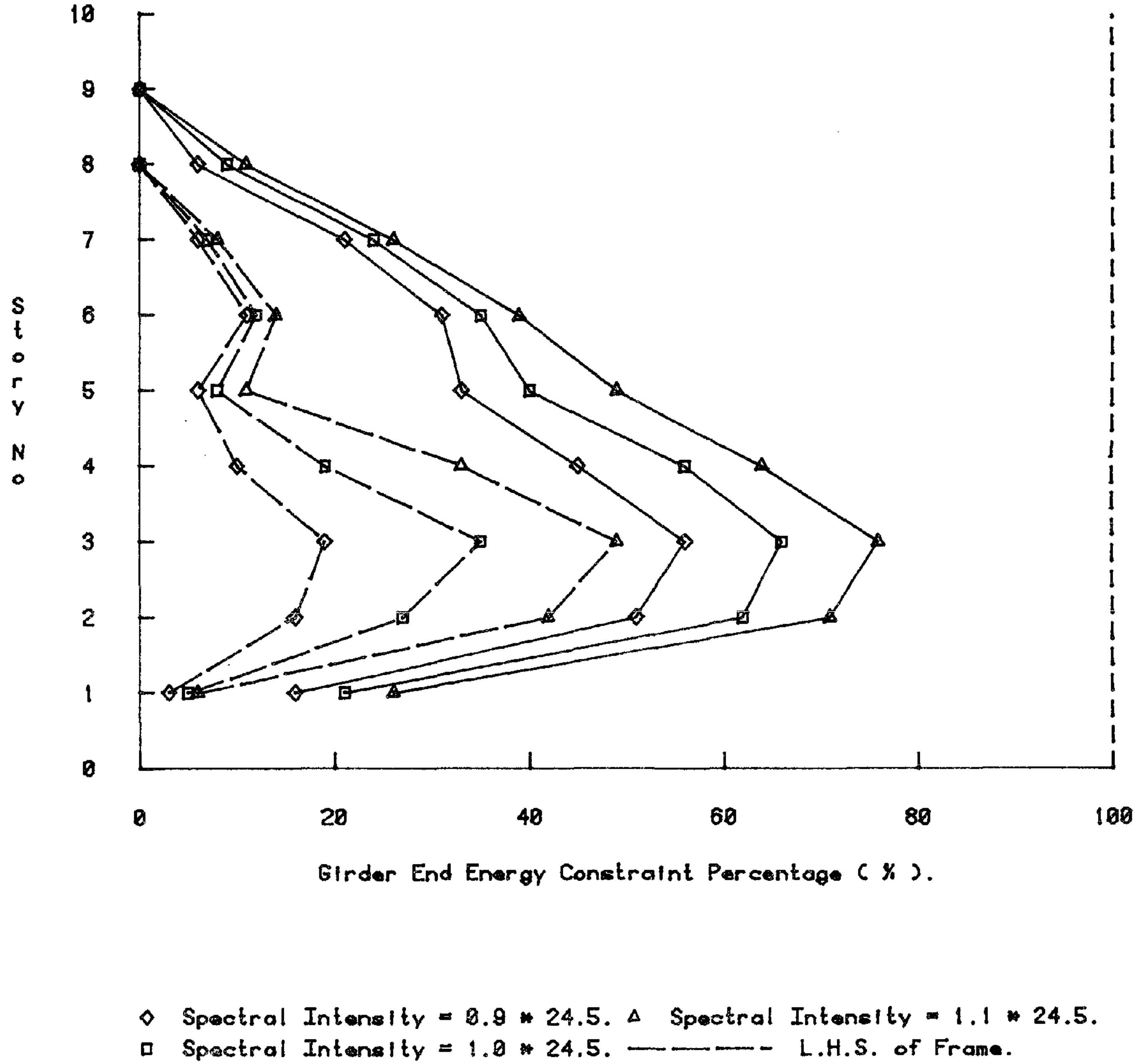


◇ 6 % Damping..
△ 5 % Damping.
□ 4 % Damping.

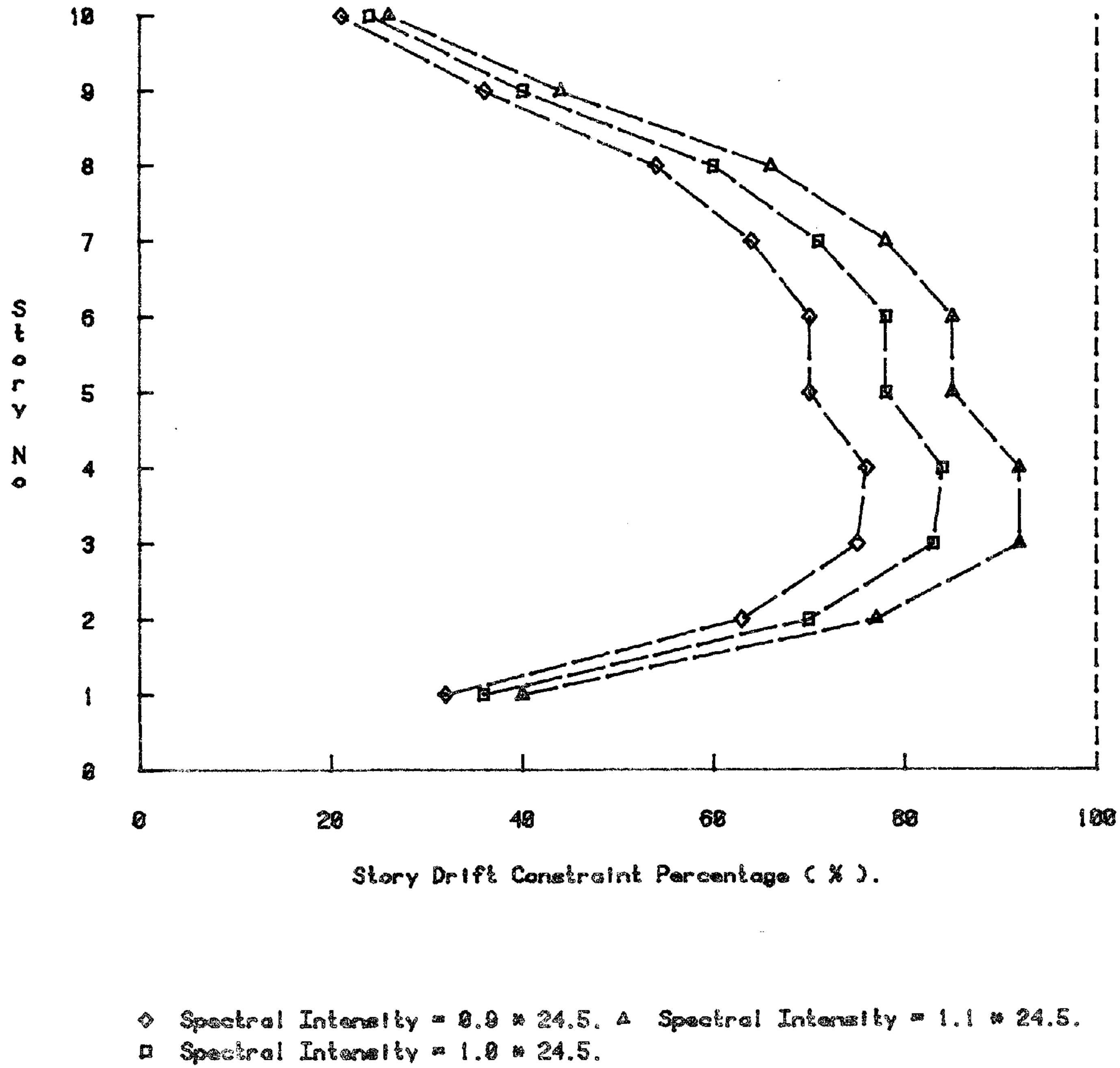
FIG(35) : Final Design : Perturbed Spectral Intensity [GiryId = 1]



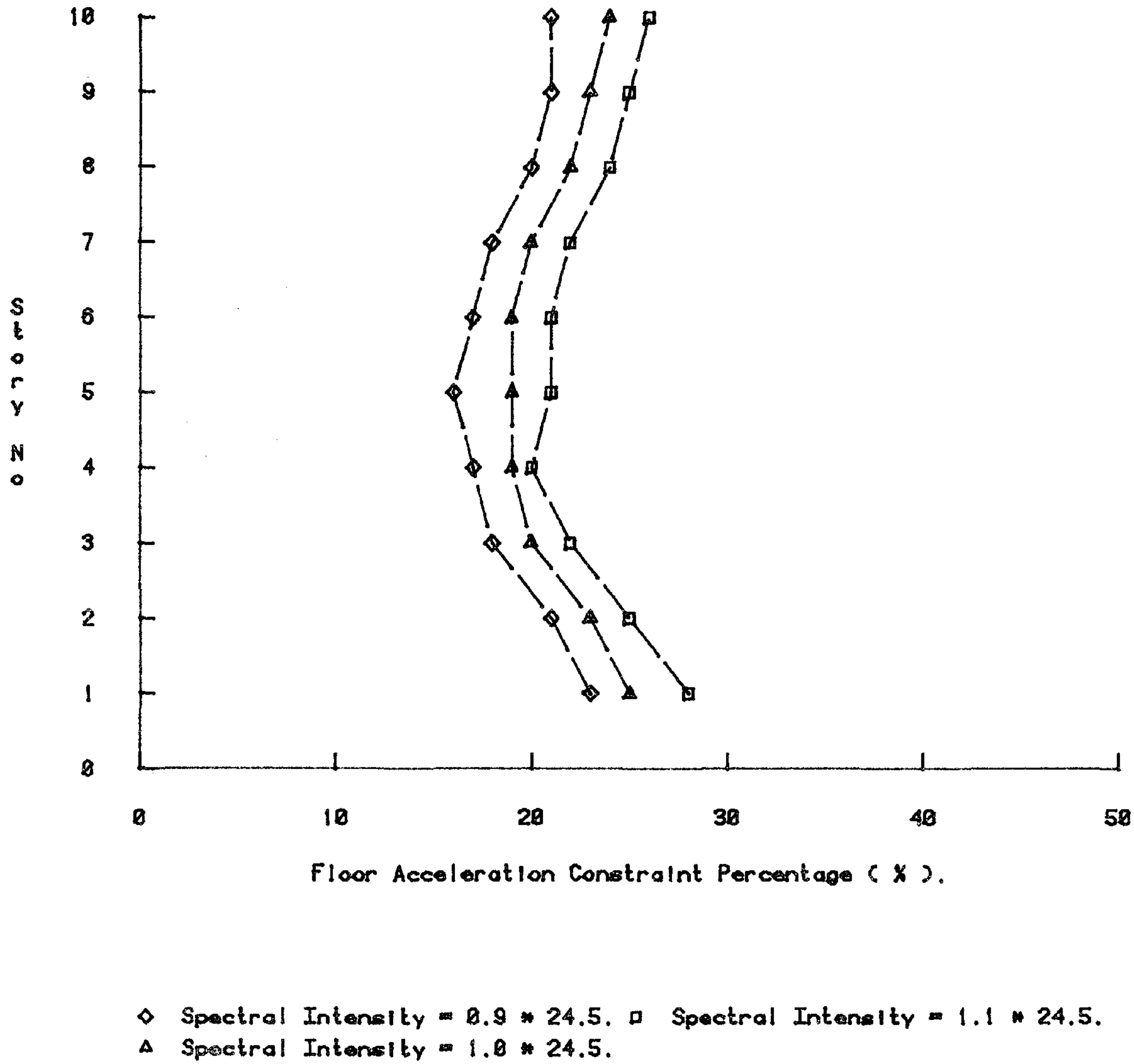
FIG(36) : Final Design : Perturbed Spectral Intensity [Girduc = 6]



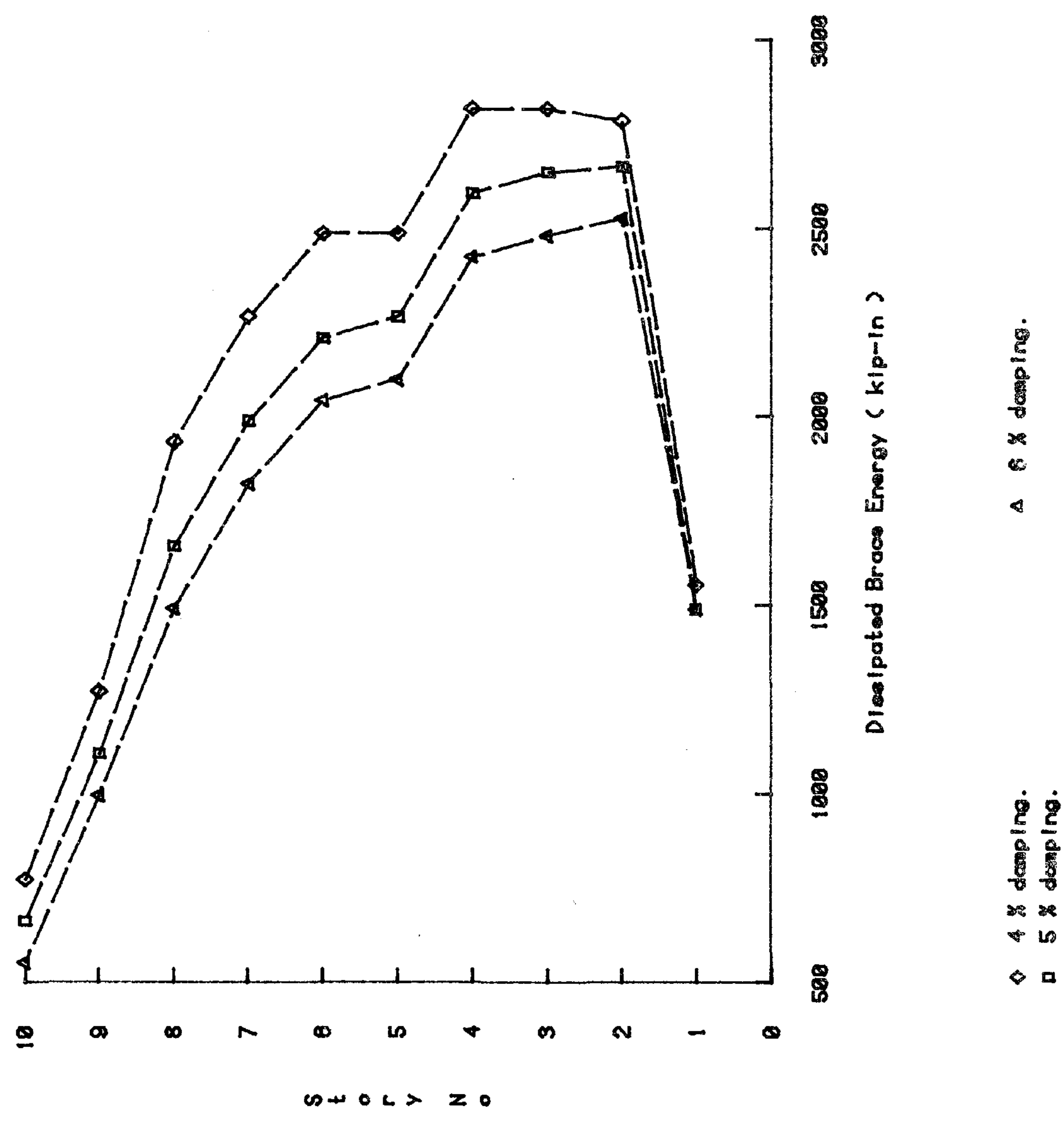
FIG(37) : Final Design : Perturbed Spectral Intensity [Drift = 0.005]



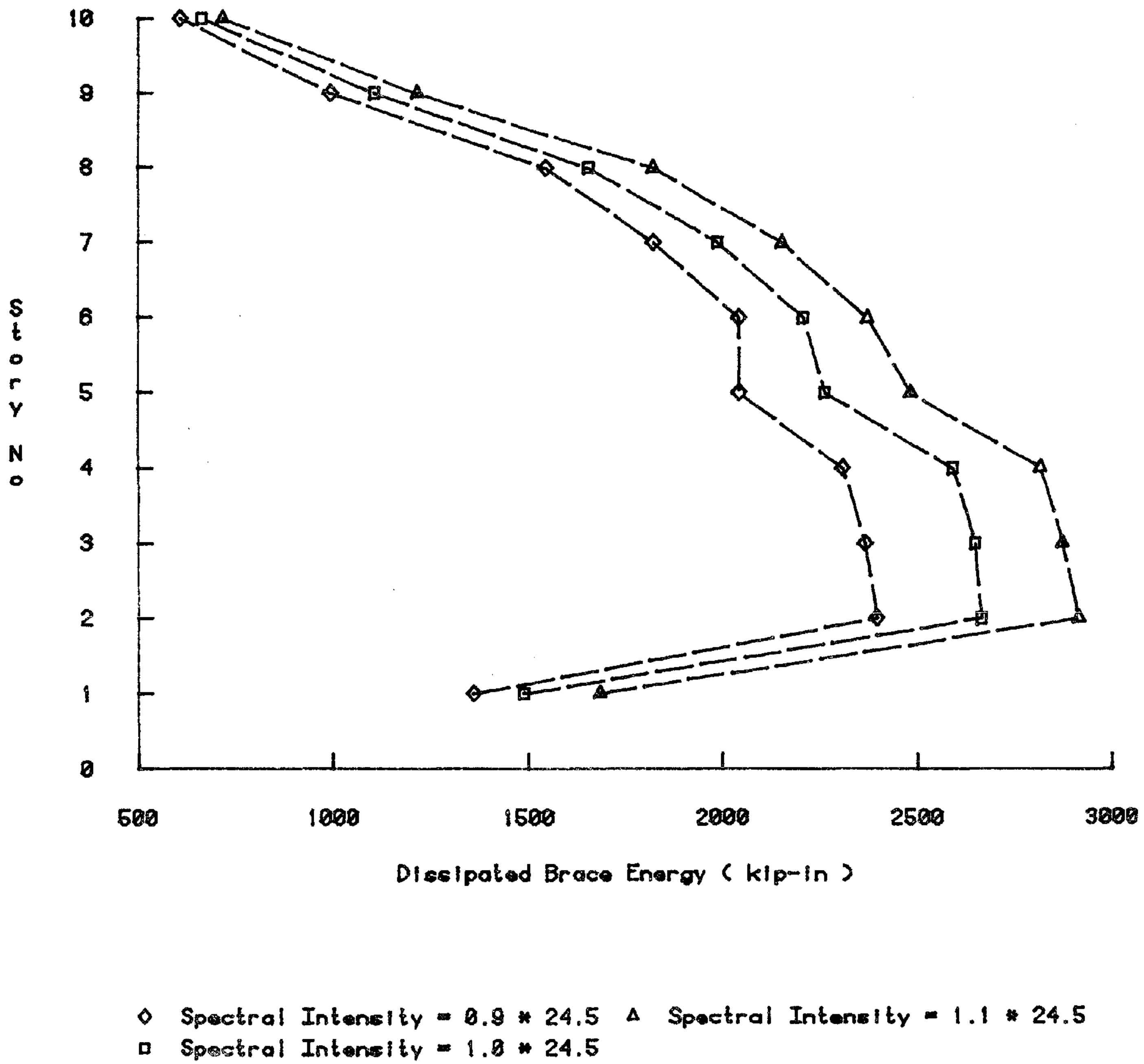
FIG(38) : Final Design : Perturbed Spectral Intensity [Accel = 0.5].



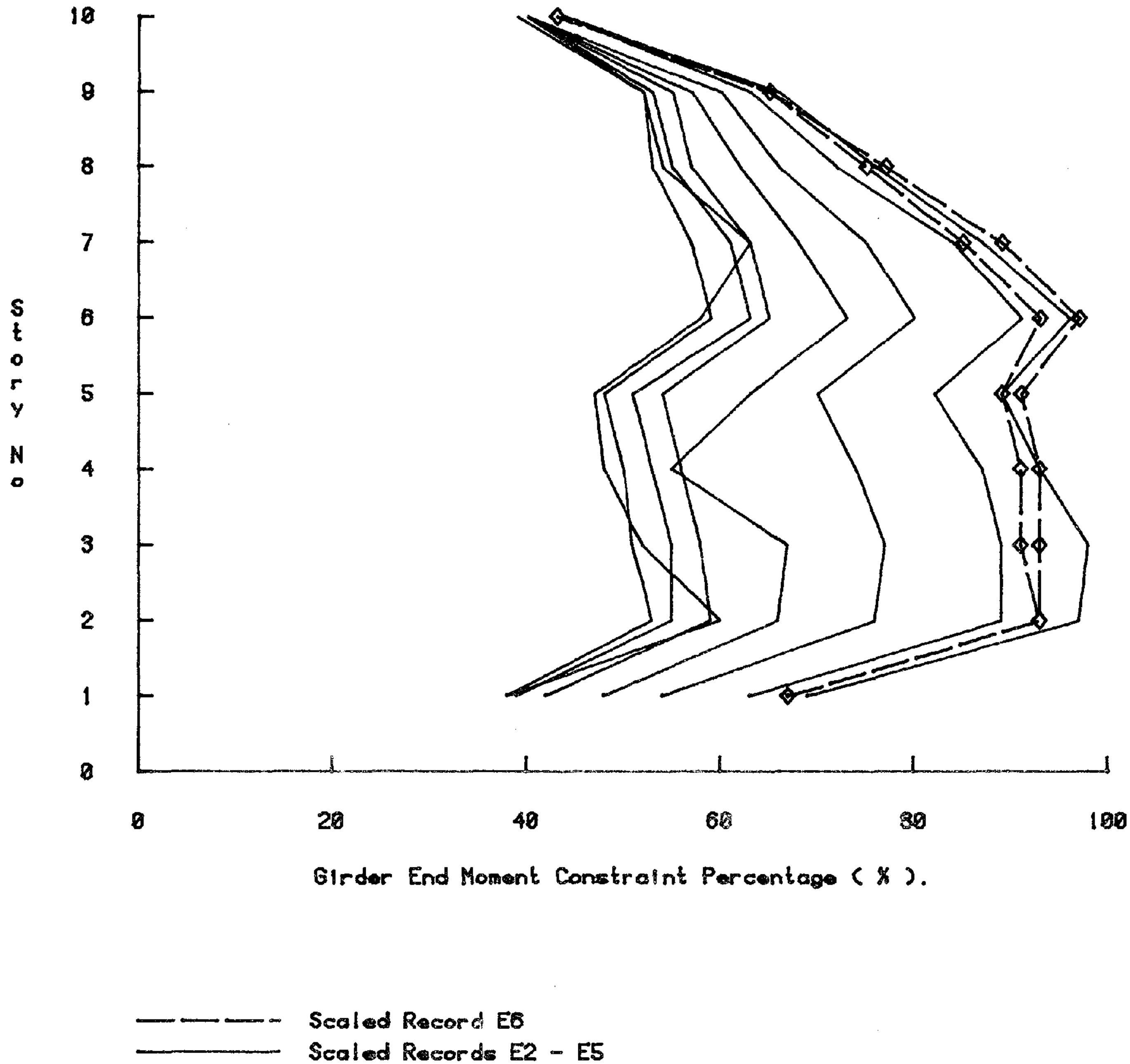
FIG(39) : Final Design : Perturbed Damping



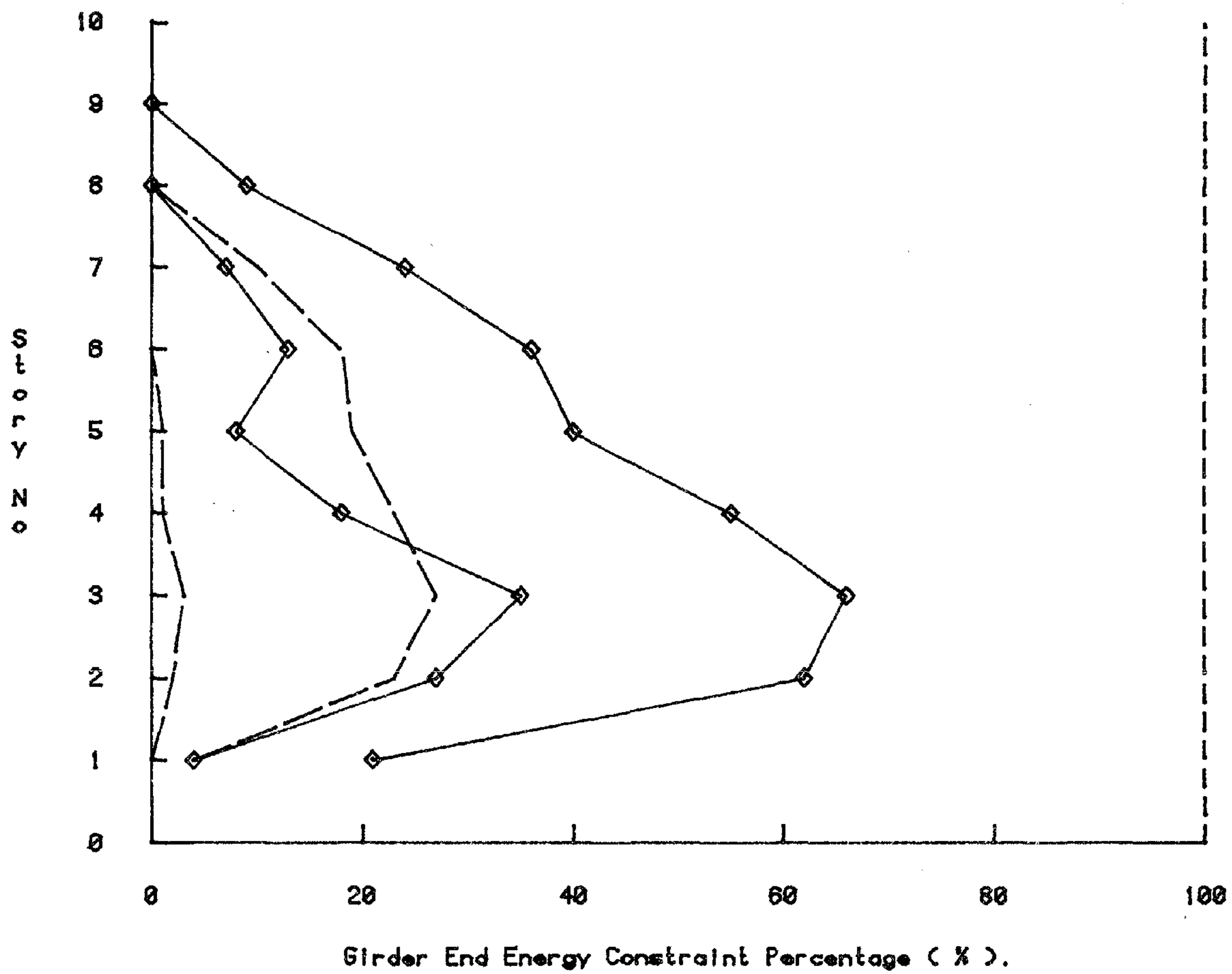
FIG(40) : Final Design : Perturbed Spectral Intensity



FIG(41) : Final Design : Different Eq. Records [GiryId = 1]



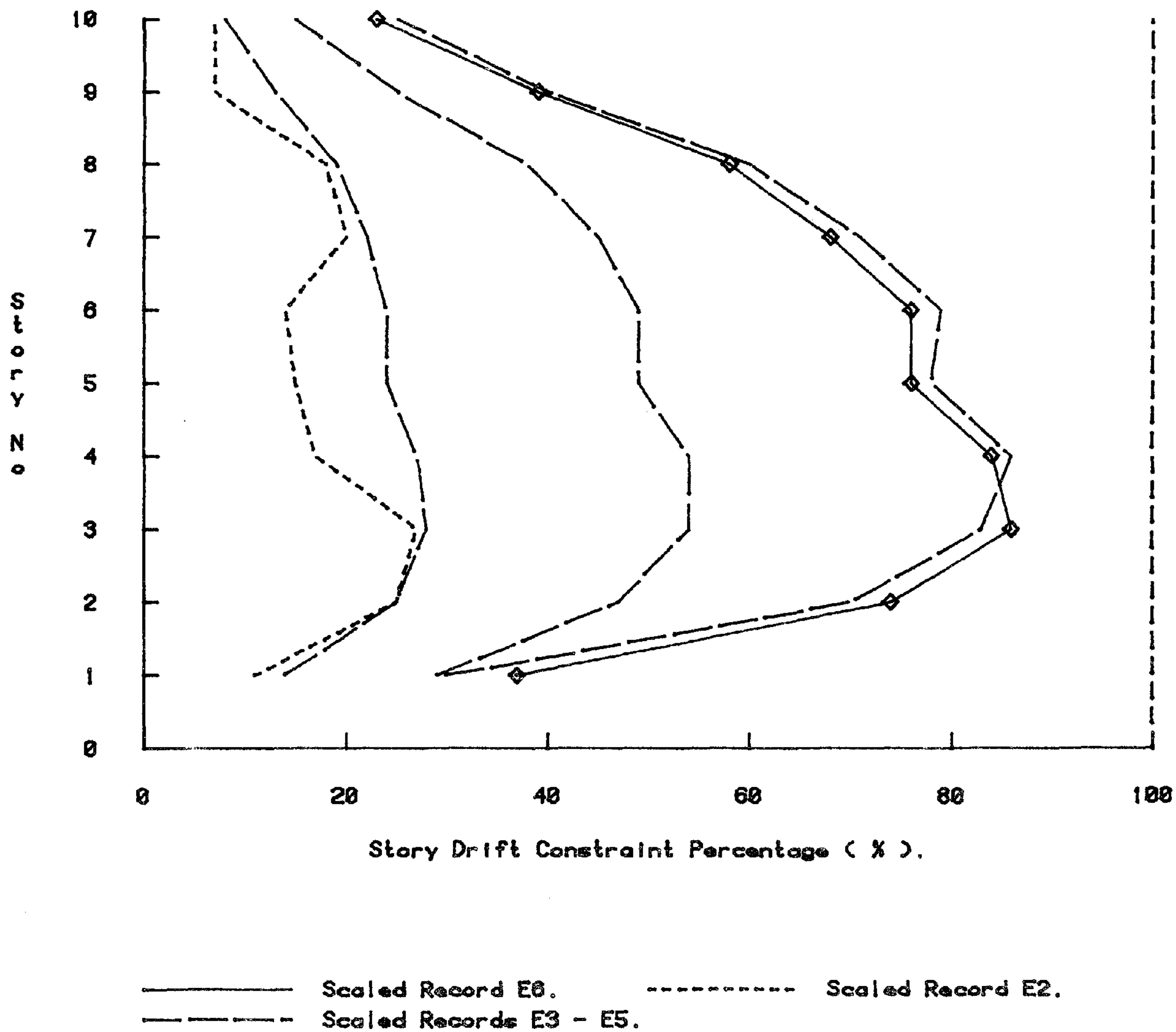
FIG(42) : Final Design : Different Eq. Records [Girduc = 6]



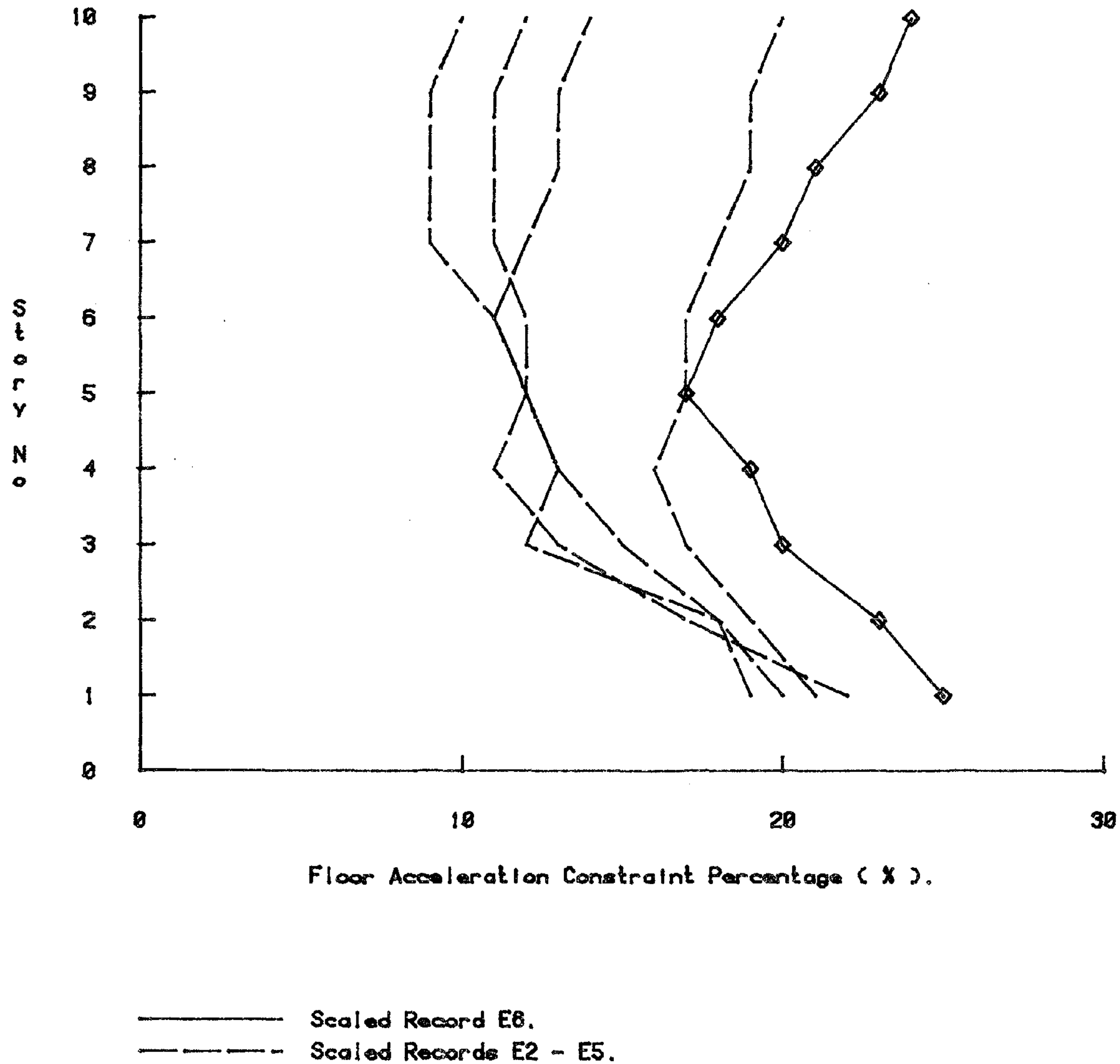
——————— Scaled Record E6

— — — — Scaled Record E5 ; Girders remain elastic for E3 and E4

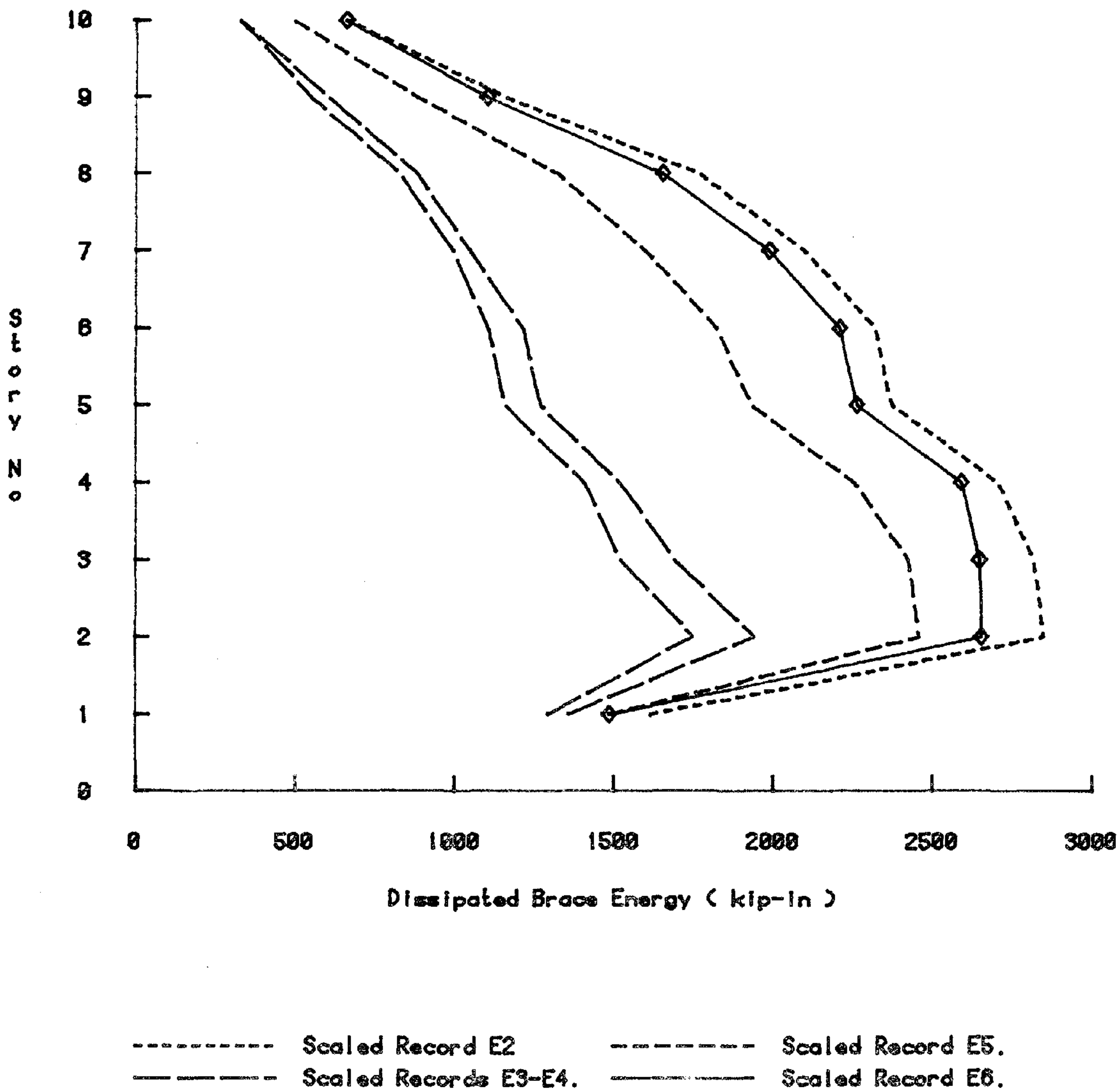
FIG(43) : Final Design : Different Eq. Records [Drift = 0.005].



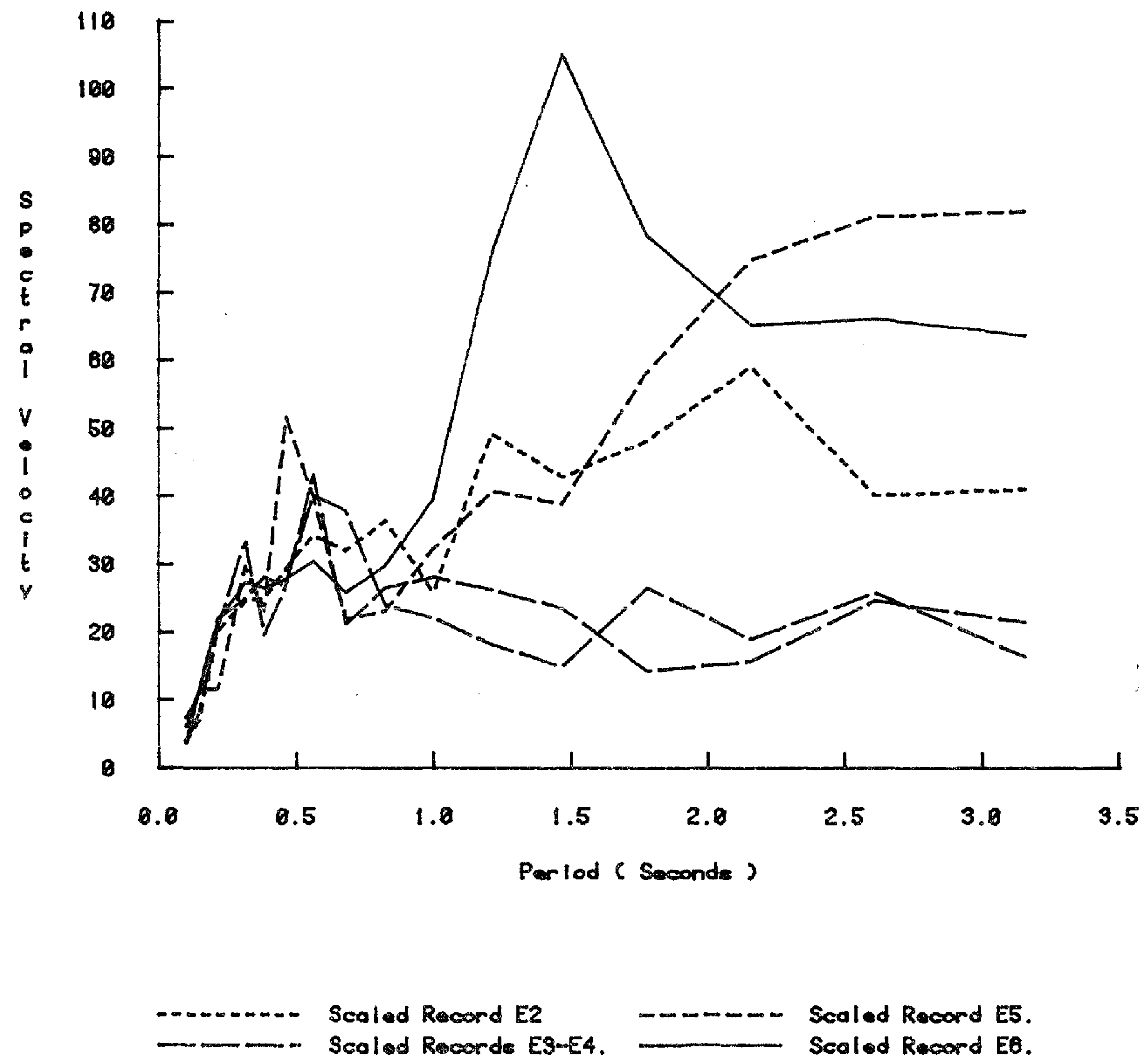
FIG(44) : Final Design : Different Eq. Records [Accel = 0.5].



FIG(45) : Final Design : Different Eq. Records.



FIG(46) : Pseudo-Spectral Velocity (in/sec) vs Period (sec).



EARTHQUAKE ENGINEERING RESEARCH CENTER REPORTS

NOTE: Numbers in parentheses are Accession Numbers assigned by the National Technical Information Service; these are followed by a price code. Copies of the reports may be ordered from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia, 22161. Accession Numbers should be quoted on orders for reports (PB --- ---) and remittance must accompany each order. Reports without this information were not available at time of printing. The complete list of EERC reports (from EERC 67-1) is available upon request from the Earthquake Engineering Research Center, University of California, Berkeley, 47th Street and Hoffman Boulevard, Richmond, California 94804.

UCB/EERC-77/01 "PLUSH - A Computer Program for Probabilistic Finite Element Analysis of Seismic Soil-Structure Interaction," by M.P. Romo Organista, J. Lysmer and H.B. Seed - 1977 (PB81 177 651)A05

UCB/EERC-77/02 "Soil-Structure Interaction Effects at the Humboldt Bay Power Plant in the Ferndale Earthquake of June 7, 1975," by J.E. Valera, H.B. Seed, C.F. Tsai and J. Lysmer - 1977 (PB 265 795)A04

UCB/EERC-77/03 "Influence of Sample Disturbance on Sand Response to Cyclic Loading," by K. Mori, H.B. Seed and C.K. Chan - 1977 (PB 267 352)A04

UCB/EERC-77/04 "Seismological Studies of Strong Motion Records," by J. Shoja-Taheri - 1977 (PB 269 655)A10

UCB/EERC-77/05 Unassigned

UCB/EERC-77/06 "Developing Methodologies for Evaluating the Earthquake Safety of Existing Buildings," by No. 1 - B. Bresler; No. 2 - B. Bresler, T. Okada and D. Zisling; No. 3 - T. Okada and B. Bresler; No. 4 - V.V. Bertero and B. Bresler - 1977 (PB 267 354)A08

UCB/EERC-77/07 "A Literature Survey - Transverse Strength of Masonry Walls," by Y. Omote, R.L. Mayes, S.W. Chen and R.W. Clough - 1977 (PB 277 933)A07

UCB/EERC-77/08 "DRAIN-TABS: A Computer Program for Inelastic Earthquake Response of Three Dimensional Buildings," by R. Guendelman-Israel and G.H. Powell - 1977 (PB 270 693)A07

UCB/EERC-77/09 "SUBWALL: A Special Purpose Finite Element Computer Program for Practical Elastic Analysis and Design of Structural Walls with Substructure Option," by D.Q. Le, H. Peterson and E.P. Popov - 1977 (PB 270 567)A05

UCB/EERC-77/10 "Experimental Evaluation of Seismic Design Methods for Broad Cylindrical Tanks," by D.P. Clough (PB 272 280)A13

UCB/EERC-77/11 "Earthquake Engineering Research at Berkeley - 1976," - 1977 (PB 273 507)A09

UCB/EERC-77/12 "Automated Design of Earthquake Resistant Multistory Steel Building Frames," by N.D. Walker, Jr. - 1977 (PB 276 526)A09

UCB/EERC-77/13 "Concrete Confined by Rectangular Hoops Subjected to Axial Loads," by J. Vallenas, V.V. Bertero and E.P. Popov - 1977 (PB 275 165)A06

UCB/EERC-77/14 "Seismic Strain Induced in the Ground During Earthquakes," by Y. Sugimura - 1977 (PB 284 201)A04

UCB/EERC-77/15 Unassigned

UCB/EERC-77/16 "Computer Aided Optimum Design of Ductile Reinforced Concrete Moment Resisting Frames," by S.W. Zagajeski and V.V. Bertero - 1977 (PB 280 137)A07

UCB/EERC-77/17 "Earthquake Simulation Testing of a Stepping Frame with Energy-Absorbing Devices," by J.M. Kelly and D.F. Tsztoo - 1977 (PB 273 506)A04

UCB/EERC-77/18 "Inelastic Behavior of Eccentrically Braced Steel Frames under Cyclic Loadings," by C.W. Roeder and E.P. Popov - 1977 (PB 275 526)A15

UCB/EERC-77/19 "A Simplified Procedure for Estimating Earthquake-Induced Deformations in Dams and Embankments," by F.I. Makdisi and H.B. Seed - 1977 (PB 276 820)A04

UCB/EERC-77/20 "The Performance of Earth Dams during Earthquakes," by H.B. Seed, F.I. Makdisi and P. de Alba - 1977 (PB 276 821)A04

UCB/EERC-77/21 "Dynamic Plastic Analysis Using Stress Resultant Finite Element Formulation," by P. Lukkunapvasit and J.M. Kelly - 1977 (PB 275 453)A04

UCB/EERC-77/22 "Preliminary Experimental Study of Seismic Uplift of a Steel Frame," by R.W. Clough and A.A. Huckelbridge 1977 (PB 278 769)A08

UCB/EERC-77/23 "Earthquake Simulator Tests of a Nine-Story Steel Frame with Columns Allowed to Uplift," by A.A. Huckelbridge - 1977 (PB 277 944)A09

UCB/EERC-77/24 "Nonlinear Soil-Structure Interaction of Skew Highway Bridges," by M.-C. Chen and J. Penzien - 1977 (PB 276 176)A07

UCB/EERC-77/25 "Seismic Analysis of an Offshore Structure Supported on Pile Foundations," by D.D.-N. Liou and J. Penzien 1977 (PB 283 180)A06

UCB/EERC-77/26 "Dynamic Stiffness Matrices for Homogeneous Viscoelastic Half-Planes," by G. Dasgupta and A.K. Chopra - 1977 (PB 279 654)A06

UCB/EERC-77/27 "A Practical Soft Story Earthquake Isolation System," by J.M. Kelly, J.M. Eidinger and C.J. Derham - 1977 (PB 276 814)A07

UCB/EERC-77/28 "Seismic Safety of Existing Buildings and Incentives for Hazard Mitigation in San Francisco: An Exploratory Study," by A.J. Meltsner - 1977 (PB 281 970)A05

UCB/EERC-77/29 "Dynamic Analysis of Electrohydraulic Shaking Tables," by D. Rea, S. Abedi-Hayati and Y. Takahashi 1977 (PB 282 569)A04

UCB/EERC-77/30 "An Approach for Improving Seismic - Resistant Behavior of Reinforced Concrete Interior Joints," by B. Galunic, V.V. Bertero and E.P. Popov - 1977 (PB 290 870)A06

UCB/EERC-78/01 "The Development of Energy-Absorbing Devices for Aseismic Base Isolation Systems," by J.M. Kelly and D.F. Tsztoo - 1978 (PB 284 978)A04

UCB/EERC-78/02 "Effect of Tensile Prestrain on the Cyclic Response of Structural Steel Connections, by J.G. Bouwkamp and A. Mukhopadhyay - 1978

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UCB/EERC-78/07 "Studies of Structural Response to Earthquake Ground Motion," by O.A. Lopez and A.K. Chopra - 1978 (PB 282 790)A05

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UCB/EERC-78/11 "Mathematical Modelling of Hysteresis Loops for Reinforced Concrete Columns," by S. Nakata, T. Sproul and J. Penzien - 1978 (PB 298 274)A05

UCB/EERC-78/12 "Damageability in Existing Buildings," by T. Blejwas and B. Bresler - 1978 (PB 80 166 978)A05

UCB/EERC-78/13 "Dynamic Behavior of a Pedestal Base Multistory Building," by R.M. Stephen, E.L. Wilson, J.G. Bouwkamp and M. Button - 1978 (PB 286 650)A08

UCB/EERC-78/14 "Seismic Response of Bridges - Case Studies," by R.A. Imbsen, V. Nutt and J. Penzien - 1978 (PB 286 503)A10

UCB/EERC-78/15 "A Substructure Technique for Nonlinear Static and Dynamic Analysis," by D.G. Row and G.H. Powell - 1978 (PB 288 077)A10

UCB/EERC-78/16 "Seismic Risk Studies for San Francisco and for the Greater San Francisco Bay Area," by C.S. Oliveira - 1978 (PB 81 120 115)A07

UCB/EERC-78/17 "Strength of Timber Roof Connections Subjected to Cyclic Loads," by P. Gülkan, R.L. Mayes and R.W. Clough - 1978 (HUD-000 1491)A07

UCB/EERC-78/18 "Response of K-Braced Steel Frame Models to Lateral Loads," by J.G. Bouwkamp, R.M. Stephen and E.P. Popov - 1978

UCB/EERC-78/19 "Rational Design Methods for Light Equipment in Structures Subjected to Ground Motion," by J.L. Sackman and J.M. Kelly - 1978 (PB 292 357)A04

UCB/EERC-78/20 "Testing of a Wind Restraint for Aseismic Base Isolation," by J.M. Kelly and D.E. Chitty - 1978 (PB 292 833)A03

UCB/EERC-78/21 "APOLLO - A Computer Program for the Analysis of Pore Pressure Generation and Dissipation in Horizontal Sand Layers During Cyclic or Earthquake Loading," by P.P. Martin and H.B. Seed - 1978 (PB 292 835)A04

UCB/EERC-78/22 "Optimal Design of an Earthquake Isolation System," by M.A. Bhatti, K.S. Pister and E. Polak - 1978 (PB 294 735)A06

UCB/EERC-78/23 "MASH - A Computer Program for the Non-Linear Analysis of Vertically Propagating Shear Waves in Horizontally Layered Deposits," by P.P. Martin and H.B. Seed - 1978 (PB 293 101)A05

UCB/EERC-78/24 "Investigation of the Elastic Characteristics of a Three Story Steel Frame Using System Identification," by I. Kaya and H.D. McNiven - 1978 (PB 296 225)A06

UCB/EERC-78/25 "Investigation of the Nonlinear Characteristics of a Three-Story Steel Frame Using System Identification," by I. Kaya and H.D. McNiven - 1978 (PB 301 363)A05

UCB/EERC-78/26 "Studies of Strong Ground Motion in Taiwan," by Y.M. Hsiung, B.A. Bolt and J. Penzien - 1978 (PB 298 436)A06

UCB/EERC-78/27 "Cyclic Loading Tests of Masonry Single Piers: Volume 1 - Height to Width Ratio of 2," by P.A. Hidalgo, R.L. Mayes, H.D. McNiven and R.W. Clough - 1978 (PB 296 211)A07

UCB/EERC-78/28 "Cyclic Loading Tests of Masonry Single Piers: Volume 2 - Height to Width Ratio of 1," by S.-W.J. Chen, P.A. Hidalgo, R.L. Mayes, R.W. Clough and H.D. McNiven - 1978 (PB 296 212)A09

UCB/EERC-78/29 "Analytical Procedures in Soil Dynamics," by J. Lysmer - 1978 (PB 298 445)A06

UCB/EERC-79/01 "Hysteretic Behavior of Lightweight Reinforced Concrete Beam-Column Subassemblages," by B. Forzani, E.P. Popov and V.V. Bertero - April 1979(PB 298 267)A06

UCB/EERC-79/02 "The Development of a Mathematical Model to Predict the Flexural Response of Reinforced Concrete Beams to Cyclic Loads, Using System Identification," by J. Stanton & H. McNiven - Jan. 1979(PB 295 875)A10

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UCB/EERC-79/04 "A Mathematical Model of Masonry for Predicting its Linear Seismic Response Characteristics," by Y. Mengi and H.D. McNiven - Feb. 1979(PB 298 266)A06

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UCB/EERC-79/07 "The Design of Steel Energy Absorbing Restrainers and Their Incorporation into Nuclear Power Plants for Enhanced Safety: Volume 1 - Summary Report," by P.N. Spencer, V.F. Zackay, and E.R. Parker - Feb. 1979(UCB/EERC-79/07)A09

UCB/EERC-79/08 "The Design of Steel Energy Absorbing Restrainers and Their Incorporation into Nuclear Power Plants for Enhanced Safety: Volume 2 - The Development of Analyses for Reactor System Piping,""Simple Systems" by M.C. Lee, J. Penzien, A.K. Chopra and K, Suzuki "Complex Systems" by G.H. Powell, E.L. Wilson, R.W. Clough and D.G. Row - Feb. 1979(UCB/EERC-79/08)A10

UCB/EERC-79/09 "The Design of Steel Energy Absorbing Restrainers and Their Incorporation into Nuclear Power Plants for Enhanced Safety: Volume 3 - Evaluation of Commercial Steels," by W.S. Owen, R.M.N. Pelloux, R.O. Ritchie, M. Faral, T. Ohhashi, J. Toplosky, S.J. Hartman, V.F. Zackay and E.R. Parker - Feb. 1979(UCB/EERC-79/09)A04

UCB/EERC-79/10 "The Design of Steel Energy Absorbing Restrainers and Their Incorporation into Nuclear Power Plants for Enhanced Safety: Volume 4 - A Review of Energy-Absorbing Devices," by J.M. Kelly and M.S. Skinner - Feb. 1979(UCB/EERC-79/10)A04

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UCB/EERC-79/12 "Cyclic Loading Tests of Masonry Single Piers; Volume 3 - Height to Width Ratio of 0.5," by P.A. Hidalgo, R.L. Mayes, H.D. McNiven and R.W. Clough - May 1979(PB 301 321)A08

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UCB/EERC-79/15 "Optimal Design of Localized Nonlinear Systems with Dual Performance Criteria Under Earthquake Excitations," by M.A. Bhatti - July 1979(PB 80 167 109)A06

UCB/EERC-79/16 "OPTDYN - A General Purpose Optimization Program for Problems with or without Dynamic Constraints," by M.A. Bhatti, E. Polak and K.S. Pister - July 1979(PB 80 167 091)A05

UCB/EERC-79/17 "ANSR-II, Analysis of Nonlinear Structural Response, Users Manual," by D.P. Mondkar and G.H. Powell July 1979(PB 80 113 301)A05

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UCB/EERC-79/23 "Shaking Table Study of Single-Story Masonry Houses, Volume 1: Test Structures 1 and 2," by P. Gülkan, R.L. Mayes and R.W. Clough - Sept. 1979 (HUD-000 1763)A12

UCB/EERC-79/24 "Shaking Table Study of Single-Story Masonry Houses, Volume 2: Test Structures 3 and 4," by P. Gülkan, R.L. Mayes and R.W. Clough - Sept. 1979 (HUD-000 1836)A12

UCB/EERC-79/25 "Shaking Table Study of Single-Story Masonry Houses, Volume 3: Summary, Conclusions and Recommendations," by R.W. Clough, R.L. Mayes and P. Gülkan - Sept. 1979 (HUD-000 1837)A06

UCB/EERC-79/26 "Recommendations for a U.S.-Japan Cooperative Research Program Utilizing Large-Scale Testing Facilities," by U.S.-Japan Planning Group - Sept. 1979(PB 301 407)A06

UCB/EERC-79/27 "Earthquake-Induced Liquefaction Near Lake Amatitlan, Guatemala," by H.B. Seed, I. Arango, C.K. Chan, A. Gomez-Masso and R. Grant de Ascoli - Sept. 1979(NUREG-CR1341)A03

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UCB/EERC-79/34 "Interaction Effects of Simultaneous Torsional and Compressional Cyclic Loading of Sand," by P.M. Griffin and W.N. Houston - Dec. 1979(ADA 092 352)A15

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UCB/EERC-80/06 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 1A): Piping with Energy Absorbing Restrainers: Parameter Study on Small Systems," by G.H. Powell, C. Oughourlian and J. Simons - June 1980

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UCB/EERC-80/19 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 1B): Stochastic Seismic Analyses of Nuclear Power Plant Structures and Piping Systems Subjected to Multiple Support Excitations," by M.C. Lee and J. Penzien - June 1980

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UCB/EERC-80/30 "Dynamic Properties of an Eight-Story Prefabricated Panel Building," by J.G. Bouwkamp, J.P. Kollegger and R.M. Stephen - October 1980(PB81 200 313)A05

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UCB/EERC-80/33 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 4): Shaking Table Tests of Piping Systems with Energy-Absorbing Restrainers," by S.F. Stiemer and W.G. Godden - Sept. 1980

UCB/EERC-80/34 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 5): Summary Report," by P. Spencer

UCB/EERC-80/35 "Experimental Testing of an Energy-Absorbing Base Isolation System," by J.M. Kelly, M.S. Skinner and K.E. Beucke - October 1980(PB81 154 072)A04

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UCB/EERC-80/43 "Modelling of Soil-Structure Interaction by Finite and Infinite Elements," by F. Medina - December 1980(PB81 229 270)A04

UCB/EERC-81/01 "Control of Seismic Response of Piping Systems and Other Structures by Base Isolation," edited by J.M. Kelly - January 1981 (PB81 200 735)A05

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