50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NCEER-92-0022	2.	PB93-227502

4. Title and Subtitle IDARC Version 3.0: A Program for the Inelastic Damage Analysis of reinforced Concrete Structures	5. Report Date August 31, 1992
	6.
7. Author(s) S.K. Kunnath, A.M. Reinhorn and R.F. Lobo	8. Performing Organization Rept. No.
9. Performing Organization Name and Address Dept. of Civil Engineering SUNY at Buffalo Buffalo, New York 14260 Dept. of Civil & Environmental Engineering University of Central Florida Orlando, Florida 32816-0450	10. Project/Task/Work Unit No. 11. Contract(C) or Grant(G) No. (C) BCS 90-25010 NEC-91029 (G)
12. Sponsoring Organization Name and Address National Center for Earthquake Engineering Research State University of New York at Buffalo Red Jacket Quadrangle Buffalo, New York 14261	13. Type of Report & Period Covered Technical report 14.

15. Supplementary Notes
This research was conducted at the State University of New York at Buffalo and the University of Central Florida and was partially supported by the National Science Foundation under Grant No. BCS 90-25010 and the New York State Science and Technology Foundation under Grant No. NEC-91029.

16. Abstract (Limit: 200 words)
This report summarizes significant modeling and program enhancements to the computer code, IDARC (see NCEER-87-0008) for inelastic damage analysis of reinforced concrete frame-wall structures. As described here, IDARC is extended to include members with tapered cross-sections and the ability to specify different envelope characteristics at each end of the member. In addition, it is now able to handle moment releases at either end of a member to model perfect hinge connections. Two new element types are available: a circular column element with circumferential arrangement of longitudinal reinforcement and spiral hoops; and an inelastic discrete spring element which can be used to model nonlinear flexible connections, or indirectly, the effect of bar pull-out and joint distortions. P-Delta effects are now included in the step-by-step analysis, and a single-step correction to control unbalanced forces during event transition is incorporated. Also, it is now possible to specify applied force or displacement histories, typical in laboratory testing of components and subassemblages. The computation of damage indices has been considerably enhanced. Numerous input and output enhancements have also been incorporated. The program is validated using several available experimental results of dynamic and quasi-static testing of components, frames and model structures. Several sample problems are included, along with corresponding IDARC data files. A User Manual for the new version of the program accompanies this report.

17. Document Analysis a. Descriptors

b. Identifiers/Open-Ended Terms

IDARC. Computer programs. Reinforced concrete structures. Damage analysis. Frame wall systems. Inelastic response analysis. Case studies. Earthquake engineering.

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
Release Unlimited	Unclassified	146
	20. Security Class (This Page) Unclassified	22. Price

(See ANSI-Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4-77) (Formerly NTIS-35)

PB93-227502

NATIONAL CENTER FOR EARTHQUAKE
ENGINEERING RESEARCH
State University of New York at Buffalo

IDARC Version 3.0: A Program for the Inelastic Damage Analysis of Reinforced Concrete Structures

by

S.K. Kunnath, A.M. Reinhorn and R.F. Lobo
Department of Civil Engineering
State University of New York at Buffalo
Buffalo, New York 14260

and

Department of Civil and Environmental Engineering
University of Central Florida
Orlando, Florida 32816-0450

Technical Report NCEER-92-0022

August 31, 1992

This research was conducted at the State University of New York at Buffalo and the University of Central Florida and was partially supported by the National Science Foundation under Grant No. BCS 90-25010 and the New York State Science and Technology Foundation under Grant No. NEC-91029.

NOTICE

IDARC Version 3.0:
A Program for the Inelastic Damage
Analysis of Reinforced Concrete Structures

by

S.K. Kunnath[1], A.M. Reinhorn[2] and R.F. Lobo[3]

August 31, 1992

Technical Report NCEER-92-0022

NCEER Project Number 91-3115B

NSF Master Contract Number BCS 90-25010
and
NYSSTF Grant Number NEC-91029

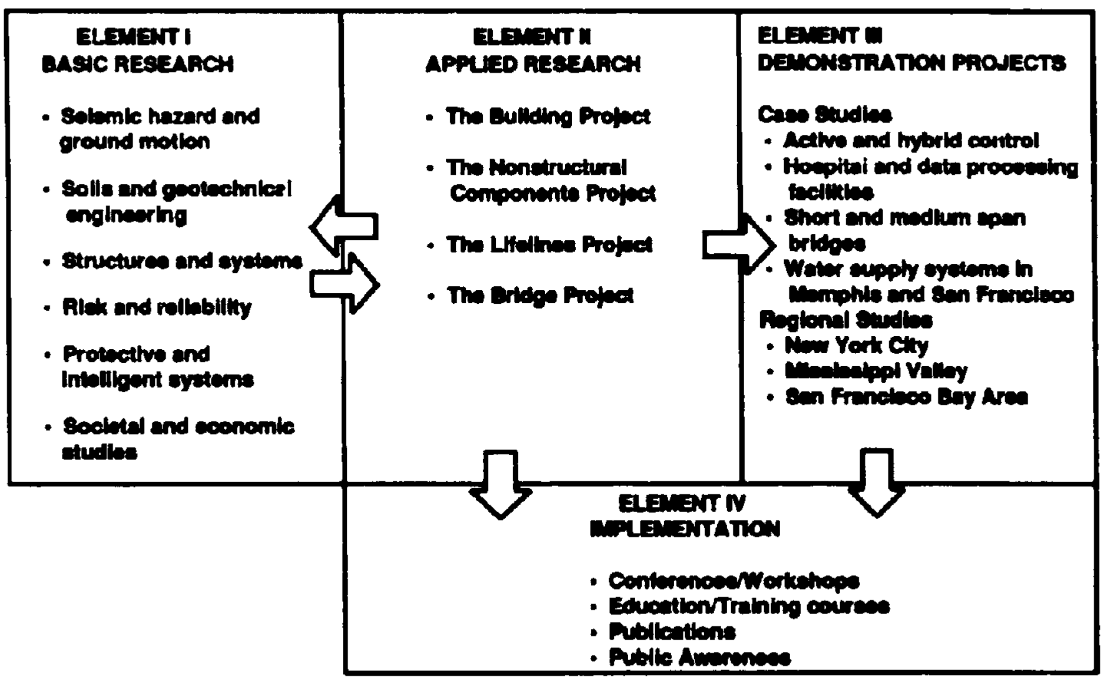
1 Assistant Professor, Department of Civil and Environmental Engineering, University of Central Florida
2 Professor, Department of Civil Engineering, State University of New York at Buffalo
3 Graduate Student, Department of Civil Engineering, State University of New York at Buffalo

NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
State University of New York at Buffalo
Red Jacket Quadrangle, Buffalo, NY 14261

PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established to expand and disseminate knowledge about earthquakes, improve earthquake-resistant design, and implement seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures in the eastern and central United States and lifelines throughout the country that are found in zones of low, moderate, and high seismicity.

NCEER's research and implementation plan in years six through ten (1991-1996) comprises four interlocked elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten. Element III, Demonstration Projects, have been planned to support Applied Research projects, and will be either case studies or regional studies. Element IV, Implementation, will result from activity in the four Applied Research projects, and from Demonstration Projects.



Research in the **Building Project** focuses on the evaluation and retrofit of buildings in regions of moderate seismicity. Emphasis is on lightly reinforced concrete buildings, steel semi-rigid frames, and masonry walls or infills. The research involves small- and medium-scale shake table tests and full-scale component tests at several institutions. In a parallel effort, analytical models and computer programs are being developed to aid in the prediction of the response of these buildings to various types of ground motion.

ABSTRACT

This report summarizes the significant modeling and program enhancements to the computer code, IDARC (see Technical Report NCEER-87-0008) for inelastic damage analysis of reinforced concrete frame-wall structures. The base program is capable of analyzing structures in the inelastic range subjected to combined horizontal and vertical excitations, quasi-static cyclic loading, and incrementally applied static loads.

The distributed flexibility model originally resident in IDARC was based on prismatic members with constant cross-sections and identical properties at both ends of a member. This model has now been extended to include members with tapered cross-sections and the ability to specify different envelope characteristics at each end of the member. In addition, it is possible to prescribe moment releases at either end of a member to model perfect hinge connections.

Two new element types are available: a circular column element with circumferential arrangement of longitudinal reinforcement and spiral hoops; and an inelastic discrete spring element which can be used to model nonlinear flexible connections, or indirectly, the effect of bar pull-out and joint distortions. The trilinear moment-curvature properties at critical sections may be specified in two ways: either directly as user-specified nonsymmetric trilinear envelopes; or by specification of cross-section data, in which case the moment-curvature envelopes are automatically generated by the program using a generalized fiber model, thus replacing the empirical formulations of the previous version.

P-Delta effects are included in the step-by-step analysis, and a single-step correction to control unbalanced forces during event transition (stiffness changes during loading and unloading) is incorporated. In addition to input of transient loads, it is now possible to specify applied force or displacement histories, typical in laboratory testing of components and subassemblages. In this case, the system is assumed to respond quasi-statically without influence of inertia or damping.

The computation of damage indices has been considerably enhanced. Several indicators of damage using energy, stiffness and ductility based representations are included, and the progression of damage as a function of time can be monitored. Numerous input and output enhancements have also been incorporated to make the task of data-input and output-interpretation simple and meaningful.

The program is validated using several available experimental results of dynamic and quasi-static testing of components, frames and model structures. While a certain degree of model tuning may be necessary to match experimental results, it was established that the only essential parameter to be calibrated is the initial stiffness of the structural members which collectively provides a good estimate of the fundamental system period.

Several sample problems are included, along with corresponding IDARC data files. A User Manual for the new version of the program accompanies this report.

ACKNOWLEDGEMENTS

Financial support provided for the development of IDARC by the National Center for Earthquake Engineering Research (which in turn is supported by the National Science Foundation) and the State of New York is gratefully acknowledged. Some of the enhancements included in this report were sponsored by the National Institute of Standards and Technology (NIST), Gaithersburg, Maryland. The authors also wish to express their thanks to Dr. John Gross (NIST) for his contribution in the analysis of the Cypress Viaduct (Case Study #5), and to Ms. Xiaoyi Chang for assisting in the computer runs of the other numerical examples presented in this report.

TABLE OF CONTENTS

LIST OF FIGURES

SECTION 1
INTRODUCTION

The need for computational tools to facilitate inelastic analysis of reinforced concrete structures under transient loads has led to the development of several programs during the last few decades. The most popular of these programs has been DRAIN-2D (Kanaan and Powell, 1973) which has been used successfully in several applications, and has seen only one major enhancement recently (Allahabadi and Powell, 1988). Apart from its original release version, a number of researchers have also adopted the basic DRAIN-2D framework for their respective developments through the incorporation of either new element modules or new hysteretic models. A case in point is program SARCF (Chung et al.,1988; Gomez et al.,1990) which also contained damage modeling features and options for automated damage design. Other programs such as SAKE (Otani, 1974) and unpublished versions of the computer code written to support the Tsukuba tests of the full-scale 7-story building (Wight, 1985) were limited in scope to find as wide an application as DRAIN-2D.

The release of IDARC in 1987 (Park et al.,1987) introduced a number of significant enhancements to conventional modeling schemes, such as those in the DRAIN-2D-based programs, for reinforced concrete structural analysis in the inelastic range. IDARC developments were based primarily on the need to fill a vacuum between experimental research and analytical simulation. While dozens of quasi-static and shaking-table tests were being carried out to study the performance of reinforced concrete components and structures, little progress was being made in incorporating observed aspects of concrete behavior into analytical tools for global structural evaluation. Hence, IDARC was conceived as a platform for reinforced concrete structural analysis in which various aspects of concrete behavior could be modeled, tested and improved upon. Some highlights of the program which make it particularly attractive for modeling of reinforced concrete structures are as follows:

(1) It is well established from laboratory testing that inelasticity in reinforced concrete is not confined to a concentrated point but rather tends to spread into the member. Hence, a distributed flexibility model in which the effects of spread plasticity are

somehow included would represent a more realistic approach to constructing the element stiffness matrix. IDARC provides a basis for including a variety of distributed models.

(2) Another vital aspect in predicting the inelastic behavior of reinforced concrete is modeling the hysteretic force-deformation response. Depending upon the level of axial load, the effects of high shear, the amount and distribution of reinforcement, and numerous other factors, the resultant force-deformation behavior may exhibit vastly different loop patterns. Hence, the need for a versatile force-deformation hysteretic model which can simulate stiffness degradation, strength deterioration and pinching behavior (either bond-slip or crack-closing) is essential. IDARC provides a non-symmetric trilinear envelope with the ability to model all of the above hysteretic characteristics.

(3) The presence of shear walls in most concrete buildings make it necessary to adequately model the behavior of these panels and their interaction with moment-resisting frames. In particular, the behavior of walls in shear is considerably different from their response in flexure. IDARC provides a means to model flexure and shear independently. Consequently, the effects of shear yielding or impending shear failure can be predicted.

(4) A great deal of effort in typical program input goes toward the preparation of primary moment-curvature envelopes. IDARC provides a module to carry out this preprocessing task by building all of the required envelopes from basic cross-section data that can be read directly from engineering drawings of building plans.

(5) Finally, IDARC introduced the idea of including a qualitative assessment of the inelastic dynamic analysis through damage indices. These indices are representative of the damage distribution throughout the system in a physical sense. While the assessed damage magnitudes cannot currently be related to damage limit states, such as repairable, irreparable or collapse, there exists the possibility of calibrating the model, through comparison with available dynamic experiments or damaged building data, using IDARC.

1.1 Organization of Report

This report is organized into three parts. The first part, covering Sections 2-4, presents various aspects of the IDARC program highlighting specific enhancements to the code from the earlier release version, and accompanying modeling details. Improvements and added features in the member-by-member modeling of structures is detailed in Section 2. Evaluation and stipulation of envelope characteristics at member cross-sections is considered a vital part of the modeling process, and is presented in Section 3. Section 4 outlines the numerical processes involved in performing the step-by-step inelastic analysis. This section also describes the task of post-processing, in which response quantities are expressed as damage indices.

The second part, presented in Section 5, is devoted exclusively to program validation. Several available experimental results of dynamic and quasi-static testing are used to demonstrate the effectiveness of the program to reproduce real-world results. The performance of IDARC in simulating experiments is compared with two existing tools for nonlinear seismic analysis: DRAIN2D and SARCF-II.

The final part comprises the User Manual for the program. Several user input guidelines are provided and every attempt is made to show clearly the meaning and effect of critical input parameters. The sample problems are accompanied by input data files.

SECTION 2
MODELING OF STRUCTURAL SYSTEM

IDARC is a computer program for two-dimensional analysis of 3D building systems in which a set of frames parallel to the loading direction are inter-connected by transverse elements to permit flexural-torsional coupling. The structural model is capable of integrating ductile moment-resisting frames with shear wall models and out-of-plane elements thereby enabling a more realistic modeling of the overall structural system.

A reinforced concrete building is idealized as a series of plane frames linked together by transverse beams. Each frame must lie in the same vertical plane. Consequently, a building is modeled using the following element types:

(i) Beam-Column Elements
(ii) Shear Walls
(iii) Inelastic Axial (or Edge Column) Elements
(iv) Transverse Beams
(v) Discrete Spring Elements

A discretized section of a building using all of the above element types (except the discrete springs) is shown in Figure 2.1. Beams and columns are modeled as inelastic single component elements with distributed flexibility. Shear and flexure are combined at the element flexibility level (Kunnath et al., 1990). Shear walls are modeled using a combination of shear and flexure springs connected in series. This enables the modeling of shear cracking and yielding. *Since shear wall elements, as modeled in IDARC, can be represented as line elements, it is possible to use them for modeling short columns or other vertical elements in which inelastic shear behavior needs to be modeled independently.* In addition, edge columns of shear walls or any other axial element can be modeled separately using inelastic axial springs. Transverse elements which contribute to the stiffness of the building are assumed to have an effect on both the vertical and rotational deformation of the shear walls or main beams to which they are connected and are modeled using elastic linear and rotational springs. Discrete inelastic springs may also be attached anywhere in the structure to represent local behavior that cannot otherwise be incorporated into the structural model.

2.1 Summary of IDARC Element Library

Details of the element types that currently exist in the IDARC library can be found in the earlier manual (Park et al., 1987). A brief summary is presented here for completeness.

Beam-Columns: Main beam-column elements form a vertical plane in the axis of loading. They are modeled as simple flexural springs in which shear-deformation effects are also included. Axial deformation effects are included in columns but ignored in beams. Interaction between bending moment and axial load is presently not considered directly in the step-by-step analysis, but the effect of axial load in the moment capacity computations is included.

Shear Walls: Walls may be modeled in two ways: (1) With reference to Figure A-10, the entire wall, including the edge column, may be modeled as a single element ***in which case it is not necessary to input the edge column data in Section H***; (2) The boundary columns may be modeled separately as axial elements in which case the wall input in Section G1 should contain the central section ONLY, and the boundary columns are modeled as edge columns in Section H. **Note that the input of boundary edge columns should not be duplicated.** The ability to treat each wall as an equivalent column with inelastic axial springs at the edges allows for the bending deformation of the wall element to be caused by the vertical movements of the boundary columns. The motivation for such a modeling scheme is based on experimental studies conducted during the U.S.-Japan Research Program (Wight, 1985).

Inelastic Axial Elements: Studies on the behavior of columns subjected to axial load reversals are limited, hence no attempt was made to develop a new model for the inelastic response of the axial spring of edge columns tied to shear walls. Instead, the model developed as part of the U.S.-Japan Research Program was implemented without modification. The details of the model can be found in Kabeyasawa et al.(1983).

Transverse Beams: To incorporate the effects of transverse elements on the in-plane response of the main frames, each transverse T-beam is modeled using elastic springs with one vertical and one rotational (torsional) degree-of-freedom as shown in Figure 2.1. Transverse elements are basically of two types: beams which connect to shear walls; and beams connected to the main beams in the direction of loading. Direct stiffness contributions arising from these springs are simply added to corresponding terms in the overall structure stiffness matrix. The purpose of modeling transverse beams in this fashion is to account for their restraining action due to two effects, should they become significant: (a) the axial movements of vertical elements, especially edge columns in shear walls; (b) flexural-torsional coupling with main elements.

Details of the inelastic discrete spring element is described in the next section on Program Enhancements.

2.2 Program Enhancements

The distributed flexibility model originally resident in IDARC was based on prismatic members with constant cross-sections. This model is extended in this version to include (1) members with tapered cross-sections; (2) members in which the cross-section properties are different at each end; and (3) members requiring specification of different hysteretic properties at either end.

One of the limitations of the earlier IDARC release was its inability to handle internal member hinges, which though uncommon in buildings, is encountered frequently in laboratory testing of beam-slab-column subassemblages. The present version provides this capability. Also, as a general case of the preceding option, a discrete spring element is implemented wherein a range of stiffness and inelastic behavior patterns may be specified.

2.2.1 General Distributed Flexibility Model

The moment distribution along a frame member under the action of lateral loads, such as those arising from seismic forces, is linear, as shown in Figure 2.2. The presence of gravity loads will alter the distribution somewhat, but the linear distribution is valid for lateral load moments which far exceed the gravity load moments. *If gravity load moments are significant, then it is imperative to subdivide the beam into an adequate number of sub-elements.* When the member experiences inelastic deformations, cracks tend to spread from the joint interface resulting in a curvature distribution as shown in Figure 2.2. In the IDARC flexibility formulation, both a linear and nonlinear variation of curvature is assumed, depending upon whether the member cross-section is constant or tapered. This assumption is more realistic than the figure suggests, since the additional inelastic curvature due to yield penetration in the joint and possible diagonal tension cracking are not shown.

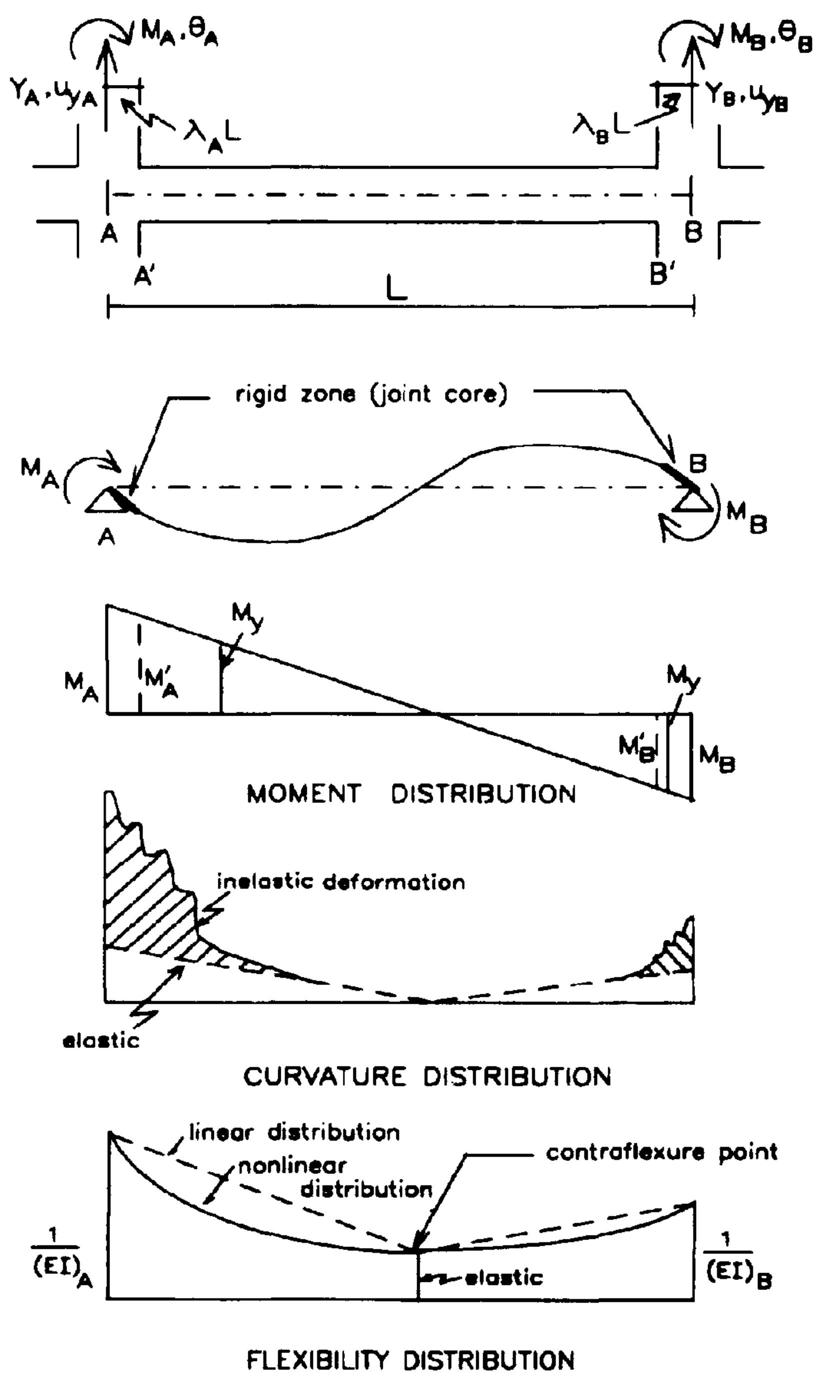


Figure 2.2 Spread Plasticity Component Model

Once the flexibility distribution is established, the 2x2 flexibility matrix is derived from virtual work principles. Flexibility coefficients are obtained from the following relationship:

$$f_{ij} = \int_0^L m_i(x) m_j(x) \frac{1}{EI(x)} dx \tag{2.1}$$

The integration can be carried out in closed form for the assumed linear variation in curvature for two possible cases: (i) members bent in double curvature with a contraflexure point within the member; and (ii) members in single curvature without any contraflexure point.

A typical inelastic single component element model is shown in Figure 2.2. Two degrees-of-freedom are considered per node. For columns and shear walls, an additional axial degree-of-freedom is considered at each node. For members with constant cross-section, the flexibility factor, *1/EI*, is assumed to have a linear variation along the member between the end sections and the point of contraflexure. Flexibility coefficients for this case are reported in Kunnath et al. (1992).

A procedure is now described to consider a more general case in which the ratio of the flexural rigidity term *(1/EI)* may be assumed to vary nonlinearly as shown in Figure 2.2. This will require a complete numerical integration along the member to determine the flexibility coefficients. The sequence of operations to determine the 2x2 flexibility matrix is as follows:

(1) From the end moment of the member, determine the contraflexure point.

(2) If the contraflexure point lies outside the element, subdivide the member into 2*(NSP-1) equal segments. (where NSP is an integer variable used to specify the number of segments to be used in the numerical integration)

(3) If the contraflexure point lies within the element, determine the zero crossing of the moment diagram. Divide each part of the member (about the contraflexure point) into (NSP-1) equal segments.

(4) An explicit integration scheme must now be employed to determine the integral of the (1/EI) diagram about each set of NSP points.

Note: *Two aspects must be noted in the flexibility formulation: (1) If both ends of the section yield and the contraflexure point is located outside the element, it is obvious that the entire member has plastified, hence the computation in Step (2) must be modified accordingly; (2) If the moment distribution in a member having double curvature causes one of the shear spans to be very small (say, less than 10% of the member length), then the elastic zone must be extended further than that computed in Step (3) based on the moment distribution in the adjacent shear span.*

The flexibility coefficients are obtained from the following integrals:

$$f_{11} = \int_0^L \frac{1}{EI_x}\left(1 - 2\frac{x}{L} + \frac{x^2}{L^2}\right)dx \tag{2.2}$$

$$f_{12} = -f_{21} = \int_0^L \frac{1}{EI_x}\left(-\frac{x}{L} + \frac{x^2}{L^2}\right)dx \tag{2.3}$$

$$f_{22} = \int_0^L \frac{1}{EI_x}\left(\frac{x^2}{L^2}\right)dx \tag{2.4}$$

where:

$$EI_x = EI_A + \frac{x}{L}(EI_B - EI_A) \tag{2.5}$$

The subsequent steps in constructing the stiffness matrix follows the procedure outlined in Kunnath et al. (1992).

2.2.2 Modeling of Perfect Hinge

A perfect member hinge is modelled by setting the hinge moment to zero and condensing out the corresponding degree-of-freedom. With reference to Figure 2.3, the relationship between the moments at the center of the joint and the face of the member is given by:

$$M_A = \left[\frac{1}{1-\lambda_A}\right] M'_A \tag{2.6}$$

The element stiffness equation relating moment and rotation is:

$$\{M_A\} = k_s\{\theta_A\} \tag{2.7}$$

where:

$$k_s = k_{11} - \frac{(k_{12})^2}{k_{22}} \tag{2.8}$$

where k_{ij} are the coefficients of the inverted flexibility matrix. Finally, from equilibrium of forces, the 3 x 3 element stiffness matrix is constructed as follows:

$$\begin{pmatrix} Y_A \\ M_A \\ Y_B \end{pmatrix} = \{R_B\}\left[\frac{1}{1-\lambda_A}\right] k_s \left[\frac{1}{1-\lambda_A}\right]\{R_B\}^T \begin{pmatrix} u_{YA} \\ \theta_A \\ u_{YB} \end{pmatrix} \tag{2.9}$$

where:

$$\{R_B\} = \{\ -1/L \quad 1 \quad 1/L\ \}^T \tag{2.10}$$

2.2.3 Modeling of Discrete Inelastic Spring

A discrete spring with user-specified moment-rotation characteristics may be located at any node in the structure. In the IDARC nodal convention, this refers to an L-I-J position. Figure 2.4 shows four elements framing into a joint with the possible maximum of three springs. In general, more than one spring may be specified at the same location, though the total number of locations at which springs may be specified at a particular joint must be one less than the number of elements framing into it.

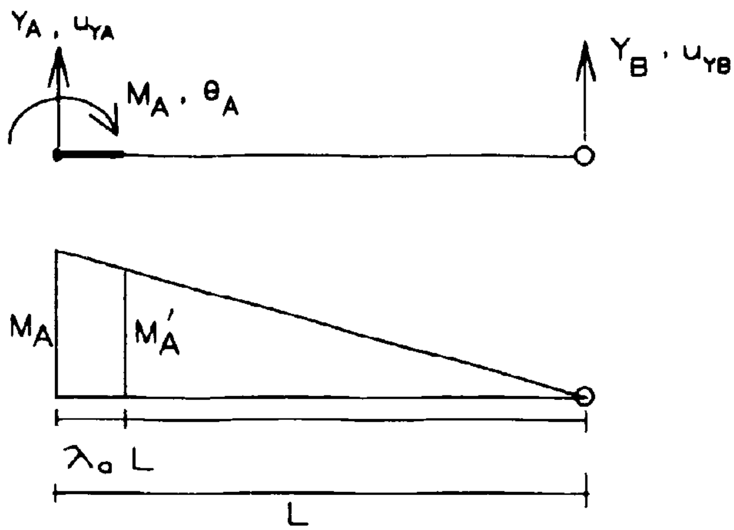


Figure 2.3 Element Hinge Modeling

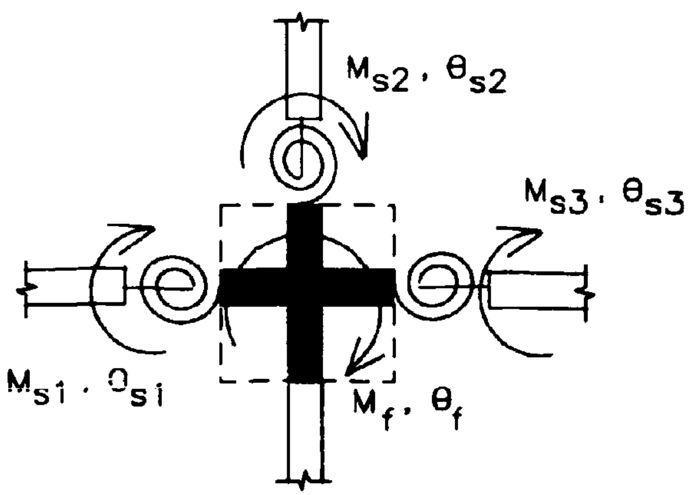


Figure 2.4 Modeling of Discrete Spring

The characteristics of the spring can be specified as a nonsymmetric trilinear envelope with degrading parameters. Alternatively, the spring stiffness may be specified either as a relatively small quantity or an infinitely large value to simulate a hinge or rigid connection respectively. With reference to Figure 2.4, the spring stiffness is incorporated into the overall structural stiffness matrix as follows:

$$\begin{pmatrix} M_{si} \\ M_f \end{pmatrix} = k_{\theta i} \begin{pmatrix} 1 & -1 \\ -1 & 1 \end{pmatrix} \begin{pmatrix} \theta_{si} \\ \theta_f \end{pmatrix} \tag{2.11}$$

where M_{si} and M_f refer to the spring moment and the fixed joint moment respectively, θ_{si} and θ_f are the corresponding rotations, and $k_{\theta i}$ is the current tangent stiffness of the spring element. Spring rotations are expressed as a function of the joint rotation.

The introduction of discrete springs in this manner results in the possibility of having upto 4 rotational degrees-of-freedom per node. Modeling of joint distortions and bar pull-out is thus accommodated, but not directly implemented. Modeling of joint distortions, for example, will still require the incorporation of a new element module which relates joint shear to the independent rotational degrees-of-freedom. The modeling of bar pull-out can be accomplished in the present framework by apportioning the total element stiffness between the spring and the element itself. Spring yielding is then initiated at the impending bar pull-out strength.

SECTION 3
COMPONENT PROPERTY IDENTIFICATION

The specification of moment-curvature envelopes of member cross-sections forms an essential and important part of the analysis. The earlier IDARC version incorporated an identification module that computed the necessary envelopes from cross-section data. However, most of the expressions used to compute the moment-curvature envelopes were based in part on empirical models derived from statistical analysis of experimental data. Consequently, these models were not suitable for a variety of cross-sections with non-standard details such as non-ductile frames.

The present IDARC version replaces the empirical identification module with a mechanical one in which all cross-section properties are computed from a fiber model analysis using concrete and reinforcement stress-strain properties. Alternatively, IDARC now also provides an option for users to input their own cross-section properties directly. Details of the fiber model computations are described in the next section.

3.1 Moment-Curvature Envelopes

Figure 3.1 shows a typical rectangular section subjected to a combination of an axial load and a moment. The procedure outlined here is general and applicable to all types of cross-sections: T-beams, shear-wall sections and circular column sections. Some simplifying assumptions are made in the analysis and are summarized here:

- plane sections are assumed to remain plane after bending of the cross-section has taken place;
- the tensile strength in concrete is ignored beyond the tensile cracking capacity;
- the effects of bond-slip between the reinforcement and concrete is not accounted;
- the difference in properties between confined core and concrete cover is ignored;
- the stress-strain properties of concrete and steel are modeled as shown in Figures A-3 and A-4 (see User Guide in Appendix A).

The procedure used is adopted from Mander (1984). The moment-curvature analysis is carried out on the cross-section by dividing the concrete area into a number of strips or fibers. Steel areas and their respective locations are identified separately. With reference to Figure 3-1, the strain at any section is given by:

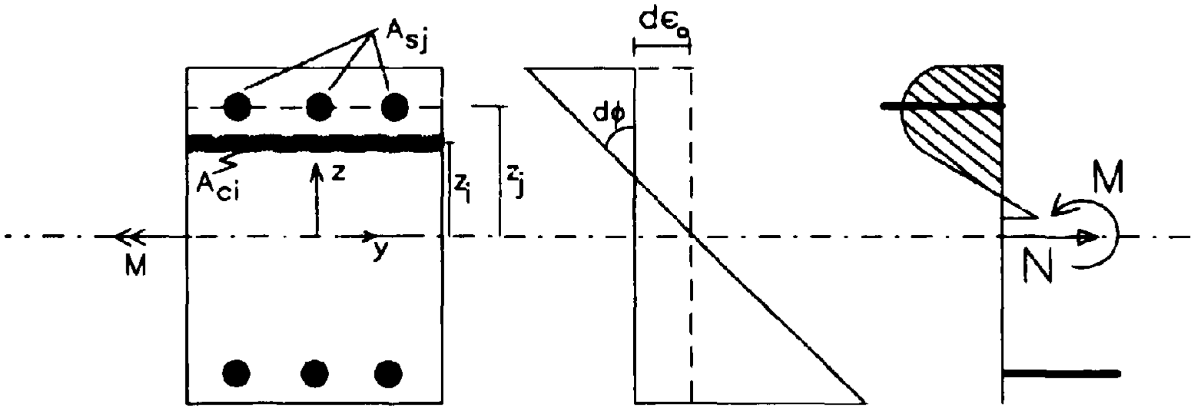


Figure 3.1 Section Detail for Fiber Model Analysis

$$\varepsilon(z) = d\varepsilon_o \quad : d\phi \tag{3.1}$$

where $d\varepsilon_o$ is the centroidal strain, z is the distance from the reference axis, and ϕ is the curvature of the cross-section. The resulting axial load and moment on the cross-section can be computed from:

$$N = \int E \; d\varepsilon \; dA \tag{3.2}$$

$$M = \int E \; d\varepsilon \; z \; dA \tag{3.3}$$

where N is the axial force, E is the elastic modulus of the corresponding concrete or steel fiber, $d\varepsilon$ is the strain in the fiber, and z_i is the distance to the fiber from the reference axis.

Substituting Equation (3.1) into Equation (3.2) and replacing the integral by a finite summation over the discretized fibers, the following expression is obtained:

$$\begin{aligned} \Delta N = & \left(\sum_{i=1}^{NCC} f_{ci} A_{ci} + \sum_{j=1}^{NSS} f_{sj} A_{sj} \right) d\varepsilon \; + \\ & \left(\sum_{i=1}^{NCC} f_{ci} A_{ci} z_i + \sum_{j=1}^{NSS} f_{sj} A_{sj} z_j \right) d\phi \end{aligned} \tag{3.4}$$

where NCC, NSS are the number of concrete strips and steel areas considered in the section respectively, f_{ci} , f_{si} are the stress in the concrete and steel sections respectively, and A_{ci} , A_{si} are the areas of the concrete strip and steel respectively. The complete procedure for developing the moment-curvature envelope is as follows:

1. Apply a small incremental curvature to the previous value.

$$d\phi_{i+1} = d\phi_i + \Delta\phi \tag{3.5}$$

2. The change in the centroidal strain to provide equilibrium is determined from Equation (3.4) due to the out-of-balance axial load (in the first step, this will be the total axial load, and in subsequent steps, the unbalanced axial force), as follows:

$$\Delta\varepsilon_o = (\Delta N - E_s \Delta\phi) \; / \; E_a \tag{3.6}$$

where:

$$E_a = \left(\sum_{i=1}^{NCC} f_{ci} \; A_{ci} + \sum_{j=1}^{NSS} f_{sj} \; A_{sj} \right) \tag{3.7a}$$

$$E_x = \left(\sum_{i=1}^{NCC} f_{ci} \; A_{ci} \; z_i + \sum_{j=1}^{NSS} f_{sj} \; A_{sj} \; z_j \right) \tag{3.7b}$$

3. The incremental centroidal strain computed above is added to the previous value of the centroidal strain, and the revised strain profile of the section is established from Equation (3.1).

4. The new axial load and moment are then computed from discretized forms of the Equations (3.2) and (3.3). If the computed axial load is close to the applied axial load (specified by some tolerance limit), the established strain profile is correct, and a new increment of curvature is applied. If any unbalanced axial load exists, return to Step 2 after setting the curvature increment to zero.

The above procedure works very well with very few iterations required to obtain convergence. IDARC uses this procedure to set up moment-curvature envelopes for columns (rectangular or circular), beams (rectangular or T-sections) and shear walls with or without edge columns. Shear walls may be irregular and include such sections as U- and L-shaped core walls.

Effect of hoop spacing on column capacity of circular sections: The effect of hoop spacing on the moment-curvature envelope is introduced in the following manner. It is assumed that the capacity of the column remains unchanged after the concrete cover has spalled. Hence,

$$0.85 f_c' A_g = f_{cc}' A_{cc} \tag{3.8}$$

where f_{cc}' is the confined compressive strength, A_{cc} is the area of core concrete, and A_g is the gross concrete area. An expression relating confined to unconfined strength of concrete is given by Park and Paulay (1975) and is based on the confining stress relation of Richart et al (1928):

$$f_{cc}' = f_c' + 2.05 \rho_s f_y \tag{3.9}$$

where ρ_s is the volumetric ratio of confinement steel to core concrete, given by:

$$\rho_s = \frac{A_h \pi d_c}{s A_{cc}} \tag{3.10}$$

where A_h is the cross-sectional area of the hoop steel, A_{cc} is the diameter of the concrete core, and s is the spacing of hoops. The modified compressive stress of concrete is finally obtained from substitution of Equation (3.9) into Equation (3.8):

$$f_{cm}' = \frac{(f_c' + 2.05\rho_s f_y)\ A_{cc}}{0.85A_g} \tag{3.11}$$

3.2 Ultimate Deformation Capacity

The ultimate deformation capacity is expressed through the ultimate curvature of the section as determined from the fiber model analysis of the cross-section. The incremental curvature that is applied to the section as described in Equation (3.5) is continued until one of the following conditions is reached:

- the specified ultimate compressive strain in the extreme concrete fiber is reached; or
- the specified ultimate strength of one of the reinforcement bars is attained.

The attained curvature of the section when either of the two above conditions is reached is recorded as the ultimate curvature. This parameter forms an important part of the damage analysis, and hence, the specification of the ultimate strain of concrete must be done with reasonable certainty.

The only factor considered to influence the ultimate deformation capacity of the section is the degree of confinement. Since confinement does not significantly effect the maximum compressive stress, the present formulation only considers the effect of confinement on the downward slope of the concrete stress-strain curve. With reference to Figure A-3 (see Appendix), the factor ZF defines the shape of the descending branch. The expression developed by Kent and Park (1971) is used:

$$ZF = \frac{0.5}{\varepsilon_{50u} + \varepsilon_{50h} - \varepsilon_o} \tag{3.12}$$

where:

$$\varepsilon_{50u} = \frac{3.0 + \varepsilon_o\ f_c'}{f_c' - 1000.0} \tag{3.13}$$

$$\varepsilon_{50h} = 0.75 \ \rho_s \sqrt{\frac{\bar{b}}{s_h}} \tag{3.14}$$

in which the concrete strength is prescribed in *psi*, ρ_s is the volumetric ratio of confinement steel to core concrete, $\bar{b}$ is the width of the confined core, and s_h is the spacing of hoops. The effect of introducing this parameter to define the descending branch of the concrete stress-strain curve is to provide additional ductility to well-confined columns. *Improved formulations for stress-strain behavior of confined concrete can be found in a recent publication by Paulay and Priestley (1992).*

3.3 Hysteretic Modeling

The hysteretic model used for the inelastic analysis is a general-purpose versatile model which uses *four* hysteretic control parameters in conjunction with a non-symmetric trilinear curve to establish the rules under which inelastic loading reversals take place. The three main characteristics represented in the model are: *stiffness degradation; strength deterioration,* and *crack-closure/ bond-slip* or *pinching.* The control parameters can be combined in various ways to achieve a range of hysteretic patterns typical of reinforced concrete sections.

Stiffness Degrading Parameter: This input parameter has the same meaning as the previous IDARC version, and defines the amount of stiffness decay as an indirect function of the attained ductility. As shown in Figure 3.2, all unloading paths on the primary curve target a common point. This introduces the effect of increased stiffness degradation at larger deformation levels. The parameter, *HC,* can be obtained quite effectively from a review of experimental data of components that are typical to the structure being analyzed. The typical range for *HC,* based on observations of test data, lies between 1.5 and 3.0.

Strength Deteriorating Parameters: These parameters have been redefined in the present version. The loss in strength, as indicated in Figure 3.2, is obtained from the following expression:

$$F_{new} = F_{max}(1.0 - HBE*\bar{E} - HBD*\mu_c) \tag{3.15}$$

where *HBE* and *HBD* are user-input control parameters that determine the amount of strength decay as a function of dissipated energy and ductility, respectively:

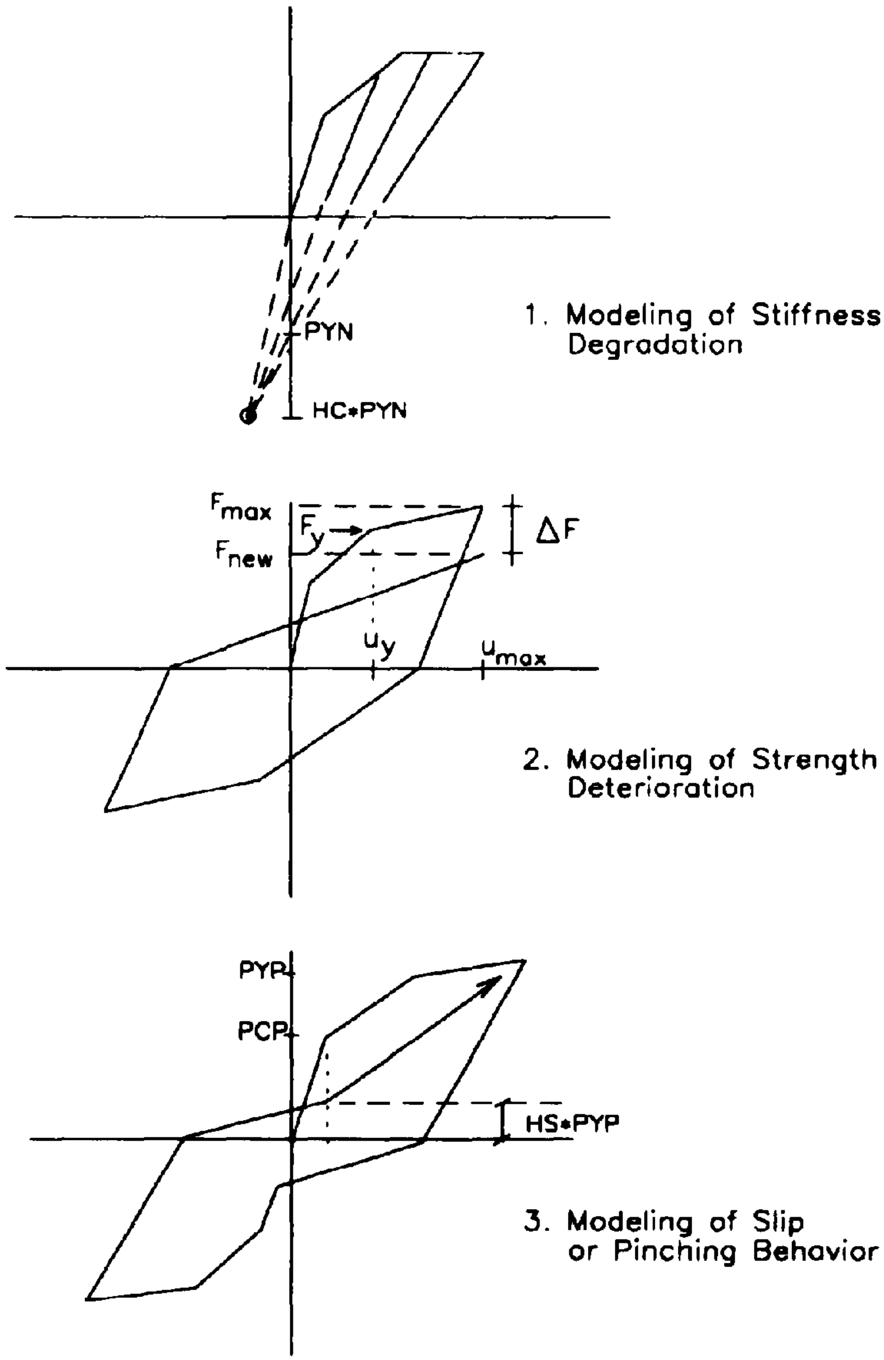


Figure 3.2 Hysteretic Model Control Parameters

$$\overline{E} = \frac{A_T}{M_y \phi_u} \qquad (3.16)$$

$$\mu_c = \frac{\phi_{max}}{\phi_y} \qquad (3.17)$$

where A_T is the total area under the $M-\phi$ loops, M_y is the yield moment, ϕ_y is the yield curvature and ϕ_{max} is the maximum attained curvature. *The advantage of this formulation over the previous IDARC model is the fact that strength decay can now be controlled as a function of either ductility or energy or both.*

Typical values to be used, when access to test data is not readily available, are as follows: 0.1 for *HBE*, 0.0 for *HBD*. For substantially increased degradation, either or both parameters may be increased upto a maximum of approximately 0.5 to obtain a conservative (highly degraded) response.

Slip or Pinching Control Parameter: This parameter remains unchanged from the previous IDARC version. Unloading paths, upon crossing the zero moment axis, aim a lower target point specified by *(HS*PYP) or (HS*PYN)* and retain this smaller stiffness until the path crosses the cracking deformation, as shown in Figure 3.2. Upon crossing the cracking deformation point, the loading paths aim the previous maximum point, unless strength deterioration is also specified, in which case a lower target point is used. A value of *HS=0.5* can be used to simulate typical effects of crack opening and closing.

The modeling of the independent parameters, as shown in Figure 3.2, capture the effects of each parameter independent of the others. When combined in an appropriate manner, these control parameters are capable of reproducing a variety of hysteretic shapes that are typical of most RC sections.

SECTION 4
ANALYSIS MODULES

The inelastic response analysis is carried out on the assembled element stiffness matrices in conjunction with the force-deformation hysteretic model. The following options are now available:

(a) **nonlinear static analysis** for computation of initial stress states under dead and live loads;

(b) **failure/collapse mode analysis** under monotonic lateral loading;

(c) **quasi-static cyclic analysis** under load or displacement control;

(d) incremental **dynamic response analysis** under horizontal and vertical seismic excitations;

(e) a comprehensive **damage analysis.**

The sequence and interaction of the various modules is shown in the flow chart in Figure 4.1. In all cases, the final equilibrium equation to be solved assumes the following form:

$$[K]\{u\} = \{F\} \tag{4.1}$$

where [K] is the overall stiffness matrix, {u} is the vector of unknown nodal displacements, and {F} is the vector of applied equivalent forces on the system. Since the stiffness matrix is symmetric and banded, a compact scheme is used to store the resultant matrix in which the main diagonal is offset to the first column and only the remaining half band width is saved.

Stiffness matrices are stored at the element level. These matrices are then assembled onto the global stiffness matrix. The load vector corresponding to the right-hand side of Eq.(4.1) is established, depending upon the type of analysis being performed (static, monotonic, cyclic, or dynamic). Following the solution of the equilibrium equation, the inelastic moments at the ends of each element are computed from the recovered member nodal displacements. Element sub-matrices are stored in a manner to enable direct computation of inelastic end moments at the face of the element across the rigid panel zone. The updating of stiffness matrices is carried out only in the event of a stiffness change.

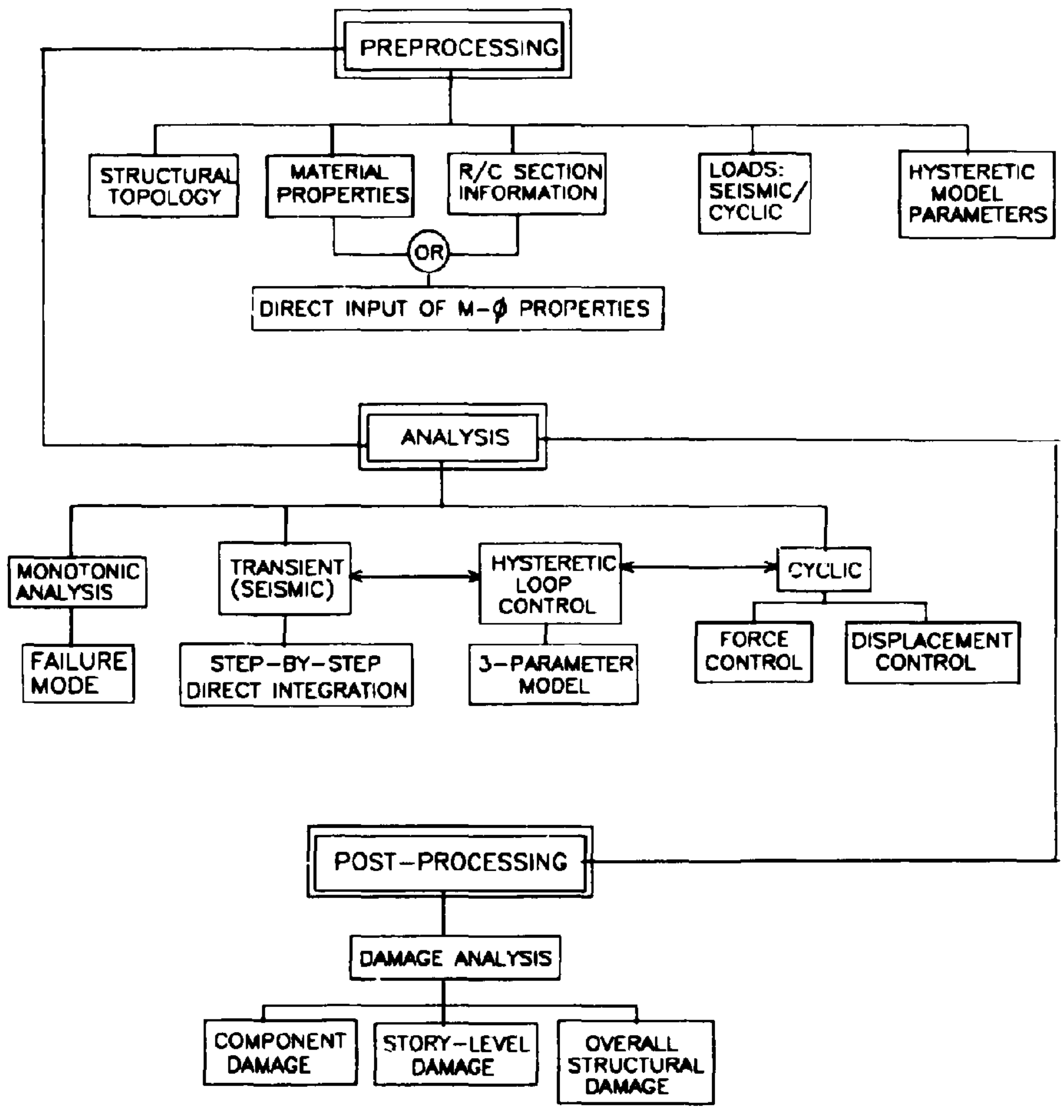


Figure 4.1 Program Organization

A single step force-equilibrium correction procedure was incorporated into the cyclic and seismic analysis routines. Finally, a simple equivalent force method is used to account for P-delta effects due to inter-story drift. Details of these procedures, and the general features of the analysis modules are presented in the following sections.

4.1 Incremental Nonlinear Static Analysis

The analysis phase begins with the evaluation of the initial stress states of members under equivalent dead and live loads that exist in the structure prior to application of cyclic or earthquake loads. The same initial state is assumed prior to the failure mode analysis under monotonically increasing lateral load. For the static analysis option, loads can be specified in two ways: (a) uniformly distributed loads; and (b) nodal forces and/or moments. If a uniform load is specified, equivalent nodal values with fixed-end forces are computed.

Due to the assumed linear moment distribution in the flexibility matrix, stress levels in the members due to initial loads must be relatively small so that the assumed moment distribution pattern is not seriously violated. Otherwise, beams must be subdivided into sub-elements so that the dead load moment distribution is captured effectively. An example of such an application is shown in Section 5 (see case study #5).

The prescribed initial static loads can be applied in small increments. If the system is expected to remain elastic, the entire load can be applied in a single step. Care must be taken to sub-divide the load into reasonably small increments so as to trace the nonlinear behavior accurately. A simple technique to assure convergence is to keep increasing the number of load steps (parameter *JSTP* in the User Guide) till consistent results are obtained. *Note that this module may be used independently to carry out nonlinear static and monotonic analyses.*

4.2 Collapse Mode Analysis

A collapse mode analysis is a simple and efficient technique to predict seismic response behavior prior to a full dynamic analysis. The method provides a means to assess adequacy of strength, determine potential ductility capacity and establish sequence of component yielding. The monotonic analysis involves an incremental solution procedure whereby the structure is loaded laterally under an inverted triangular load. The force vector corresponding to the lateral floor degree-of-freedom is computed as follows:

$$F(i) = \overline{w}_b \frac{\Sigma w(i)}{\Sigma w(i) h(i)} w(i) h(i) \tag{4.2}$$

where w, h and $\overline{w}_b$ are the weight, height and factored base shear estimate and the subscript i refers to the story level under consideration. The lateral load distribution, as computed above, is then applied to the structure in small increments, as a function of building weight. The stress state of each member is evaluated at the end of each step of load application. Analysis proceeds till the deflection of the top of the structure exceeds 2% of the total building height.

4.3 Nonlinear Quasistatic Cyclic Analysis

A common application in laboratory testing of R/C components and sub-assemblages is reversed cyclic loading using force or deformation control. IDARC provides these options with the following features: (a) specified force history at one or more story levels; and (b) specified displacement history at one or more story levels. In both cases, the program will interpolate linearly between user-specified points for a more accurate analysis. The cyclic analysis routine is identical to the transient analysis module with the exception that inertia and damping terms are not included in the computation of the restoring force vector.

4.4 Incremental Dynamic Analysis under Earthquake Loads

The incremental solution of the assembled system of equations involves the following dynamic equation of equilibrium:

$$[M]\{\Delta\ddot{u}\} + [C]\{\Delta\dot{u}\} + \{R(u_t)\} = \{\Delta F(t)\} \tag{4.3}$$

in which:

$[M]$ is the lumped mass matrix

$[C]$ is the viscous damping matrix

$\{R(u_t)\}$ is the restoring force vector at the start of the time step

u is the relative displacment

$\{\Delta F(t)\}$ is the effective incremental load vector

The solution of Eq.(4.3) is accomplished by a direct step-by-step integration procedure using the Newmark's β method. Assuming linear acceleration:

$$\{\dot{u}\}_{t+\Delta t} = \{\dot{u}\}_t + \Delta t[(1-\delta)\{\ddot{u}\}_t + \delta\{\ddot{u}\}_{t+\Delta t}] \qquad (4.4)$$

$$\{u\}_{t+\Delta t} = \{u\}_t + \Delta t\{\dot{u}\}_t + (\Delta t)^2[(1/2-\beta)\{\ddot{u}\}_t + \beta\{\ddot{u}\}_{t+\Delta t}] \qquad (4.5)$$

Newmark (1959) proposed an unconditionally stable algorithm with $\delta = 1/2$ and $\beta = 1/4$ which reduces the above scheme to a constant-average-acceleration method. Substitution of these coefficients and rearranging of Eqs.(4.3)-(4.5) yield the following expressions for incremental velocity and acceleration:

$$\{\Delta\dot{u}\}_{t+\Delta t} = \frac{\Delta t}{2}\{\ddot{u}\}_t + \frac{2}{\Delta t}\{\Delta u\}_{t+\Delta t} - 2\{\dot{u}\}_t - \Delta t\{\ddot{u}\}_t \qquad (4.6)$$

$$\{\Delta\ddot{u}\}_{t+\Delta t} = \frac{4}{(\Delta t)^2}\{\Delta u\}_{t+\Delta t} - \frac{4}{\Delta t}\{\dot{u}\}_t - 2\{\ddot{u}\}_t \qquad (4.7)$$

Substituting the above expressions into the dynamic equation of equilibrium (Eq.4.3), it is possible to solve for the incremental displacements at the current time step:

$$\{\Delta u\}_{t+\Delta t} = [K^*]\{\Delta F^*\}_{t+\Delta t} \qquad (4.8)$$

where K^* and ΔF^* are the equivalent dynamic stiffness and load vector given by:

$$[K^*] = \frac{4}{\Delta t^2}[M] + \frac{2}{\Delta t}[C] + [K] \qquad (4.9)$$

$$\{\Delta F^*\} = \{\Delta F\}_{t+\Delta t} + (4/\Delta t[M] + 2[C])\{\dot{u}\}_t + 2[M]\{\ddot{u}\}_t \qquad (4.10)$$

Once the displacement at time $t + \Delta t$ is known, it is possible to compute the corresponding velocities and accelerations by direct substitution in Eqs.(4.6)-(4.7).

Equilibrium correction: The solution is performed incrementally assuming that the properties of the structure do not change during the time step of analysis. However, since the stiffness of some element is likely to change state during some calculation step, the new configuration may not satisfy equilibrium. A compensation procedure is adopted to minimize this error by applying a one-step unbalanced force correction.

At the end of some given time step, t_{i+1}, the computed restoring force using the current stiffness k_1 recorded at time t_i may lead to an unbalanced force as shown in Figure 4.2 :

$$\{\Delta R\} = \{R\}_{i+1} - \{R\}_i \tag{4.11}$$

where R_{i+1} is computed using the current stiffness instead of accounting for the event transition to a new stiffness k_2 . This corrective force is then applied at the next time step of analysis. The unbalanced forces are computed when moments, shears and stiffnesses are being updated in the hysteretic model. Such a procedure was first adopted in DRAIN2D [Kannan and Powell, 1973] since the total cost of performing an iterative nonlinear analysis would become prohibitive, especially for large building systems.

It must be pointed out that this technique is not physically accurate, since adding the unbalanced forces at the next time step has the effect of modifying the input load history and thereby altering the overall response. Such a procedure generally works well for single degree-of-freedom systems and may or may not improve numerical drifts associated with unbalanced forces in multi-degree-of-freedom systems. ***Hence this corrective technique is provided as a user option in the IDARC program.***

4.5 Analysis of P-Delta Effects

The additional overturning moments generated by relative inter-story drift are generally referred to as P-delta effects. It arises essentially due to gravity loads and is usually taken into consideration by evaluating axial forces in the vertical elements and computing a geometric stiffness matrix which is added to the element stiffness matrix.

In the present IDARC version, P-delta effects are represented by equivalent lateral forces, equal in magnitude to the overturning moment caused by eccentric gravity forces due to inter-story drift (Wilson and Habibullah, 1987). Consider a typical vertical element between two story levels shown in Figure 4.3. Taking moments about the lower story level, the following equilibrium equation is obtained:

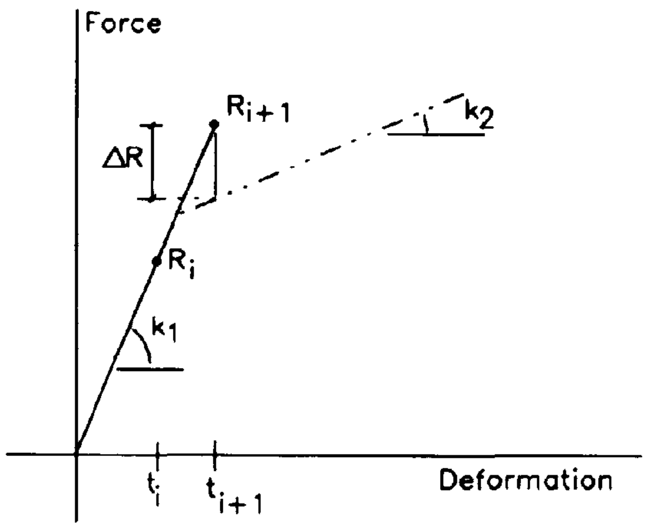


Figure 4.2 Unbalanced Force Correction

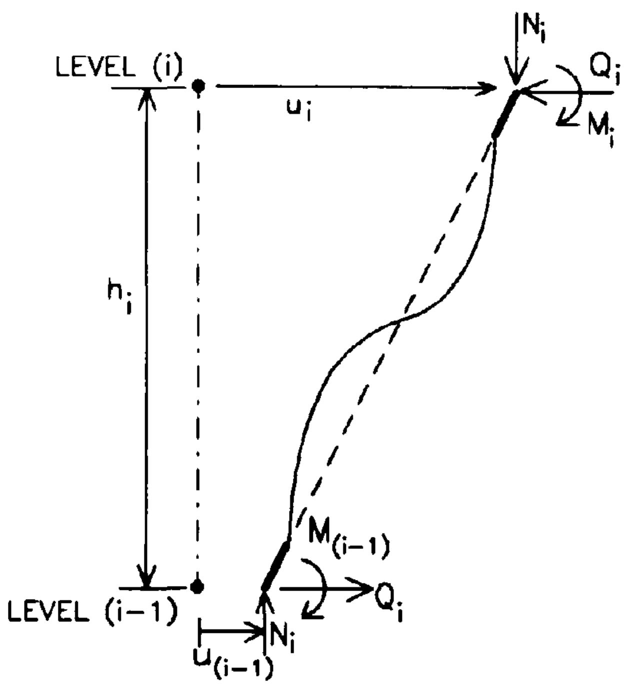


Figure 4.3 Computation of Shear Due to P-Delta Effects

$$Q_i h_i - (M_i + M_{i-1}) - N_i(u_i - u_{i-1}) = 0.0 \tag{4.12}$$

Considering equilibrium of the additional gravity load shears at story level i, the following expression is obtained:

$$P_i = N_i(u_i - u_{i-1}) \,/\, h_i \;-\; N_{i+1}(u_{i+1} - u_i) \,/\, h_i \tag{4.13}$$

The above equations can be written in the following form for each component:

$$\{P^*\} = [K]_G \{\Delta u\} \tag{4.14}$$

where $[K]_G$ is a tridiagonal matrix similar to the geometric stiffness matrix in finite elements. This matrix is added to the overall stiffness matrix prior to the start of a new analysis step.

4.6 Damage Analysis

The damage model resident in the original release version of IDARC is the model developed by Park, Ang and Wen (1984) wherein structural damage is expressed as a linear combination of *ductility (deformation)* damage and that contributed by hysteretic *energy dissipation* due to repeated cyclic loading. Direct application of the model to structural systems requires determination of an overall member deformation. Since inelastic behavior is confined within plastic zones near the ends of a member, the relationship between overall member deformation, local plastic rotations and the damage index is difficult to correlate. Moreover, the presence of internal member hinges renders the model unusable. A modified version of the model was, therefore, developed, based on moment, rotation, and dissipated hysteretic energy, as follows:

$$D = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} E_T \tag{4.15}$$

where:

θ_m = maximum rotation attained during load history

θ_u = ultimate rotation capacity of section

θ_r = recoverable rotation at unloading

β = strength degrading parameter = HBE (see User Manual)

M_y = Yield moment of section

E_T = Dissipated hysteretic energy

The original Park model used different strength degrading parameters for damage and local member hysteresis. Since the intent of the β parameter was to provide a correlation between strength loss and damage, the present version uses the same parameter for both damage computations and hysteretic modeling, i.e., $\beta = HBE$.

The above damage index can be used directly to determine damage at each member cross-section. Dissipated hysteretic energy is used as a weighting factor to compute the component damage index. As in the original IDARC program, two additional indices are also reported: a story level damage index; and an overall structural damage index. Both indices are computed using weighting factors based on dissipated hysteretic energy at the component and story levels, respectively.

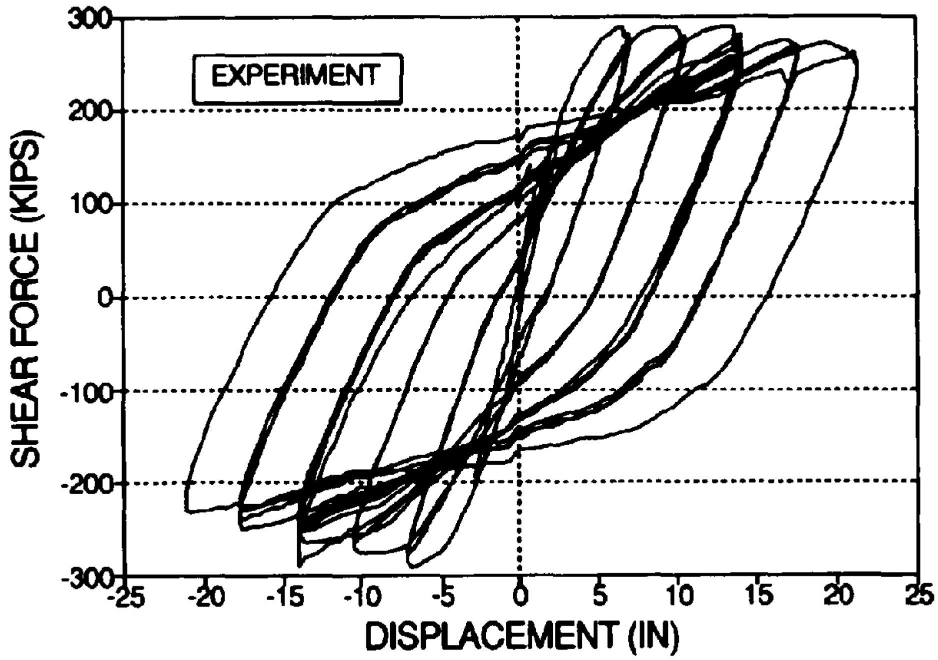
SECTION 5
PROGRAM VALIDATION: CASE STUDIES

The new version of IDARC has been verified extensively for accuracy of results through simulation of experimentally recorded behavior. The case studies include verification of observed performance of full-scale and scaled model structures. The structures are subjected to different loading types, encompass a variety of structural properties, and present most of the modeling, input and output features of the new IDARC program. The results obtained from the IDARC analyses are compared to analytical results obtained by other computer codes and by experimental results obtained in laboratory testing. This section presents a representative sample of case studies, emphasizing geometric and material descriptions, the input excitation (either quasi-static or earthquake), and selected results which illustrate the capabilities of the program.

5.1 Case Study #1: Component Testing - Full-Scale Bridge Pier Under Reversed Cyclic Loading

A series of full-scale and scale model circular columns were tested at the laboratories of the National Institute of Standards and Technology (Stone and Cheok, 1989; Cheok and Stone, 1990). These columns represent typical bridge piers designed in accordance with CALTRANS specifications. The piers were tested by applying both axial and lateral loads as shown in the experimental set-up in Fig. 5-1. The column analyzed in this sample investigation is a full-scale circular bridge pier measuring 30 feet with an aspect ratio of 6.0. The tests were performed using a displacement controlled quasistatic history as shown in Fig. 5-1. The column was made of 5.2 ksi concrete (measured compressive strength at 28 days) and had a modulus of elasticity of approximately 4110 ksi. Grade 60 steel with an actual yield stress of 68.9 ksi and an elasticity modulus of 27438 ksi was used as longitudinal reinforcement. The steel exhibited good ductility in the material testing with a 2% strain and a strain hardening of 1454 ksi before actual rupture. The cross-section in Fig. 5-1 also shows the reinforcement details. The experiment was analyzed using data presented in the Input Data Sheet for Case Study #1 (see Appendix B).

The purpose of this analysis is to simulate the essential characteristics of the hysteretic behavior and compare it with the experimental recorded response. The modified three parameter hysteretic model was used with a stiffness degradation coefficient HC=9.0, strength degradation coefficients HBE=0.05; HBD=0.0 (very little deterioration in



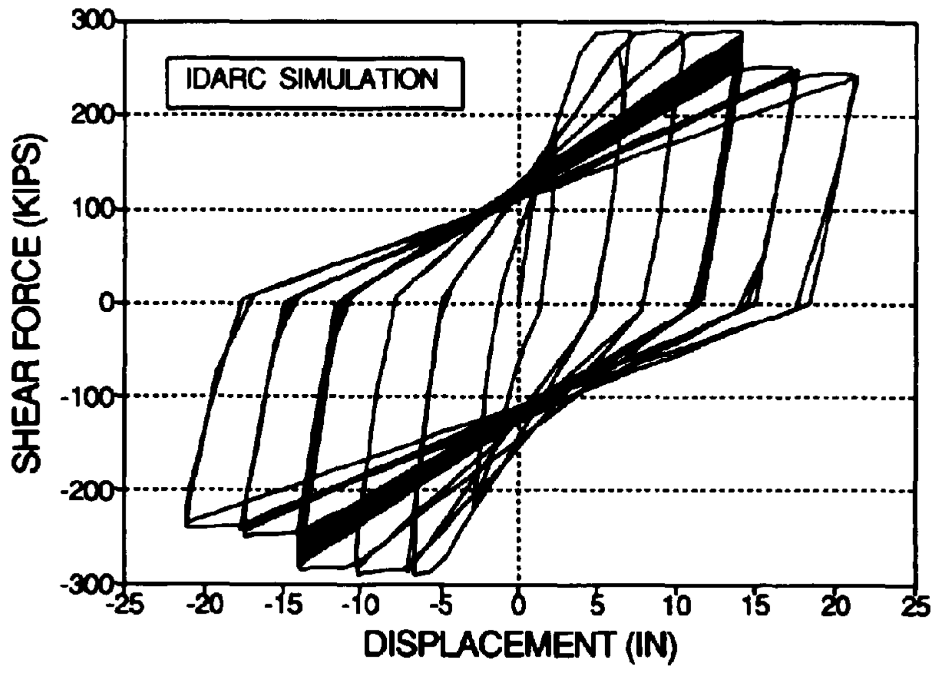


Figure 5.2 Comparison of Observed vs. Computed Response

strength), and a pinching coefficient HS=1.0 (indicating no pinching). These parameters were estimated from the observed experimental loops, and could be used to represent well-detailed sections. The response obtained from the analysis is compared with the test results in Fig. 5-2. The maximum loads attained in the analysis, 290 kips and 316 kips (positive and negative) compare well with those observed in the tests, 284 kips and 296 kips, respectively.

The damage evaluated using the analytical model is presented in Figure 5-3. Part of the damage is due to permanent deformations while part is due to strength deterioration from hysteretic behavior. Note that the deformation damage stays constant during the phase in which the column was cycled repeatedly at a ductility of 4.0. The total damage reaches approximately 0.9, which is indicative of extremely large damage, usually beyond repair as was the case for the tests presented here. It must also be pointed out that the specimen was able to sustain an additional one and half cycles before failure at a ductility of 8.0.

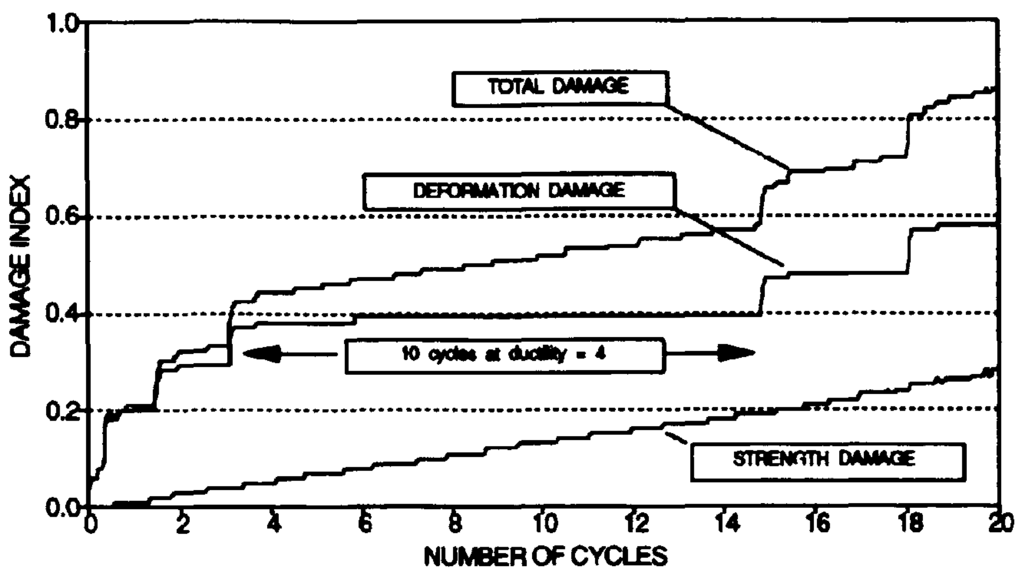


Figure 5.3 Progressive Damage History During Cyclic Testing

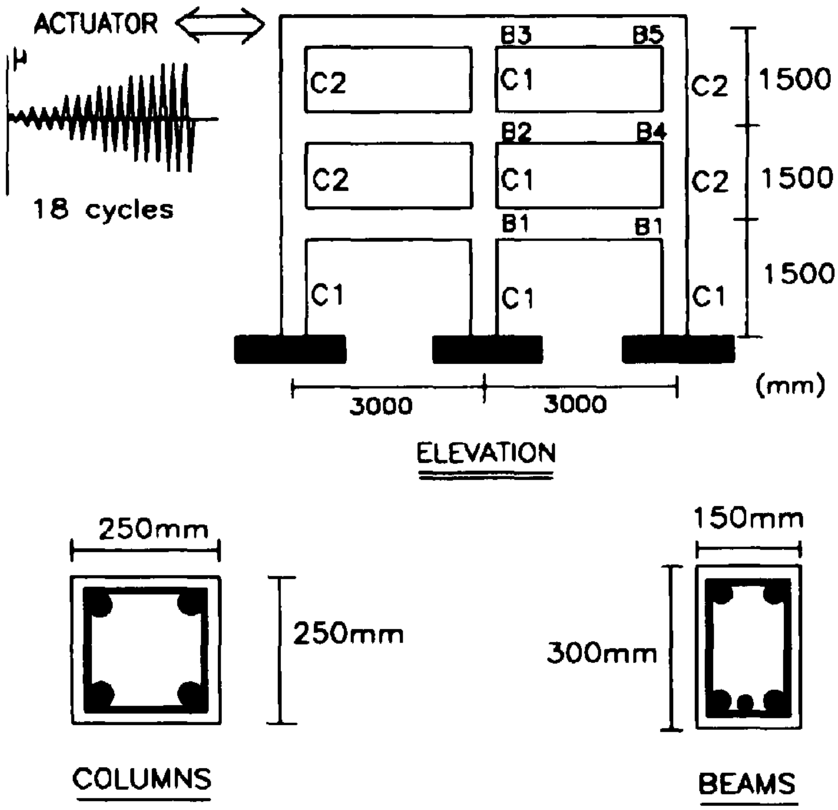
5.2 Case Study #2: Subassemblage Testing - 1:2 Scaled Three-Story Frame

A 1:2 scaled model of a three-story frame typical to construction practice of reinforced concrete structures in China was tested in the laboratory by Yunfei et al (1986). The structure was tested using a displacement controlled loading as shown in Fig. 5-4. The geometry of the frame and the essential reinforcement used for the analysis is also shown in Fig. 5-4. The frame is made of 40.2 MPa concrete and is reinforced by Grade 40 steel (400 MPa yield strength). Default parameters were used for the remainder of the material property information (see zero input in data set for Case Study #2, Appendix B). The first three cycles of loading produced cracking and first yielding. Subsequent loadings of three cycles at the same ductility were applied until the frame collapsed.

The model was analyzed using data specified in the data sheet for Case Study #2 in Appendix B. The hysteretic parameters were initially assigned based on well-detailed ductile sections obtained from the previous case study. These parameters were found to be adequate in reproducing the overall system response, however, a better estimate was obtained by increasing the strength degrading parameter. The final parameters, HC=8 for stiffness degradation, HBE=0.1 for strength deterioration and HS=1.0 for bond slip (pinching), produced excellent agreement of force levels at the larger amplitude cycles as shown in Fig. 5-5.

The choice of hysteretic parameters is important but not critical in establishing the overall system response. For example, values of HC between 4.0 and 9.0 and values of HBE between 0.05 and 0.10 would have produced almost comparable results. As will be pointed out later, a proper choice of hysteretic parameters becomes important only for cases where local failures, due to effects of bar pull-out, punching shear, etc., are expected, or when microconcrete is used for small-scale models (1:4 or greater). In this case study, no special connection behavior was modeled.

The present version of the program calculates dissipated hysteretic energy of components that can be used as an identification target for the choice of hysteretic parameters. In the current analysis, the identification was directed towards the maximum force levels which involves only the strength deterioration parameter. Hysteretic energy is also a known measure of structural damage. Fig.5-6 presents a comparative representation of dissipated energy and total system damage. A maximum damage of about 0.6 was achieved in the analysis, indicating that the global damage index is less sensitive to local damage accumulated at individual sections. Therefore, it will be necessary to calibrate global indices before they can be used in damage assessment.



(dimensions in mm)

SECTION	NO OF BARS & BAR DIA	HOOPS
COLUMN C1	4 # 14	12mm@75 mm
COLUMN C2	4 # 12	8mm@75 mm
BEAM B1	TOP: 2 # 16 BOT: 2 # 16	6mm@75 mm
BEAM B2	TOP: 2 # 16 BOT: 2 # 16, 1 # 10	6mm@75 mm
BEAM B3	TOP: 2 # 12 BOT: 2 # 14	6mm@75 mm
BEAM B4	TOP: 2 # 18 BOT: 2 # 16	6mm@75 mm
BEAM B5	TOP: 2 # 14 BOT: 2 # 14	6mm@75 mm

Figure 5.4 Details of Half-Scale Model Frame

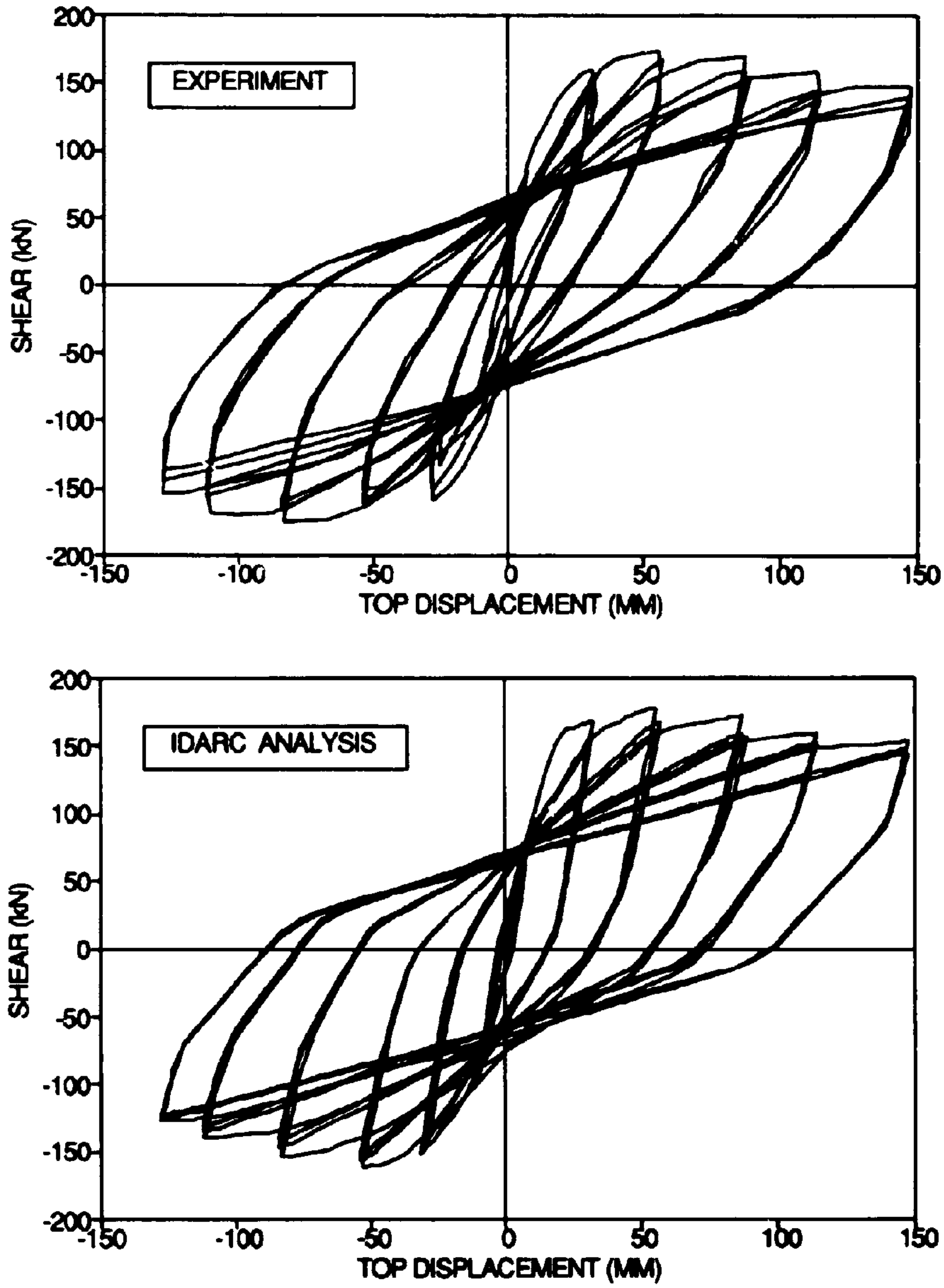


Figure 5.5 Comparison of Observed vs. Simulated Force-Deformation Response

Another feature of the IDARC program is the push-over analysis under monotonically increasing lateral loads. This feature was used to determine the correspondence of the observed collapse mechanism. The frame developed a beam side sway collapse mechanism that was clearly documented in the experimental records through measured rebar yielding in the critical beam-column interface and column-base sections, and identified by visual observations. Figure 5-7 shows the damaged frame with observed plastic hinge locations and computed sequence of hinge formation using IDARC.

Finally, the progression of damage history is shown in Fig. 5-8 for each of the story levels. The upper two levels did not experience any column damage. Studies of this nature can be used to calibrate damage models using ductility demand and dissipated hysteretic energy as controlling criteria.

The two case studies presented this far were both based on displacement controlled loading, which is generally typical in laboratory testing of components and subassemblages. IDARC can also be used for force-controlled loading histories.

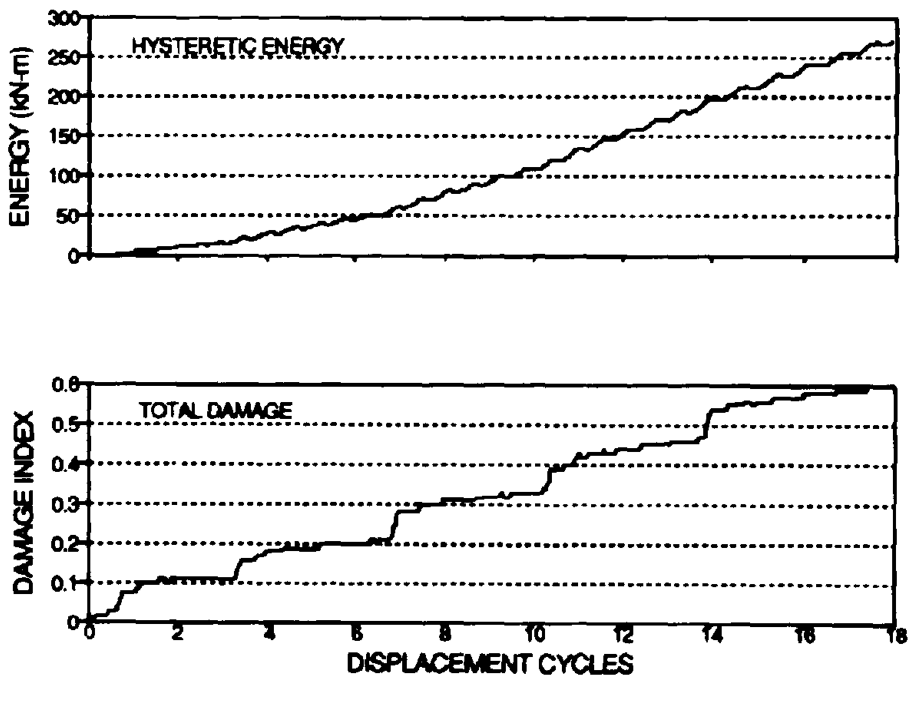


Figure 5.6 Correlation of Dissipated Energy and Global Damage

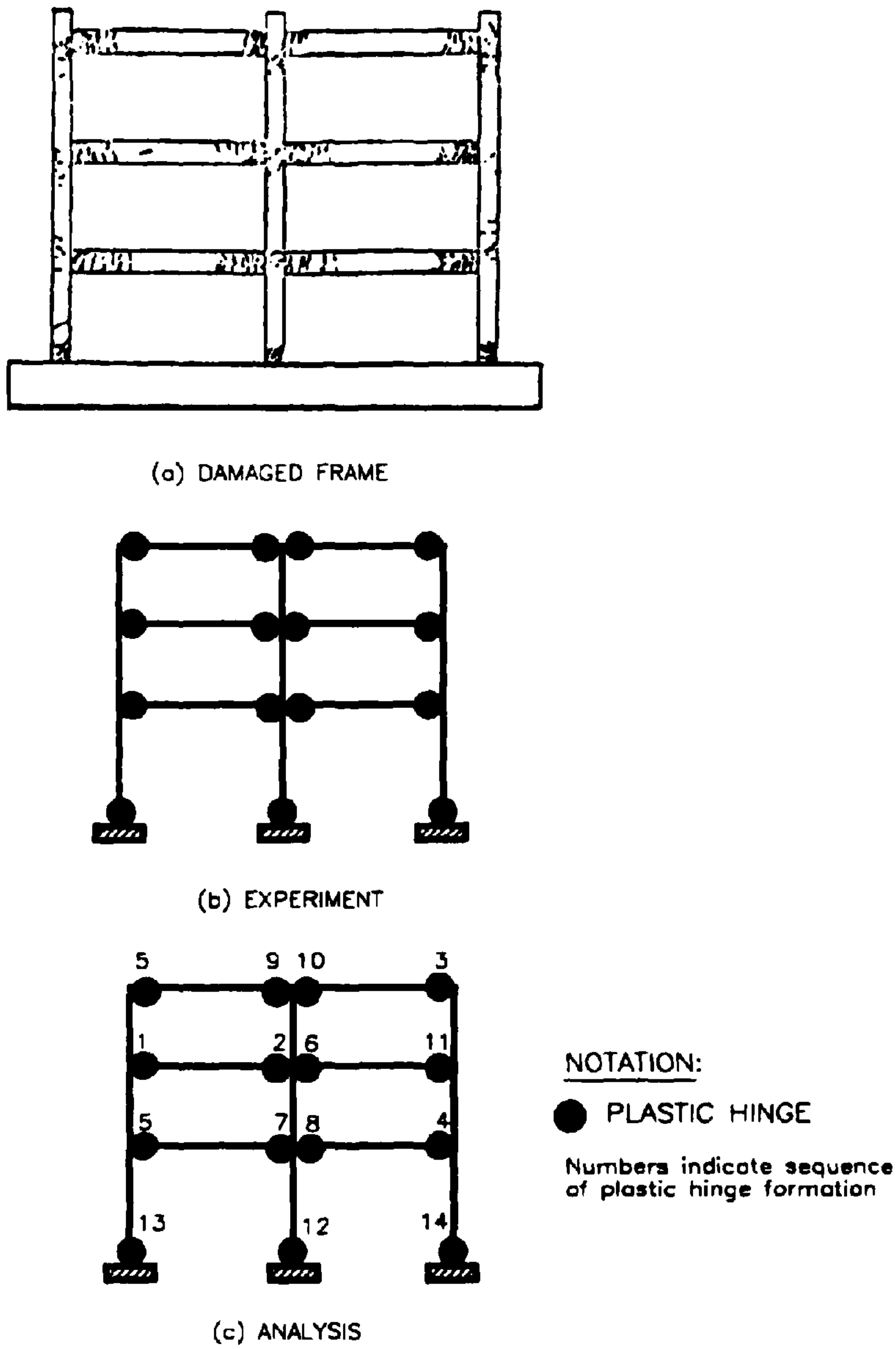


Figure 5.7 Study of Collapse Mechanism

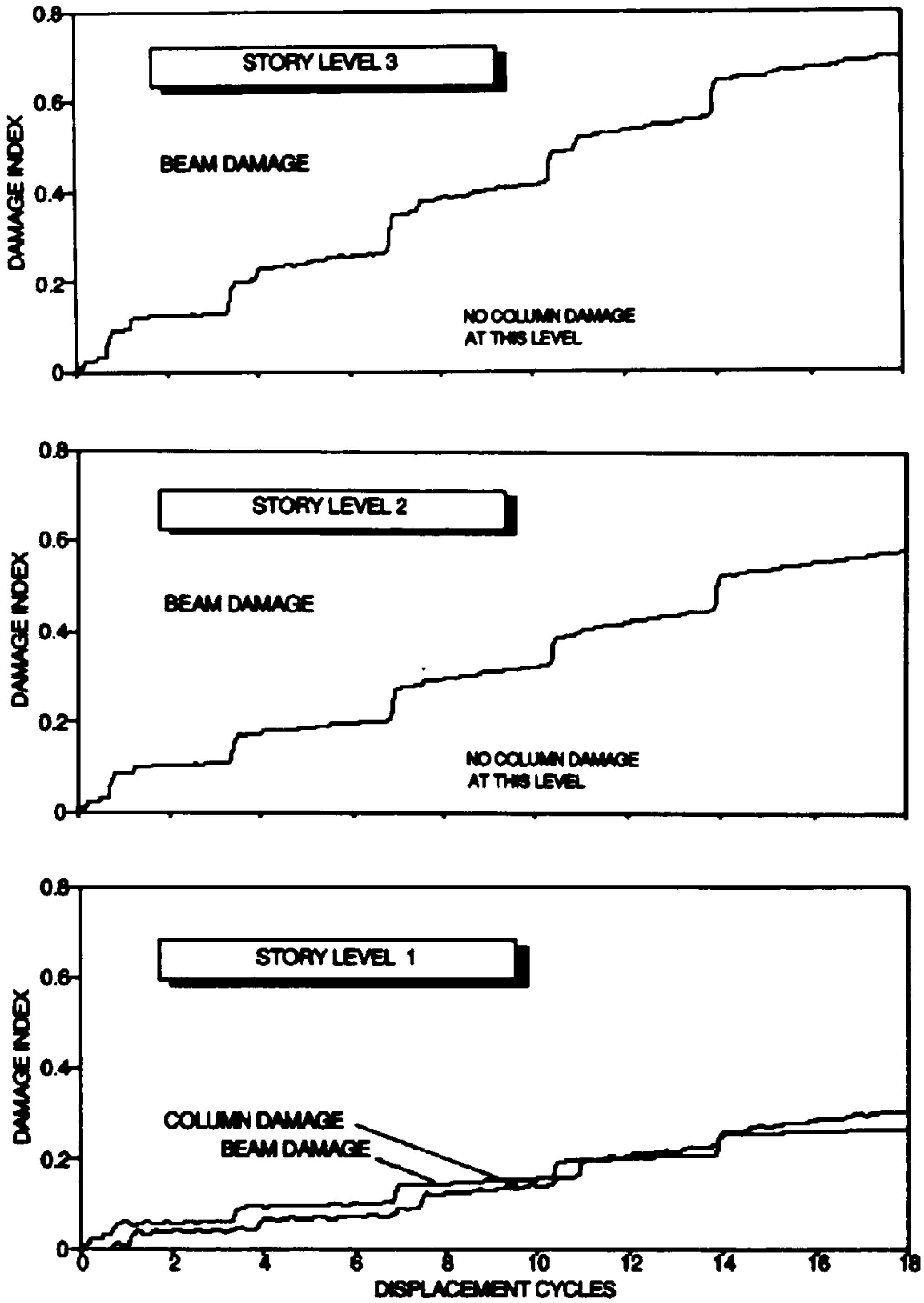


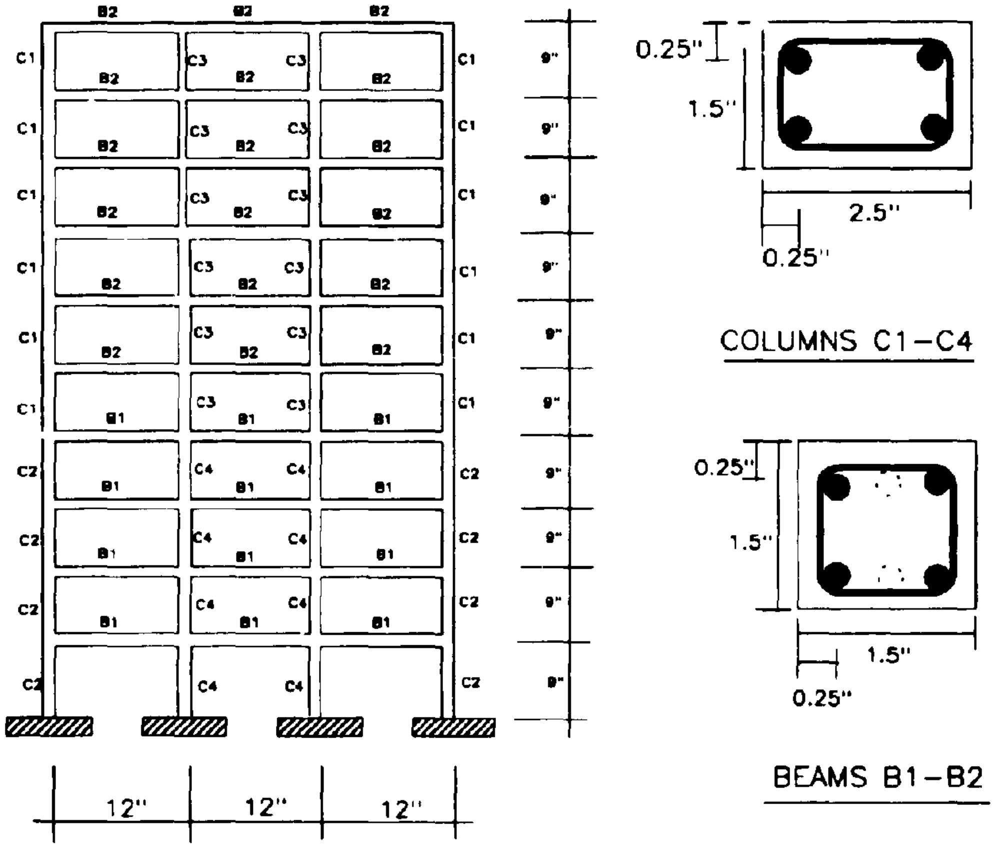
Figure 5.8 Progressive Story Level Damage

5.3 Case Study #3: Seismic Simulation of Ten-Story Model Structure

This study is based on shaking table tests of a scaled model ten-story, three-bay frame structure conducted at the University of Illinois, Urbana (Cecen, 1979). The model was subjected to simulated earthquake ground motions at levels that produce strong inelastic behavior and damage. The geometrical configuration, element designation, dimensions and reinforcement details are shown in Fig. 5-9. The model is made of 4350 psi concrete and Grade 60 steel with a measured yielding strength of 70 ksi and modulus of elasticity of 29000 ksi. The initial concrete modulus was adjusted to provide a fundamental period consistent with observed response. This is an important consideration when initial conditions, such as cracking resulting from gravity loads or model construction, produce a system that is not consistent with gross moment of inertia computations.

The model was subjected to scaled ground excitations with time compression of 2.5 of the 1940 El Centro accelerogram. The peak base accelerations of the three successive seismic inputs were: 0.36g, 0.84g, and 1.6g repsectively, as shown in Fig. 5-10. The purpose of this case study is to compare the analytical response with the experiment in case of severe nonlinearities resulting from progressive damage. The second objective of the study is to compare the analytical performance with other analytical programs that perform similar tasks. The analysis was done using the information presented in the input data sheets for Case Study #3 (see Appendix B). The structure is modeled by mass similitude with a total floor weight of 1000 lbs per floor. The dynamic analysis is done using an analysis time step of 0.001 sec. Hysteretic parameters used are listed in the input data sheet. There was no predetermined basis for the choice of hysteretic parameters. The program default values were used for both beams and columns, with the exception of the stiffness degrading parameter for columns. The program assigned deafault for this parameter is 2.0. However, results of testing on relatively small scale components (1:4 or greater) indicate that the parameter HC is much smaller. It is suggested to use $HC = 0.5 - 1.0$ in such cases.

The comparison of the analytical and experimental results in terms of (i) peak accelerations is shown in Figure 5-11; and (ii) peak displacements is shown in Fig. 5-12. The maximum displacements reported in Cecen (1979) are based on one-half the double amplitudes, while the IDARC values are absolute peaks. The entire displacement histories compare more favorably as will be seen shortly.



SECTION	LONG. REINF (each face)	TRANS. REINF.
BEAM B1	2#13	#16 gage@ 0.3"
BEAM B2	3#16	– " –
COLUMN C1	2#10	#16 gage@ 0.35"
COLUMN C2	2#7	– " –
COLUMN C3	2#13	– " –
COLUMN C4	2#8	– " –

Figure 5.9 Configuration and Reinforcement Details for Model Structure

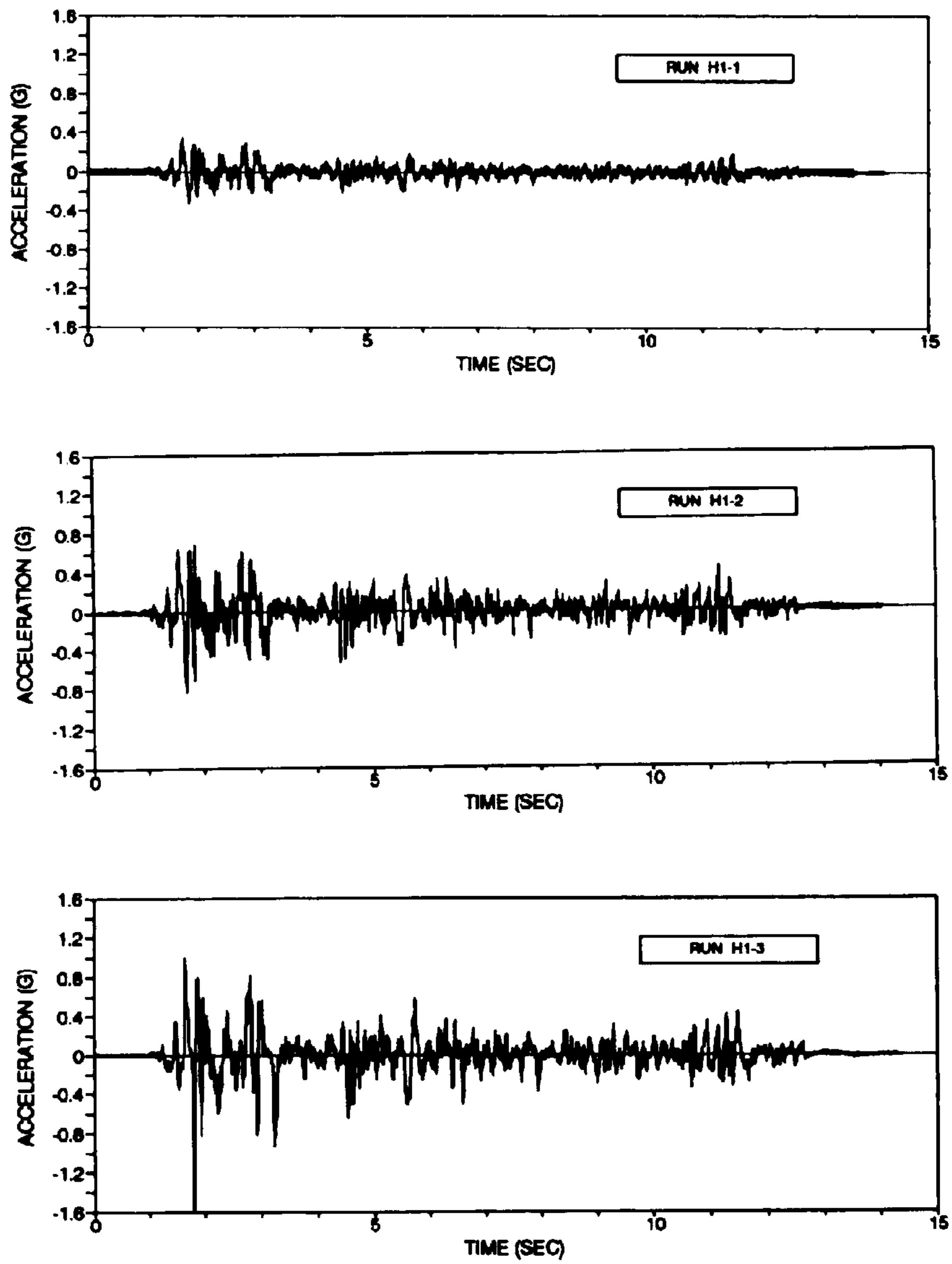


Figure 5.10 Achieved Table Motions for Seismic Testing

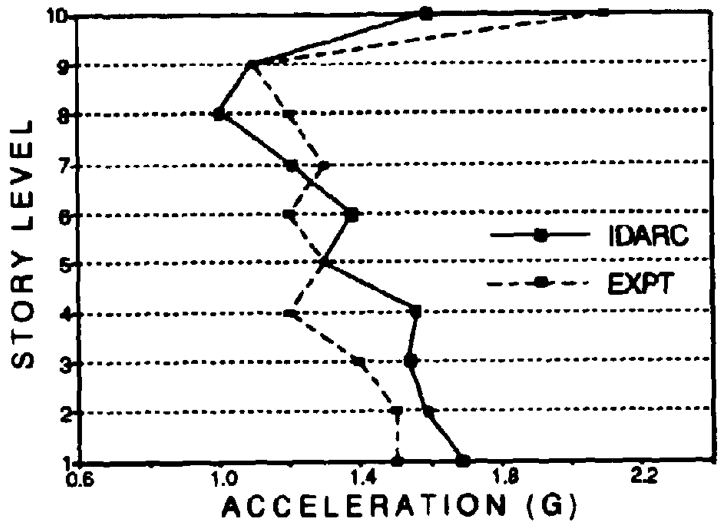


Figure 5.11 Computed vs. Observed Peak Acceleration Response

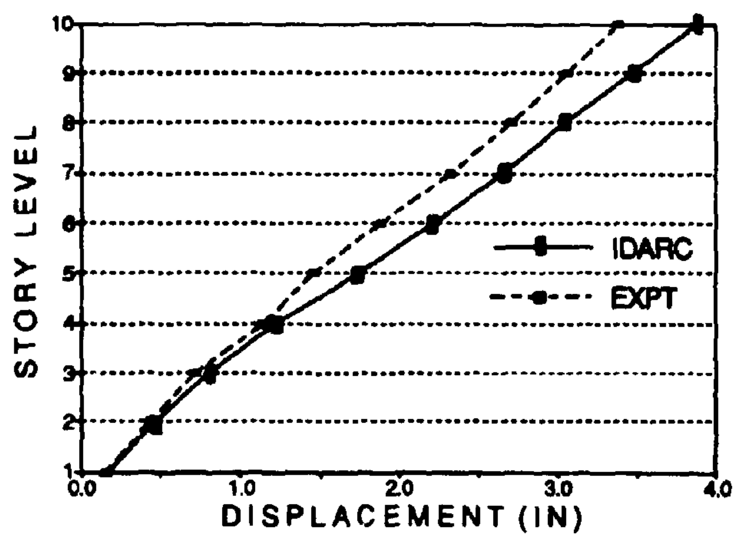


Figure 5.12 Computed vs. Observed Peak Displacement Response

The analysis is also compared with two other available computer programs: (i) SARCF-III (Gomez et al.,1990) and (ii) DRAIN-2D (Kaanan and Powell, 1971). Since both SARCF and DRAIN use bilinear envelopes, only the initial stiffness and yield moments were provided as basic input. The default Takeda degrading model was used in DRAIN while the damage-based hysteresis model was used in SARCF. The results are presented in Figs. 5-13 through 5-15. IDARC shows peak differences ranging between 3% to 10% of experimentally observed values. It can also be observed that an excellent agreement is obtained using IDARC for RUN H1-3 which has the largest inelastic response.

In all three programs, the three successive seismic inputs were provided as a continuous ground motion so that the effects of each run were carried forth to the next without returning the system to zero conditions. Recording instruments, on the other hand, are typically reset to zero conditions between tests thereby making it difficult to track permanent deformations, if any.

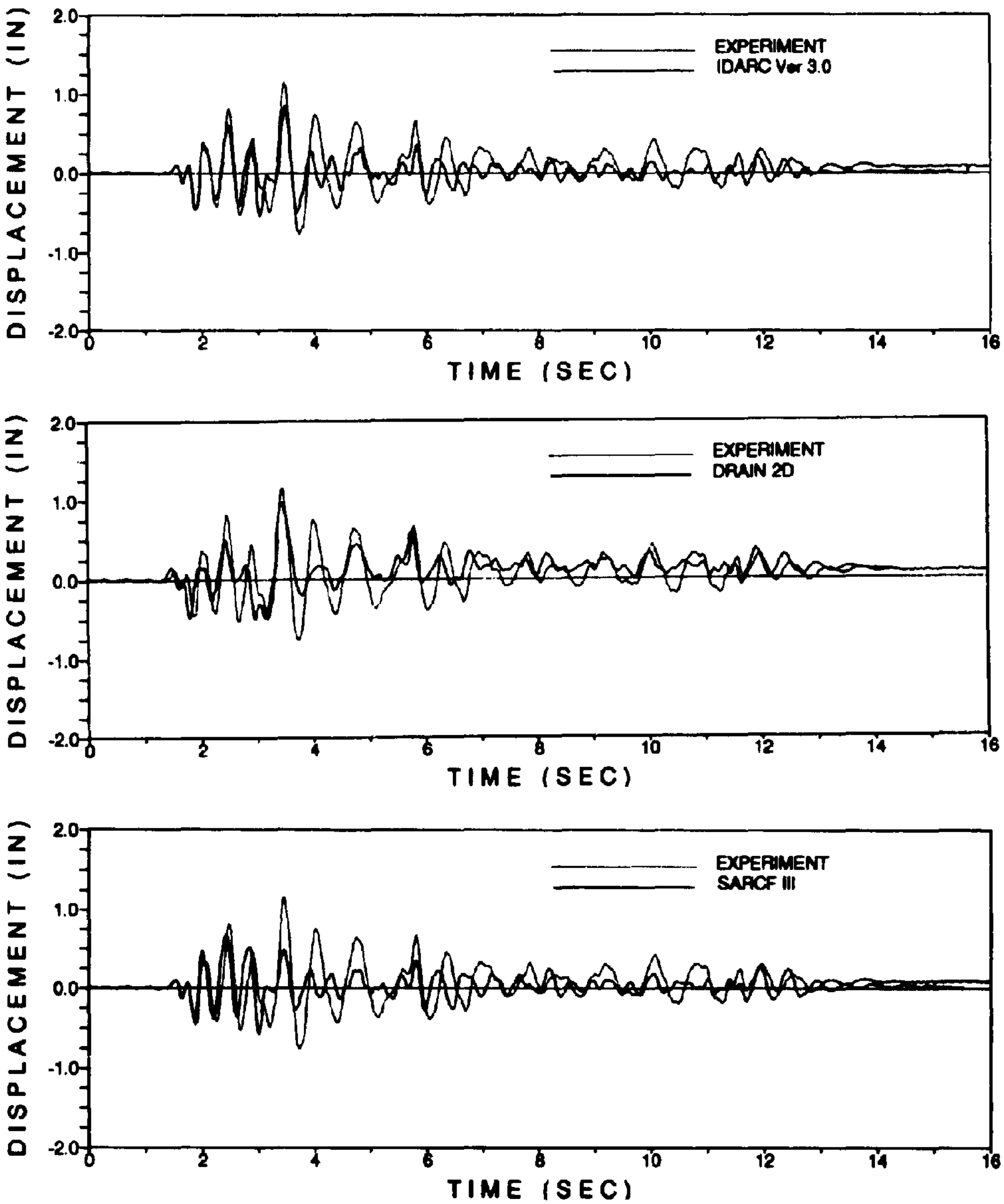


Figure 5.13 Comparison with Other Programs (Low Intensity)

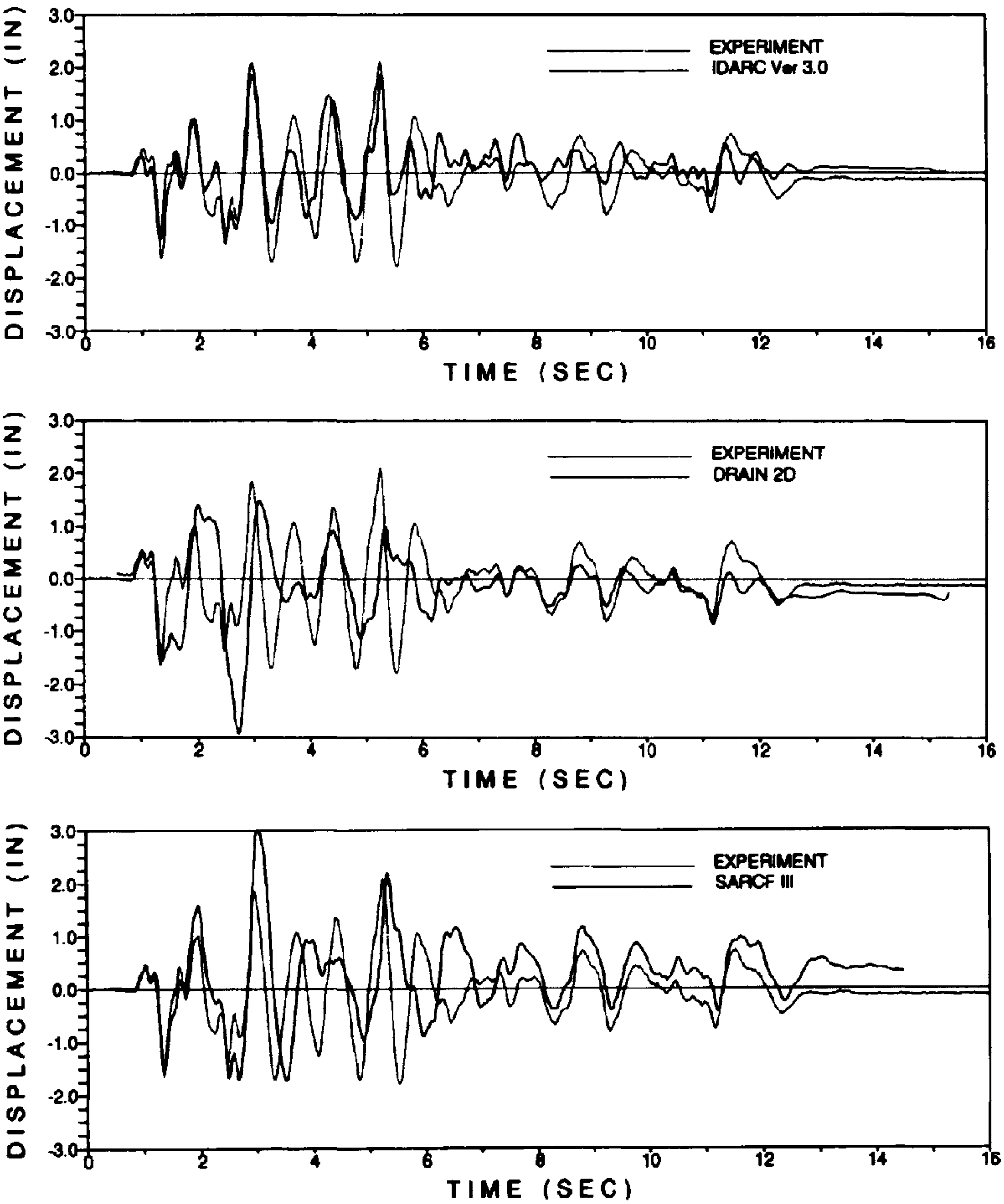


Figure 5.14 Comparison with Other Programs (Moderate Intensity: Inelastic)

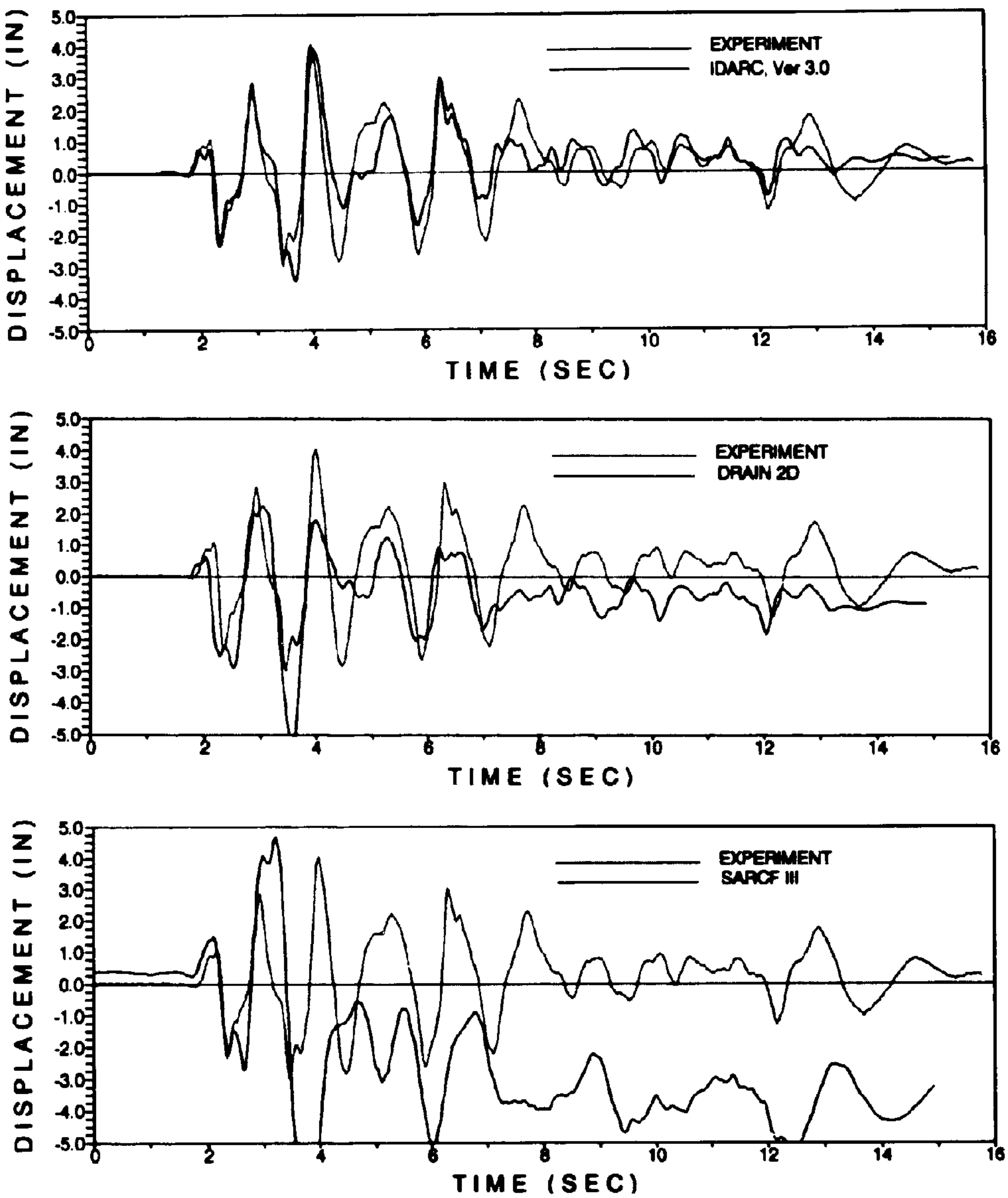


Figure 5.15 Comparison with Other Programs (Highly Inelastic)

5.4 Case Study #4: Seismic Response of 1:3 Scale Model Lightly Reinforced Concrete Structure

A comprehensive study of lightly reinforced concrete frame structures was the subject of numerous investigations at the State University of New York at Buffalo (Bracci, 1992), and at Cornell University (El-Altar, 1990). A 1:3 scaled model was constructed, tested, retrofitted, and retested using simulated earthquake motion generated by the shaking table at SUNY/Buffalo. The model reflects a slice of a long structure with three-bay frames in the transverse direction. The "slice" has two parallel lightly reinforced frames as indicated by the model representation in the plan view in Fig. 5-16. Essential geometrical data and reinforcement details are also shown in the figure. Attained concrete strengths were 4000 psi, 3000 psi, and 3500 psi at the first, second, and third story levels respectively, with an elastic modulus of 2700 ksi, 2300 ksi, and 2530 ksi, respectively. The steel had an average yielding strength of 65 ksi after annealing with a modulus of elasticity of approximately 29000 ksi. Additional details about the structure and the testing can be found in Bracci (1992).

The model was tested by a sequence of ground (table) motions reflecting a low level earthquake (PGA=0.05g), a moderate earthquake (PGA=0.20g) and a severe earthquake (PGA-0.30g). The ground motion is obtained by scaling the acceleration time history of Taft (1952) N21E component. Only two sets of results are presented here.

The main purpose of this study was to investigate the effectiveness of using identified component properties from separate sub-assemblage tests in predicting the dynamic response of the total structure. The data set used for in this example is presented in Appendix B. Only the second run at a measured peak acceleration of 0.22g is included, since the basic data is the same for both runs, with the exception of the initial stiffness and the input ground motion. As indicated, the data was derived entirely from the results of separate interior and exterior beam-column sub-assemblage tests which provided information on yield strengths and hysteretic behavior. *No attempt was made to fit the observed shaking table response.*

The comparison of response displacements for the top story for the mild and moderate earthquake are shown in Figs. 5-17 and 5-18. IDARC predictions show good agreement for both peak values and the total response history. The comparison includes predictions by DRAIN-2D and SARCF-II. More data on observed behavior in terms of deformations, stresses, and damage mechanisms are reported in Bracci (1992).

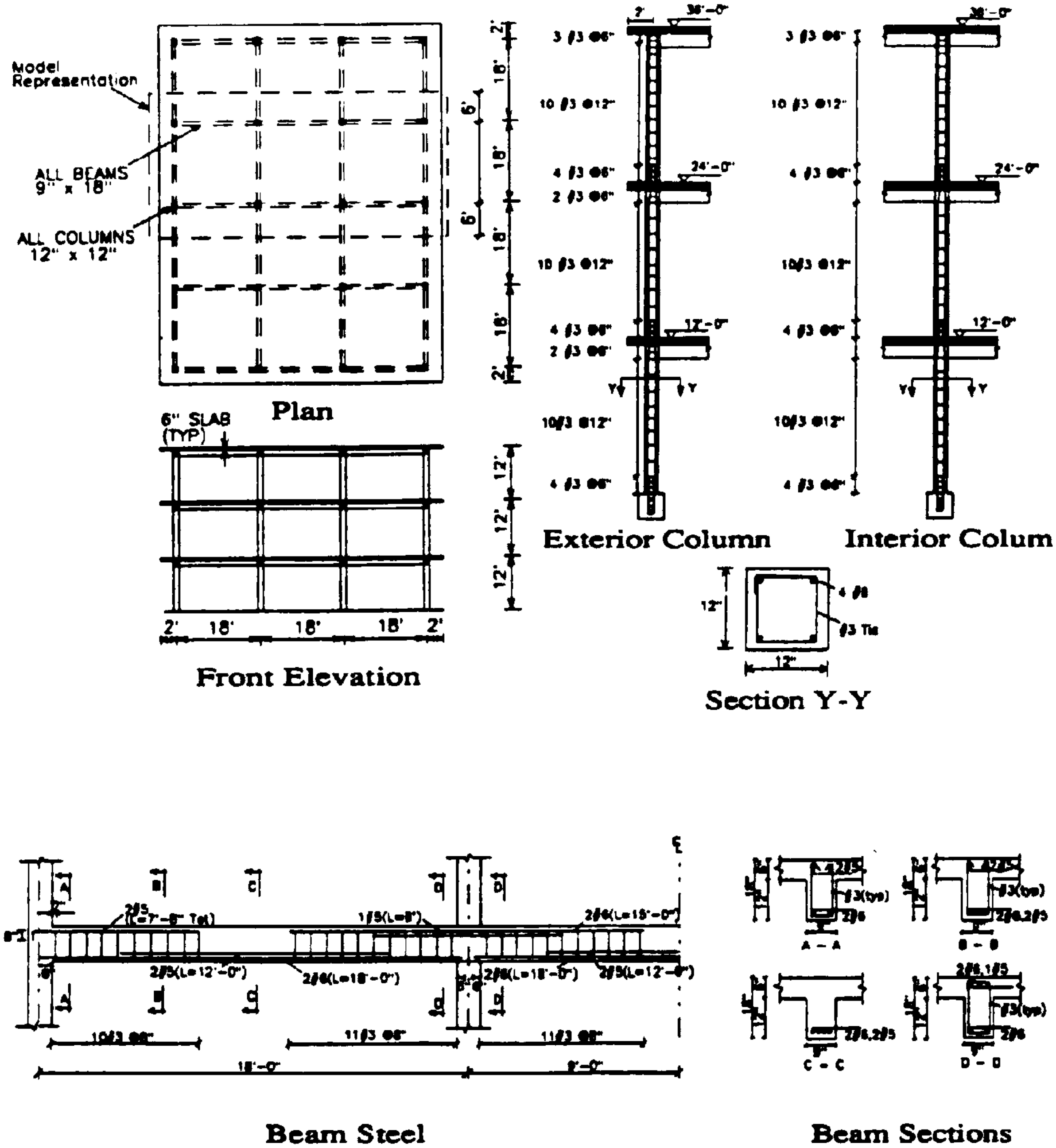


Figure 5.16 Details of Gravity-Load-Designed Frame Building

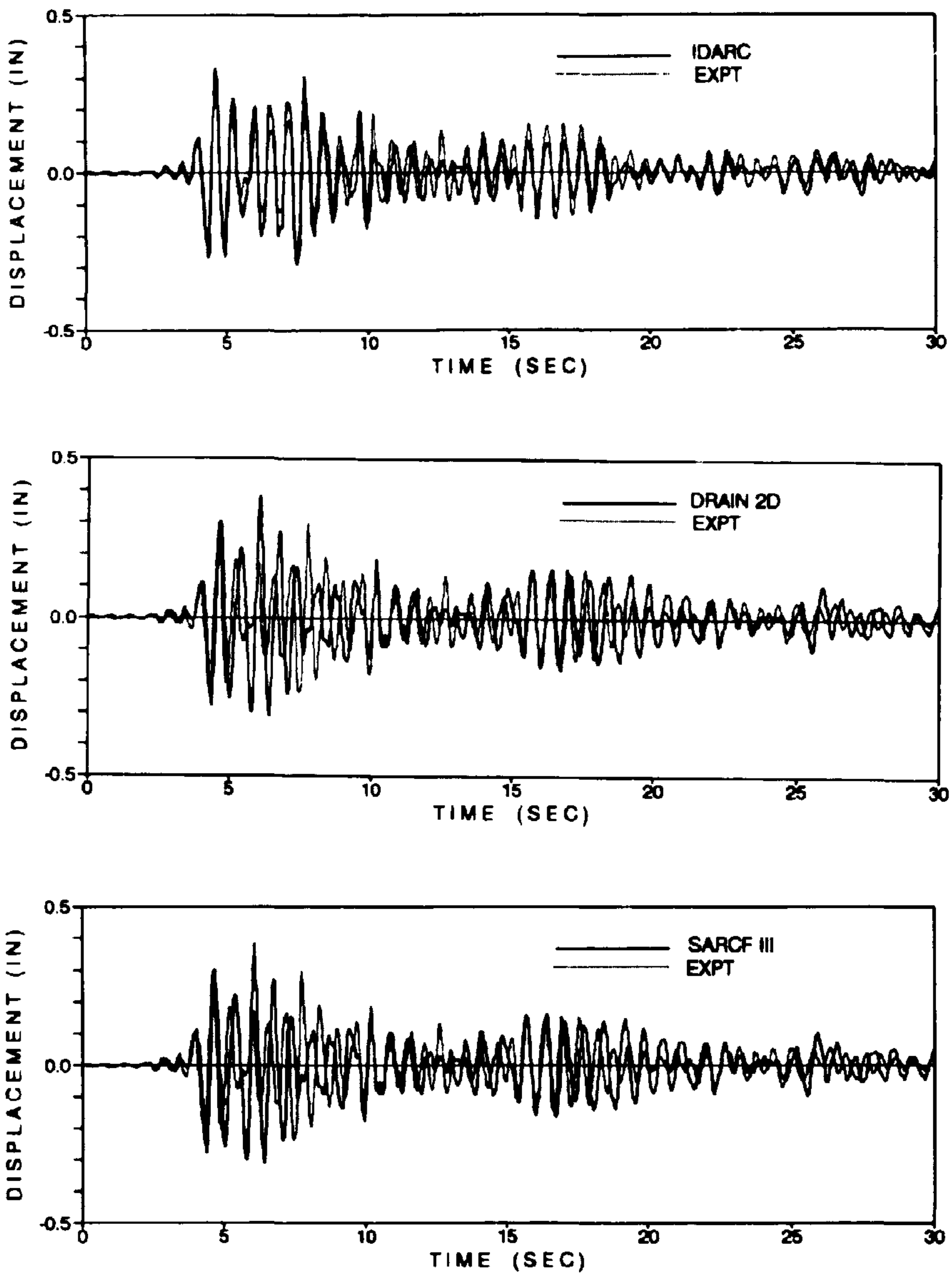


Figure 5.17 Comparison with Other Programs - Low Intensity (0.05g)

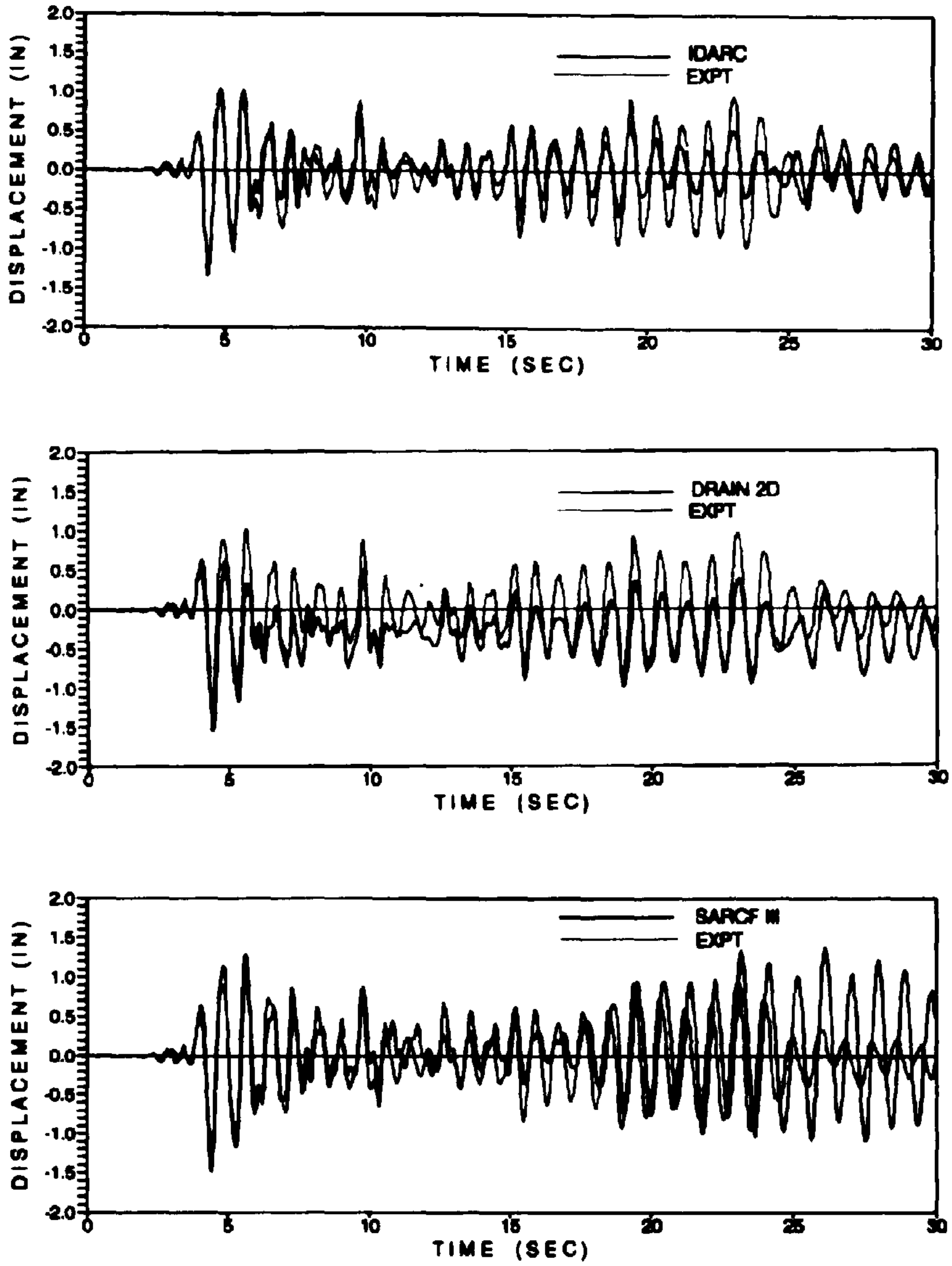


Figure 5.18 Comparison with Other Programs - Moderate Intensity (0.22g)

5.5 Case Study #5: Damage Analysis of the Cypress Viaduct Collapse During the 1989 Loma Prieta Earthquake

The collapse of the Cypress Viaduct during Loma Prieta Earthquake in 1989 provided an excellent opportunity to verify IDARC in seismic damage evaluation of an existing structure. The Cypress structure consisted of a boxed girder roadway supported by a series of 83 reinforced concrete two-story bents. Eleven types of bents were used in the construction of the viaduct. Fifty-three of these bents were designated as Type B1, which consists of two portal frames, one mounted on top of the other (Fig.5-19). The upper frame is connected to the lower by shear keys (hinges). The dimensions of a typical B1 bent and its reinforcement details are shown in Fig. 5-19. B1 bents suffered the most damage and seemed to have failed in the same consistent manner throughout the freeway.

The structure was modeled using a combination of tapered column, shear-panel and beam elements. The pedestal region was modeled as a squat shear wall so that its impending shear failure could be monitored. The Outer Harbor Wharf horizontal strong-motion records were transformed to 94° which is transverse to the alignment of the collapsed portion of the viaduct. The influence of gravity loads on the structure was simulated by imposing a ramp load in the form of a vertical excitation with magnitude of 1 g. The actual ground motions were introduced after the resulting free vibrations had damped out. The data used for the analysis is presented in the data sheet for Case Study #5 in Appendix B.

The purpose of this analysis is to demonstrate the use of the program in practical analysis of existing structures. The IDARC model of the bent is shown in Fig.5-20. The imposed vertical and horizontal motions on the structure are shown along with the top level displacement response in Fig.5-21. The IDARC analysis revealed that the first element to fail was the left-side pedestal after approximately 12.5 seconds into the earthquake *(Note that the plot shown in Fig.5-22 includes an initial 4 seconds of gravity load input).* A plot of the damage history of this pedestal is shown in Figure 5-22, in which the horizontal input motion and the pedestal shear history are also shown for reference. Complete details of the analysis of the Cypress Viaduct using IDARC is reported in a separate publication (Gross and Kunnath, 1992).

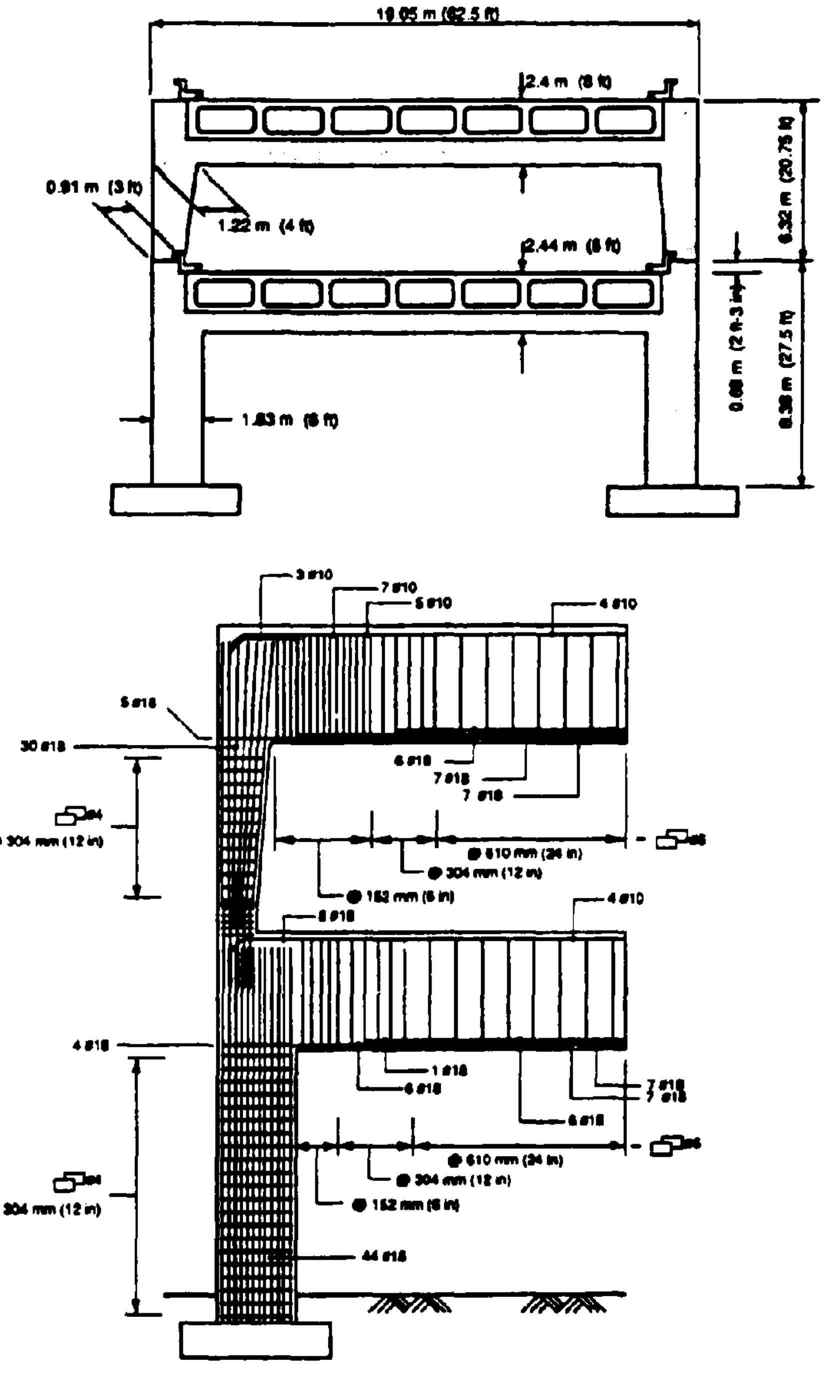


Figure 5.19 Structural Configuration and Reinforcement Details of Type B1 Bent

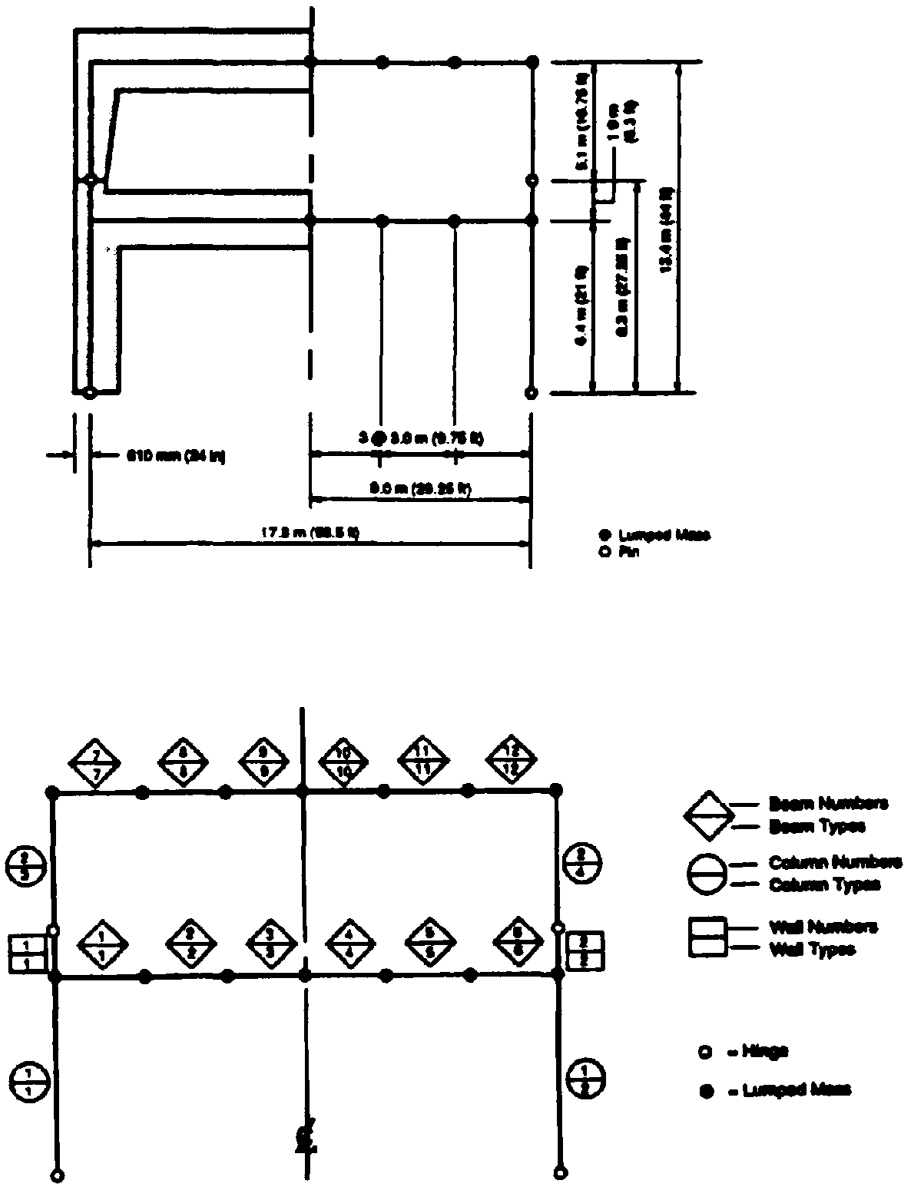


Figure 5.20 IDARC Model Used in Damage Analysis

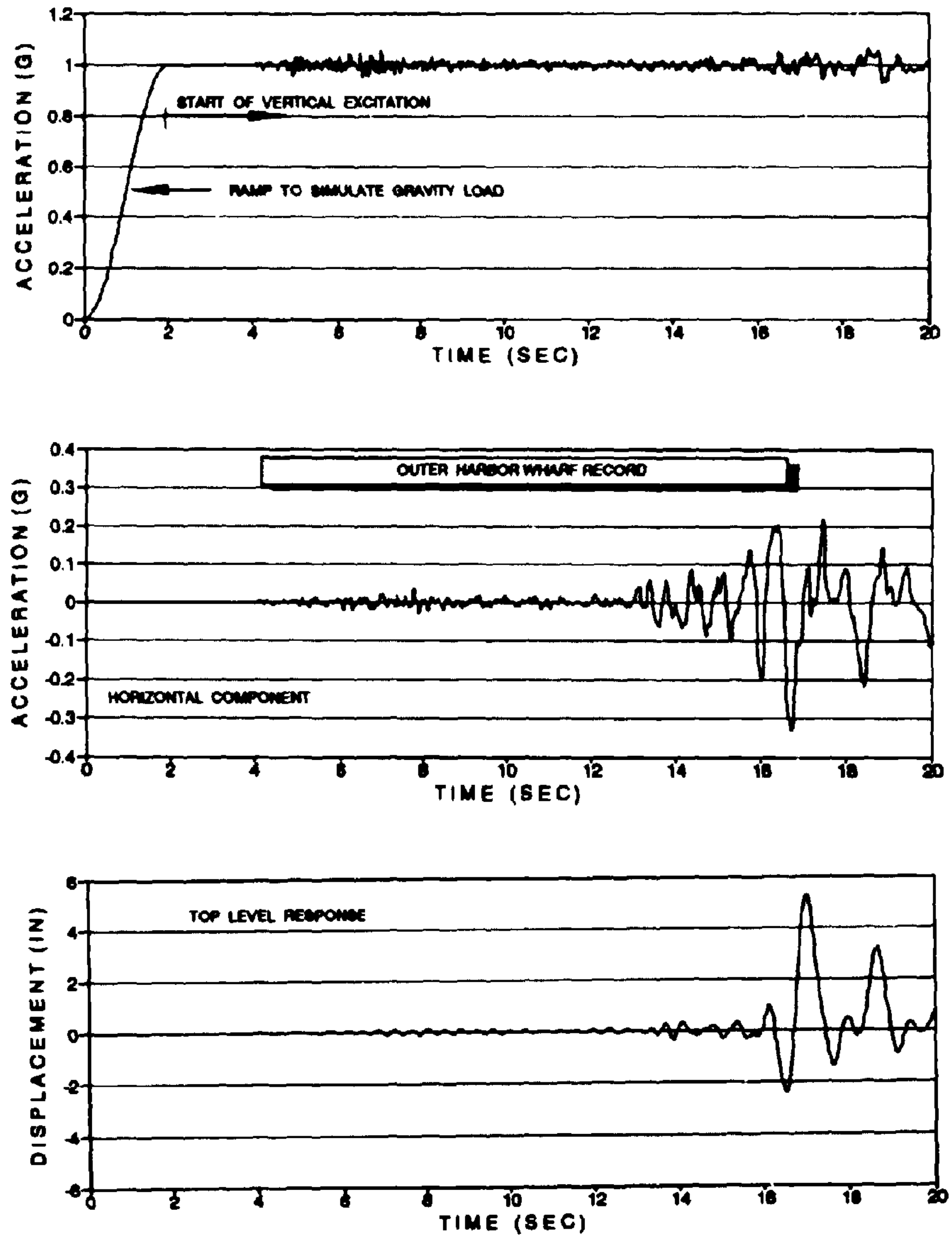


Figure 5.21 Displacement Response of B1 Bent

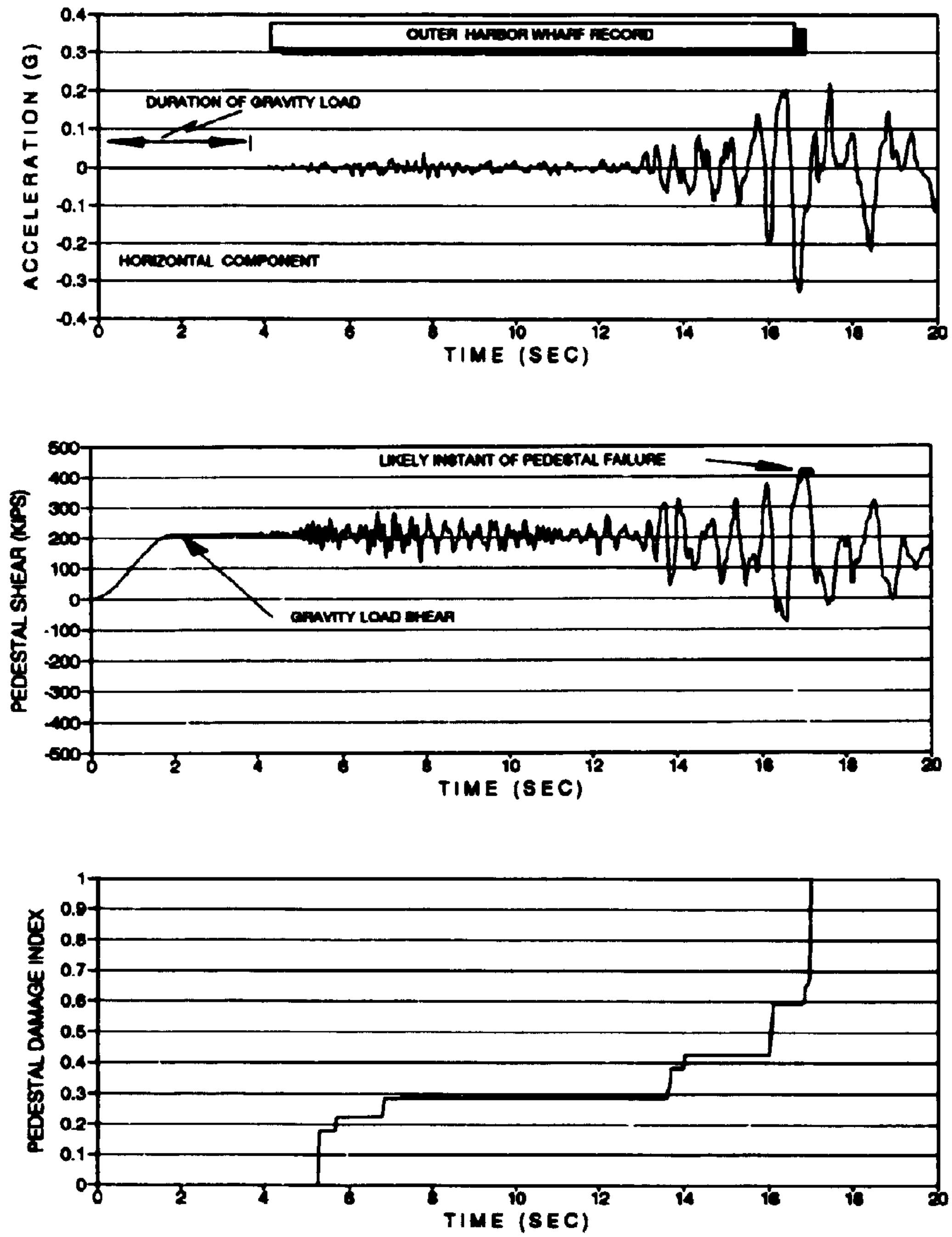


Figure 5.22 Damage History of Pedestal Region

The case studies presented in this Section are only meant to show a representative sample of IDARC capabilities. The task of modeling different structures vary from case to case, depending upon the degree of complexity in structural configuration and member connections. While IDARC must still be regarded as a special-purpose program, it can be used with generality in analysis of structures ranging from buildings to bridges and partial subassemblages used in laboratory testing.

The input parameters to the program are obtained directly from engineering drawings or from separate computations of member properties. The only exceptions are the input of hysteretic parameters and the assigned viscous damping for the dynamic analysis. The case studies presented here cover a range of different structures from single components to scaled model frame buildings to full scale existing structures. They also include well-detailed ductile joints to gravity-load-designed nonductile connections. The parameters used here can serve as a reference for the choice of appropriate parameters. It is recommended to use data from component tests when available, either by actual testing of from the literature of past testing of similar configurations and details.

The choice of hysteretic parameters is critical only in the prediction of local failures at a beam-column interface. For systems with a large number of elements, the overall response is less sensitive to local behavior. Consequently, the prediction of global damage states is more reliable for single components, such as single bridge piers, and structures where the damage is more evenly distributed.

SECTION 6
CONCLUSIONS

The success of an analysis is dependent largely on the adequacy of the modeling. Given the complexities of RC behavior, the task of assembling a structural model with reliable capacity estimates (both strength and deformation) is formidable. Yet, in the simple framework of member-by-member modeling, as illustrated in the previous section, it is possible to predict overall system response with reasonable accuracy.

The primary basis of the member-by-member modeling of RC elements is derived from distributed flexibility concepts. Inelastic action is accounted in terms of hysteretic moment-curvature behavior at critical sections. The resulting instantaneous system is then ready to accept a variety of loading options: incremental static, lateral monotonic, quasistatic cyclic, and transient dynamic.

A significant portion of this report deals with validation studies, in which analytical predictions are compared with experimentally observed response. If experimental results are to be reproduced with great precision, a certain degree of model tuning may be necessary. The tuning process is sometimes referred to as "identification". System identification for the prediction of inelastic response requires that the following estimates be as precise as the degree of precision expected:

- strength and deformation capacity
- hysteretic control parameters
- constant mass-dependent damping
- variable stiffness-dependent damping (not available in IDARC)
- initial stiffness of components

Based on studies conducted this far using IDARC, it has been established that the only essential parameter to be calibrated is the initial stiffness of the structural members which collectively provides a good estimate of the fundamental system period.

A summary of the major modeling and program enhancements to the computer code, IDARC, was presented. Though this is the second official release of the program, it has been labeled Version 3.0, since a number of intermediate versions have been distributed with ad-hoc changes and improvements. The intent of Release 3.0 is to supersede all existing versions of the code.

SECTION 7
REFERENCES

Allahabadi, R. and Powell, G.H. (1988). "DRAIN-2DX User Guide." Report No. UCB/EERC-88/06. University of California, Berkeley.

Bracci, J.M. (1992). "Experimental and Analytical Study of Seismic Damage and Retrofit of Lightly Reinforced Concrete Structures in Low Seismicity Zones." Ph.D. Dissertation. Department of Civil Engineering, State University of New York at Buffalo, New York.

Cecen, H. (1979). "Response of Ten Story Reinforced Concrete Model Frames to Simulated Earthquakes." Ph.D. Dissertation. Department of Civil Engineering, University of Illinois, Urbana.

Cheok, G.S. and Stone, W.C. (1990). "Behavior of 1/6 Scale Model Bridge Columns Subjected to Inelastic Cyclic Loading." ACI Structural Journal, Vol.87, No.6, 630-638.

Chung, Y.S., Meyer, C. and Shinozuka, M. (1988). "SARCF User's Guide: Seismic Analysis of Reinforced Concrete Frames." Technical Report NCEER-88-0044, State University of New York at Buffalo.

El-Attar, A.G. (1991). "A Study of the Seismic Behavior of Lightly Reinforced Concrete Structures." Ph.D. Dissertation. Department of Civil Engineering, Cornell University, Ithaca.

Gomez, S.R., Chung, Y.S. and Meyer, C. (1990). "SARCF-II User's Guide: Seismic Analysis of Reinforced Concrete Frames." Technical Report NCEER-90-0027, State University of New York at Buffalo.

Gross, J.L. and Kunnath, S.K. (1992). "Application of Inelastic Damage Analysis to Double Deck Highway Structures." Technical Report NISTIR-4857, U.S. Department of Commerce, National Institute of Standards and Technology, Gaithersburg, MD.

Kabeyasawa, T., Shiohara, H., Otani, S. and Aoyama, H. (1983). "Analysis of the Full-Scale Seven-Story Reinforced Concrete Test Structure." Journal of the Faculty of Engineering, University of Tokyo, Vol.XXXVII, No.2.

Kanaan, A.E. and Powell, G.H. (1973) "DRAIN-2D - A General Purpose Computer Program for Dynamic Analysis of Inelastic Plane Structures." Report No.UCB/EERC/73/06 and 73/22. University of California, Berkeley.

Kent, D.C. and Park, R. (1971). "Flexural Members with Confined Concrete." Journal of Structural Division, ASCE, Vol.97, ST7, 1969-1990.

Kunnath, S.K., Reinhorn, A.M. and Abel, J.F. (1992). "A Computational Tool for Seismic Performance of Reinforced Concrete Buildings." Computers and Structures, Pergamon Press, Vol.41, No.1, 157-173.

Kunnath, S.K., Reinhorn, A.M. and Park, Y.J. (1990). "Analytical Modeling of Inelastic Seismic Response of R/C Structures." Journal of Structural Engineering, ASCE, Vol.116, No.4, 996-1017.

Mander, J.B. (1984). "Seismic Design of Bridge Piers." Ph.D. Dissertation. Department of Civil Engineering. University of Canterbury, New Zealand.

Otani, S. (1974). "SAKE: A Computer Program for Inelastic Response of RC Frames Subject to Earthquakes." Civil Engineering Studies. Technical Report No.SRS 413, University of Illinois, Urbana.

Park, R. and Paulay, T. (1975). "Reinforced Concrete Structures." John Wiley.

Park, Y.J., Reinhorn, A.M. and Kunnath, S.K. (1987). "IDARC: Inelastic Damage Analysis of Reinforced Concrete Frame - Shear-Wall Structures." Technical Report NCEER-87-0008, State University of New York at Buffalo.

Park, Y.J. Ang, A.H-S. and Wen, Y.K. (1984). "Seismic Damage Analysis and Damage-Limiting Design of R/C Buildings." Civil Engineering Studies. Technical Report No.SRS 516, University of Illinois, Urbana.

Paulay, T. and Priestley, M.J.N. (1992). "Seismic Design of Reinforced Concrete and Masonry Buildings." John Wiley.

Richart, F.E., Brandtzaeg, A. and Brown, R.L. (1928). "A Study of the Failure of Concrete Under Combined Compressive Stresses." University of Illinois Engineering Experimental Station, Bulletin No.185.

Stone, W.C. and Cheok, G.C. (1989). "Inelastic Behavior of Full-Scale Bridge Columns Subjected to Cyclic Loading." NIST Building Science Series 166, National Institute of Standards and Technology, Gaithersburg, MD.

Wight, J.K. (Editor) (1985). "Earthquake Effects on Reinforced Concrete Structures", U.S.-Japan Research, ACI Special Publication SP-84, American Concrete Institute, Detroit.

Wilson, E.L. and Habibullah, A. (1987). "Static and Dynamic Analysis of Multistory Buildings, Including P-Delta Effects." Earthquake Spectra, Vol.3, No.2.

Yunfei, H., Yufeng, C., Chang, S. and Bainian, H. (1986). "The Experimental Study of a Two-Bay Three Story Reinforced Concrete Frame Under Cyclic Loading." Proc., 8th Symposium on Earthquake Engineering, Roorkee, India.

APPENDIX A

USER GUIDE

IDARC, Version 3.0
USER GUIDE

INPUT FORMAT

A free format is used to read all input data. Hence, conventional delimiters (commas, blanks) may be used to separate data items. Standard FORTRAN variable format is used to distinguish integers and floating point numbers. Input data must, therefore, conform to the specified variable type.

NOTE: ***Provision is made for a line of text between each set of data items. Refer to the sample data files accompanying this Manual. No blank lines are to be input. A zero input will result in program default values, where applicable.***

DATA SET A: GENERAL INFORMATION

TITLE OF PROBLEM

TITLE	Alpha-numeric title, upto 80 characters.

CONTROL DATA **(SEE FIGURE A-1)**

USER_TEXT	***Reference information*: upto 80 characters of text**	
NSO,NFR,NCON,NSTL,NPDEL	NSO =	Number of stories
	NFR =	Number of frames
	NCON =	Number of different concrete material properties
	NSTL =	Number of different steel reinforcement properties
	NPDEL =	0 (ignore P-Delta effects)
	=	1 (include P-Delta effects)

NOTES: A structure must be decomposed into a series of parallel frames. Input is required only for non-identical frames, denoted here by the integer variable NFR. The entire group of frames can be defined in the IDARC L-I-J nodal locater system. This concept is shown graphically in Figure A-1. Three examples of different frame defintions are shown. In Figure A-1a, the four-story building made up of a total of four frames is assumed to have two pairs of identical frames, hence, only two of them need be input in IDARC (NFR=2). The cantilever beam/column shown in Figure A-1b is defined as a single-story structure with one column line. Likewise, the subassemblage shown in Figure 1c is defined as a 2-story structure with three column lines. The number of concrete and steel properties refer to the number of stress-strain envelopes to be input in Set B and Set C respectively.

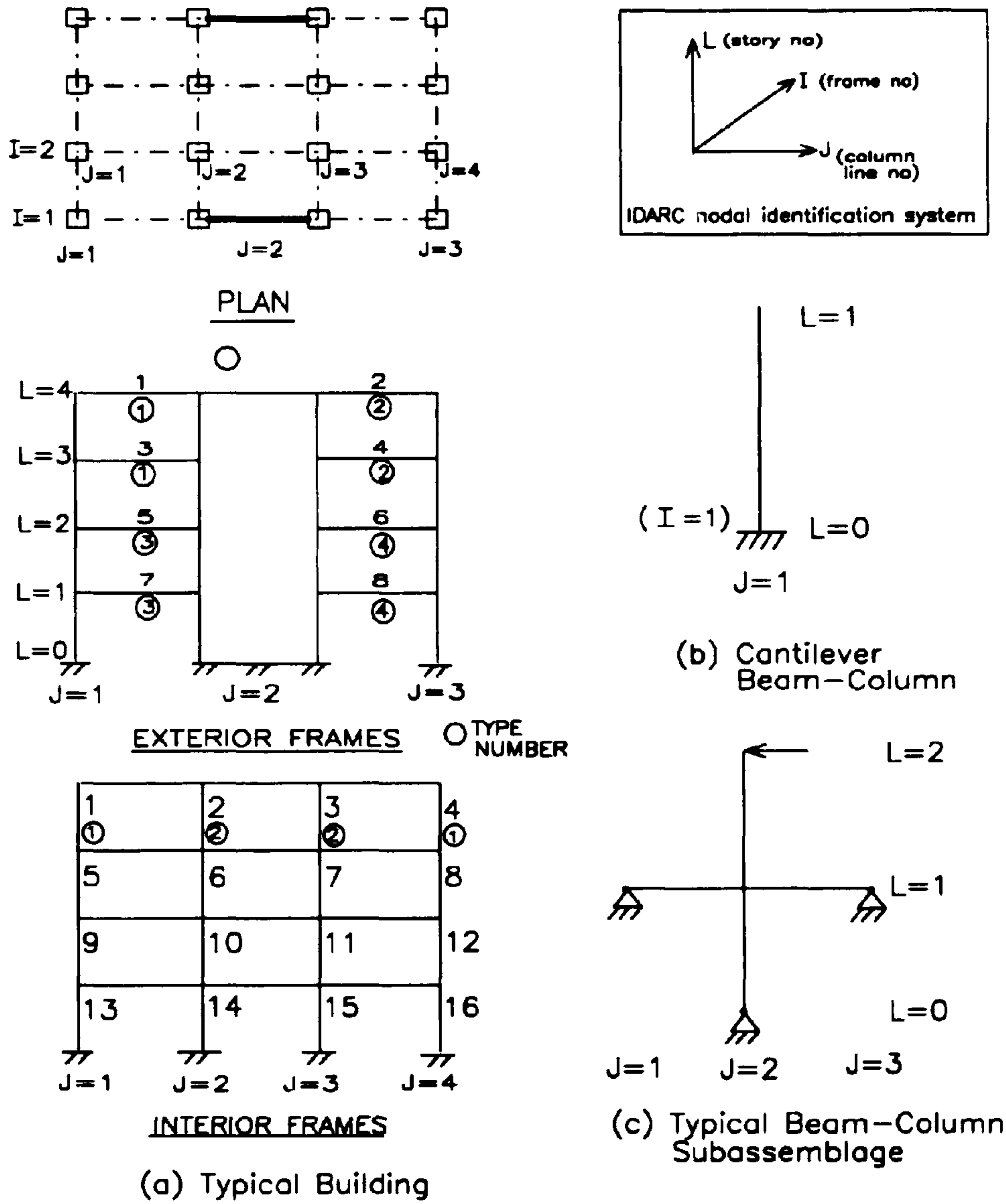


Figure A-1 Frame Discretization and Nodal Identification

ELEMENT TYPES (SEE FIGURE A-1)

USER_TEXT	***Reference information*: upto 80 characters of text**
MCOL,MBEM,MWAL,MEDG, MTRN,MSPR	MCOL = No. of types of columns MBEM = No. of types of beams MWAL = No. of types of shear walls MEDG = No. of types of edge columns MTRN = No. of types of transverse beams MSPR = No. of types of rotational springs

NOTES: Elements are grouped into identical sets based on cross-section data and initial conditions such as axial loads. For example, in the exterior frame shown in Figure A-1a, there are 8 columns. Typically, the exterior columns at each level will be identical, hence, only 4 column types need to be defined. The interior frame, assuming identical interior and exterior columns in each floor, will require only 8 column types to define all 16 elements, i.e., 2 types per each level as shown in the Figure.

ELEMENT DATA

USER_TEXT	***Reference information*: upto 80 characters of text**
NCOL,NBEM,NWAL,NEDG, NTRN,NSPR,NMR	NCOL = No. of columns NBEM = No. of beams NWAL = No. of shear walls NEDG = No. of edge columns NTRN = No. of transverse beams NSPR = No. of rotational springs NMR = No. of moment releases

NOTES: NMR is used to specify moment releases (hinge locations) at member ends. Releasing a moment at a member end results in a hinge condition at that end thereby disallowing moments to develop at the section.

UNIT SYSTEM

USER_TEXT	***Reference information*: upto 80 characters of text**
IU	System of units = 1, inch,kips = 2, mm, kN

DEFAULT SYSTEM OF UNITS: inch, kip

A zero input for IU will result in the use of *inch* and *kip* units.

FLOOR ELEVATIONS (SEE FIGURE A-2)

USER_TEXT — ***Reference information*: upto 80 characters of text**

HIGT(1),HIGT(2)...HIGT(NSO) — Elevation of each story from the base, beginning with the first floor level.

DESCRIPTION OF IDENTICAL FRAMES

USER_TEXT — ***Reference information*: upto 80 characters of text**

NDUP(1),NDUP(2)...NDUP(NFR) — Number of duplicate frames for each of the NFR frames

NOTES: In the sample structure shown in Figure A-1, there are four frames. However, the two interior frames are identical as are the exterior frames. In this case, NFR=2, and NDUP(1) = NDUP(2) = 2.

PLAN CONFIGURATION

USER_TEXT — ***Reference information*: upto 80 characters of text**

NVLN(1),NVLN(2)...NVLN(NFR) — Number of column lines (or J-locater points) in each frame.

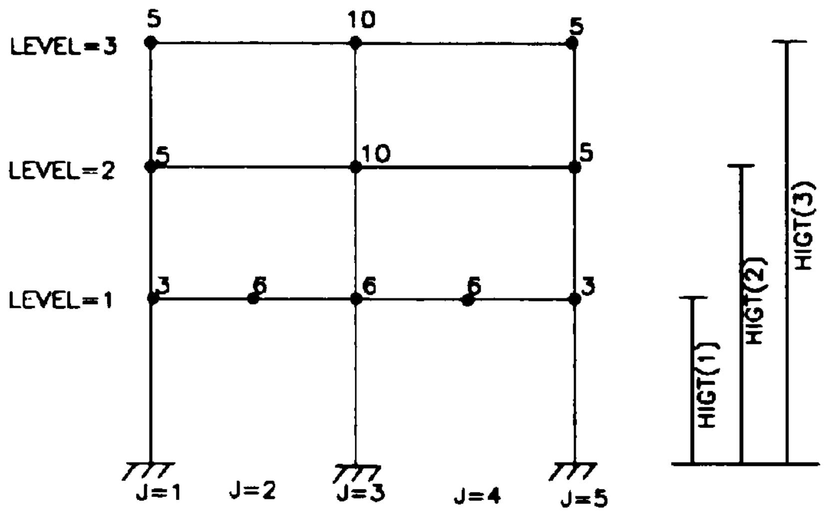
NOTES: *A set of NVLN points for each frame should define completely the column lines necessary to specify every vertical element in that frame. If a beam element is subdivided into two or more segments, then the number of column lines specified must include these internal beam nodes as well.*

NODAL WEIGHTS **(SEE FIGURE A-2)**

USER_TEXT — ***Reference information*: upto 80 characters of text**

LEVEL, IFR(1), WVT(1)...WVT(NVLN(1))	**LEVEL = Story level number**
IFR(2), WVT(1)...WVT(NVLN(2))	**IFR(J) = Frame number**
.....repeat for NFR frames	**WVT(K) = Nodal weight**

(next level)
repeat for NSO levels



FRAME #1
(numbers shown at nodes = nodal weights)

INPUT DATA:

```
1,  1,  3.0, 6.0,  6.0, 6.0, 3.0
2,  1,  5.0, 0.0, 10.0, 0.0, 5.0
3,  1,  5.0, 0.0, 10.0, 0.0, 5.0
```

Figure A-2 Floor Heights and Nodal Weights

ENVELOPE GENERATION OPTION

USER_TEXT	***Reference information*: upto 80 characters of text**
IUSER	Code for specification of user properties = 0, requires IDARC generated envelopes for atleast one element = 1, complete moment-curvature envelope data to be provided by user

DATA SET B: CONCRETE PROPERTIES (SEE FIGURE A-3)

SKIP THIS INPUT IF IUSER .NE. 0

USER_TEXT	***Reference information*: upto 80 characters of text**
IM,FC,EC,EPS0,FT,EPSU,ZF	*Characteristics of concrete stress-strain curve*: IM = Concrete type number FC = Unconfined compressive strength EC = Initial Young's Modulus of concrete EPS0 = Strain at max. strength of concrete (%) FT = Stress at tension cracking EPSU = Ultimate strain in compression (%) ZF = Parameter defining slope of falling branch

repeat for each of the NCON concrete types

DEFAULT VALUES:

EC $= 57 \cdot \sqrt{(FC \cdot 1000)}$ *ksi ; EPS0 = 0.2% ; FT = 0.12*FC ;*

EPSU and ZF are derived from Equation (3.12) and depends on section data.

DATA SET C : REINFORCEMENT PROPERTIES (SEE FIGURE A-4)

SKIP THIS INPUT IF IUSER .NE. 0

USER_TEXT	***Reference information*: upto 80 characters of text**
IM,FS,FSU,ES,ESH,EPSH	*Characteristics of steel stress-strain curve*: IM = Steel type number. FS = Yield strength FSU = Ultimate strength ES = Modulus of elasticity ESH = Modulus of strain hardening EPSH = Strain at start of hardening (%)

repeat for each of the NSTL steel types

DEFAULT VALUES:

*FSU = 1.4 * FS ; ES = 29,000 ksi ; ESH = (ES / 60) ksi ; EPSH = 3.0%*

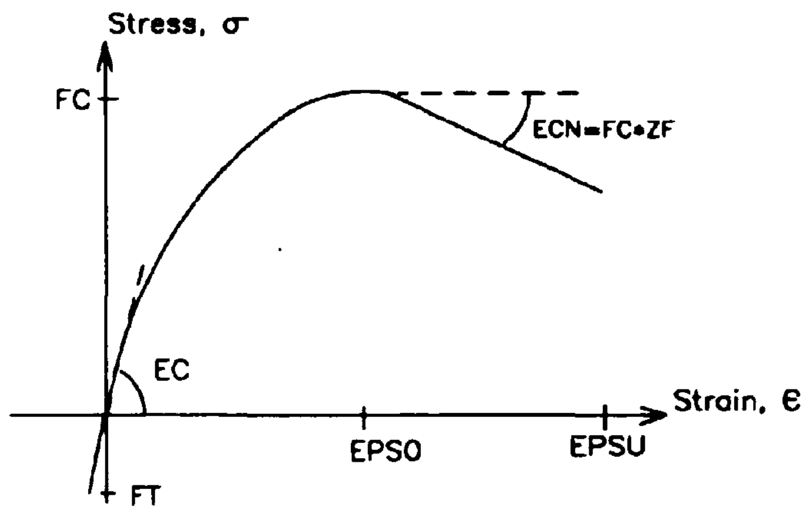


Figure A-3 Stress-Strain Curve for Unconfined Concrete

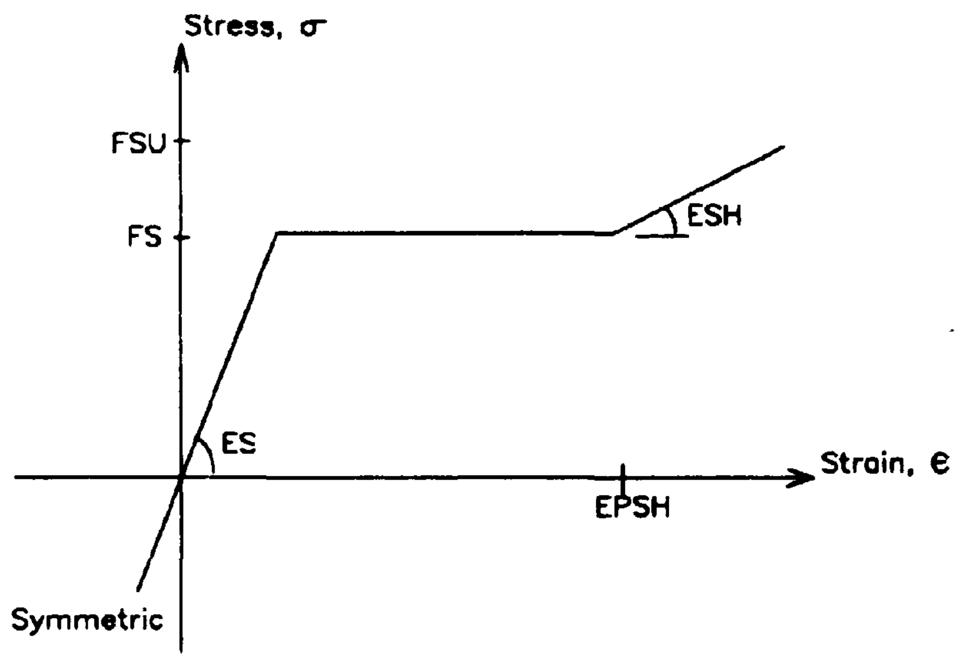


Figure A-4 Stress-Strain Curve for Reinforcing Bars

DATA SET D: <u>HYSTERETIC MODELING RULES</u> **(SEE FIGURE A-5)**

USER_TEXT — ***Reference information*: upto 80 characters**

NHYS — Number of types of hysteretic rules

<u>HYSTERETIC MODEL PARAMETERS</u>

IR, HC, HBE, HBD, HS

IR = Parameter Set Number
HC = Stiffness degrading coefficient
HBE = Energy-based strength decay parameter
HBD = Ductility-based strength decay parameter
HS = Target slip or crack-closing parameter

(NHYS lines of data)

<u>DEFAULT VALUES:</u> HC = 2.0 ; HBE = 0.10 ; HBD = 0.0; HS = 1.0 ;
(assigned only if HC,HBE,HBD and HS are all input as zero)

NOTES: *Hysteretic behavior is specified at both ends of each member. Access to experimental results of the cyclic force-deformation characteristics of components typical to the structure being analysed provides the best means of specifying the above degrading parameters. Table A-1 and Figure A-5 provide a number of qualitative insights into modeling of the hysteretic parameters. The loops shown in Figure A-5 are only meant to show the relative effects of changing the parameters. The general meaning of the parameters can be characterised as follows: An increase in **HC** retards the amount of stiffness degradation; an increase in **HBD,HBE** accelerates the strength deterioration; and an increase in **HS** reduces the amount of slip. (Also refer to Section 3.3 of this report)*

Table A-1. Typical Range of Values for Hysteretic Parameters

Parameter	Meaning	Value	Effect
HC	Stiffness degrading parameter	0.10 2.00 10.0	Severe degradation Nominal degradation (<u>default</u>) Negligible degradation
HBE	Strength degrading parameter (energy-controlled)	0.0 0.10 0.40	No deterioration Nominal deterioration (<u>default</u>) Severe deterioration
HBD	Strength degrading parameter (ductility-based)	0.0 0.10 0.40	(<u>default</u>) Nominal deterioration Severe deterioration
HS	Slip or crack-closing parameter	0.1 0.5 1.0	Extremely pinched loops Nominal pinching No pinching (<u>default</u>)

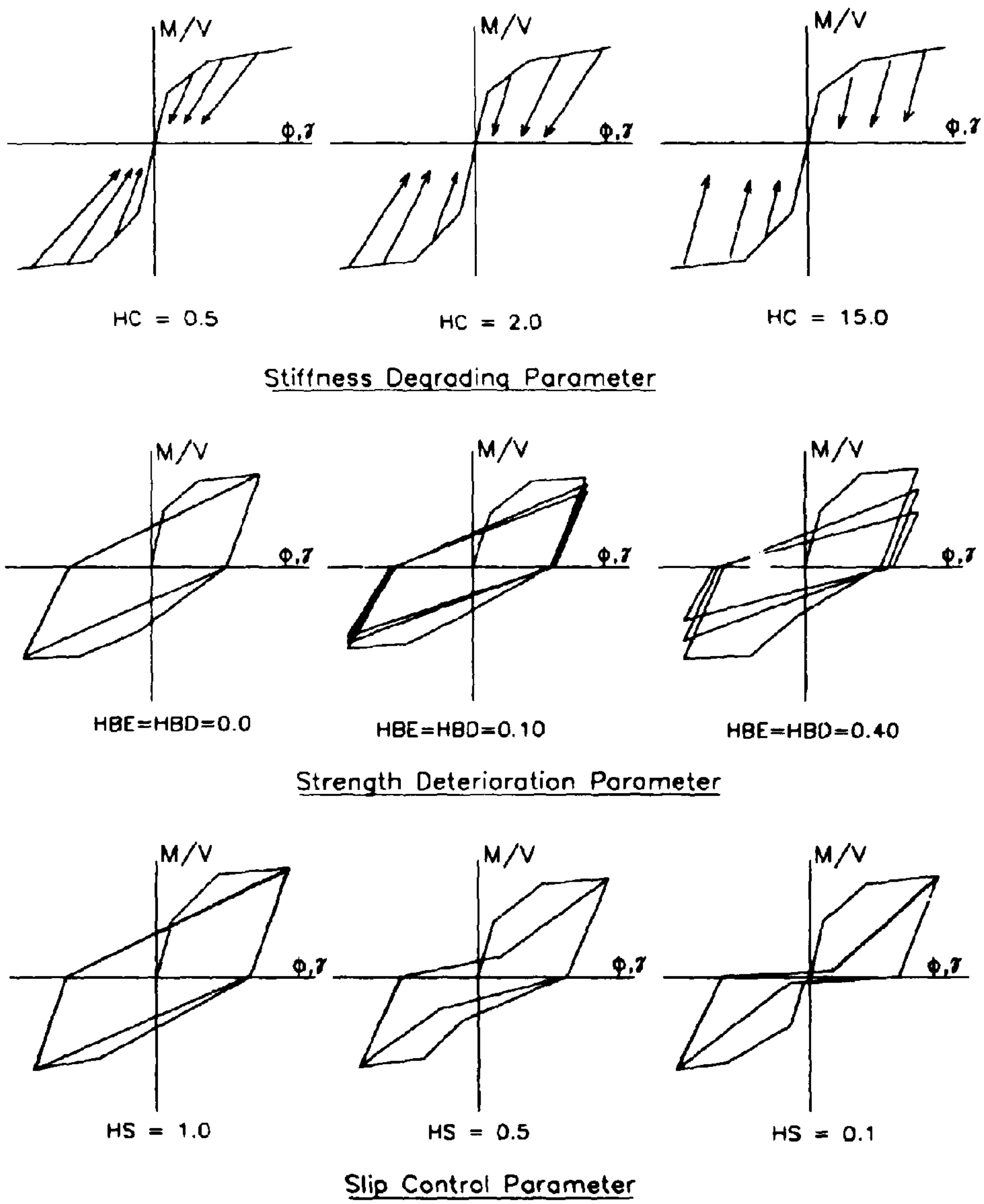


Figure A-5 Qualitative View of Effects of Degrading Parameters on Hysteretic Behavior

DATA SET E: COLUMN PROPERTIES

SKIP THIS INPUT IF THE STRUCTURE HAS NO COLUMNS

USER_TEXT — ***Reference information*: upto 80 characters of text**

IUCOL — Type of column input
= 0; Section dimensions and reinf to be specified
= 1; Moment-curvature envelope to be specified

IF IUCOL = 1, GO TO SET E3

USER_TEXT — ***Reference information*: upto 80 characters of text**

For each column type, input the following:

ICTYPE — Type of column
= 1; rectangular *(DEFAULT)*
= 2; circular

IF ICTYPE = 2, GO TO SET E2

SET E1: ICTYPE=1: RECTANGULAR COLUMN DATA **(SEE FIGURE A-6)**

General data: KC,IMC,IMS,AN,AMLC,RAMC1,RAMC2
Bottom section: KHYSC, D, B, DC, AT, HBD, HBS, CEF
Top section: If KHYSC for bottom section is input with negative sign, section is symmetric, hence do not input top section data ELSE, repeat as above, starting with *KHYSC*

KC = Column type number
IMC = Concrete type number
IMS = Steel type number
AN = Axial load
AMLC = Center-to-center column height
RAMC1 = Rigid zone length at bottom
RAMC2 = Rigid zone length at top

Column data for bottom section:

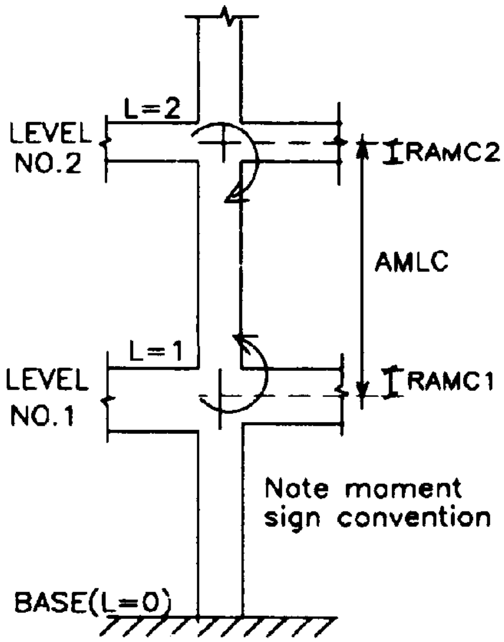
KHYSC = Hysteretic rule number **(may be negative)***
D = Depth of column
B = Width of column
DC = Distance from centroid of reinforcement to face of column
AT = Area of reinforcement on one face
HBD =Hoop bar diameter
HBS =Hoop bar spacing
CEF = Effectiveness of column confinement

Column data for top section: (similar to bottom)

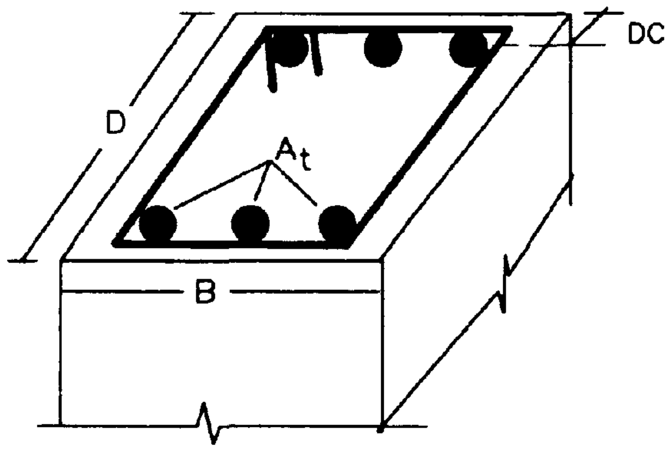
Skip this input if KHYSC is negative for bot section

*** An input value of KHYSC with negative sign for the bottom section will result in symmetric values being assigned to the top section.**

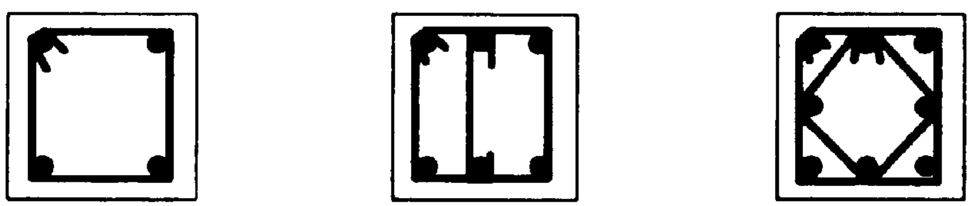
Return to input of ICTYPE. When done, go to SET F



Typical Column Line



Typical Column Cross-Section



Minimal Confinement
CEFF = 0.5

Nominal Confinement
CEFF = 0.66

Well Confined
CEFF = 1.0

Effectiveness of Confinement for Some Typical Hoop Arrangements

Figure A-6 Rectangular Column Input Details

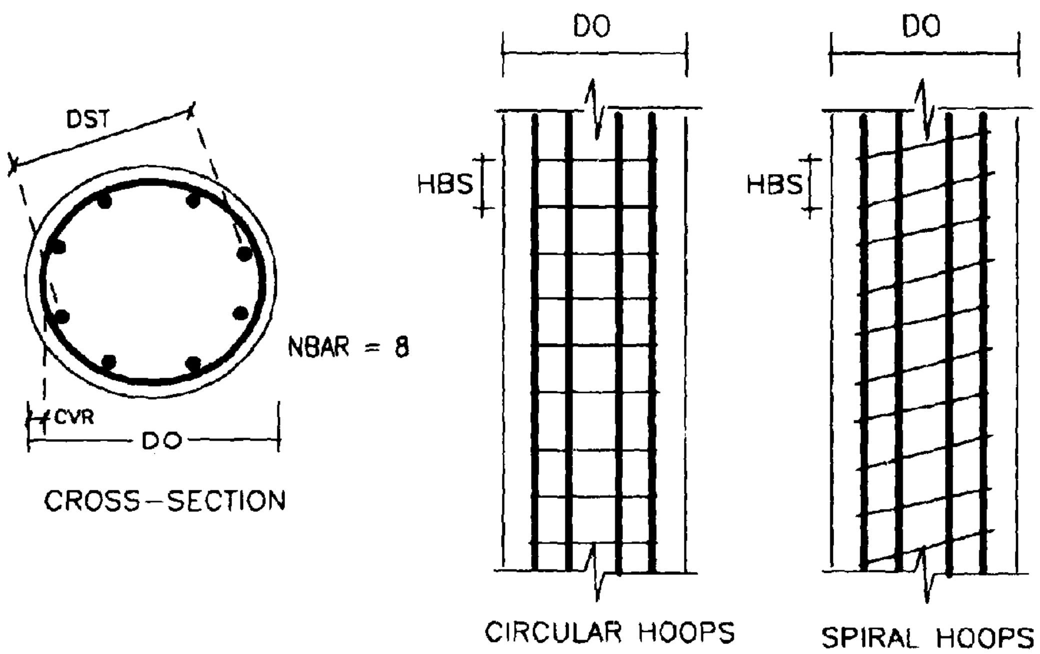


Figure A-7 Circular Column Input Details

__SET E2: ICTYPE = 2: CIRCULAR COLUMN INPUT__ **(SEE FIGURE A-7)**

KC,IMC,IMS,KHYSC,AMLC,RAMC1,RAMC2
AN,DO,CVR,DST,NBAR,BDIA,HBD,HBS

KC = Colum Type number
IMC = Concrete type number
IMS = Steel type number
KHYSC = Hysteretic Rule number
AMLC = Center-to-center column height
RAMC1 = Rigid arm bottom
RAMC2 = Rigid arm top
AN = Axial load on the column
DO = Outer diameter of column
CVR = Cover to center of hoop bar
DST = Distance between centers of long. bars
NBAR = Number of longitudinal bars
BDIA = Diameter of longitudinal bar
HBD = Diameter of hoop bar
HBS = Spacing of hoop bars

Return to input of ICTYPE. When done go to SET F.

SET E3: USER INPUT PROPERTIES (Rectangular or Circular) **(SEE FIGURE A-8)**

USER_TEXT ***Reference information*: upto 80 characters of text**

General Data: KC, AMLC, RAMC1, RAMC2

Bottom section: KHYSC, EI,EA,GA, PCP,PYP,UYP,UUP,EI3P,
PCN,PYN,UYN,UUN,EI3N

Top section: If KHYSC for bottom section is input with negative sign, section is symmetric, hence do not input top section data
ELSE, repeat as above, starting with *KHYSC*

KC = Column type number
AMLC = Column Length
RAMC1 = Rigid Arm (Bottom)
RAMC2 = Rigid Arm (Top)

Data for column bottom

KHYSC = Hysteretic rule number (may be negative)*
EI = Initial Flexural Rigidity (EI)
EA = Axial Stiffness (EA/L)
GA = Shear Stiffness (Shear modulus*Shear Area)

Positive properties:

PCP = Cracking Moment (positive)
PYP = Yield Moment (positive)
UYP = Yield Curvature (positive)
UUP = Ultimate Curvature (positive)
EI3P = Post Yield Flexural Stiffness (positive)

Negative properties:

PCN = Cracking Moment (negative)
PYN = Yield Moment (negative)
UYN = Yield Curvature (negative)
UUN = Ultimate Curvature (negative)
EI3N = Post yield Flexural Stiffness (negative)

Data for column top (similar to BOT section)

Skip this input if KHYSC is negative for bottom section

*** An input value of KHYSC with negative sign for the bottom section will result in symmetric values being assigned to the top section.**

Repeat for each column type, starting with General Data (SET E3)

SET F: BEAM PROPERTIES

SKIP THIS INPUT IF THE STRUCTURE HAS NO BEAMS

USER_TEXT ***Reference information*: upto 80 characters of text**

IUBEM Type of beam input
= 0; Section dimensions, and reinf details specified
= 1; Moment-curvature envelope directly specified

IF IUBEM = 1, GO TO SET F2

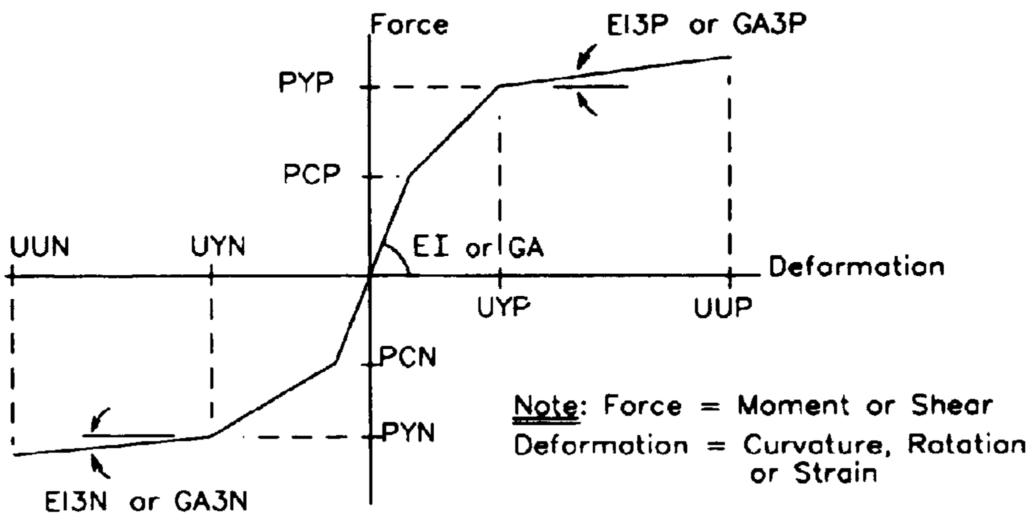


Figure A-8 Notation for User-Input Trilinear Envelopes

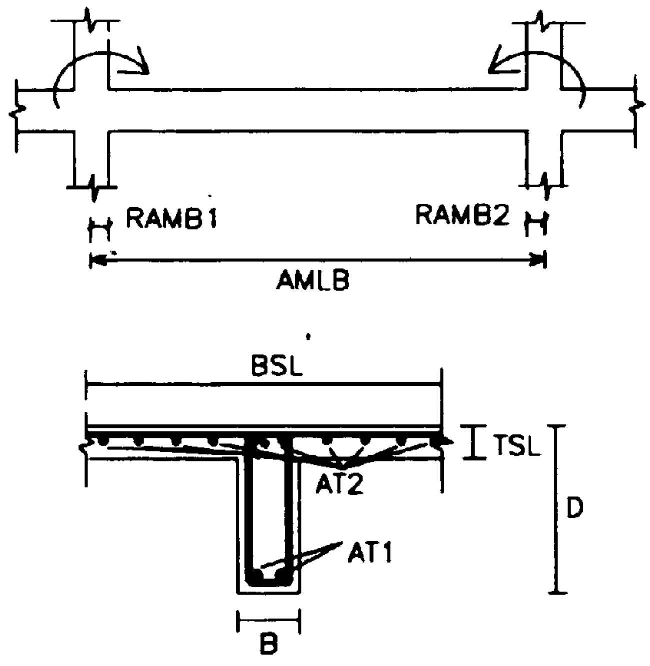


Figure A-9 Input Details for Beam-Slab Sections

SET F1: SECTION DIMENSIONS SPECIFIED **(SEE FIGURE A-9)**

USER_TEXT — ***Reference information*: upto 80 characters of text**

General data: KB,IMC,IMS,AMLB,RAMB1,RAMB2,

Left section: KHYSB, D, B, BSL TSL, BC, AT1, AT2, HBD, HBS

Right section: If KHYSB for left section is input with negative sign, section is symmetric, hence do not input right section data, ELSE, input right section data starting with *KHYSB*

KB = Beam type number
IMC = Concrete type number
IMS = Steel type number
AMLB = Member length
RAMB1 = Rigid zone length (left)
RAMB2 = Rigid zone length (right)

Data for beam section on LEFT

KHYSB = Hysteretic rule number
(may be negative)*
D = Overall depth
B = Lower width
BSL = Effective slab width
TSL = Slab thickness
BC = Cover to centroid of steel
AT1 = Area of bottom bars
AT2 = Area of top bars
HBD = Diameter of stirrup bars
HBS = Spacing of stirrups

Data for beam section on RIGHT

(Similar to definitions for left section)
Skip this input if KHYSB is negative for left section

***An input value of KHYSB with negative sign for the left section will result in symmetric values being assigned to the right section.**

Repeat for each beam type starting with General Data (SET F1)
When done, go to **SET** G

SET F2: USER INPUT PROPERTIES **(SEE FIGURE A-8)**

USER_TEXT *Reference information*: **upto 80 characters of text**

General Data: KB, AMLB, RAMB1, RAMB2
Left section: KHYSB, EI,GA, PCP,PYP,UYP,UUP,EI3P, PCN,PYN,UYN,UUN,EI3N
Right section: If KHYSB for left section is input with negative sign, section is symmetric, hence do not input right section data
ELSE, repeat as above, starting with *KHYSB*

KB = Beam type number
AMLB = Column Length
RAMB1 = Rigid Arm (Left)
RAMB2 = Rigid Arm (Right)

Data for beam - left section

KHYSB = Hysteretic rule number (may be negative)*
EI = Initial Flexural Rigidity
GA = Shear Stiffness (Shear modulus*Shear Area)

Positive properties:

PCP = Cracking Moment (positive)
PYP = Yield Moment (positive)
UYP = Yield Curvature (positive)
UUP = Ultimate Curvature (positive)
EI3P = Post Yield Flexural Stiffness (positive)

Negative properties:

PCN = Cracking Moment (negative)
PYN = Yield Moment (negative)
UYN = Yield Curvature (negative)
UUN = Ultimate Curvature (negative)
EI3N = Post yield Flexural Stiffness (negative)

Data for beam - right section

(Similar to definitions for left section)
Skip this input if KHYSB is negative for left section

*** An input value of KHYSB with negative sign for the left section will result in symmetric values being assigned to the right section.**

Repeat for each beam type, starting with General Data (SET F2)

SET G: SHEAR WALL PROPERTIES **(SEE FIGURE A-10 AND A-11)**

SKIP THIS INPUT IF THE STRUCTURE HAS NO SHEAR WALLS

USER_TEXT *Reference information*: **upto 80 characters of text**

IUWAL Type of wall input
= 0; Section dimensions and reinf details
= 1; Moment-curvature and shear-strain envelopes

IF IUWAL = 1, GO TO SET G2

SET G1: SECTION DIMENSIONS SPECIFIED

USER_TEXT *Reference information*: **upto 80 ch racters of text**

General Data: KW,IMC,KHYSW(1),KHYSW(2),KHYSW(3),AN,AMLW,NSECT

KW = Shear wall type number
IMC = Concrete type number
KHYSW(1) = Hysteretic Rule Number (bottom)
KHYSW(2) = Hysteretic Rule Number (top)
KHYSW(3) = Hysteretic Rule Number (shear)
AN = Axial load
AMLW = Height of shear wall
NSECT = Number of Sections

For each of the *NSECT* sections, input the following

KS,IMS,DWAL,BWAL,PT,PW

KS = Section number
IMS = Steel type number
DWAL = Depth of section
BWAL = Width of section
PT = Vertical reinforcement ratio (%)
PW = Horizontal reinf ratio (%)

.repeat NSECT times

Repeat for each wall type starting with General Data; When done go to **SET H**

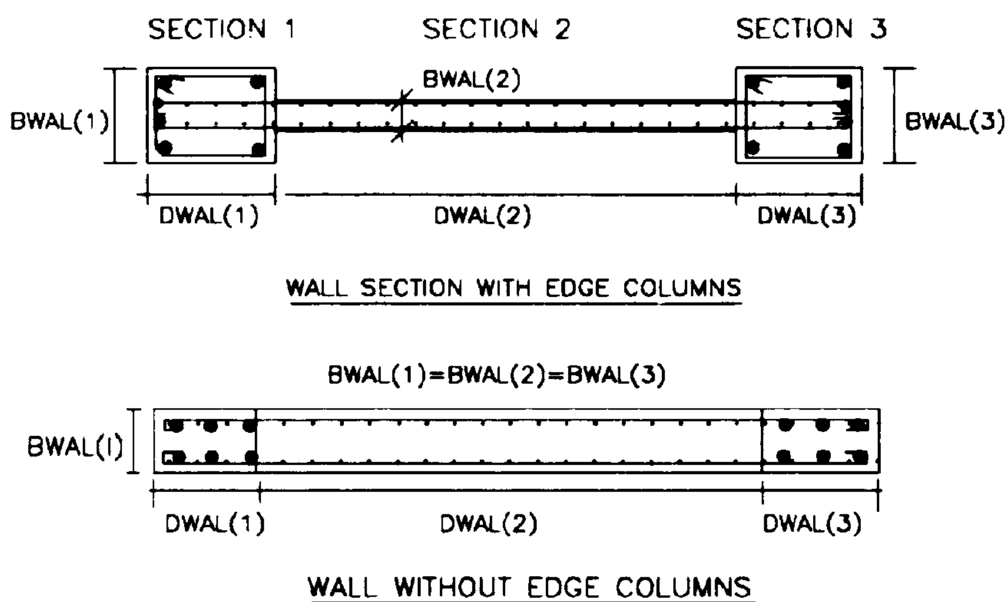


Figure A-10 Typical Input Detail for Shear Wall Sections

SET G2: USER INPUT PROPERTIES **(SEE FIGURE A-8)**

USER_TEXT ***Reference information*: upto 80 characters of text**

General Data: KW, AMLW, EAW

Flexure: BOT: KHYSW, EI,PCP,PYP,UYP,UUP,EI3P, PCN,PYN,UYN,UUN,EI3N

Flexure: TOP: If KHYSW for bottom section is input with negative sign, section is symmetric, hence, do not input top section data
ELSE, repeat as above, starting with *KHYSW*

Shear: KHYSW, GA,PCP,PYP,UYP,UUP,GA3P, PCN,PYN,UYN,UUN,GA3N

KW = Wall type number
AMLW = Wall length
EAW = Axial Stiffness (EA/L)

Data for wall section at bottom

KHYSW = Hysteretic rule number (may be negative)*
EI = Initial flexural stiffness (EI)

Positive properties:

PCP = Cracking Moment (positive)
PYP = Yield Moment (positive)
UYP = Yield Curvature (positive)
UUP = Ultimate Curvature (positive)
EI3P = Post Yield Flexural Stiffness (positive)

Negative properties:

PCN = Cracking Moment (negative)
PYN = Yield Moment (negative)
UYN = Yield Curvature (negative)
UUN = Ultimate Curvature (negative)
EI3N = Post yield Flexural Stiffness (negative)

Data for wall section at top (similar to bottom section)

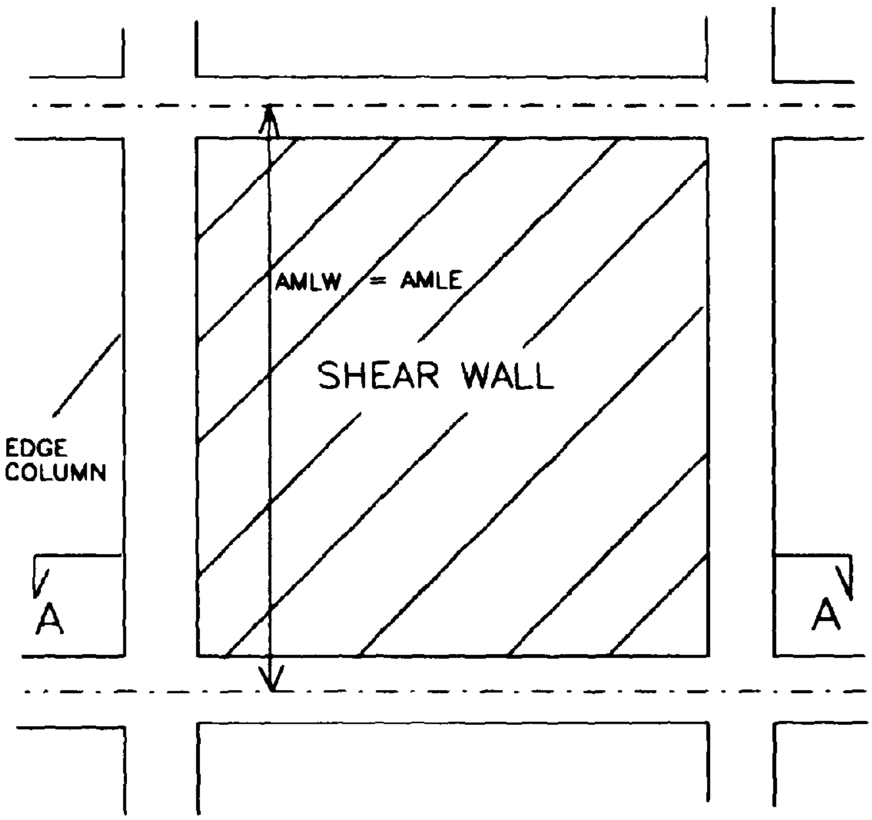
Skip this input if KHYSW is negative for bot section

*** An input value of KHYSW with negative sign for the bottom section will result in symmetric values being assigned to the top section.**

Data for shear properties:

KHYSW = Hysteretic Rule Number
GA = Initial Shear Stiffness (shear modulus*area)
PCP = Cracking Shear (positive)
PYP = Yield Shear (positive)
UYP = Yield Shear strain (positive)
UUP = Ultimate Shear strain (positive)
GA3P =Post Yield Shear Stiffness (positive)
PCN = Cracking Shear (negative)
PYN = Yield Shear (negative)
UYN = Yield Shear strain (negative)
UUN = Ultimate Shear strain (negative)
GA3N =Post Yield Shear Stiffness (negative)

Return to start of General Data (SET G2). Repeat for each wall type



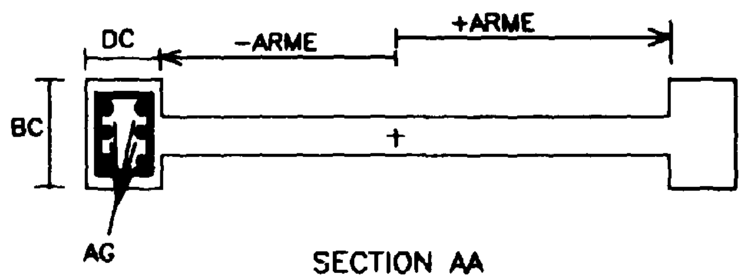


Figure A-11 Shear Wall and Edge Column Details

SET H: **EDGE COLUMN PROPERTIES** **(SEE FIGURE A-11)**

SKIP THIS INPUT IF THE STRUCTURE HAS NO EDGE COLUMNS

Do not duplicate edge column data if already input in SHEAR WALL data. See Section 2.1 for information pertaining to modeling walls with edge columns.

USER_TEXT — ***Reference information*: upto 80 characters of text**

KE,IMC,IMS,AN,DC,BC,AG,AMLE,ARME

- KE = Edge column type number
- IMC = Concrete type number
- IMS = Steel type number
- AN = Axial load
- DC = Depth of edge column
- BC = Width of edge column
- AG = Gross area of main bars
- AMLE = Member length
- ARME = Arm length

Repeat for each of MEDG elements starting with edge column type number.

SET I: **TRANSVERSE BEAM PROPERTIES** **(SEE FIGURE A-12)**

THIS INPUT NOT REQUIRED IF STRUCTURE HAS NO TRANSVERSE BEAMS

USER_TEXT — ***Reference information*: upto 80 characters of text**

KT,AKV,ARV,ALV

- KT = Transverse beam type number
- AKV = Vertical Stiffness
- ARV = Torsional Stiffness
- ALV = Arm length

Repeat for each of MTRN elements

NOTES:

1. Transverse elements are assumed to remain elastic. The degree of fixity at the ends will depend on the state of the joint and the state of the members that frame into the joint before and during the application of load. If the entire region is expected to stay elastic, then the vertical stiffness should be computed as : $AKV = 12EI/L^3$ *. In the extreme case that one of ends do not transmit stiffness due to yielding of adjoining members or deterioration of the joint, then* $AKV = 3EI/L^3$ *. An intermediate value is a good average approximation.*

2. If duplicate frames are present, extreme care should be taken in specifying transverse beam properties. The program multiplies the input values by the number of duplicate frames to which they are attached. For example, for the frames shown in Figure A-1, NDUP(1) = NDUP(2) = 2. The program will factor the input stiffness values by (NDUP(1)+NDUP(2))=4.0. Input stiffnesses should, therefore, be modified to account for this effect. If the modeling of transverse elements is crucial to the analysis, the use of duplicate frames should be avoided.

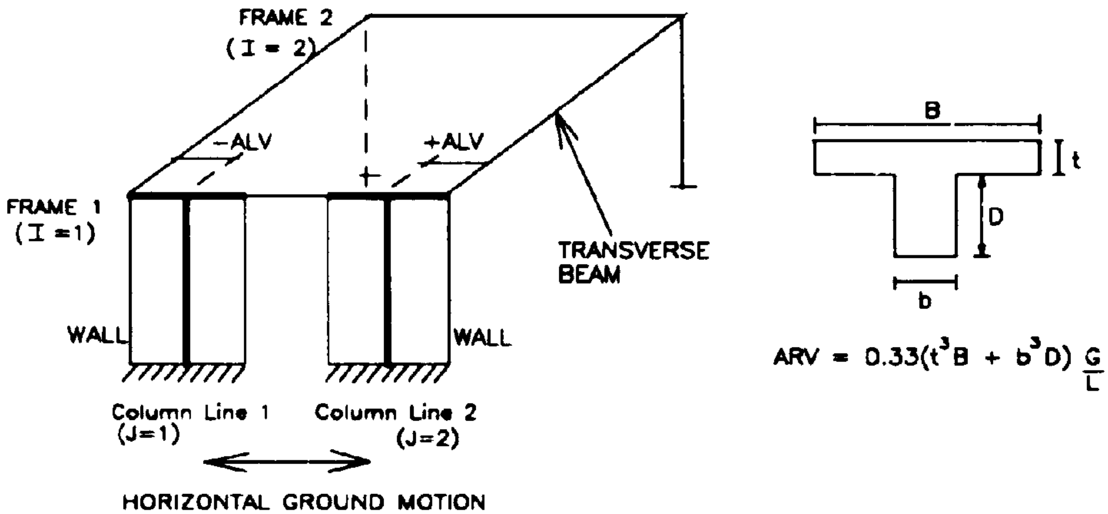


Figure A-12 Transverse Beam Input

SET J: <u>ROTATIONAL SPRING PROPERTIES</u> (SEE FIGURE A-8)

THIS INPUT NOT REQUIRED IF ROTATIONAL SPRINGS ARE NOT SPECIFIED

USER_TEXT **_Reference information_: upto 80 characters of text**

General Data: KHYSR, EI,PCP,PYP,UYP,UUP,EI3P,
PCN,PYN,UYN,UUN,EI3N

KHYSR = Hysteretic Rule Number
EI = Initial Rotational Stiffness (M/θ)

<u>Positive properties</u>:

PCP = Cracking moment (positive)
PYP = Yield moment (positive)
UYP = Yield rotation (positive, radians)
UUP = Ultimate rotation (positive, radians)
EI3P = Post-yield stiffness ratio (positive)

<u>Negative properties</u>:

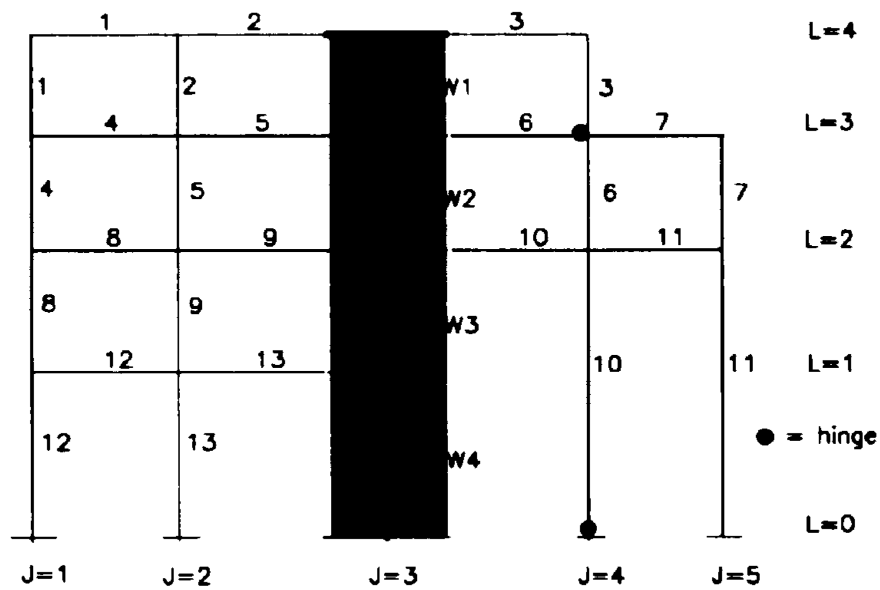
PCN = Cracking moment (negative)
PYN = Yield moment (negative)
UYN = Yield rotation (negative)
UUN = Ultimate rotation capacity (negative)
EI3N = Post yield stiffness ratio (negative)

repeat for each spring type

NOTES: Spring properties, unlike other element types, are specified in terms of moment and rotation (in radians). The envelope follows the same nonsymmetric trilinear pattern as shown in Figure A-8.

ELEMENT CONNECTIVITY INPUT

NOTE: *Element connectivity is established through the 3 positional locaters described in Figure A-1: a story level, a frame number and a column line. The L position locater (or story level) varies from 0 to the number of stories; the I position locater (or frame number) varies from 1 to the number of frames; and the J locater varies from 1 to the number of NVLN positions (column lines) for each frame. The hypothetical structure shown below is used to demonstrate the input format. Only a representative data set is shown.*



Element Type	Number	Type	IC	JC	LBC	LTC
COLUMNS	1	1	1	1	3	4
	2	2	1	2	3	4
	10	8	1	4	0	2
	Number	Type	LB	IB	JLB	JRB
BEAMS	1	1	4	1	1	2
	2	2	4	1	2	3
	6	3	3	1	3	4
	Number	Type	IW	JW	LBW	LTW
WALLS	1	1	1	3	3	4
	2	2	1	3	2	3

Figure A-13. Element Connectivity for Sample Structure

SET K: COLUMN CONNECTIONS **(SEE FIGURE A-13)**

SKIP THIS INPUT IF THE STRUCTURE HAS NO COLUMNS

USER_TEXT	***Reference information*: upto 80 characters of text**
M,ITC,IC,JC,LBC,LTC	M = Column number ITC = Column type number IC = Frame number JC = Column Line number LBC = Story level at bottom of column LTC = Story level at top of column

(NCOL lines of data)

NOTES: Input is required for each of the NCOL columns.

SET L: BEAM CONNECTIVITY **(SEE FIGURE A-13)**

SKIP THIS INPUT IF STRUCTURE HAS NO BEAMS

USER_TEXT	***Reference information*: upto 80 characters of text**
M,ITB,LB,IB,JLB,JRB	M = Beam number ITB = Beam type number LB = Story level IB = Frame number JLB = Column Line number of left section JRB = Column Line number of right section

(NBEM lines of data)

NOTES: Input is required for each of the NBEM beams.

SET M: SHEAR WALL CONNECTIVITY **(SEE FIGURE A-13)**

SKIP THIS INPUT IF STRUCTURE HAS NO SHEAR WALLS

USER_TEXT	***Reference information*: upto 80 characters of text**
M,ITW,IW,JW,LBW,LTW	M = Wall number ITW = Wall type number IW = Frame number JW = Column line number LBW = Story level at bottom LTW = Story level at top

(NWAL lines of data)

NOTES: Input is required for each of the NWAL shear walls.

SET N: EDGE COLUMN CONNECTIVITY

SKIP THIS INPUT IF STRUCTURE HAS NO EDGE COLUMNS

USER_TEXT	***Reference information*: upto 80 characters of text**
M,ITE,IE,JE,LBE,LTE	M = Edge column number ITE = Edge column type number IE = Frame number JE = Column line number LBE = Story level at bottom of column LTE = Story level at top of column

(NEDG lines of data)

SET O: TRANSVERSE BEAM CONNECTIVITY

SKIP THIS INPUT IF STRUCTURE HAS NO TRANSVERSE BEAMS

USER_TEXT	***Reference information*: upto 80 characters of text**
M,ITT,LT,IWT,JWT,IFT,JFT	M = Transverse beam number ITT = Transverse beam type number LT = Story level IWT = Frame number of origin of transverse beam* JWT = Column line of origin of transverse beam* IFT = Frame number of connecting wall or column JFT = Column line of connecting wall or column

(NTRN lines of data)

NOTES: **For beam-to-wall connections, IWT and JWT refer to the I,J locations of the wall.*

SET P: <u>SPRING LOCATIONS</u> **(SEE FIGURE A-14)**

SKIP THIS INPUT IF ROTATIONAL SPRINGS ARE NOT SPECIFIED

USER_TEXT	***Reference information***: **upto 80 characters of text**
M, ISP, JSP, LSP, KSPL	M = Spring number ISP = Frame number JSP = Column line number LSP = Story level KSPL = Relative spring location as follows: *Code for KSPL ->* = 1, spring on beam, left of joint = 2, spring on column, top of joint = 3, spring on beam, right of joint = 4, spring on column, bottom of joint

(NSPR lines of data)

NOTE: The number of springs at a joint is limited to one less than the total number of members framing into the joint)

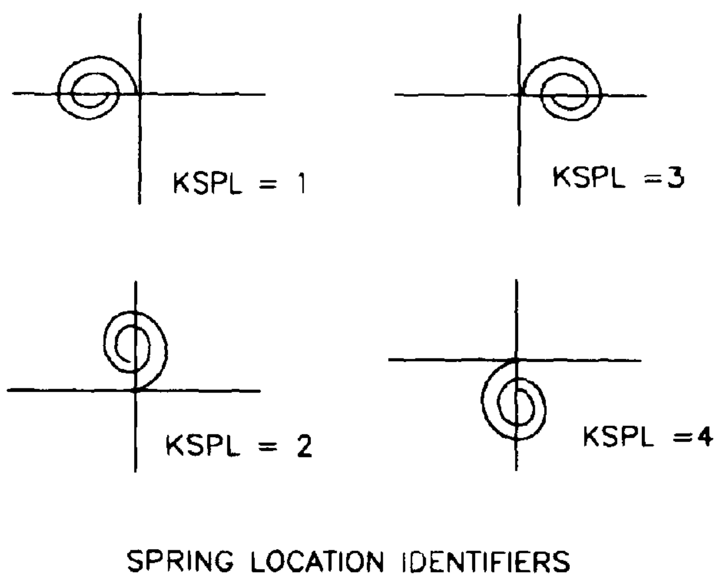


Figure A-14 Specification of Discrete Inelastic Springs

SET Q: <u>MOMENT RELEASES</u> (SEE FIGURE A-15)

SKIP THIS INPUT IF MOMENT RELEASES ARE NOT REQUIRED, NMR = 0

USER_TEXT	***Reference information*: upto 80 characters of text**	
IDM, IHTY, INUM, IREG	IDM =	ID number
	IHTY =	Element type using following code CODE: 1 = COLUMN 2 = BEAM 3 = WALL
	INUM =	Column, Beam or Wall number
	IREG =	Location of hinge or moment release = 1, BOTTOM or LEFT = 2, TOP or RIGHT
(NMR lines of data)		

<u>Sample Input (with reference to Fig A-13)</u>

IDM	IHTY	INUM	IREG
1	1 (col)	10 (col #)	1 (bot)
2	2 (beam)	6 (beam#)	2 (right)

Figure A-15 Specification of Moment Releases

ANALYSIS OPTIONS:

USER_TEXT	***Reference information*: upto 80 characters of text**
IOPT	Option for continuing analysis = 0 , STOP (Data check mode) = 1 , Inelastic incremental analysis with static loads = 2 , Monotonic "pushover" analysis including static loads (if specified) = 3 , Inelastic dynamic analysis including static loads (if specified) = 4 , Quasi-static cyclic analysis including static loads (if specified)

Notes: *It is generally advisable to use the "data check" mode for the first trial run of a new data set. The program performs only minimal checking of input data. Structural elevation plots generated by IDARC help identify errors in connectivity specification. Since IDARC prints all input data almost immediately after they are read, the task of detecting the source of input errors is generally expedited. It is also important to verify all printed output, especially section properties such as flexural stiffness and yield moment.*

OPTION 1 permits an independent nonlinear static analysis. Static loads are input in data set R1. OPTIONS 2 - 4 may be combined with long-term static loads which is input in data set R1.

SET R1: LONG-TERM LOADING (STATIC LOADS)

Control Information

USER_TEXT	***Reference information*: upto 80 characters of text**
NLU,NLJ,NLM,NLC	NLU = No. of uniformly loaded beams NLJ = No. of laterally loaded joint NLM = No. of specified nodal moments NLC = No. of concentrated vertical loads

NOTE: THIS INPUT IS REQUIRED FOR ALL ANALYSIS OPTIONS.
IF NLU = NLJ = NLM = NLC = 0, and IOPT = 2, CONTINUE TO SET R2.
IF NLU = NLJ = NLM = NLC = 0, and IOPT = 3, CONTINUE TO SET R3.
IF NLU = NLJ = NLM = NLC = 0, and IOPT = 4, CONTINUE TO SET R4.

Next Data Set:

JSTP,IOCRL	JSTP = No. of incremental steps in which to apply the static loads (default = 1 step) IOCRL = Steps between printing output *(If IOCRL=0, only final results will be printed; if IOCRL=2, printout will result every 2 steps, and so on)*

NOTES: Dead and live loads that exist prior to the application of seismic or quasi-static cyclic loads can be input in this section. Such loads are typically specified through uniformly loaded beam members. An option is also available for lateral load analysis and the specification of nodal loads at joints. When used in conjenction with Options 2-4, the resulting forces are carried forward to the monotonic, dynamic and quasi-static analysis.

Uniformly Loaded Beam Data

SKIP THIS INPUT SECTION IF NLU=0

USER_TEXT	***Reference information*: upto 80 characters of text**
IL, IBN, FU	IL = Load number IBN = Beam number FU = Magnitude of load (Force/length)

NLU lines of data required in this section

Laterally Loaded Joints

SKIP THIS INPUT SECTION IF NLJ=0

USER_TEXT	***Reference information*: upto 80 characters of text**
IL, LF, IF, FL	IL = Load number LF = Story level number IF = Frame number FL = Magnitude of load

NLJ lines of data required in this section

Nodal Moment Data

SKIP THIS INPUT SECTION IF NLM=0

USER_TEXT	***Reference information*: upto 80 characters of text**
IL, IBM, FM1, FM2	IL = Load number IBM = Beam number FM1 = Nodal moment (left) FM2 = Nodal moment (right)

NLM lines of data required in this section. See Figure A-9 for beam moment sign convention.

Data on Concentrated Vertical Loads

SKIP THIS INPUT SECTION IF NLC=0

USER_TEXT	***Reference information*: upto 80 characters of text**
IL, IFV, LV, JV, FV	IL = Load number IFV = Frame number LV = Story level number JV = Column line number FV = Magnitude of load

NLC lines of data required in this section

IF IOPT = 2, CONTINUE TO SET R2.
IF IOPT = 3, CONTINUE TO SET R3.
IF IOPT = 4, CONTINUE TO SET R4.

SET R2: MONOTONIC PUSH-OVER ANALYSIS (FOR IOPT = 2 ONLY)

USER_TEXT	***Reference information*: upto 80 characters of text**
PMAX, MSTEPS	PMAX = Estimate of base shear strength coefficient (ratio of lateral load capacity to total weight) MSTEPS = Number of steps in which to apply the monotonically increasing load

DEFAULT VALUES: PMAX = 1/NSO + 0.01*NSO ; MSTEPS = 40

NOTES: *The program uses the PMAX value only to determine the load steps for the push-over analysis. The prescribed base shear (product of PMAX and total structure weight) is applied incrementally in MSTEPS steps as an inverted triangular load, until the top story displacement reaches 2% of the total structure height OR the specified PMAX is reached. If the program output shows a linear shear vs. deformation plot, the base shear estimate is too low. If the maximum displacement is reached too quickly (indicated by too few points in the plot), the estimate is too high.*

FOR IOPT = 2, STOP HERE

SET R3: DYNAMIC ANALYSIS CONTROL PARAMETERS (FOR IOPT = 3 ONLY)

USER_TEXT	***Reference information*: upto 80 characters of text**

GMAXH,GMAXV,DTCAL,TDUR,DAMP

GMAXH = Peak horizontal acceleration (g's)
GMAXV = Peak vertical acceleration (g's)
DTCAL = Time step for response analysis (secs)
TDUR = Total duration of analysis (secs)
DAMP = Damping coefficient (% of critical)

NOTES: *The input accelerogram is scaled uniformly to achieve the specified peak acceleration. DTCAL should not exceed the time interval of the input wave, DTINP.*
The ratio (DTINP/DTCAL) must yield an integer number.
TDUR may be less than the total duration of the earthquake. If TDUR is greater than the total time duration of the input wave, a free vibration analysis of the system will result for the remaining time.

INPUT WAVE

USER_TEXT	***Reference information*: upto 80 characters of text**
IWV,NDATA,DTINP	IWV = 0, Vertical component of acceleration not included = 1, Vertical component of acceleration is included NDATA = Number of points in earthquake wave files DTINP = Time interval of input wave

WAVE TITLE

NAMEW	Alpha-numeric title for input wave upto 80 characters

FILENAME - HORIZONTAL COMPONENT

WHFILE	Name of file (with extension) from which to read horizontal component of earthquake record *Note: Filename should not exceed 12 characters*
WINPH(I),I=1,NDATA	Horizontal component of earthquake wave (NDATA points) *NOTE: This data is read from the file **WHFILE** specified in the previous data item*

WAVE DATA - VERTICAL COMPONENT

SKIP THIS INPUT IF IWV .EQ. 0

FILENAME - VERTICAL COMPONENT

WVFILE	Name of file (with extension) from which to read vertical component of earthquake record *Note: Filename should not exceed 12 characters*
WINPV(I),I=1,NDATA	Vertical component of earthquake wave (NDATA points) *NOTE: This data is read from the file **WVFILE** specified in the previous data item.*

NOTES: Accelerogram data may be input in any system of units. The accelerogram is scaled uniformly to achieve the specified peak values of GMAXH and GMAXV. Since data is read in free format, as many lines as necessary to read the entire wave must be input. The data points of the input wave may, therefore, be entered sequentially until the last (or NDATA) point.

GO TO DATA SET S

SET R4: <u>QUASI-STATIC CYCLIC ANALYSIS</u> (FOR IOPT=4 ONLY)

USER_TEXT	***Reference information*: upto 80 characters of text**
ICNTRL	Cyclic Analysis option = 0, Force controlled input = 1, displacement controlled input
NLDED	Number of story levels at which the force or displacement is applied
NSTLD(I),I=1,NLDED	List of story levels at which the force or displacement is applied
NPTS	Number of points to be read in force or displacement history
F(I,1),I=1,NPTS	first data set (NPTS) at story level NSTLD(1)
F(I,2),I=1,NPTS	next data set (NPTS) at story level NSTLD(2)
	. .
F(I,K),I=1,NPTS	repeat till last set, where K = NSTLD(NLDED)
DTCAL	Analysis step (fraction of input steps) The analysis is performed between (1/DTCAL) interpolated points on the input history

SET S: <u>OUTPUT CONTROL</u>

USER_TEXT	***Reference information*: upto 80 characters**
NSOUT,DTOUT,ISO(I),I=1,NSOUT	NSOUT = No of output histories DTOUT = Output time interval ISO(I) output story numbers
FNAMES(1)	Filename to store time history output for story number ISO(1)
FNAMES(2)	Filename for story number ISO(2)
FNAMES(NSOUT)	Filename for story number ISO(NSOUT)

NOTES: If the quasi-static cyclic analysis option is used, DTOUT refers to the number of steps between output printing; for example, DTOUT=2 will print results every 2 steps.

SET T: ELEMENT HYSTERESIS OUTPUT

USER_TEXT — ***Reference information*: upto 80 characters**

KCOUT, KBOUT, KWOUT, KSOUT — Control Data for Element Output:

KCOUT = Number of columns for which hysteresis output is required
KBOUT = Number of beams for which hysteresis output is required
KWOUT = Number of walls for which hysteresis output is required
KSOUT = Number of springs for which hysteresis output is required

COLUMN OUTPUT SPECIFICATION

SKIP THIS INPUT IF KCOUT = 0

USER_TEXT — ***Reference information*: upto 80 characters**

ICLIST(I), I=1,KCOUT — List of column numbers for which moment-curvature hysteresis is required

BEAM OUTPUT SPECIFICATION

SKIP THIS INPUT IF KBOUT = 0

USER_TEXT — ***Reference information*: upto 80 characters**

IBLIST(I), I=1,KBOUT — List of beam numbers for which moment-curvature hysteresis is required

SHEAR WALL OUTPUT SPECIFICATION

SKIP THIS INPUT IF KWOUT = 0

USER_TEXT — ***Reference information*: upto 80 characters**

IWLIST(I), I=1,KWOUT — List of shear wall numbers for which moment-curvature and shear-strain hysteresis is required

DISCRETE SPRING OUTPUT SPECIFICATION

SKIP THIS INPUT IF KSOUT = 0

USER_TEXT — ***Reference information*: upto 80 characters**

ISLIST(I), I=1,KSOUT — List of spring numbers for which moment-rotation hysteresis is required

NOTES: *All the output generated in this section refers to moment-curvature hysteresis for beams, columns and shear-walls; in addition shear vs. shear strain history is generated for walls; whereas moment-rotation hysteresis is produced for the discrete spring elements. Output filenames are generated as follows:*

IF KCOUT = 2, AND ICLIST(1) = 3 AND ICLIST(2) = 12, THEN THE FOLLOWING FILES WILL BE CREATED:

COL_003.PRN and COL_012.PRN

(where 3 and 12 refer to the element numbers for which output is requested)

END OF DATA INPUT

APPENDIX B

SAMPLE INPUT DATA SHEETS

CASE STUDY #1

```
CASE STUDY # 1  :  Full-Scale Circular Bridge Pier (NIST)
CONTROL DATA
1, 1, 1, 1, 1, 0
ELEMENT TYPES
1, 0, 0, 0, 0, 0
ELEMENT DATA
1, 0, 0, 0, 0, 0, 0
UNIT SYSTEM (KIPS/INCH)
1
FLOOR ELEVATIONS
360.0
DESCRIPTION OF IDENTICAL FRAMES
1
PLAN CONFIGURATION (SINGLE COLUMN LINE)
1
NODAL WEIGHTS
1, 1, 300.0
CODE FOR SPECIFICATION OF USER PROPERTIES
0
CONCRETE PROPERTIES
1, 5.2, 4110.0,  0.2, 0.624, 0.0, 0.0
REINFORCEMENT PROPERTIES
1, 68.9, 103.6,  27438.0, 0.0, 0.0
HYSTERETIC MODELING RULES
1
1, 9.0, 0.00, 0.05, 1.0
MOMENT CURVATURE ENVELOPE GENERATION
0
COLUMN DIMENSIONS
2
1, 1,1,1, 360.0, 0.0, 0.0, 1000.0, 60.0, 2.5, 54.5,
  25, 1.69, 0.625, 3.5
COLUMN CONNECTIVITY
1,1,1,1,0,1
ANALYSIS TYPE
4
LONG TERM LOADING (none)
0,0,0,0
```

CASE STUDY #2

```
CASE STUDY # 2:  1:2 SCALE THREE STORY FRAME
CONTROL DATA
3,1,1,1,0
ELEMENT TYPES
4,5,0,0,0,0
ELEMENT DATA
9,6,0,0,0,0,0
UNITS SYSTEM :  KN - MM
2
FLOOR ELEVATIONS
1500.0, 3000.0, 4500.0
DESCRIPTION OF IDENTICAL FRAMES
1
PLAN CONFIGURATION: NO OF COLUMN LINES
3
NODAL WEIGHTS
1,1, 22.24, 22.24, 22.24
2,1, 22.24, 22.24, 22.24
3,1, 22.24, 22.24, 22.24
CODE FOR SPECIFICATION OF USER PROPERTIES
0
CONCRETE PROPERTIES
1, 0.0402, 0.0, 0.0, 0.0, 0.0, 0.0
REINFORCEMENT PROPERTIES
1, 0.4, 0.0, 0.0, 0.0, 0.0
HYSTERETIC MODELING RULES
2
1, 8.0, 0.00, 0.10, 1.0
2, 8.0, 0.00, 0.10, 1.0
MOMENT CURVATURE ENVELOPE GENERATION
0
COLUMN DIMENSIONS
1
1,1,1, 594.2, 1498.6, 149.86, 149.86,
    1, 250.0, 250.0, 15.0, 226.2, 8.0, 75.0, 0.5
    1, 250.0, 250.0, 15.0, 226.2, 8.0, 75.0, 0.5
1
2,1,1, 990.6, 1498.6, 149.86, 149.86,
    1, 250.0, 250.0, 15.0, 307.7, 12.0, 75.0, 0.5
    1, 250.0, 250.0, 15.0, 307.7, 12.0, 75.0, 0.5
1
3,1,1, 594.2, 1498.6, 0.0, 149.86,
    1, 250.0, 250.0, 15.0, 307.7, 12.0, 75.0, 0.5
    1, 250.0, 250.0, 15.0, 307.7, 12.0, 75.0, 0.5
1
4,1,1, 990.6, 1498.6, 0.0, 149.86,
    1, 250.0, 250.0, 15.0, 307.7, 12.0, 75.0, 0.5
    1, 250.0, 250.0, 15.0, 307.7, 12.0, 75.0, 0.5
BEAM MOMENT CURVATURE ENVELOPE GENERATION
0
```

```
BEAM DIMENSIONS
1,1,1, 3000.0, 125.0, 125.0
    2, 300.0,150.0,150.0,0.0,15.0, 401.9,401.9, 6.0, 75.0
    2, 300.0,150.0,150.0,0.0,15.0, 401.9,401.9, 6.0, 75.0
2,1,1, 3000.0, 125.0, 125.0
    2, 300.0,150.0,150.0,0.0,15.0, 480.6,401.9, 6.0, 75.0
    2, 300.0,150.0,150.0,0.0,15.0, 401.9,509.0, 6.0, 75.0
3,1,1, 3000.0, 125.0, 125.0
    2, 300.0,150.0,150.0,0.0,15.0, 401.9,509.0, 6.0, 75.0
    2, 300.0,150.0,150.0,0.0,15.0, 480.6,401.9, 6.0, 75.0
4,1,1, 3000.0, 125.0, 125.0
    2, 300.0,150.0,150.0,0.0,15.0, 307.7,226.5, 6.0, 75.0
    2, 300.0,150.0,150.0,0.0,15.0, 307.7,307.7, 6.0, 75.0
5,1,1, 3000.0, 125.0, 125.0
    2, 300.0,150.0,150.0,0.0,15.0, 307.7,226.5, 6.0, 75.0
    2, 300.0,150.0,150.0,0.0,15.0, 307.7,307.7, 6.0, 75.0
COLUMN CONNECTIVITY
1,1,1,1,2,3
2,2,1,2,2,3
3,1,1,3,2,3
4,1,1,1,1,2
5,2,1,2,1,2
6,1,1,3,1,2
7,3,1,1,0,1
8,4,1,2,0,1
9,3,1,3,0,1
BEAM CONNECTIVITY
1,5,3,1,1,2
2,4,3,1,2,3
3,3,2,1,1,2
4,2,2,1,2,3
5,1,1,1,1,2
6,1,1,1,2,3
ANALYSIS TYPE
4
LONG TERM LOADING (none)
0,0,0,0
QUASI-STATIC CYCLIC ANALYSIS
1
1
3
249
     0.0000     6.8580     0.0000    -6.8580     0.0000    10.1600
     0.0000   -10.1600     0.0000    12.7000    25.4000    32.4104
    25.4000    12.7000     0.0000   -12.7000   -25.4000   -32.0802
   -25.4000   -12.7000     0.0000    12.7000    25.4000    31.9024
    25.4000    12.7000     0.0000   -12.7000   -25.4000   -29.7180
   -25.4000   -12.7000     0.0000    12.7000    25.4000    30.0482
    25.4000    12.7000     0.0000   -12.7000   -25.4000   -28.7020
   -25.4000   -12.7000     0.0000    20.3200    40.6400    50.8000
    55.8800    50.8000    40.6400    20.3200     0.0000   -20.3200
   -40.6400   -50.8000   -53.3400   -50.8000   -40.6400   -20.3200
     0.0000    20.3200    40.6400    50.8000    57.4040    50.8000
    40.6400    20.3200     0.0000   -20.3200   -40.6400   -50.8000
   -54.3560   -50.8000   -40.6400   -20.3200     0.0000    20.3200
    40.6400    50.8000    56.1340    50.8000    40.6400    20.3200
```

```
    0.0000     -20.3200     -40.6400     -50.8000     -54.1020     -50.8000
  -40.6400     -20.3200       0.0000      25.4000      50.8000      76.2000
   86.8680      76.2000      50.8000      25.4000       0.0000     -25.4000
  -50.8000     -76.2000     -83.3120     -76.2000     -50.8000     -25.4000
    0.0000      25.4000      50.8000      76.2000      87.1220      76.2000
   50.8000      25.4000       0.0000     -25.4000     -50.8000     -76.2000
  -84.5820     -76.2000     -50.8000     -25.4000       0.0000      25.4000
   50.8000      76.2000      88.6460      76.2000      50.8000      25.4000
    0.0000     -25.4000     -50.8000     -76.2000     -84.5820     -76.2000
  -50.8000     -25.4000       0.0000      38.1000      76.2000     106.6800
  114.3000     106.6800      76.2000      38.1000       0.0000     -38.1000
  -76.2000    -106.6800    -111.7600    -106.6800     -76.2000     -38.1000
    0.0000      38.1000      76.2000     106.6800     114.3000     106.6800
   76.2000      38.1000       0.0000     -38.1000     -76.2000    -106.6800
 -112.2680    -106.6800     -76.2000     -38.1000       0.0000      38.1000
   76.2000     106.6800     113.5380     106.6800      76.2000      38.1000
    0.0000     -38.1000     -76.2000    -106.6800    -112.2680    -106.6800
  -76.2000     -38.1000       0.0000      38.1000      76.2000     114.3000
  139.7000     147.8280     139.7000     114.3000      76.2000      38.1000
    0.0000     -38.1000     -76.2000    -114.3000    -121.9200    -127.7620
 -121.9200    -114.3000     -76.2000     -38.1000       0.0000      38.1000
   76.2000     114.3000     139.7000     147.0660     139.7000     114.3000
   76.2000      38.1000       0.0000     -38.1000     -76.2000    -114.3000
 -121.9200    -128.0160    -121.9200    -114.3000     -76.2000     -38.1000
    0.0000      38.1000      76.2000     114.3000     139.7000     147.3200
  139.7000     114.3000      76.2000      38.1000       0.0000     -38.1000
  -76.2000    -114.3000    -121.9200    -127.0000    -121.9200    -114.3000
  -76.2000     -38.1000       0.0000
0.02
OUTPUT CONTROL
3,10,1,2,3
LEVEL1.OUT
LEVEL2.OUT
LEVEL3.OUT
MISCELLANEOUS OUTPUT INFORMATION
0,0,0,0
```

CASE STUDY #3

```
CASE STUDY # 3  : TEN STORY FRAME MODEL (ILLINOIS TEST)
CONTROL DATA
10,1,1,1,1
ELEMENT TYPES
20,2,0,0,0,0
ELEMENT DATA
40,30,0,0,0,0,0
UNITS SYSTEM
1
FLOOR ELEVATIONS
9.0,18.0,27.0,36.0,45.0,54.0,63.0,72.0,81.0,90.0
DESCRIPTION OF IDENTICAL FRAMES
2
PLAN CONFIGURATION
4
NODAL WEIGHTS
1, 1, 0.125, 0.125, 0.125, 0.125
2, 1, 0.125, 0.125, 0.125, 0.125
3, 1, 0.125, 0.125, 0.125, 0.125
4, 1, 0.125, 0.125, 0.125, 0.125
5, 1, 0.125, 0.125, 0.125, 0.125
6, 1, 0.125, 0.125, 0.125, 0.125
7, 1, 0.125, 0.125, 0.125, 0.125
8, 1, 0.125, 0.125, 0.125, 0.125
9, 1, 0.125, 0.125, 0.125, 0.125
10, 1, 0.125, 0.125, 0.125, 0.125
CODE FOR SPECIFICATION OF USER PROPERTIES
0
CONCRETE PROPERTIES
1, 4.35, 1000.0, 0.3, 0.435, 1.2, 100.0
REINFORCEMENT PROPERTIES
1, 70.0, 72.5, 29000.0, 40.0, 2.0
HYSTERETIC MODELING RULES
2
1, 1.0,0.0,0.1,1.0
2, 2.0,0.0,0.1,1.0
COLUMN MOMENT CURVATURE ENVELOPE GENERATION
0
COLUMN DIMENSIONS
1
1,1,1,1.25,9.0,0.0,0.75, 1, 2.0,1.5,0.25,0.049,0.0625,0.35,0.5
                         1, 2.0,1.5,0.25,0.049,0.0625,0.35,0.5
1
2,1,1,1.12,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.049,0.0625,0.35,0.5
                          1, 2.0,1.5,0.25,0.049,0.0625,0.35,0.5
1
3,1,1,1.00,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.049,0.0625,0.35,0.5
                          1, 2.0,1.5,0.25,0.049,0.0625,0.35,0.5
1
4,1,1,0.88,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.049,0.0625,0.35,0.5
                          1, 2.0,1.5,0.25,0.049,0.0625,0.35,0.5
```

```
1
5,1,1,0.75,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.049,0.0625,0.35,0.5
                          1, 2.0,1.5,0.25,0.029,0.0625,0.35,0.5
1
6,1,1,0.63,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.029,0.0625,0.35,0.5
                          1, 2.0,1.5,0.25,0.029,0.0625,0.35,0.5
1
7,1,1,0.50,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.029,0.0625,0.35,0.5
                          1, 2.0,1.5,0.25,0.029,0.0625,0.35,0.5
1
8,1,1,0.38,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.029,0.0625,0.35,0.5
                          1, 2.0,1.5,0.25,0.029,0.0625,0.35,0.5
1
9,1,1,0.25,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.029,0.0625,0.35,0.5
                          1, 2.0,1.5,0.25,0.029,0.0625,0.35,0.5
1
10,1,1,0.13,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.029,0.0625,0.35,0.5
                           1, 2.0,1.5,0.25,0.029,0.0625,0.35.0.5
1
11,1,1,1.25,9.0,0.0,0.75, 1, 2.0,1.5,0.25,0.041,0.0625,0.35,0.5
                          1, 2.0,1.5,0.25,0.041,0.0625,0.35,0.5
1
12,1,1,1.13,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.041,0.0625,0.35,0.5
                           1, 2.0,1.5,0.25,0.041,0.0625,0.35,0.5
1
13,1,1,1.00,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.041,0.0625,0.35,0.5
                           1, 2.0,1.5,0.25,0.041,0.0625,0.35,0.5
1
14,1,1,0.88,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.041,0.0625,0.35,0.5
                           1, 2.0,1.5,0.25,0.041,0.0625,0.35,0.5
1
15,1,1,0.75,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.041,0.0625,0.35,0.5
                           1, 2.0,1.5,0.25,0.013,0.0625,0.35,0.5
1
16,1,1,0.63,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.013,0.0625,0.35,0.5
                           1, 2.0,1.5,0.25,0.013,0.0625,0.35,0.5
1
17,1,1,0.50,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.013,0.0625,0.35,0.5
                           1, 2.0,1.5,0.25,0.013,0.0625,0.35,0.5
1
18,1,1,0.38,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.013,0.0625,0.35,0.5
                           1, 2.0,1.5,0.25,0.013,0.0625,0.35,0.5
1
19,1,1,0.25,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.013,0.0625,0.35,0.5
                           1, 2.0,1.5,0.25,0.013,0.0625,0.35,0.5
1
20,1,1,0.13,9.0,0.75,0.75, 1, 2.0,1.5,0.25,0.013,0.0625,0.35,0.5
                           1, 2.0,1.5,0.25,0.013,0.0625,0.35,0.5
BEAM MOMENT CURVATURE ENVELOPE GENERATION
0
BEAM DIMENSIONS
1,1,1,12.0,0.75,0.75,
      2, 1.5,1.5,1.5,0.0,0.25,0.0092,0.0092,0.0625,0.3
      2, 1.5,1.5,1.5,0.0,0.25,0.0092,0.0092,0.0625,0.3
```

```
2,1,1,12.0,0.75,0.75,
       2, 1.5,1.5,1.5,0.0,0.25,0.013,0.013,0.0625,0.3
       2, 1.5,1.5,1.5,0.0,0.25,0.013,0.013,0.0625,0.3
COLUMN CONNECTIVITY
 1   1  1  1  0  1
 2   2  1  1  1  2
 3   3  1  1  2  3
 4   4  1  1  3  4
 5   5  1  1  4  5
 6   6  1  1  5  6
 7   7  1  1  6  7
 8   8  1  1  7  8
 9   9  1  1  8  9
10  10  1  1  9 10
11  11  1  2  0  1
12  12  1  2  1  2
13  13  1  2  2  3
14  14  1  2  3  4
15  15  1  2  4  5
16  16  1  2  5  6
17  17  1  2  6  7
18  18  1  2  7  8
19  19  1  2  8  9
20  20  1  2  9 10
21  11  1  3  0  1
22  12  1  3  1  2
23  13  1  3  2  3
24  14  1  3  3  4
25  15  1  3  4  5
26  16  1  3  5  6
27  17  1  3  6  7
28  18  1  3  7  8
29  19  1  3  8  9
30  20  1  3  9 10
31   1  1  4  0  1
32   2  1  4  1  2
33   3  1  4  2  3
34   4  1  4  3  4
35   5  1  4  4  5
36   6  1  4  5  6
37   7  1  4  6  7
38   8  1  4  7  8
39   9  1  4  8  9
40  10  1  4  9 10
BEAM CONNECTIVITY
 1   2  1  1  1  2
 2   2  2  1  1  2
 3   2  3  1  1  2
 4   2  4  1  1  2
 5   1  5  1  1  2
 6   1  6  1  1  2
 7   1  7  1  1  2
 8   1  8  1  1  2
 9   1  9  1  1  2
```

```
10  1 10  1  1  2
11  2  1  1  2  3
12  2  2  1  2  3
13  2  3  1  2  3
14  2  4  1  2  3
15  1  5  1  2  3
16  1  6  1  2  3
17  1  7  1  2  3
18  1  9  1  2  3
19  1  9  1  2  3
20  1 10  1  2  3
21  2  1  1  3  4
22  2  2  1  3  4
23  2  3  1  3  4
24  2  4  1  3  4
25  1  5  1  3  4
26  1  6  1  3  4
27  1  7  1  3  4
28  1  8  1  3  4
29  1  9  1  3  4
30  1 10  1  3  4
ANALYSIS TYPE
3
STATIC ANALYSIS OPTION
0,0,0,0
DYNAMIC ANALYSIS CONTROL PARAMETERS
1.6163, 0.0, 0.001, 43.5,  2.0
INPUT WAVE INFORMATION
0.0,10773,0.004
Actual Table Motion - TAFT NS Component
WAVEH.DAT
OUTPUT CONTROL
4,0.02,1,5,7,10
LEVEL3.OUT
LEVEL5.OUT
LEVEL7.OUT
LEVEL10.OUT
MISCELLANEOUS OUTPUT INFORMATION
2,2,0,0
COLUMN OUTPUT
1,37
BEAM OUTPUT
1,21
```

Note: *The earthquake record is read separately from file WAVEH.DAT as specified in the input data. This file consists of all three records, merged sequentially, thereby preserving the damaged state of the structure at the end of each test.*

CASE STUDY #4

```
Analysis of 1:3 Scale Three Story Model (Achieved PGA = 0.22 g)
Control Data
3,1,0,0,0
Element types
6,1,0,0,0,0
Element data
12,9,0,0,0,0,0
Unit system
1
Floor elevations
45.0, 93.0, 141.0
Number of duplicate frames
2
No of column lines
4
Nodal weights
1, 1, 3.375, 3.375, 3.375, 3.375
2, 1, 3.375, 3.375, 3.375, 3.375
3, 1, 3.375, 3.375, 3.375, 3.375
Env generation option
1
Hysteretic Control
2
1, 0.5, 0.0, 0.10, 1.0
2, 2.0, 0.0, 0.10, 1.0
Column input option
1
Column data
1, 48.0,3.0,3.0,
   1, 30400.0, 843.0, 19980.8, 10.0, 18.0, 0.001, 0.006, 400.0
                               10.0, 18.0, 0.001, 0.006, 400.0
   1, 30400.0, 843.0, 19980.8, 10.0, 18.0, 0.001, 0.006, 400.0
                               10.0, 18.0, 0.001, 0.006, 400.0
2, 48.0,3.0,3.0,
   1, 30400.0, 843.0, 19980.8, 10.0, 22.0, 0.001, 0.006, 400.0
                               10.0, 22.0, 0.001, 0.006, 400.0
   1, 30400.0, 843.0, 19980.8, 10.0, 22.0, 0.001, 0.006, 400.0
                               10.0, 22.0, 0.001, 0.006, 400.0
3, 48.0,3.0,3.0,
   1, 22900.0, 900.0, 24160.0, 10.0, 22.0, 0.0012, 0.006, 400.0,
                               10.0, 22.0, 0.0012, 0.006, 400.0,
   1, 22900.0, 900.0, 22528.0, 10.0, 22.0, 0.0012, 0.006, 400.0,
                               10.0, 22.0, 0.0012, 0.006, 400.0
4, 48.0,3.0,3.0,
   1, 22900.0, 900.0, 24160.0, 14.0, 29.0, 0.0013, 0.006, 400.0,
                               14.0, 29.0, 0.0013, 0.006, 400.0,
   1, 22900.0, 900.0, 22528.0, 14.0, 29.0, 0.0013, 0.006, 400.0,
                               14.0, 29.0, 0.0013, 0.006, 400.0
5, 45.0,0.0,3.0,
   1, 34000.0, 960.0, 20640.0, 12.0, 28.0, 0.0016, 0.007, 400.0,
                               12.0, 28.0, 0.0016, 0.007, 400.0,
   1, 34000.0, 960.0, 24000.0, 12.0, 28.0, 0.0016, 0.007, 400.0,
                               12.0, 28.0, 0.0016, 0.007, 400.0
6, 45.0,0.0,3.0,
   1, 34000.0, 960.0, 20640.0, 16.0, 38.0, 0.0014, 0.007, 400.0,
```

```
                                  16.0, 38.0, 0.0014, 0.007, 400.0,
   1, 34000.0, 960.0, 24000.0, 16.0, 38.0, 0.0014, 0.007, 400.0,
                                  16.0, 38.0, 0.0014, 0.007, 400.0
Beam input type
1
Beam data
1, 72.0, 2.0, 2.0
   2, 140000.0, 20000.0, 15.0, 30.0, 0.0005, 0.005, 2400.0
                         30.0, 70.0, 0.001,  0.008, 2400.0
   2, 140000.0, 20000.0, 15.0, 30.0, 0.0005, 0.005, 2400.0
                         30.0, 70.0, 0.001,  0.008, 2400.0
Column connectivity
1,1,1,1,2,3
2,2,1,2,2,3
3,2,1,3,2,3
4,1,1,4,2,3
5,3,1,1,1,2
6,4,1,2,1,2
7,4,1,3,1,2
8,3,1,4,1,2
9,5,1,1,0,1
10,6,1,2,0,1
11,6,1,3,0,1
12,5,1,4,0,1
Beam connectivity
1,1,3,1,1,2
2,1,3,1,2,3
3,1,3,1,3,4
4,1,2,1,1,2
5,1,2,1,2,3
6,1,2,1,3,4
7,1,1,1,1,2
8,1,1,1,2,3
9,1,1,1,3,4
Type of Analysis
3
Static loads
0,0,0,0
Dynamic Analysis Control Data
0.22, 0.0, 0.002, 30.0, 1.2
Wave data
0,3000,0.01
TAFT - EARTHQUAKE
WAVE23.DAT
Output options
1, 0.02, 3
JINEL.PRN
Hys output
0,0,0,0
```

Note: *The earthquake record is read separately from file WAVE23.DAT as specified in the input data.*

CASE STUDY #5

```
CASE 5: Seismic Damage Analysis of Cypress Viaduct
Control Data - 4 stories, 1 frame, 1 conc and 1 steel type
4, 1, 1, 1, 0
Element types: 2 cols, 12 beams, 2 walls
2, 12, 2, 0, 0, 0
Element data: 4 columns, 12 beams, 2 walls
4, 12, 2, 0, 0, 0, 4
System of units: k/in
1
Floor elevations
252.0  327.0  327.0  528.0
Duplicate frame info
1
No of column lines
7
Nodal weights  (Note: Story 2 & 3 are dummy levels)
1   1  116.7  233.3  233.3  233.3  233.3  233.3  116.7
2   1    0.0    0.0    0.0    0.0    0.0    0.0    0.0
3   1    0.0    0.0    0.0    0.0    0.0    0.0    0.0
4   1  116.7  233.3  233.3  233.3  233.3  233.3  116.7
Option for M-phi input
1
Hysteresis Rules
4
1  2.0, 0.0, 0.1, 1.0
2  2.0, 0.0, 0.1, 1.0
3  2.0, 0.0, 0.1, 1.0
4  1.0, 0.0, 0.2, 1.0
Option for column input
1
COLUMN DATA

1     252.0      0.0     48.0
 -1   8.38E+9  8.73e+4     0.0
      50350   266300  5.12e-5  2.19e-4  1.37e+8
      50350   266300  5.12e-5  2.19e-4  1.37e+8
2       201      0.0     48.0
  1   1.02e+9  5.82e+4     0.0
      12200    64350  1.04e-4  4.07e-4  1.85e+7
      12200    64350  1.04e-4  4.07e-4  1.85e+7
  1   2.32e+9  7.41e+4     0.0
      19200    90300  7.24e-5  3.70e-4  3.21e+7
      19200    90300  7.24e-5  3.70e-4  3.21e+7

Option for beam input
1
BEAM DATA

1     117.0     48.0      0.0
  2   2.00E+10   0.0     45700     70500  2.29E-5  8.78E-4  6.29E+7
                         47100    136800  2.51E-5  5.68E-4  1.16E+8
  2   2.00E+10   0.0     45900    117800  2.48E-5  5.68E-4  1.01E+8
                         40900     45600  2.27E-5  9.21E-4  4.23E+7
2     117.0      0.0      0.0
  2   2.00E+10   0.0     45900    117800  2.48E-5  5.68E-4  1.01E+8
                         40900     45600  2.27E-5  9.21E-4  4.23E+7
```

```
  2  2.00E+10    0.0    48500    208200  2.84E-5  3.07E-4  1.27E+8
                        18500     20600  2.11E-5  8.23E-4  2.71E+7
3    117.0       0.0      0.0
  2  2.00E+10    0.0    48500    208200  2.84E-5  3.07E-4  1.27E+8
                        18500     20600  2.11E-5  8.23E-4  2.71E+7
  2  2.00E+10    0.0    49000    222300  2.87E-5  2.89E-4  1.30E+8
                        18500     20600  2.10E-5  7.81E-4  2.90E+7
4    117.0       0.0      0.0
  2  2.00E+10    0.0    49000    222300  2.87E-5  2.89E-4  1.30E+8
                        18500     20600  2.10E-5  7.81E-4  2.90E+7
  2  2.00E+10    0.0    48500    208200  2.84E-5  3.07E-4  1.27E+8
                        18500     20600  2.11E-5  8.23E-4  2.71E+7
5    117.0       0.0      0.0
  2  2.00E+10    0.0    48500    208200  2.84E-5  3.07E-4  1.27E+8
                        18500     20600  2.11E-5  8.23E-4  2.71E+7
  2  2.00E+10    0.0    45900    117800  2.48E-5  5.68E-4  1.01E+8
                        40900     45600  2.27E-5  9.21E-4  4.23E+7
6    117.0       0.0     48.0
  2  2.00E+10    0.0    45900    117800  2.48E-5  5.68E-4  1.01E+8
                        40900     45600  2.27E-5  9.21E-4  4.23E+7
  2  2.00E+10    0.0    45700     70500  2.29E-5  8.78E-4  6.29E+7
                        47100    136800  2.51E-5  5.68E-4  1.16E+8
7    117.0      24.0      0.0
  2  2.00E+10    0.0    44800     86800  2.39E-5  6.36E-4  7.57E+7
                        44100     54500  2.31E-5  9.10E-4  4.90E+7
  2  2.00E+10    0.0    49200    224300  2.98E-5  2.96E-4  1.28E+8
                        25500     28600  2.24E-5  7.51E-4  3.62E+7
8    117.0       0.0      0.0
  2  2.00E+10    0.0    49200    224300  2.98E-5  2.96E-4  1.28E+8
                        25500     28600  2.24E-5  7.51E-4  3.62E+7
  2  2.00E+10    0.0    51200    301900  3.33E-5  2.16E-4  9.77E+7
                        21600     24000  2.14E-5  5.62E-4  5.70E+7
9    117.0       0.0      0.0
  2  2.00E+10    0.0    51200    301900  3.33E-5  2.16E-4  9.77E+7
                        21600     24000  2.14E-5  5.62E-4  5.70E+7
  2  2.00E+10    0.0    51200    301900  3.33E-5  2.16E-4  9.77E+7
                        21600     24000  2.14E-5  5.62E-4  5.70E+7
10   117.0       0.0      0.0
  2  2.00E+10    0.0    51200    301900  3.33E-5  2.16E-4  9.77E+7
                        21600     24000  2.14E-5  5.62E-4  5.70E+7
  2  2.00E+10    0.0    51200    301900  3.33E-5  2.16E-4  9.77E+7
                        21600     24000  2.14E-5  5.62E-4  5.70E+7
11   117.0       0.0      0.0
  2  2.00E+10    0.0    51200    301900  3.33E-5  2.16E-4  9.77E+7
                        21600     24000  2.14E-5  5.62E-4  5.70E+7
  2  2.00E+10    0.0    49200    224300  2.98E-5  2.96E-4  1.28E+8
                        25500     28600  2.24E-5  7.51E-4  3.62E+7
12   117.0       0.0     24.0
  2  2.00E+10    0.0    49200    224300  2.98E-5  2.96E-4  1.28E+8
                        25500     28600  2.24E-5  7.51E-4  3.62E+7
  2  2.00E+10    0.0    44800     86800  2.39E-5  6.36E-4  7.57E+7
                        44100     54500  2.31E-5  9.10E-4  4.90E+7
Option for wall input
1
WALL DATA

1  75.0  2.83e+5
  -3  9.9e+15  9.9e+15  9.99e+15  2.0  10.0  9.9e+12
               9.9e+15  9.99e+15  2.0  10.0  9.9e+12
   4  9.433+5  400      520   9.380e-4  1.600e-3  1.500e+4
               250      405   1.105e-3  5.333e-3  1.125e+4
2  75  2.83e+5
  -3  9.9e+15  9.9e+15  9.99e+15  2.0  10.0  9.9e+12
```

```
                9.9e+15  9.99e+15  2.0   10.0  9.9e+12
   4  9.433+5  250      405   1.105e-3   5.333e-3  1.125e+4
                400      520   9.380e-4   1.600e-3  1.500e+4
Column connectivity
1,   1, 1, 1, 0, 1
2,   1, 1, 7, 0, 1
3,   2, 1, 1, 2, 4
4,   2, 1, 7, 3, 4
Beam connectivity
1,   1, 1, 1, 1, 2
2,   2, 1, 1, 2, 3
3,   3, 1, 1, 3, 4
4,   4, 1, 1, 4, 5
5,   5, 1, 1, 5, 6
6,   6, 1, 1, 6, 7
7,   7, 4, 1, 1, 2
8,   8, 4, 1, 2, 3
9,   9, 4, 1, 3, 4
10, 10, 4, 1, 4, 5
11, 11, 4, 1, 5, 6
12, 12, 4, 1, 6, 7
Shear wall connectivity
1,   1, 1, 1, 1, 2
2,   2, 1, 7, 1, 3
Moment releases
1, 1, 1, 1
2, 1, 2, 1
3, 1, 3, 1
4, 1, 4, 1
Phase II option (=0, STOP; =3, Seismic; =4, Quasistatic)
3
Long term loading: static loads
0  0  0  0
Control data for dynamic analysis
0.33, 1.065, 0.001, 20.0, 3.0
Wave control data
1, 2201, 0.02
GRAVITY LOAD PLUS OUTER HARBOUR WHARF RECORD
ohw_hori.dat
ohw_vert.dat
Output control
2, 0.02, 1, 4
FIRST.PRN
SECOND.PRN
Hysteresis Output
0, 0, 2, 0
Wall numbers for output
1, 2
```

Note: The earthquake record is read separately from files:
OHW_HORI.DAT (horizontal component) and
OHW_VERT.DAT (vertical component)
as specified in the input data

APPENDIX C

PROGRAM NOTES

C.1 Installation and Execution

The current version of the program has been tested on VAX/VMS and several UNIX-based machines. The program is composed of two files: the main source code *idarc3.f* and a file which contains the global common block definitions *IDDEFN.FOR*. On a UNIX machine the following command will create an executable file called *IDARC3* :

f77 -o IDARC3 idarc3.f

On a VAX/VMS machine, it is necessary to rename the source file to *idarc3.for*. The typical two step process to create the executable file is:

for idarc3
link idarc3

On a UNIX machine, real-time execution is done by simply typing the name of the executable file (in this case, **IDARC3**). On a VAX/VMS machine the command for executing the program is : **RUN IDARC3.**

C.2 Data Files and Output Files

Upon execution, IDARC looks for a data file named
IDARC.DAT
which should contain the names of the input and output files on separate lines.
For example, to run the sample CASE STUDY #1, the file *IDARC.DAT* should contain the following lines:
CASE1.DAT
CASE1.OUT
where *CASE1.DAT* is the data file presented in Appendix B.

A number of output files are generated by the program:

1. The main program output which contains a summary of input and essential output parameters is stored in the main output file specified in the *IDARC.DAT* file.

2. Story level outputs are generated for specified story levels and are output on separate user-specified output files. These file names are specified in the data section titled *Output Control.*

3. Element hysteresis output can be generated by specifying element numbers for which output is required (see Case Study #5 which requests output of force-deformation hysteresis for Walls 1 and 2).

NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
LIST OF TECHNICAL REPORTS

The National Center for Earthquake Engineering Research (NCEER) publishes technical reports on a variety of subjects related to earthquake engineering written by authors funded through NCEER. These reports are available from both NCEER's Publications Department and the National Technical Information Service (NTIS). Requests for reports should be directed to the Publications Department, National Center for Earthquake Engineering Research, State University of New York at Buffalo, Red Jacket Quadrangle, Buffalo, New York 14261. Reports can also be requested through NTIS, 5285 Port Royal Road, Springfield, Virginia 22161. NTIS accession numbers are shown in parenthesis, if available.

NCEER-87-0001 "First-Year Program in Research, Education and Technology Transfer," 3/5/87, (PB88-134275/AS).

NCEER-87-0002 "Experimental Evaluation of Instantaneous Optimal Algorithms for Structural Control," by R.C. Lin, T.T. Soong and A.M. Reinhorn, 4/20/87, (PB88-134341/AS).

NCEER-87-0003 "Experimentation Using the Earthquake Simulation Facilities at University at Buffalo," by A.M. Reinhorn and R.L. Ketter, to be published.

NCEER-87-0004 "The System Characteristics and Performance of a Shaking Table," by J.S. Hwang, K.C. Chang and G.C. Lee, 6/1/87, (PB88-134259/AS). This report is available only through NTIS (see address given above).

NCEER-87-0005 "A Finite Element Formulation for Nonlinear Viscoplastic Material Using a Q Model," by O. Gyebi and G. Dasgupta, 11/2/87, (PB88-213764/AS).

NCEER-87-0006 "Symbolic Manipulation Program (SMP) - Algebraic Codes for Two and Three Dimensional Finite Element Formulations," by X. Lee and G. Dasgupta, 11/9/87, (PB88-219522/AS).

NCEER-87-0007 "Instantaneous Optimal Control Laws for Tall Buildings Under Seismic Excitations," by J.N. Yang, A. Akbarpour and P. Ghaemmaghami, 6/10/87, (PB88-134333/AS).

NCEER-87-0008 "IDARC: Inelastic Damage Analysis of Reinforced Concrete Frame - Shear-Wall Structures," by Y.J. Park, A.M. Reinhorn and S.K. Kunnath, 7/20/87, (PB88-134325/AS).

NCEER-87-0009 "Liquefaction Potential for New York State: A Preliminary Report on Sites in Manhattan and Buffalo," by M. Budhu, V. Vijayakumar, R.F. Giese and L. Baumgras, 8/31/87, (PB88-163704/AS). This report is available only through NTIS (see address given above).

NCEER-87-0010 "Vertical and Torsional Vibration of Foundations in Inhomogeneous Media," by A.S. Veletsos and K.W. Dotson, 6/1/87, (PB88-134291/AS).

NCEER-87-0011 "Seismic Probabilistic Risk Assessment and Seismic Margins Studies for Nuclear Power Plants," by Howard H.M. Hwang, 6/15/87, (PB88-134267/AS).

NCEER-87-0012 "Parametric Studies of Frequency Response of Secondary Systems Under Ground-Acceleration Excitations," by Y. Yong and Y.K. Lin, 6/10/87, (PB88-134309/AS).

NCEER-87-0013 "Frequency Response of Secondary Systems Under Seismic Excitation," by J.A. HoLung, J. Cai and Y.K. Lin, 7/31/87, (PB88-134317/AS).

NCEER-87-0014 "Modelling Earthquake Ground Motions in Seismically Active Regions Using Parametric Time Series Methods," by G.W. Ellis and A.S. Cakmak, 8/25/87, (PB88-134283/AS).

NCEER-87-0015 "Detection and Assessment of Seismic Structural Damage," by E. DiPasquale and A.S. Cakmak, 8/25/87, (PB88-163712/AS).

NCEER-87-0016 "Pipeline Experiment at Parkfield, California," by J. Isenberg and E. Richardson, 9/15/87, (PB88-163720/AS). This report is available only through NTIS (see address given above).

NCEER-87-0017 "Digital Simulation of Seismic Ground Motion," by M. Shinozuka, G. Deodatis and T. Harada, 8/31/87, (PB88-155197/AS). This report is available only through NTIS (see address given above).

NCEER-87-0018 "Practical Considerations for Structural Control: System Uncertainty, System Time Delay and Truncation of Small Control Forces," J.N. Yang and A. Akbarpour, 8/10/87, (PB88-163738/AS).

NCEER-87-0019 "Modal Analysis of Nonclassically Damped Structural Systems Using Canonical Transformation," by J.N. Yang, S. Sarkani and F.X. Long, 9/27/87, (PB88-187851/AS).

NCEER-87-0020 "A Nonstationary Solution in Random Vibration Theory," by J.R. Red-Horse and P.D. Spanos, 11/3/87, (PB88-163746/AS).

NCEER-87-0021 "Horizontal Impedances for Radially Inhomogeneous Viscoelastic Soil Layers," by A.S. Veletsos and K.W. Dotson, 10/15/87, (PB88-150859/AS).

NCEER-87-0022 "Seismic Damage Assessment of Reinforced Concrete Members," by Y.S. Chung, C. Meyer and M. Shinozuka, 10/9/87, (PB88-150867/AS). This report is available only through NTIS (see address given above).

NCEER-87-0023 "Active Structural Control in Civil Engineering," by T.T. Soong, 11/11/87, (PB88-187778/AS).

NCEER-87-0024 "Vertical and Torsional Impedances for Radially Inhomogeneous Viscoelastic Soil Layers," by K.W. Dotson and A.S. Veletsos, 12/87, (PB88-187786/AS).

NCEER-87-0025 "Proceedings from the Symposium on Seismic Hazards, Ground Motions, Soil-Liquefaction and Engineering Practice in Eastern North America," October 20-22, 1987, edited by K.H. Jacob, 12/87, (PB88-188115/AS).

NCEER-87-0026 "Report on the Whittier-Narrows, California, Earthquake of October 1, 1987," by J. Pantelic and A. Reinhorn, 11/87, (PB88-187752/AS). This report is available only through NTIS (see address given above).

NCEER-87-0027 "Design of a Modular Program for Transient Nonlinear Analysis of Large 3-D Building Structures," by S. Srivastav and J.F. Abel, 12/30/87, (PB88-187950/AS).

NCEER-87-0028 "Second-Year Program in Research, Education and Technology Transfer," 3/8/88, (PB88-219480/AS).

NCEER-88-0001 "Workshop on Seismic Computer Analysis and Design of Buildings With Interactive Graphics," by W. McGuire, J.F. Abel and C.H. Conley, 1/18/88, (PB88-187760/AS).

NCEER-88-0002 "Optimal Control of Nonlinear Flexible Structures," by J.N. Yang, F.X. Long and D. Wong, 1/22/88, (PB88-213772/AS).

NCEER-88-0003 "Substructuring Techniques in the Time Domain for Primary-Secondary Structural Systems," by G.D. Manolis and G. Juhn, 2/10/88, (PB88-213780/AS).

NCEER-88-0004 "Iterative Seismic Analysis of Primary-Secondary Systems," by A. Singhal, L.D. Lutes and P.D. Spanos, 2/23/88, (PB88-213798/AS).

NCEER-88-0005 "Stochastic Finite Element Expansion for Random Media," by P.D. Spanos and R. Ghanem, 3/14/88, (PB88-213806/AS).

NCEER-88-0006 "Combining Structural Optimization and Structural Control," by F.Y. Cheng and C.P. Pantelides, 1/10/88, (PB88-213814/AS).

NCEER-88-0007 "Seismic Performance Assessment of Code-Designed Structures," by H.H-M. Hwang, J-W. Jaw and H-J. Shau, 3/20/88, (PB88-219423/AS).

NCEER-88-0008 "Reliability Analysis of Code-Designed Structures Under Natural Hazards," by H.H-M. Hwang, H. Ushiba and M. Shinozuka, 2/29/88, (PB88-229471/AS).

NCEER-88-0009 "Seismic Fragility Analysis of Shear Wall Structures," by J-W Jaw and H.H-M. Hwang, 4/30/88, (PB89-102867/AS).

NCEER-88-0010 "Base Isolation of a Multi-Story Building Under a Harmonic Ground Motion - A Comparison of Performances of Various Systems," by F-G Fan, G. Ahmadi and I.G. Tadjbakhsh, 5/18/88, (PB89-122238/AS).

NCEER-88-0011 "Seismic Floor Response Spectra for a Combined System by Green's Functions," by F.M. Lavelle, L.A. Bergman and P.D. Spanos, 5/1/88, (PB89-102875/AS).

NCEER-88-0012 "A New Solution Technique for Randomly Excited Hysteretic Structures," by G.Q. Cai and Y.K. Lin, 5/16/88, (PB89-102883/AS).

NCEER-88-0013 "A Study of Radiation Damping and Soil-Structure Interaction Effects in the Centrifuge," by K. Weissman, supervised by J.H. Prevost, 5/24/88, (PB89-144703/AS).

NCEER-88-0014 "Parameter Identification and Implementation of a Kinematic Plasticity Model for Frictional Soils," by J.H. Prevost and D.V. Griffiths, to be published.

NCEER-88-0015 "Two- and Three- Dimensional Dynamic Finite Element Analyses of the Long Valley Dam," by D.V. Griffiths and J.H. Prevost, 6/17/88, (PB89-144711/AS).

NCEER-88-0016 "Damage Assessment of Reinforced Concrete Structures in Eastern United States," by A.M. Reinhorn, M.J. Seidel, S.K. Kunnath and Y.J. Park, 6/15/88, (PB89-122220/AS).

NCEER-88-0017 "Dynamic Compliance of Vertically Loaded Strip Foundations in Multilayered Viscoelastic Soils," by S. Ahmad and A.S.M. Israil, 6/17/88, (PB89-102891/AS).

NCEER-88-0018 "An Experimental Study of Seismic Structural Response With Added Viscoelastic Dampers," by R.C. Lin, Z. Liang, T.T. Soong and R.H. Zhang, 6/30/88, (PB89-122212/AS). This report is available only through NTIS (see address given above).

NCEER-88-0019 "Experimental Investigation of Primary - Secondary System Interaction," by G.D. Manolis, G. Juhn and A.M. Reinhorn, 5/27/88, (PB89-122204/AS).

NCEER-88-0020 "A Response Spectrum Approach For Analysis of Nonclassically Damped Structures," by J.N. Yang, S. Sarkani and F.X. Long, 4/22/88, (PB89-102909/AS).

NCEER-88-0021 "Seismic Interaction of Structures and Soils: Stochastic Approach," by A.S. Veletsos and A.M. Prasad, 7/21/88, (PB89-122196/AS).

NCEER-88-0022 "Identification of the Serviceability Limit State and Detection of Seismic Structural Damage," by E. DiPasquale and A.S. Cakmak, 6/15/88, (PB89-122188/AS). This report is available only through NTIS (see address given above).

NCEER-88-0023 "Multi-Hazard Risk Analysis: Case of a Simple Offshore Structure," by B.K. Bhartia and E.H. Vanmarcke, 7/21/88, (PB89-145213/AS).

NCEER-88-0024 "Automated Seismic Design of Reinforced Concrete Buildings," by Y.S. Chung, C. Meyer and M. Shinozuka, 7/5/88, (PB89-122170/AS). This report is available only through NTIS (see address given above).

NCEER-88-0025 "Experimental Study of Active Control of MDOF Structures Under Seismic Excitations," by L.L. Chung, R.C. Lin, T.T. Soong and A.M. Reinhorn, 7/10/88, (PB89-122600/AS).

NCEER-88-0026 "Earthquake Simulation Tests of a Low-Rise Metal Structure," by J.S. Hwang, K.C. Chang, G.C. Lee and R.L. Ketter, 8/1/88, (PB89-102917/AS).

NCEER-88-0027 "Systems Study of Urban Response and Reconstruction Due to Catastrophic Earthquakes," by F. Kozin and H.K. Zhou, 9/22/88, (PB90-162348/AS).

NCEER-88-0028 "Seismic Fragility Analysis of Plane Frame Structures," by H.H-M. Hwang and Y.K. Low, 7/31/88, (PB89-131445/AS).

NCEER-88-0029 "Response Analysis of Stochastic Structures," by A. Kardara, C. Bucher and M. Shinozuka, 9/22/88, (PB89-174429/AS).

NCEER-88-0030 "Nonnormal Accelerations Due to Yielding in a Primary Structure," by D.C.K. Chen and L.D. Lutes, 9/19/88, (PB89-131437/AS).

NCEER-88-0031 "Design Approaches for Soil-Structure Interaction," by A.S. Veletsos, A.M. Prasad and Y. Tang, 12/30/88, (PB89-174437/AS). This report is available only through NTIS (see address given above).

NCEER-88-0032 "A Re-evaluation of Design Spectra for Seismic Damage Control," by C.J. Turkstra and A.G. Tallin, 11/7/88, (PB89-145221/AS).

NCEER-88-0033 "The Behavior and Design of Noncontact Lap Splices Subjected to Repeated Inelastic Tensile Loading," by V.E. Sagan, P. Gergely and R.N. White, 12/8/88, (PB89-163737/AS).

NCEER-88-0034 "Seismic Response of Pile Foundations," by S.M. Mamoon, P.K. Banerjee and S. Ahmad, 11/1/88, (PB89-145239/AS).

NCEER-88-0035 "Modeling of R/C Building Structures With Flexible Floor Diaphragms (IDARC2)," by A.M. Reinhorn, S.K. Kunnath and N. Panahshahi, 9/7/88, (PB89-207153/AS).

NCEER-88-0036 "Solution of the Dam-Reservoir Interaction Problem Using a Combination of FEM, BEM with Particular Integrals, Modal Analysis, and Substructuring," by C-S. Tsai, G.C. Lee and R.L. Ketter, 12/31/88, (PB89-207146/AS).

NCEER-88-0037 "Optimal Placement of Actuators for Structural Control," by F.Y. Cheng and C.P. Pantelides, 8/15/88, (PB89-162846/AS).

NCEER-88-0038 "Teflon Bearings in Aseismic Base Isolation: Experimental Studies and Mathematical Modeling," by A. Mokha, M.C. Constantinou and A.M. Reinhorn, 12/5/88, (PB89-218457/AS). This report is available only through NTIS (see address given above).

NCEER-88-0039 "Seismic Behavior of Flat Slab High-Rise Buildings in the New York City Area," by P. Weidlinger and M. Ettouney, 10/15/88, (PB90-145681/AS).

NCEER-88-0040 "Evaluation of the Earthquake Resistance of Existing Buildings in New York City," by P. Weidlinger and M. Ettouney, 10/15/88, to be published.

NCEER-88-0041 "Small-Scale Modeling Techniques for Reinforced Concrete Structures Subjected to Seismic Loads," by W. Kim, A. El-Attar and R.N. White, 11/22/88, (PB89-189625/AS).

NCEER-88-0042 "Modeling Strong Ground Motion from Multiple Event Earthquakes," by G.W. Ellis and A.S. Cakmak, 10/15/88, (PB89-174445/AS).

NCEER-88-0043 "Nonstationary Models of Seismic Ground Acceleration," by M. Grigoriu, S.E. Ruiz and E. Rosenblueth, 7/15/88, (PB89-189617/AS).

NCEER-88-0044 "SARCF User's Guide: Seismic Analysis of Reinforced Concrete Frames," by Y.S. Chung, C. Meyer and M. Shinozuka, 11/9/88, (PB89-174452/AS).

NCEER-88-0045 "First Expert Panel Meeting on Disaster Research and Planning," edited by J. Pantelic and J. Stoyle, 9/15/88, (PB89-174460/AS).

NCEER-88-0046 "Preliminary Studies of the Effect of Degrading Infill Walls on the Nonlinear Seismic Response of Steel Frames," by C.Z. Chrysostomou, P. Gergely and J.F. Abel, 12/19/88, (PB89-208383/AS).

NCEER-88-0047 "Reinforced Concrete Frame Component Testing Facility - Design, Construction, Instrumentation and Operation," by S.P. Pessiki, C. Conley, T. Bond, P. Gergely and R.N. White, 12/16/88, (PB89-174478/AS).

NCEER-89-0001 "Effects of Protective Cushion and Soil Compliancy on the Response of Equipment Within a Seismically Excited Building," by J.A. HoLung, 2/16/89, (PB89-207179/AS).

NCEER-89-0002 "Statistical Evaluation of Response Modification Factors for Reinforced Concrete Structures," by H.H-M. Hwang and J-W. Jaw, 2/17/89, (PB89-207187/AS).

NCEER-89-0003 "Hysteretic Columns Under Random Excitation," by G-Q. Cai and Y.K. Lin, 1/9/89, (PB89-196513/AS).

NCEER-89-0004 "Experimental Study of 'Elephant Foot Bulge' Instability of Thin-Walled Metal Tanks," by Z-H. Jia and R.L. Ketter, 2/22/89, (PB89-207195/AS).

NCEER-89-0005 "Experiment on Performance of Buried Pipelines Across San Andreas Fault," by J. Isenberg, E. Richardson and T.D. O'Rourke, 3/10/89, (PB89-218440/AS).

NCEER-89-0006 "A Knowledge-Based Approach to Structural Design of Earthquake-Resistant Buildings," by M. Subramani, P. Gergely, C.H. Conley, J.F. Abel and A.H. Zaghw, 1/15/89, (PB89-218465/AS).

NCEER-89-0007 "Liquefaction Hazards and Their Effects on Buried Pipelines," by T.D. O'Rourke and P.A. Lane, 2/1/89, (PB89-218481).

NCEER-89-0008 "Fundamentals of System Identification in Structural Dynamics," by H. Imai, C-B. Yun, O. Maruyama and M. Shinozuka, 1/26/89, (PB89-207211/AS).

NCEER-89-0009 "Effects of the 1985 Michoacan Earthquake on Water Systems and Other Buried Lifelines in Mexico," by A.G. Ayala and M.J. O'Rourke, 3/8/89, (PB89-207229/AS).

NCEER-89-R010 "NCEER Bibliography of Earthquake Education Materials," by K.E.K. Ross, Second Revision, 9/1/89, (PB90-125352/AS).

NCEER-89-0011 "Inelastic Three-Dimensional Response Analysis of Reinforced Concrete Building Structures (IDARC-3D), Part I - Modeling," by S.K. Kunnath and A.M. Reinhorn, 4/17/89, (PB90-114612/AS).

NCEER-89-0012 "Recommended Modifications to ATC-14," by C.D. Poland and J.O. Malley, 4/12/89, (PB90-108648/AS).

NCEER-89-0013 "Repair and Strengthening of Beam-to-Column Connections Subjected to Earthquake Loading," by M. Corazao and A.J. Durrani, 2/28/89, (PB90-109885/AS).

NCEER-89-0014 "Program EXKAL2 for Identification of Structural Dynamic Systems," by O. Maruyama, C-B. Yun, M. Hoshiya and M. Shinozuka, 5/19/89, (PB90-109877/AS).

NCEER-89-0015 "Response of Frames With Bolted Semi-Rigid Connections, Part I - Experimental Study and Analytical Predictions," by P.J. DiCorso, A.M. Reinhorn, J.R. Dickerson, J.B. Radziminski and W.L. Harper, 6/1/89, to be published.

NCEER-89-0016 "ARMA Monte Carlo Simulation in Probabilistic Structural Analysis," by P.D. Spanos and M.P. Mignolet, 7/10/89, (PB90-109893/AS).

NCEER-89-P017 "Preliminary Proceedings from the Conference on Disaster Preparedness - The Place of Earthquake Education in Our Schools," Edited by K.E.K. Ross, 6/23/89.

NCEER-89-0017 "Proceedings from the Conference on Disaster Preparedness - The Place of Earthquake Education in Our Schools," Edited by K.E.K. Ross, 12/31/89, (PB90-207895). This report is available only through NTIS (see address given above).

NCEER-89-0018 "Multidimensional Models of Hysteretic Material Behavior for Vibration Analysis of Shape Memory Energy Absorbing Devices, by E.J. Graesser and F.A. Cozzarelli, 6/7/89, (PB90-164146/AS).

NCEER-89-0019 "Nonlinear Dynamic Analysis of Three-Dimensional Base Isolated Structures (3D-BASIS)," by S. Nagarajaiah, A.M. Reinhorn and M.C. Constantinou, 8/3/89, (PB90-161936/AS). This report is available only through NTIS (see address given above).

NCEER-89-0020 "Structural Control Considering Time-Rate of Control Forces and Control Rate Constraints," by F.Y. Cheng and C.P. Pantelides, 8/3/89, (PB90-120445/AS).

NCEER-89-0021 "Subsurface Conditions of Memphis and Shelby County," by K.W. Ng, T-S. Chang and H-H.M. Hwang, 7/26/89, (PB90-120437/AS).

NCEER-89-0022 "Seismic Wave Propagation Effects on Straight Jointed Buried Pipelines," by K. Elhmadi and M.J. O'Rourke, 8/24/89, (PB90-162322/AS).

NCEER-89-0023 "Workshop on Serviceability Analysis of Water Delivery Systems," edited by M. Grigoriu, 3/6/89, (PB90-127424/AS).

NCEER-89-0024 "Shaking Table Study of a 1/5 Scale Steel Frame Composed of Tapered Members," by K.C. Chang, J.S. Hwang and G.C. Lee, 9/18/89, (PB90-160169/AS).

NCEER-89-0025 "DYNA1D: A Computer Program for Nonlinear Seismic Site Response Analysis - Technical Documentation," by Jean H. Prevost, 9/14/89, (PB90-161944/AS). This report is available only through NTIS (see address given above).

NCEER-89-0026 "1:4 Scale Model Studies of Active Tendon Systems and Active Mass Dampers for Aseismic Protection," by A.M. Reinhorn, T.T. Soong, R.C. Lin, Y.P. Yang, Y. Fukao, H. Abe and M. Nakai, 9/15/89, (PB90-173246/AS).

NCEER-89-0027 "Scattering of Waves by Inclusions in a Nonhomogeneous Elastic Half Space Solved by Boundary Element Methods," by P.K. Hadley, A. Askar and A.S. Cakmak, 6/15/89, (PB90-145699/AS).

NCEER-89-0028 "Statistical Evaluation of Deflection Amplification Factors for Reinforced Concrete Structures," by H.H.M. Hwang, J-W. Jaw and A.L. Ch'ng, 8/31/89, (PB90-164633/AS).

NCEER-89-0029 "Bedrock Accelerations in Memphis Area Due to Large New Madrid Earthquakes," by H.H.M. Hwang, C.H.S. Chen and G. Yu, 11/7/89, (PB90-162330/AS).

NCEER-89-0030 "Seismic Behavior and Response Sensitivity of Secondary Structural Systems," by Y.Q. Chen and T.T. Soong, 10/23/89, (PB90-164658/AS).

NCEER-89-0031 "Random Vibration and Reliability Analysis of Primary-Secondary Structural Systems," by Y. Ibrahim, M. Grigoriu and T.T. Soong, 11/10/89, (PB90-161951/AS).

NCEER-89-0032 "Proceedings from the Second U.S. - Japan Workshop on Liquefaction, Large Ground Deformation and Their Effects on Lifelines, September 26-29, 1989," Edited by T.D. O'Rourke and M. Hamada, 12/1/89, (PB90-209388/AS).

NCEER-89-0033 "Deterministic Model for Seismic Damage Evaluation of Reinforced Concrete Structures," by J.M. Bracci, A.M. Reinhorn, J.B. Mander and S.K. Kunnath, 9/27/89.

NCEER-89-0034 "On the Relation Between Local and Global Damage Indices," by E. DiPasquale and A.S. Cakmak, 8/15/89, (PB90-173865).

NCEER-89-0035 "Cyclic Undrained Behavior of Nonplastic and Low Plasticity Silts," by A.J. Walker and H.E. Stewart, 7/26/89, (PB90-183518/AS).

NCEER-89-0036 "Liquefaction Potential of Surficial Deposits in the City of Buffalo, New York," by M. Budhu, R. Giese and L. Baumgrass, 1/17/89, (PB90-208455/AS).

NCEER-89-0037 "A Deterministic Assessment of Effects of Ground Motion Incoherence," by A.S. Veletsos and Y. Tang, 7/15/89, (PB90-164294/AS).

NCEER-89-0038 "Workshop on Ground Motion Parameters for Seismic Hazard Mapping," July 17-18, 1989, edited by R.V. Whitman, 12/1/89, (PB90-173923/AS).

NCEER-89-0039 "Seismic Effects on Elevated Transit Lines of the New York City Transit Authority," by C.J. Costantino, C.A. Miller and E. Heymsfield, 12/26/89, (PB90-207887/AS).

NCEER-89-0040 "Centrifugal Modeling of Dynamic Soil-Structure Interaction," by K. Weissman, Supervised by J.H. Prevost, 5/10/89, (PB90-207879/AS).

NCEER-89-0041 "Linearized Identification of Buildings With Cores for Seismic Vulnerability Assessment," by I-K. Ho and A.E. Aktan, 11/1/89, (PB90-251943/AS).

NCEER-90-0001 "Geotechnical and Lifeline Aspects of the October 17, 1989 Loma Prieta Earthquake in San Francisco," by T.D. O'Rourke, H.E. Stewart, F.T. Blackburn and T.S. Dickerman, 1/90, (PB90-208596/AS).

NCEER-90-0002 "Nonnormal Secondary Response Due to Yielding in a Primary Structure," by D.C.K. Chen and L.D. Lutes, 2/28/90, (PB90-251976/AS).

NCEER-90-0003 "Earthquake Education Materials for Grades K-12," by K.E.K. Ross, 4/16/90, (PB91-113415/AS).

NCEER-90-0004 "Catalog of Strong Motion Stations in Eastern North America," by R.W. Busby, 4/3/90, (PB90-251984)/AS.

NCEER-90-0005 "NCEER Strong-Motion Data Base: A User Manual for the GeoBase Release (Version 1.0 for the Sun3)," by P. Friberg and K. Jacob, 3/31/90 (PB90-258062/AS).

NCEER-90-0006 "Seismic Hazard Along a Crude Oil Pipeline in the Event of an 1811-1812 Type New Madrid Earthquake," by H.H.M. Hwang and C-H.S. Chen, 4/16/90(PB90-258054).

NCEER-90-0007 "Site-Specific Response Spectra for Memphis Sheahan Pumping Station," by H.H.M. Hwang and C.S. Lee, 5/15/90, (PB91-108811/AS).

NCEER-90-0008 "Pilot Study on Seismic Vulnerability of Crude Oil Transmission Systems," by T. Ariman, R. Dobry, M. Grigoriu, F. Kozin, M. O'Rourke, T. O'Rourke and M. Shinozuka, 5/25/90, (PB91-108837/AS).

NCEER-90-0009 "A Program to Generate Site Dependent Time Histories: EQGEN," by G.W. Ellis, M. Srinivasan and A.S. Cakmak, 1/30/90, (PB91-108829/AS).

NCEER-90-0010 "Active Isolation for Seismic Protection of Operating Rooms," by M.E. Talbott, Supervised by M. Shinozuka, 6/8/9, (PB91-110205/AS).

NCEER-90-0011 "Program LINEARID for Identification of Linear Structural Dynamic Systems," by C-B. Yun and M. Shinozuka, 6/25/90, (PB91-110312/AS).

NCEER-90-0012 "Two-Dimensional Two-Phase Elasto-Plastic Seismic Response of Earth Dams," by A.N. Yiagos, Supervised by J.H. Prevost, 6/20/90, (PB91-110197/AS).

NCEER-90-0013 "Secondary Systems in Base-Isolated Structures: Experimental Investigation, Stochastic Response and Stochastic Sensitivity," by G.D. Manolis, G. Juhn, M.C. Constantinou and A.M. Reinhorn, 7/1/90, (PB91-110320/AS).

NCEER-90-0014 "Seismic Behavior of Lightly-Reinforced Concrete Column and Beam-Column Joint Details," by S.P. Pessiki, C.H. Conley, P. Gergely and R.N. White, 8/22/90, (PB91-108795/AS).

NCEER-90-0015 "Two Hybrid Control Systems for Building Structures Under Strong Earthquakes," by J.N. Yang and A. Danielians, 6/29/90, (PB91-125393/AS).

NCEER-90-0016 "Instantaneous Optimal Control with Acceleration and Velocity Feedback," by J.N. Yang and Z. Li, 6/29/90, (PB91-125401/AS).

NCEER-90-0017 "Reconnaissance Report on the Northern Iran Earthquake of June 21, 1990," by M. Mehrain, 10/4/90, (PB91-125377/AS).

NCEER-90-0018 "Evaluation of Liquefaction Potential in Memphis and Shelby County," by T.S. Chang, P.S. Tang, C.S. Lee and H. Hwang, 8/10/90, (PB91-125427/AS).

NCEER-90-0019 "Experimental and Analytical Study of a Combined Sliding Disc Bearing and Helical Steel Spring Isolation System," by M.C. Constantinou, A.S. Mokha and A.M. Reinhorn, 10/4/90, (PB91-125385/AS).

NCEER-90-0020 "Experimental Study and Analytical Prediction of Earthquake Response of a Sliding Isolation System with a Spherical Surface," by A.S. Mokha, M.C. Constantinou and A.M. Reinhorn, 10/11/90, (PB91-125419/AS).

NCEER-90-0021 "Dynamic Interaction Factors for Floating Pile Groups," by G. Gazetas, K. Fan, A. Kaynia and E. Kausel, 9/10/90, (PB91-170381/AS).

NCEER-90-0022 "Evaluation of Seismic Damage Indices for Reinforced Concrete Structures," by S. Rodriguez-Gomez and A.S. Cakmak, 9/30/90, PB91-171322/AS).

NCEER-90-0023 "Study of Site Response at a Selected Memphis Site," by H. Desai, S. Ahmad, E.S. Gazetas and M.R. Oh, 10/11/90, (PB91-196857/AS).

NCEER-90-0024 "A User's Guide to Strongmo: Version 1.0 of NCEER's Strong-Motion Data Access Tool for PCs and Terminals," by P.A. Friberg and C.A.T. Susch, 11/15/90, (PB91-171272/AS).

NCEER-90-0025 "A Three-Dimensional Analytical Study of Spatial Variability of Seismic Ground Motions," by L-L. Hong and A.H.-S. Ang, 10/30/90, (PB91-170399/AS).

NCEER-90-0026 "MUMOID User's Guide - A Program for the Identification of Modal Parameters," by S. Rodriguez-Gomez and E. DiPasquale, 9/30/90, (PB91-171298/AS).

NCEER-90-0027 "SARCF-II User's Guide - Seismic Analysis of Reinforced Concrete Frames," by S. Rodriguez-Gomez, Y.S. Chung and C. Meyer, 9/30/90, (PB91-171280/AS).

NCEER-90-0028 "Viscous Dampers: Testing, Modeling and Application in Vibration and Seismic Isolation," by N. Makris and M.C. Constantinou, 12/20/90 (PB91-190561/AS).

NCEER-90-0029 "Soil Effects on Earthquake Ground Motions in the Memphis Area," by H. Hwang, C.S. Lee, K.W. Ng and T.S. Chang, 8/2/90, (PB91-190751/AS).

NCEER-91-0001 "Proceedings from the Third Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures for Soil Liquefaction, December 17-19, 1990," edited by T.D. O'Rourke and M. Hamada, 2/1/91, (PB91-179259/AS).

NCEER-91-0002 "Physical Space Solutions of Non-Proportionally Damped Systems," by M. Tong, Z. Liang and G.C. Lee, 1/15/91, (PB91-179242/AS).

NCEER-91-0003 "Seismic Response of Single Piles and Pile Groups," by K. Fan and G. Gazetas, 1/10/91, (PB92-174994/AS).

NCEER-91-0004 "Damping of Structures: Part 1 - Theory of Complex Damping," by Z. Liang and G. Lee, 10/10/91, (PB92-197235/AS).

NCEER-91-0005 "3D-BASIS - Nonlinear Dynamic Analysis of Three Dimensional Base Isolated Structures: Part II," by S. Nagarajaiah, A.M. Reinhorn and M.C. Constantinou, 2/28/91, (PB91-190553/AS).

NCEER-91-0006 "A Multidimensional Hysteretic Model for Plasticity Deforming Metals in Energy Absorbing Devices," by E.J. Graesser and F.A. Cozzarelli, 4/9/91, (PB92-108364/AS).

NCEER-91-0007 "A Framework for Customizable Knowledge-Based Expert Systems with an Application to a KBES for Evaluating the Seismic Resistance of Existing Buildings," by E.G. Ibarra-Anaya and S.J. Fenves, 4/9/91, (PB91-210930/AS).

NCEER-91-0008 "Nonlinear Analysis of Steel Frames with Semi-Rigid Connections Using the Capacity Spectrum Method," by G.G. Deierlein, S-H. Hsieh, Y-J. Shen and J.F. Abel, 7/2/91, (PB92-113828/AS).

NCEER-91-0009 "Earthquake Education Materials for Grades K-12," by K.E.K. Ross, 4/30/91, (PB91-212142/AS).

NCEER-91-0010 "Phase Wave Velocities and Displacement Phase Differences in a Harmonically Oscillating Pile," by N. Makris and G. Gazetas, 7/8/91, (PB92-108356/AS).

NCEER-91-0011 "Dynamic Characteristics of a Full-Size Five-Story Steel Structure and a 2/5 Scale Model," by K.C. Chang, G.C. Yao, G.C. Lee, D.S. Hao and Y.C. Yeh," 7/2/91.

NCEER-91-0012 "Seismic Response of a 2/5 Scale Steel Structure with Added Viscoelastic Dampers," by K.C. Chang, T.T. Soong, S-T. Oh and M.L. Lai, 5/17/91 (PB92-110816/AS).

NCEER-91-0013 "Earthquake Response of Retaining Walls; Full-Scale Testing and Computational Modeling," by S. Alampalli and A-W.M. Elgamal, 6/20/91, to be published.

NCEER-91-0014 "3D-BASIS-M: Nonlinear Dynamic Analysis of Multiple Building Base Isolated Structures," by P.C. Tsopelas, S. Nagarajaiah, M.C. Constantinou and A.M. Reinhorn, 5/28/91, (PB92-113885/AS).

NCEER-91-0015 "Evaluation of SEAOC Design Requirements for Sliding Isolated Structures," by D. Theodossiou and M.C. Constantinou, 6/10/91, (PB92-114602/AS).

NCEER-91-0016 "Closed-Loop Modal Testing of a 27-Story Reinforced Concrete Flat Plate-Core Building," by H.R. Somaprasad, T. Toksoy, H. Yoshiyuki and A.E. Aktan, 7/15/91, (PB92-129980/AS).

NCEER-91-0017 "Shake Table Test of a 1/6 Scale Two-Story Lightly Reinforced Concrete Building," by A.G. El-Attar, R.N. White and P. Gergely, 2/28/91, (PB92-222447/AS).

NCEER-91-0018 "Shake Table Test of a 1/8 Scale Three-Story Lightly Reinforced Concrete Building," by A.G. El-Attar, R.N. White and P. Gergely, 2/28/91.

NCEER-91-0019 "Transfer Functions for Rigid Rectangular Foundations," by A.S. Veletsos, A.M. Prasad and W.H. Wu, 7/31/91.

NCEER-91-0020 "Hybrid Control of Seismic-Excited Nonlinear and Inelastic Structural Systems," by J.N. Yang, Z. Li and A. Danielians, 8/1/91, (PB92-143171/AS).

NCEER-91-0021 "The NCEER-91 Earthquake Catalog: Improved Intensity-Based Magnitudes and Recurrence Relations for U.S. Earthquakes East of New Madrid," by L. Seeber and J.G. Armbruster, 8/28/91, (PB92-176742/AS).

NCEER-91-0022 "Proceedings from the Implementation of Earthquake Planning and Education in Schools: The Need for Change - The Roles of the Changemakers," by K.E.K. Ross and F. Winslow, 7/23/91, (PB92-129998/AS).

NCEER-91-0023 "A Study of Reliability-Based Criteria for Seismic Design of Reinforced Concrete Frame Buildings," by H.H.M. Hwang and H-M. Hsu, 8/10/91, (PB92-140235/AS).

NCEER-91-0024 "Experimental Verification of a Number of Structural System Identification Algorithms," by R.G. Ghanem, H. Gavin and M. Shinozuka, 9/18/91, (PB92-176577/AS).

NCEER-91-0025 "Probabilistic Evaluation of Liquefaction Potential," by H.H.M. Hwang and C.S. Lee," 11/25/91, (PB92-143429/AS).

NCEER-91-0026 "Instantaneous Optimal Control for Linear, Nonlinear and Hysteretic Structures - Stable Controllers," by J.N. Yang and Z. Li, 11/15/91, (PB92-163807/AS).

NCEER-91-0027 "Experimental and Theoretical Study of a Sliding Isolation System for Bridges," by M.C. Constantinou, A. Kartoum, A.M. Reinhorn and P. Bradford, 11/15/91, (PB92-176973/AS).

NCEER-92-0001 "Case Studies of Liquefaction and Lifeline Performance During Past Earthquakes, Volume 1: Japanese Case Studies," Edited by M. Hamada and T. O'Rourke, 2/17/92, (PB92-197243/AS).

NCEER-92-0002 "Case Studies of Liquefaction and Lifeline Performance During Past Earthquakes, Volume 2: United States Case Studies," Edited by T. O'Rourke and M. Hamada, 2/17/92, (PB92-197250/AS).

NCEER-92-0003 "Issues in Earthquake Education," Edited by K. Ross, 2/3/92, (PB92-222389/AS).

NCEER-92-0004 "Proceedings from the First U.S. - Japan Workshop on Earthquake Protective Systems for Bridges," 2/4/92, to be published.

NCEER-92-0005 "Seismic Ground Motion from a Haskell-Type Source in a Multiple-Layered Half-Space," A.P. Theoharis, G. Deodatis and M. Shinozuka, 1/2/92, to be published.

NCEER-92-0006 "Proceedings from the Site Effects Workshop," Edited by R. Whitman, 2/29/92, (PB92-197201/AS).

NCEER-92-0007 "Engineering Evaluation of Permanent Ground Deformations Due to Seismically-Induced Liquefaction," by M.H. Baziar, R. Dobry and A-W.M. Elgamal, 3/24/92, (PB92-222421/AS).

NCEER-92-0008 "A Procedure for the Seismic Evaluation of Buildings in the Central and Eastern United States," by C.D. Poland and J.O. Malley, 4/2/92, (PB92-222439/AS).

NCEER-92-0009 "Experimental and Analytical Study of a Hybrid Isolation System Using Friction Controllable Sliding Bearings," by M.Q. Feng, S. Fujii and M. Shinozuka, 5/15/92, (PB93-150282/AS).

NCEER-92-0010 "Seismic Resistance of Slab-Column Connections in Existing Non-Ductile Flat-Plate Buildings," by A.J. Durrani and Y. Du, 5/18/92.

NCEER-92-0011 "The Hysteretic and Dynamic Behavior of Brick Masonry Walls Upgraded by Ferrocement Coatings Under Cyclic Loading and Strong Simulated Ground Motion," by H. Lee and S.P. Prawel, 5/11/92, to be published.

NCEER-92-0012 "Study of Wire Rope Systems for Seismic Protection of Equipment in Buildings," by G.F. Demetriades, M.C. Constantinou and A.M. Reinhorn, 5/20/92.

NCEER-92-0013 "Shape Memory Structural Dampers: Material Properties, Design and Seismic Testing," by P.R. Witting and F.A. Cozzarelli, 5/26/92.

NCEER-92-0014 "Longitudinal Permanent Ground Deformation Effects on Buried Continuous Pipelines," by M.J. O'Rourke, and C. Nordberg, 6/15/92.

NCEER-92-0015 "A Simulation Method for Stationary Gaussian Random Functions Based on the Sampling Theorem," by M. Grigoriu and S. Balopoulou, 6/11/92, (PB93-127496/AS).

NCEER-92-0016 "Gravity-Load-Designed Reinforced Concrete Buildings: Seismic Evaluation of Existing Construction and Detailing Strategies for Improved Seismic Resistance," by G.W. Hoffmann, S.K. Kunnath, J.B. Mander and A.M. Reinhorn, 7/15/92, to be published.

NCEER-92-0017 "Observations on Water System and Pipeline Performance in the Limón Area of Costa Rica Due to the April 22, 1991 Earthquake," by M. O'Rourke and D. Ballantyne, 6/30/92, (PB93-126811/AS).

NCEER-92-0018 "Fourth Edition of Earthquake Education Materials for Grades K-12," Edited by K.E.K. Ross, 8/10/92.

NCEER-92-0019 "Proceedings from the Fourth Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures for Soil Liquefaction," Edited by M. Hamada and T.D. O'Rourke, 8/12/92, (PB93-163939/AS).

NCEER-92-0020 "Active Bracing System: A Full Scale Implementation of Active Control," by A.M. Reinhorn, T.T. Soong, R.C. Lin, M.A. Riley, Y.P. Wang, S. Aizawa and M. Higashino, 8/14/92, (PB93-127512/AS).

NCEER-92-0021 "Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction-Induced Lateral Spreads," by S.F. Bartlett and T.L. Youd, 8/17/92.

NCEER-92-0022 "IDARC Version 3.0: Inelastic Damage Analysis of Reinforced Concrete Structures," by S.K. Kunnath, A.M. Reinhorn and R.F. Lobo, 8/31/92.