SEISMIC BEHAVIOR OF FLAT SLAB HIGH-RISE BUILDINGS IN THE NEW YORK CITY AREA

by

Paul Weidlinger[1] and Mohammed M. Ettouney[1]

October 15, 1988

Technical Report NCEER-88-0039

NCEER Contract Number 87-1002

NSF Master Contract Number ECE 86-07591

1 Weidlinger Associates, Consulting Engineers, New York City, New York

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

0272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NCEER-88-0039	2.	3. Recipient's Accession No. PB90 145681/AS

4. Title and Subtitle Seismic Behavior of Flat Slab High-Rise Buildings in the New York City Area	5. Report Date October 15, 1988
	6.
7. Author(s) Paul Weidlinger and Mohammed M. Ettouney	8. Performing Organization Rept. No:
9. Performing Organization Name and Address	10. Project/Task/Work Unit No.
	11. Contract(C) or Grant(G) No. (C) 87-1002 ECE 86-07591 (G)X
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
	14.

15. Supplementary Notes

This research was conducted at Weidlinger Associates and was partially supported by the National Science Foundation under Grant No. ECE 86-07591.

16. Abstract (Limit: 200 words)

This report examines the seismic response of typical high rise reinforced concrete residential structures. The findings are of importance in:

- the evaluation of the inherent seismic resistance of such structures, designed only for wind induced lateral forces, and
- in the evaluation on the effect of implementation of specific design requirements.

The conclusions, based on the examination of two existing high rise flat plate structures show that although specific design parameters (drift, base shear, overturning moment) exceed those induced by static wind pressure, the inherent capacity of these structures may be sufficient to resist moderate seismic excitation, if the structure satisfies wind drift design criteria.

17. Document Analysis a. Descriptors

b. Identifiers/Open-Ended Terms

EARTHQUAKE ENGINEERING
RESIDENTIAL STRUCTURES
REINFORCED CONCRETE
WIND INDUCED LATERAL FORCES
EARTHQUAKE RESISTANT DESIGN
HIGH RISE STRUCTURES
FLAT SLAB BUILDINGS
SEISMIC ACTIVITY
SHEAR
DUCTILITY

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
Release Unlimited	Unclassified	66
	20. Security Class (This Page) Unclassified	22. Price

PREFACE

The National Center for Earthquake Engineering Research (NCEER) is devoted to the expansion and dissemination of knowledge about earthquakes, the improvement of earthquake-resistant design, and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures and lifelines that are found in zones of moderate to high seismicity throughout the United States.

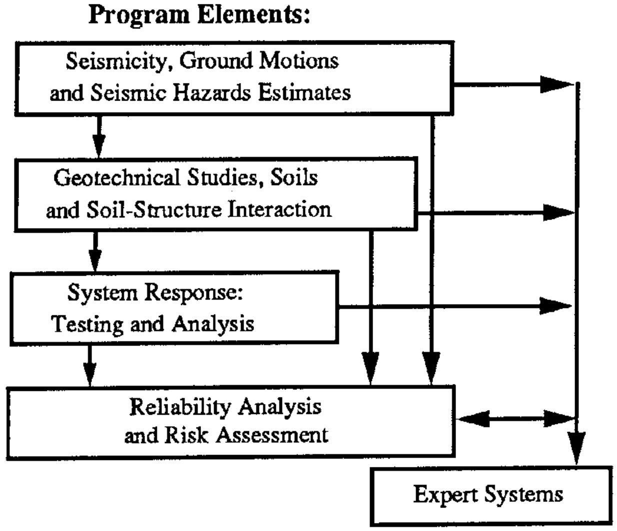
NCEER's research is being carried out in an integrated and coordinated manner following a structured program. The current research program comprises four main areas:

- Existing and New Structures
- Secondary and Protective Systems
- Lifeline Systems
- Disaster Research and Planning

This technical report pertains to Program 1, Existing and New Structures, and more specifically to Reliability Analysis and Risk Assessment.

The long term goal of research in Existing and New Structures is to develop seismic hazard mitigation procedures through rational probabilistic risk assessment for damage or collapse of structures, mainly existing buildings, in regions of moderate to high seismicity. This work relies on improved definitions of seismicity and site response, experimental and analytical evaluations of systems response, and more accurate assessment of risk factors. This technology will be incorporated in expert systems tools and improved code formats for existing and new structures. Methods of retrofit will also be developed. When this work is completed, it should be possible to characterize and quantify societal impact of seismic risk in various geographical regions and large municipalities. Toward this goal, the program has been divided into five components, as shown in the figure below:

Program Elements:



Tasks:

Earthquake Hazards Estimates,
Ground Motion Estimates,
New Ground Motion Instrumentation,
Earthquake & Ground Motion Data Base.

Site Response Estimates,
Large Ground Deformation Estimates,
Soil-Structure Interaction.

Typical Structures and Critical Structural Components:
Testing and Analysis;
Modern Analytical Tools.

Vulnerability Analysis,
Reliability Analysis,
Risk Assessment,
Code Upgrading.

Architectural and Structural Design,
Evaluation of Existing Buildings.

Reliability Analysis and Risk Assessment research constitutes one of the important areas of Existing and New Structures. Current research addresses, among others, the following issues:

1. Code issues - Development of a probabilistic procedure to determine load and resistance factors. Load Resistance Factor Design (LRFD) includes the investigation of wind vs. seismic issues, and of estimating design seismic loads for areas of moderate to high seismicity.
2. Response modification factors - Evaluation of RMFs for buildings and bridges which combine the effect of shear and bending.
3. Seismic damage - Development of damage estimation procedures which include a global and local damage index, and damage control by design; and development of computer codes for identification of the degree of building damage and automated damage-based design procedures.
4. Seismic reliability analysis of building structures - Development of procedures to evaluate the seismic safety of buildings which includes limit states corresponding to serviceability and collapse.
5. Retrofit procedures and restoration strategies.
6. Risk assessment and societal impact.

Research projects concerned with Reliability Analysis and Risk Assessment are carried out to provide practical tools for engineers to assess seismic risk to structures for the ultimate purpose of mitigating societal impact.

As part of the study of seismic risk to existing buildings designed only for wind loads, this report analyzes the forces in typical flat slab buildings for wind and seismic effects. The elastic and nonlinear seismic responses are compared with the provided capacities for two existing buildings. The need for better definition of seismic input in the New York City area is demonstrated.

ABSTRACT

This report examines the seismic response of typical high rise reinforced concrete residential structures. The findings are of importance in:

- the evaluation of the inherent seismic resistance of such structures, designed only for wind induced lateral forces, and
- in the evaluation on the effect of implementation of specific design requirements.

The conclusions, based on the examination of two existing high rise flat plate structures, show that although specific design parameters (drift, base shear, overturning moment) exceed those induced by static wind pressure, the inherent capacity of these structures may be sufficient to resist moderate seismic excitation, if the structure satisfies wind drift design criteria.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

SECTION 1
INTRODUCTION

This report examines the seismic resistance of reinforced concrete, high rise, residential structures not designed to resist seismic excitation. This class of structures is frequently constructed by using flat plate structures, where lateral force resistance is provided by shear walls in stairs and elevator cores, and occassionally by additional rigid frames on the perimeter. Two existing buildings were chosen for this study, which contain most of the typical and relevant features of these classes of structures in the New York city area. The lateral force resistance of the structures satisfies building code requirements with respect to wind. The report compares the response to seismic excitation with that of static wind response.

Depending on the intensity of the postulated seismic excitation (response spectrum), the amplitude of the seismic response was found generally to be much larger than that due to wind. On the other hand, because wind design is governed not by strength, but by drift requirements, (i.e. stiffness) it was observed that the required lateral force resistance against seismic excitation, which is sensitive to the input spectrum, is frequently well within the capacity of the structure. The implication of these findings are significant for future consideration in the evaluation of the economic impact of seismic design requirements in the New York region.

The seismic analysis was executed by taking into account the non-linear response of the structure. A simple non-linear model was used for this purpose and it was found that the ductility factor method provides reliable results, thereby simplifying the analysis of this class of structures.

SECTION 2
WIND AND SEISMIC BEHAVIOR

High rise residential buildings in New York City are usually constructed of reinforced concrete flat plate systems, where the floor slabs are supported directly on the columns. These high rise buildings are designed to resist lateral wind loadings, usually through a system of shear wall core and an outside framing system consisting of vertical columns and horizontal spandrel beams. These buildings are neither designed nor detailed to resist lateral seismic loads. In this section of the report, a comparison between the response of typical flat plate/slab buildings to wind and seismic loads is made.

In order to make such a comparison, two existing buildings of the type under consideration have been chosen. They are analyzed for wind and seismic loads. A comparison between the responses of the two buildings is made, using appropriate response measures. Finally, some conclusions are drawn from these comparisons.

2-1 Choice of Typical NYC Buildings

Two actual buildings were chosen for this study, referred to as NY1 and NY2. These two buildings contain most of the features relevant to this class of structures. This insures that the results and conclusions of this study are typical for a wide range of buildings. The features of the two buildings include, among others, the presence or absence of spandrel beams, height aspect ratio, torsional effects and typical spans between the columns. In what follows, a brief description is given of the different features of the two buildings.

2-1-1 Description of NY1

Figures 2-1 and 2-2 show an elevation view and a typical plan of NY1. It is a residential building of forty stories in New York City. The lateral load resisting system is mainly the elevator core in the center of the building. Each floor is a reinforced concrete flat slab resting directly on the vertical columns. The slab directly carries the weight of the outside walls, so there are no spandrel beams used in the building. The spans between the columns are typical for this type of construction. The building has almost symmetrical floor plan, so torsional modes are not expected to be important for either seismic or wind behavior. The global coordinate system (X,Y,Z, with Z being the vertical axis) of figure 2-2 is used throughout this study. Table 2-I shows the general overall dimensions of the building. The height to width ratio of the building makes it very slender in the Y-direction, while being moderately slender in the X-direction.

TABLE 2-I General Dimensions - NY1

Direction	Dimension (feet)
X-Direction	144
Y-Direction	183
Z-Direction (Height)	222

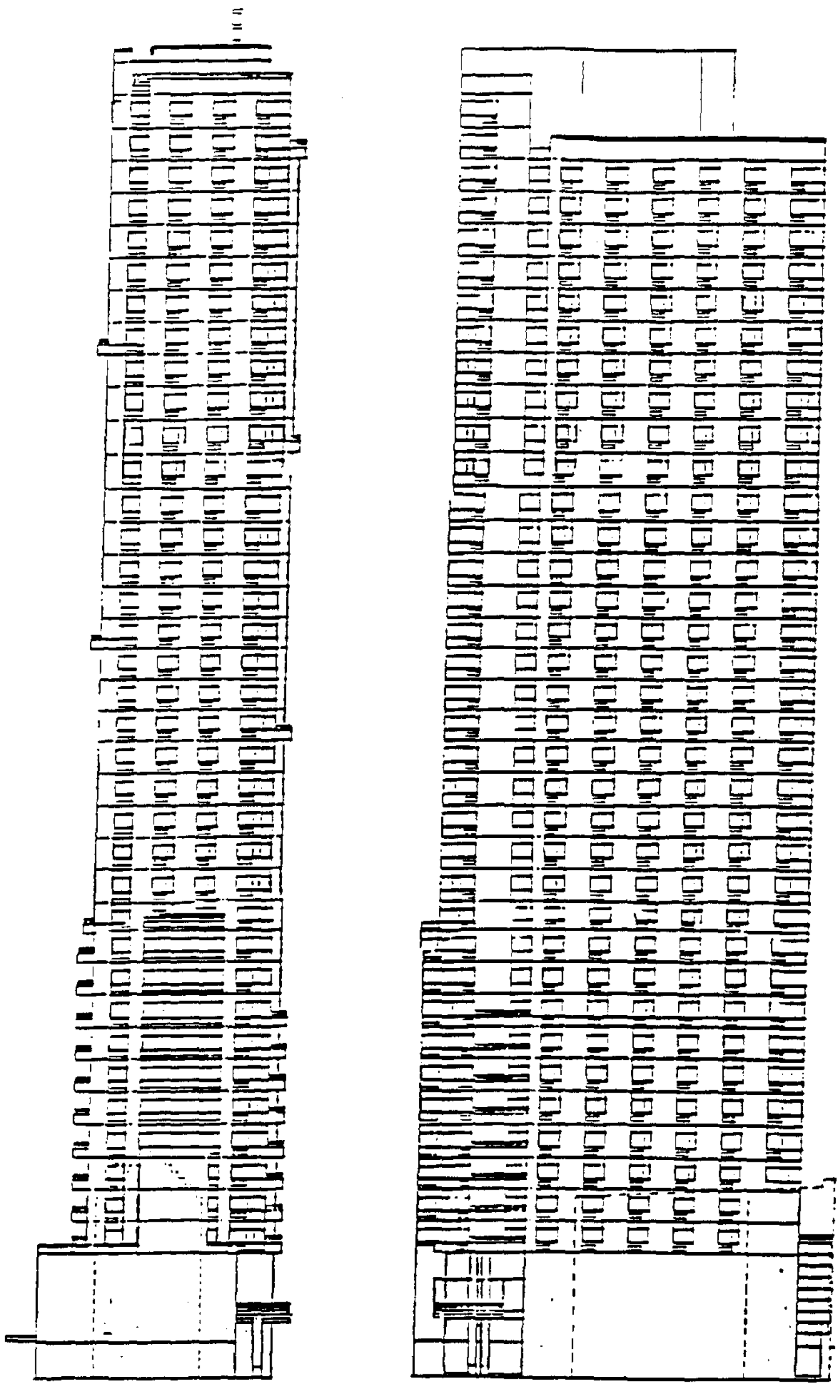


Fig. 2-1 View Elevation of NY1

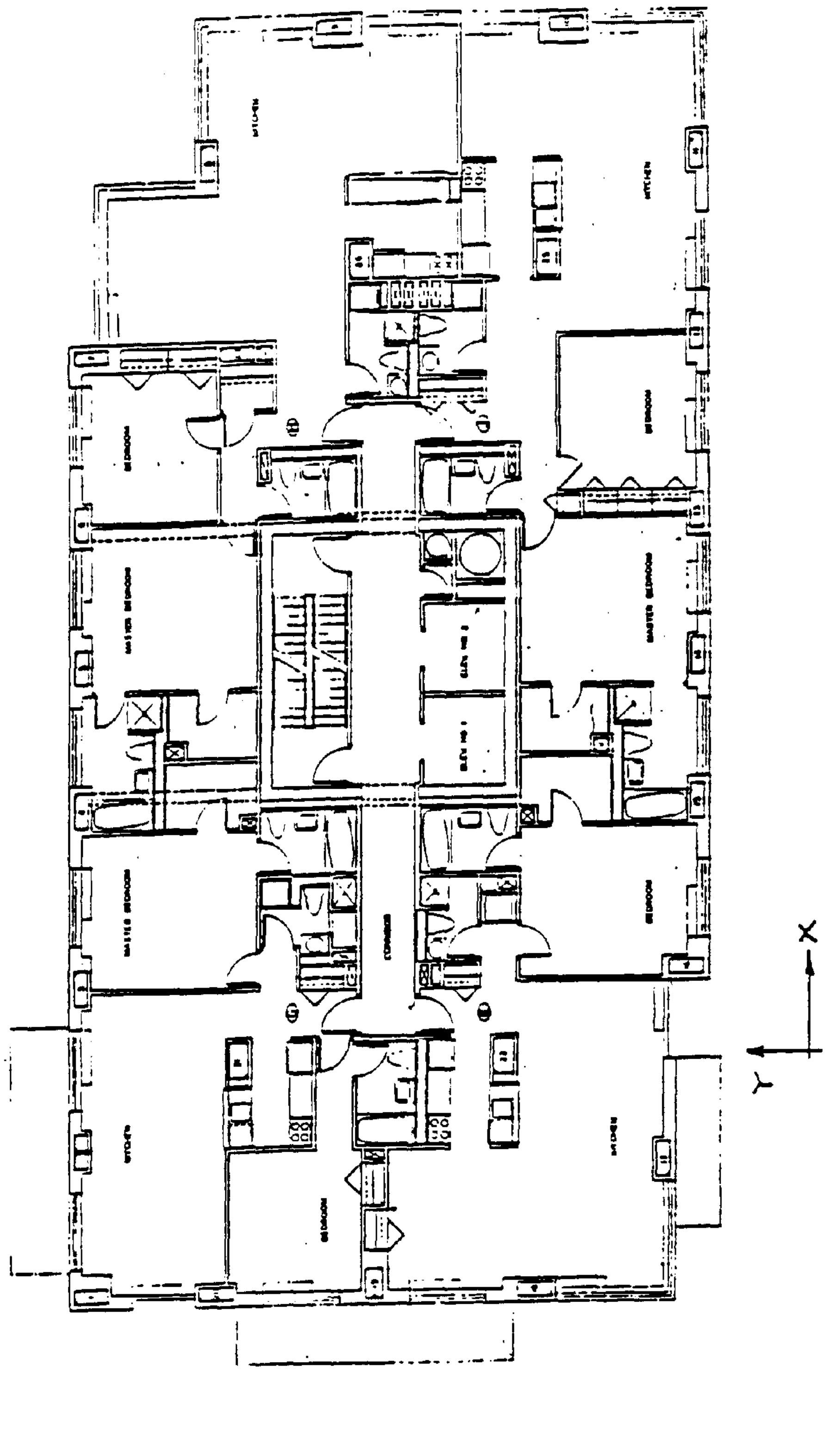


Fig. 2-2 Typical Plan of NY1

The building was designed to resist lateral wind loads as required by the New York City building code [11]. As it turned out, the steel reinforcement in the shear wall core was the minimum required by the ACI-83 design code [1]. The critical direction for the wind design was the Y-direction, as expected.

2-1-2 Description of NY2

Figure 2-3 shows a typical plan of the second building of this study, NY2, which has twenty four stories. The lateral load carrying mechanism is the sum of the following two systems:

i. Shear walls representing the elevator and stair cores.
ii. A frame system on the outside perimeter. It consists of vertical columns and horizontal spandrel beams, which carry the outside wall of the building.

Figure 2-3 also shows the global coordinate system which is referred to in the rest of this study. The building has an 'L' shaped floor plan which indicates possible major torsional effects during seismic or wind loading events. The floor system is a flat plate/slab system which is supported directly on the vertical interior columns. Table 2-II shows the general overall dimensions of the building. It is not as slender as NY1 in both the X- and the Y-directions.

The building was designed to resist lateral wind loads as required by the BOCA building code [6]. Similar to NY1, the steel reinforcement in the shear wall core was the minimum required by the ACI-83 design code [1]. The critical direction for the wind design was the X-direction, as expected.

2-2 Mathematical Model of the Two Buildings

The public domain computer code ETABS [16] was utilized for this study. The program is specialized for the linear static and dynamic analysis of multi-story structures. It has a wide range of structural modeling capabilities, including the ability to model shear walls, columns and beams. It can handle static wind and vertical loads, modal analysis as well as seismic spectral and time history analysis. Three dimensional modeling of buildings is a standard feature of the program.

A three dimensional mathematical model was prepared for each of the two buildings under consideration. All shear walls, columns, girders and structural slabs were included in the model of each building. The reinforced concrete structural elements were assumed to be uncracked, and the steel reinforcements were ignored, which is the customary way of modeling reinforced concrete buildings for linear analysis.

TABLE 2-II General Dimensions - NY2

Direction	Dimension (feet)
X-Direction	110
Y-Direction	56
Z-Direction (Height)	397

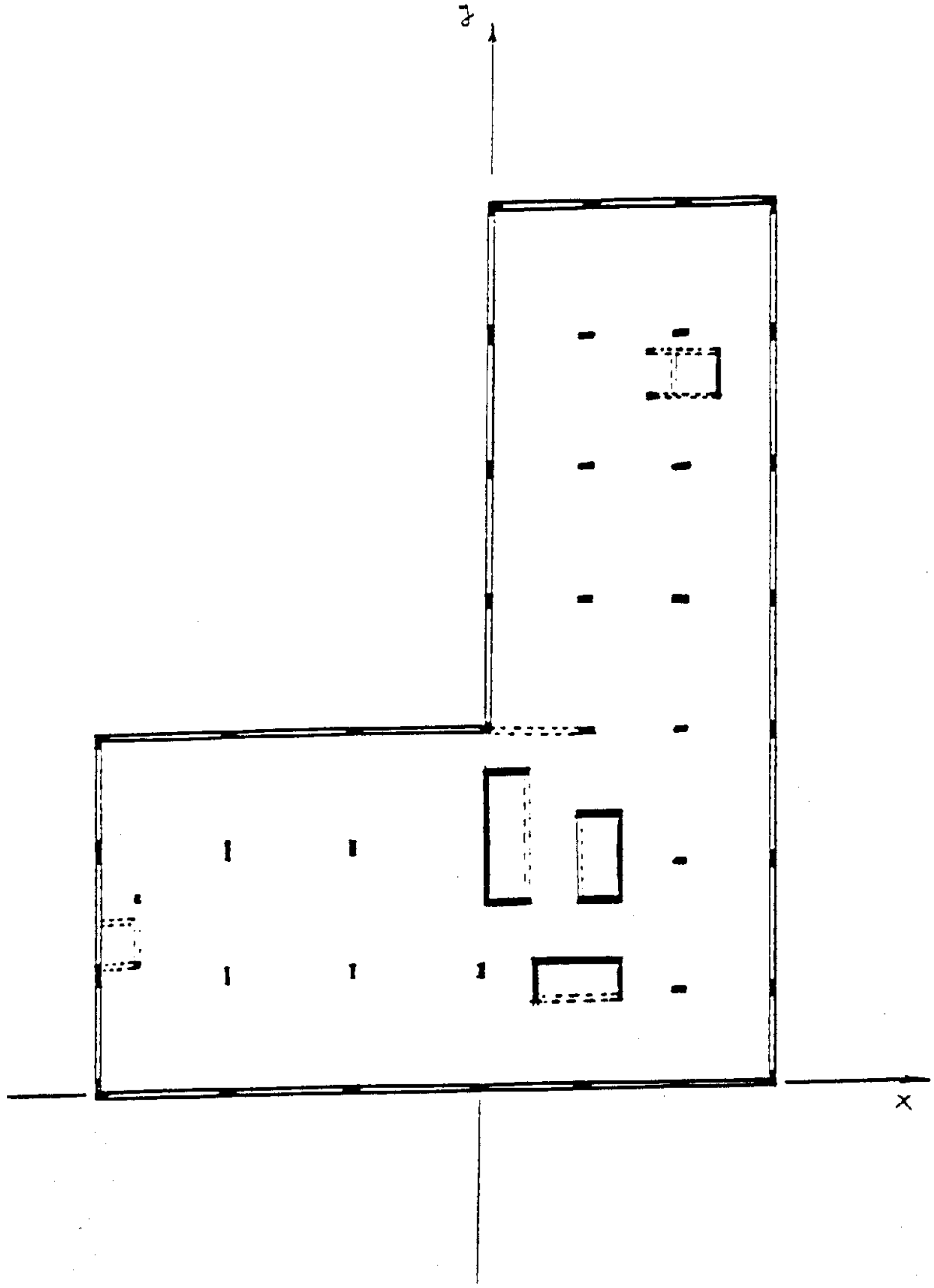


Fig. 2-3 Typical Plan of NY2

Three sets of computations were performed using ETABS. They are:

i. Modal analysis. This was needed in order to gain some insight into the dynamic properties of the systems under consideration.
ii. Wind analysis. These were static analysis runs. The deformations as well as the internal forces due to wind were calculated for the two buildings.
iii. Seismic spectral analysis. Seismic deformations as well as seismic internal forces were calculated. Several input design spectra were used in the analysis.

2-3 Seismic Input Spectra

Three design seismic spectra categories were considered for the purpose of this study. These categories are described below:

i. National design codes. There are several national design seismic spectra available for design engineers. These codes include the ATC-3 [3] and the UBC [14] among others. Although the two buildings were analyzed using both ATC-3 and the UBC (both 1985 and the 1988 recommended modifications), only the results using the ATC-3 design spectrum are reported here.
ii. Existing structures. The newly published ATC-14 [4] recommends the use of a design spectrum during the checking of, and retrofitting of, existing structures. The ATC-14 requirements are, of course, less stringent than those of ATC-3.
iii. Site dependent spectra. One of the major problems with using a general purpose 'national' seismic code is that it may overlook the special conditions of particular city or site. This is true for New York City in particular, and the east coast of the United States in general, [5], [12] and [13]. The 'national' seismic design spectra were developed for site conditions similar to those of the west coast of the United States. Studies are under way, and many more are needed, to establish the specific seismic design requirements of the east coast of the United States. One of the few available site dependent response spectra for New York City is reported in reference [15]. This spectrum is used in the present study and is referred to as the GWB spectrum.

Figure 2-4 shows the various design spectra as calculated for New York City site. Note that all the design spectra which were used in this study, including those of figure 2-4, are "elastic" spectra, i.e. buildings designed using any of them will have no ductility demands. The use of "elastic" spectra is needed since NY1 and NY2 were designed only for wind loading, and no special ductility provisions were observed in the design.

2-4 Modal Analysis Results

Figure 2-5 shows the first two fundamental modes of NY1, while figure 2-6 shows the first two fundamental modes of NY2. The taller building, NY1, shows more bending type behavior than shear in the first mode, while NY2, the shorter of the two buildings, shows a more dominant shear effect in its first mode. The bending effect increases for both buildings in the second mode.

An interesting, but not surprising, observation from the modal analysis of the two structures is that the fundamental periods (in each of the horizontal directions) are about twice those predicted by empirical formulae predicted by national building codes, such as the Uniform Building Code (UBC) [14]. Table 2-3 show the calculated fundamental periods of the two buildings using the present 3-D mathematical model and using the UBC empirical formulae. This is an understandable result since flat plate/slab type construction were not considered in developing seismic design codes.

Another important observation is that the coupling between torsional and lateral modes is of no practical significance (even for the asymmetrical building NY2) for this class of buildings.

2-5 Wind vs. Seismic Responses

The relative responses of the buildings due to wind and different seismic spectra is of importance. Both buildings (NY1 and NY2) were designed to resist wind loads, so if the response due to seismic forces was found to be less than that of wind, then wind forces govern the design, and no special provisions for seismic conditions are needed. On the other hand, if seismic responses were found to exceed those of wind responses, then special seismic provisions would have to be made for New York City site conditions for this class of buildings[1] .

For the purpose of this report, three response measures are considered. They are:

i. Horizontal displacement of each floor.
ii. Total story horizontal shear.
iii. Total story overturning moment.

Figures 2-7 through 2-9 show the ratio of the response measures (displacements, shears

[1] The use of "elastic" spectra makes a direct comparison between wind and seismic design possible, without having to include the detailing effects of seismic ductility demands.

TABLE 2-III Comparison of Natural Periods
(Seconds)

Building	Analysis (3D-Model)	Code (UBC)
NY1	5.8	2.7
NY2	3.4	1.8

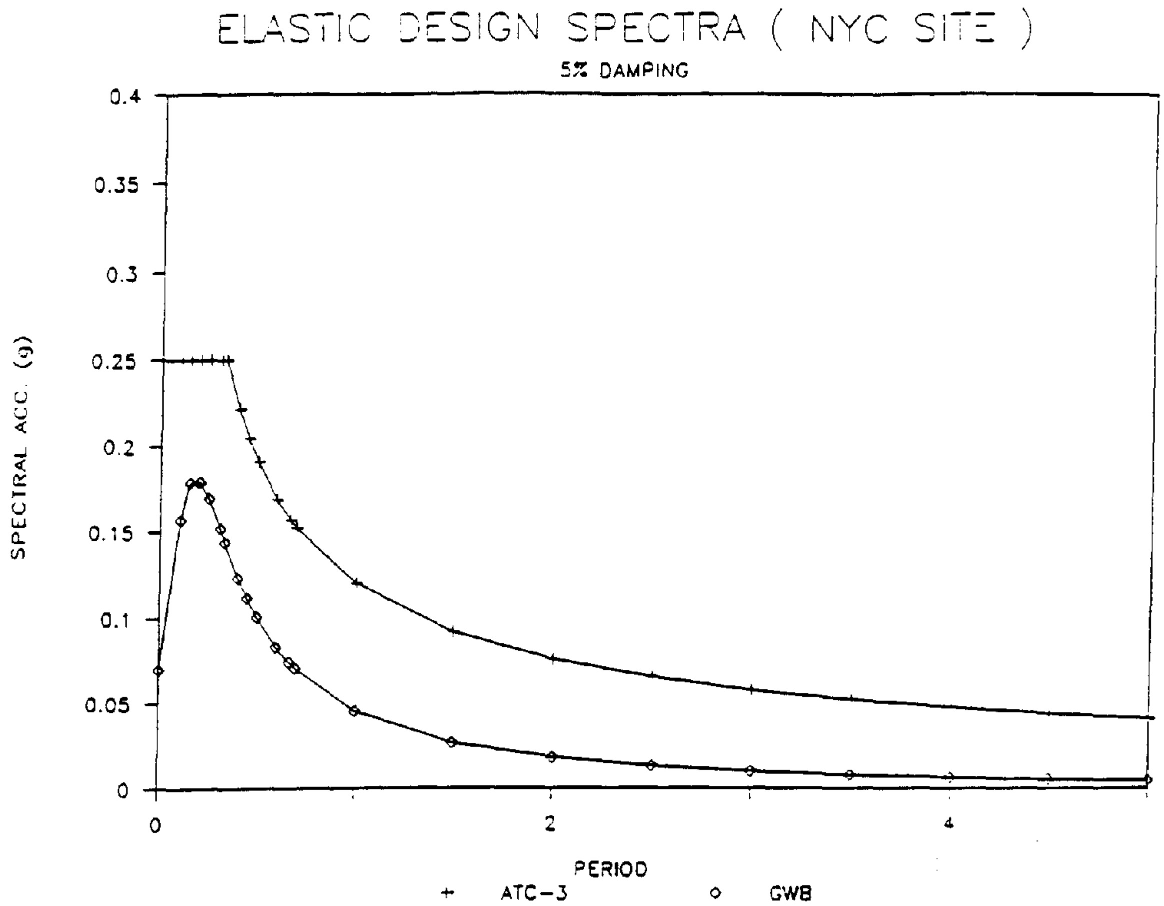


Fig. 2-4 Design Spectra

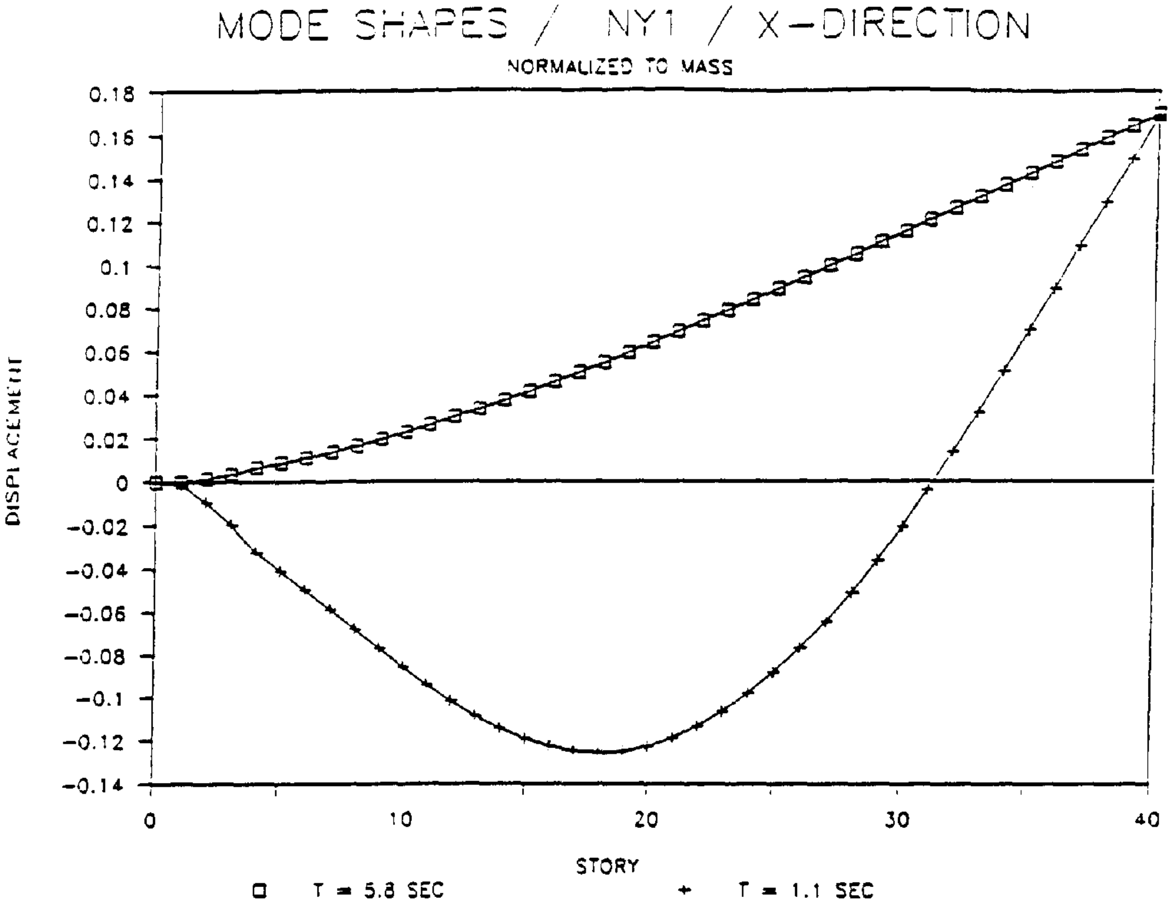


Fig. 2-5 Fundamental Modes, NY1

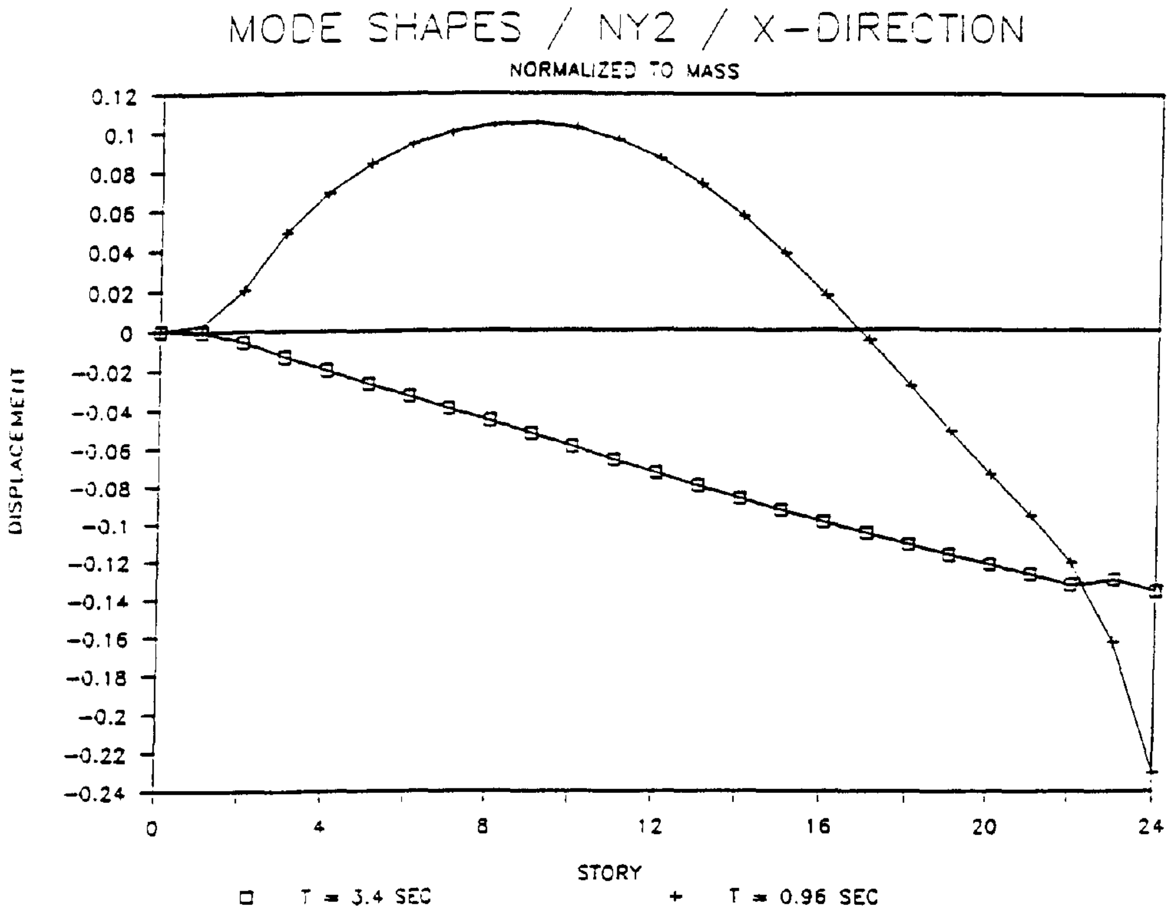


Fig. 2-6 Fundamental Modes, NY2

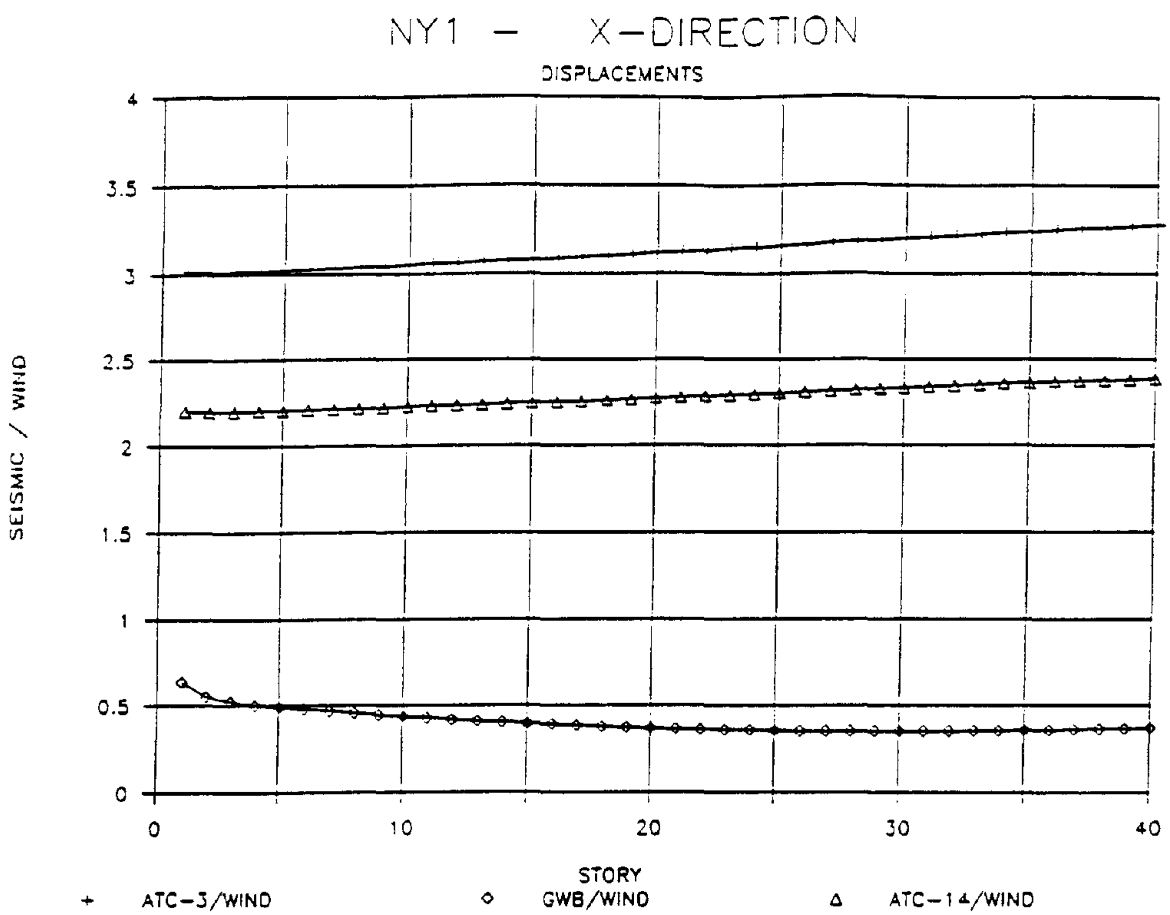


Fig. 2-7 Normalized Displacements, NY1, X-Direction

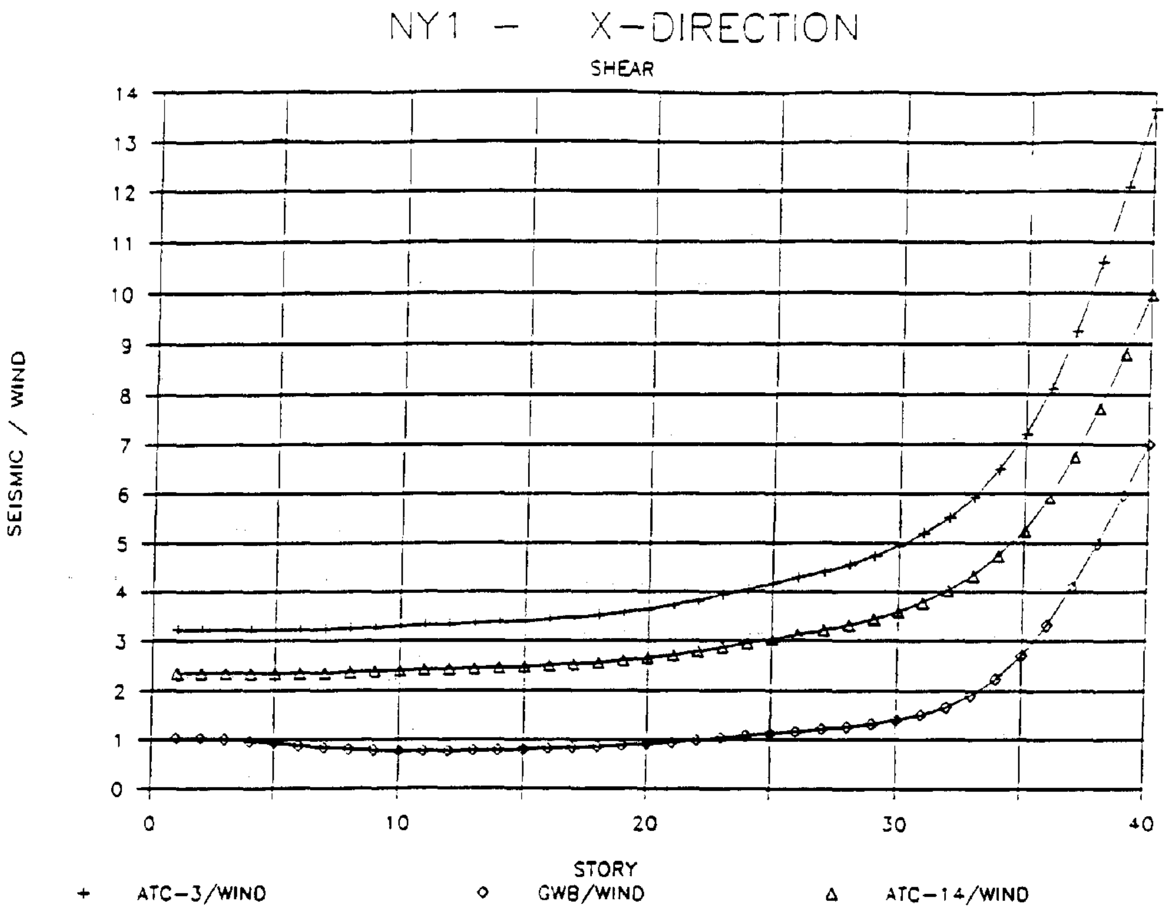


Fig. 2-8 Normalized Shear, NY1, X-Direction

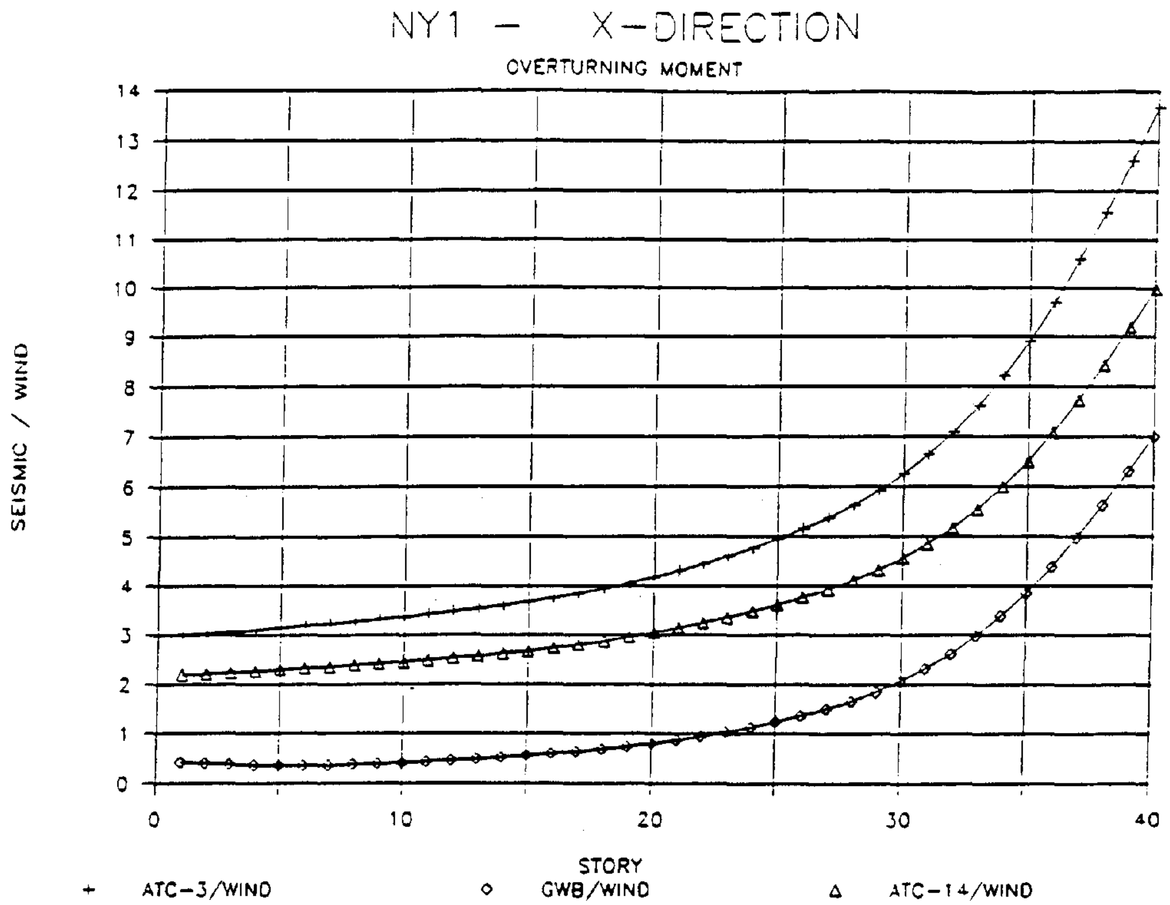


Fig. 2-9 Normalized OTM, NY1, X-Direction

and overturning moments) due to different seismic input spectra (ATC-3, ATC-14 and GWB) when divided by the responses due to wind loads, for NY1, in the X-direction. Figures 2-10 to 2-12 show similar comparison for NY1 in the Y-direction. Figures 2-13 through 2-15 show the seismic vs. wind ratios for NY2 in the X-direction, while figures 2-16 through 2-18 show the same ratios for NY2 in the Y-direction.

These results show that seismic responses exceed, by a wide margin, the wind responses for both buildings. The exceedance ratio changes with height, direction or type of building. As expected, the ATC-3 requirements are more demanding than those of the ATC-14 requirements, while the site dependent spectrum, GWB, has the least demands of the three.

The large seismic vs. wind response ratios, as shown in figures 2-7 through 2-18, are only part of the story. These buildings were designed not only to accommodate the three response measures due to wind, but also to accommodate several other design requirements from all applicable design codes. Thus, the as-built capacities of the buildings should be considered and are studied in the next section.

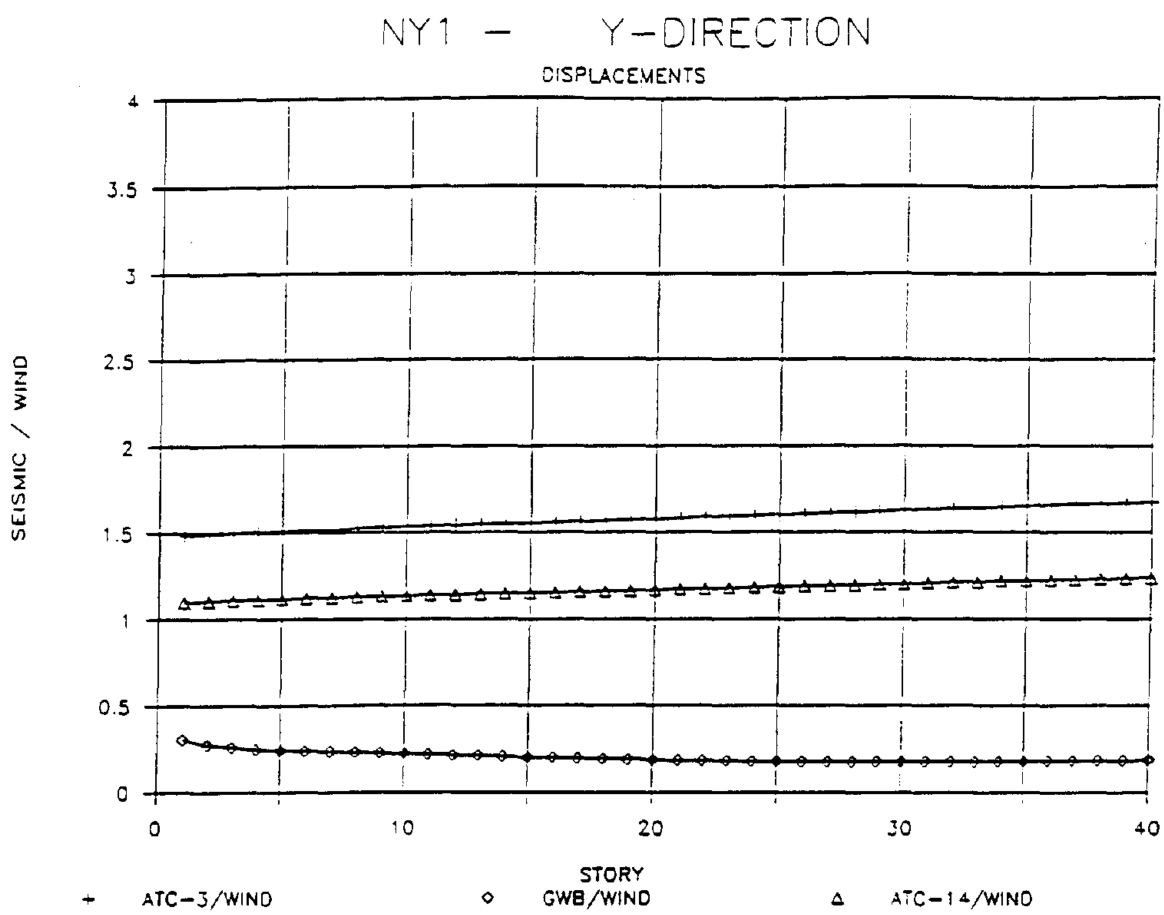


Fig. 2-10 Normalized Displacements, NY1, Y-Direction

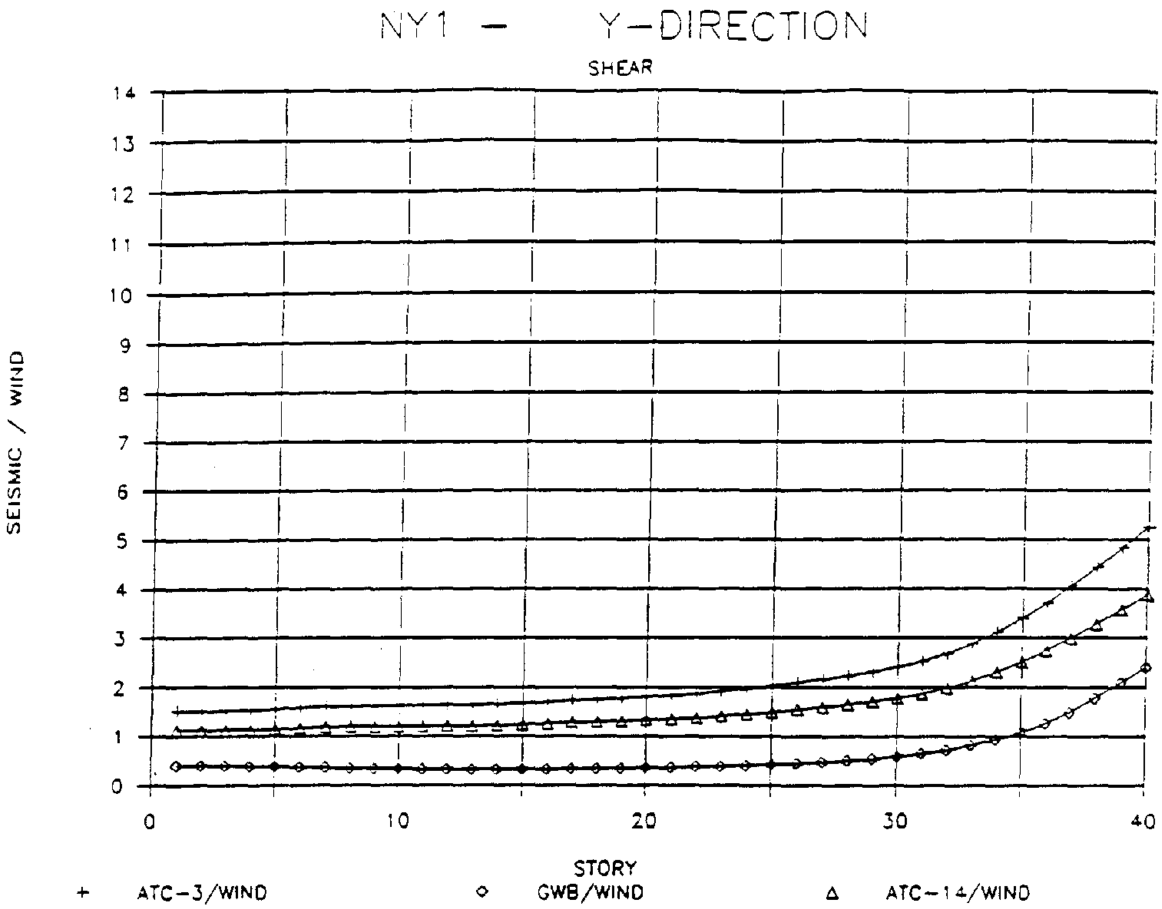


Fig. 2-11 Normalized Shear, NY1, Y-Direction

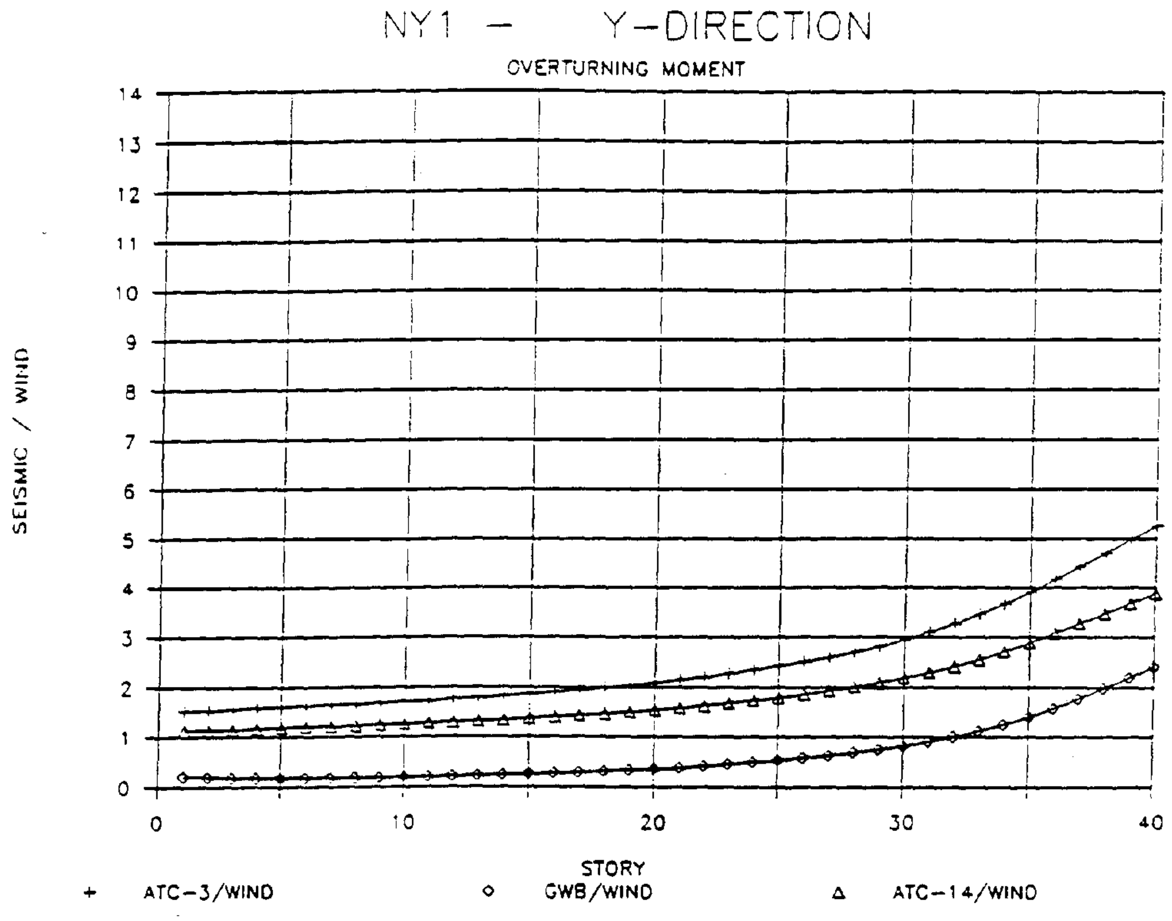


Fig. 2-12 Normalized OTM, NY1, Y-Direction

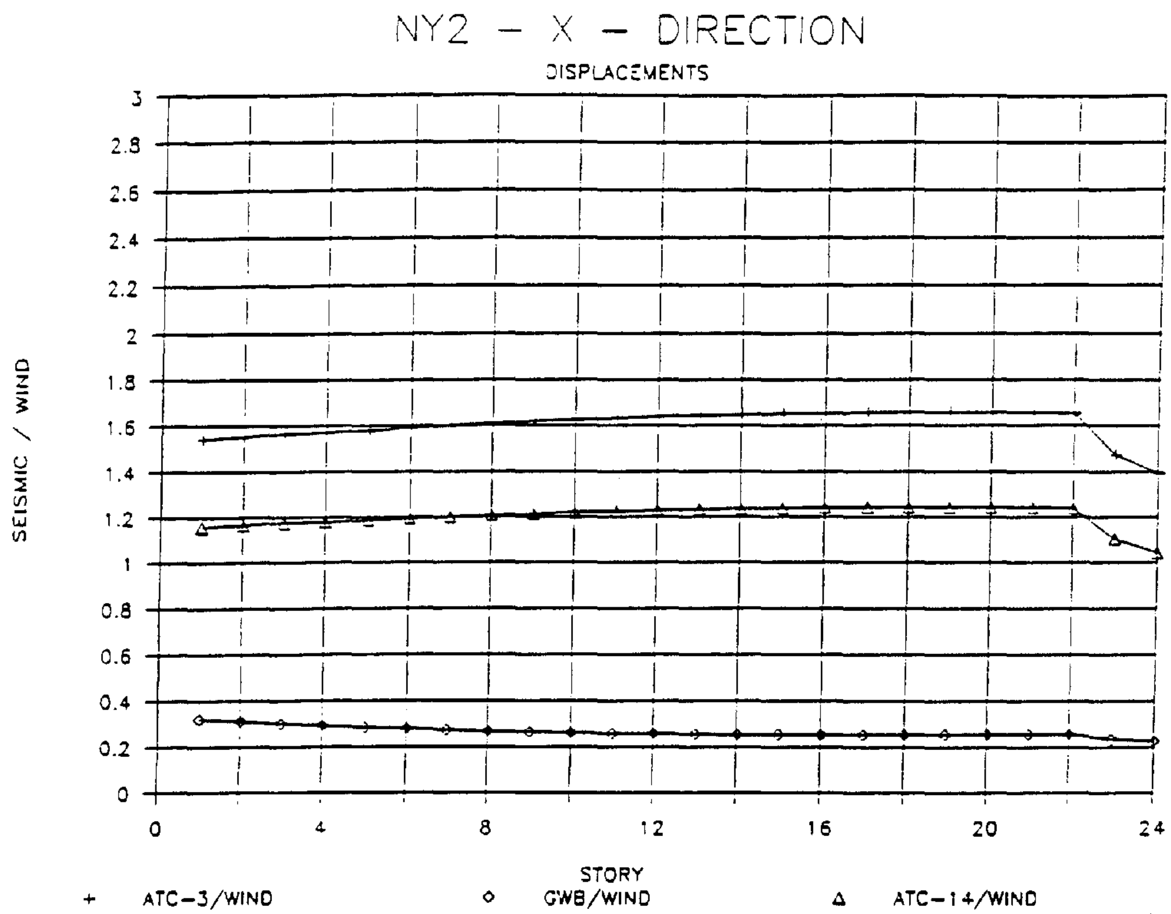


Fig. 2-13 Normalized Displacements, NY2, X-Direction

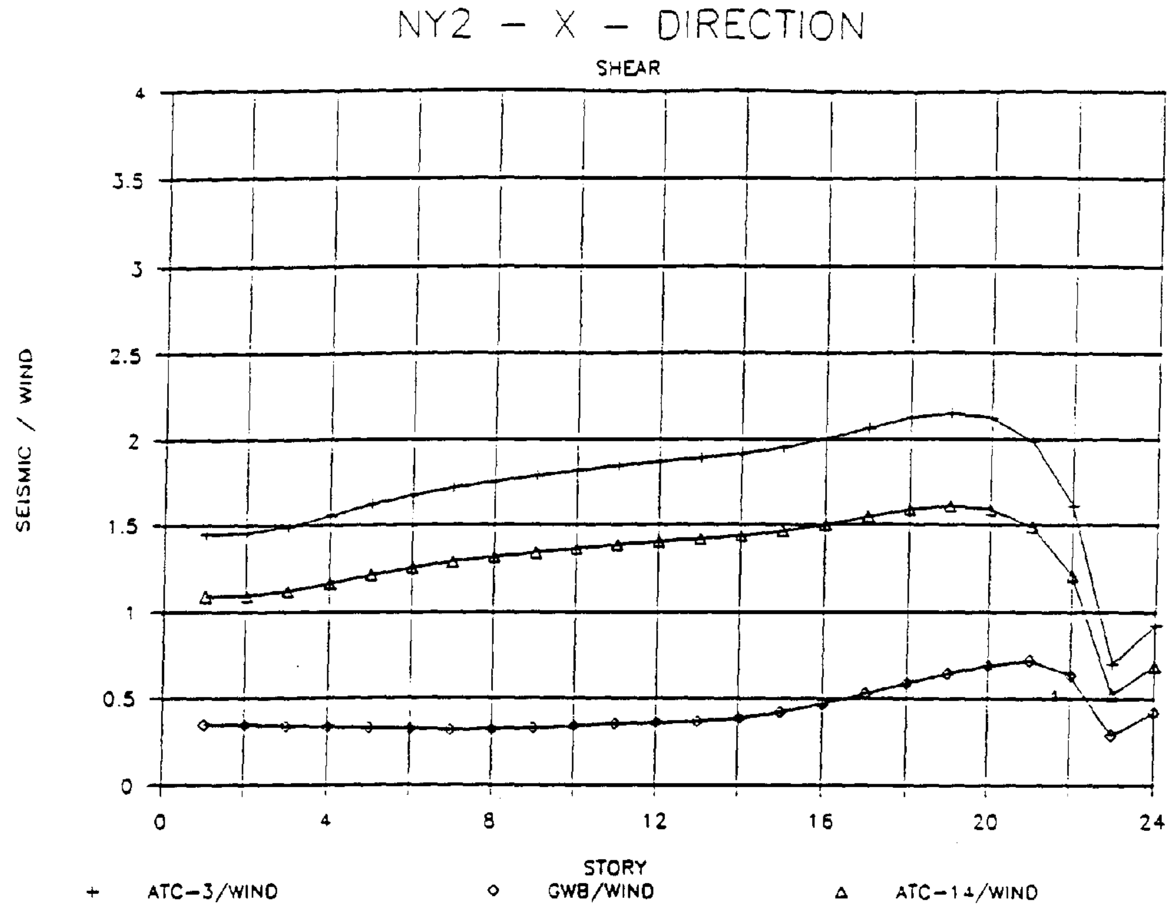


Fig. 2-14 Normalized Shear, NY2, X-Direction

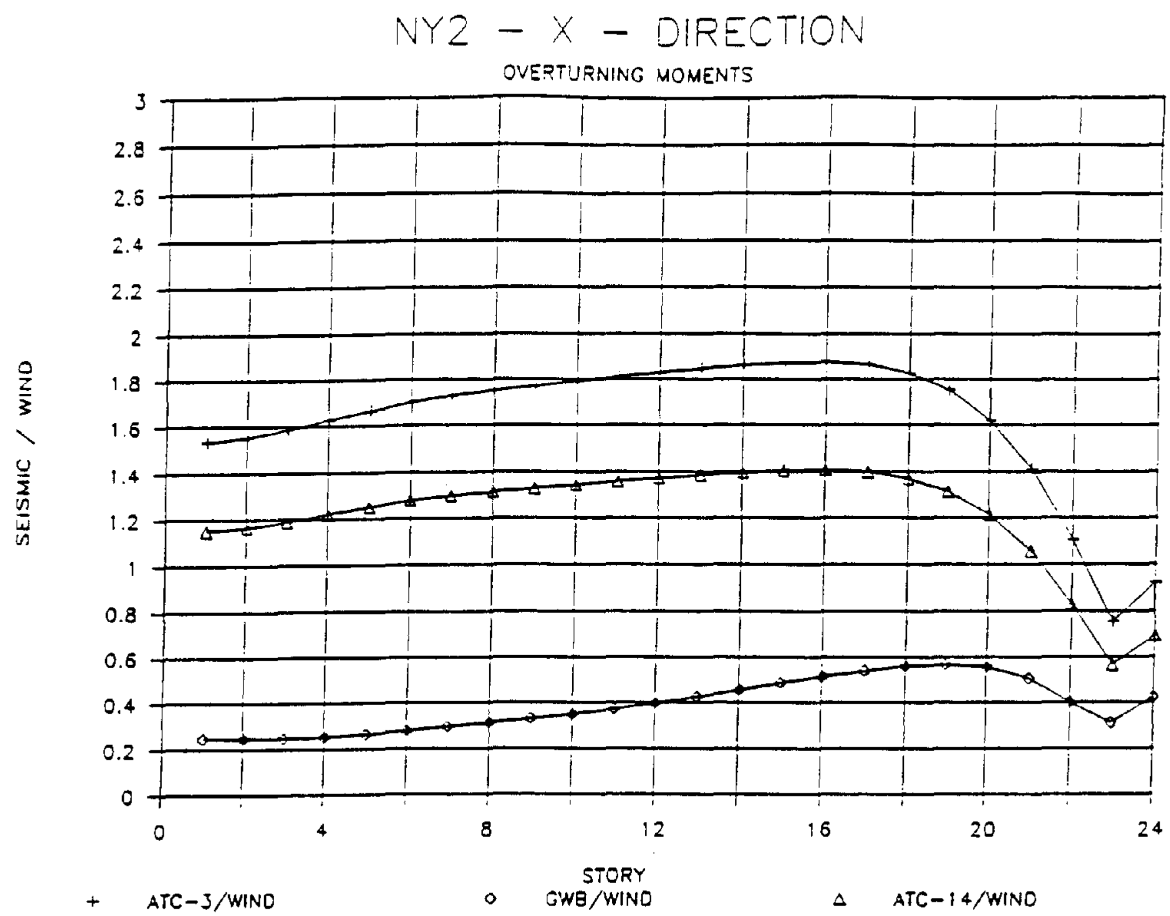


Fig. 2-15 Normalized OTM, NY2, X-Direction

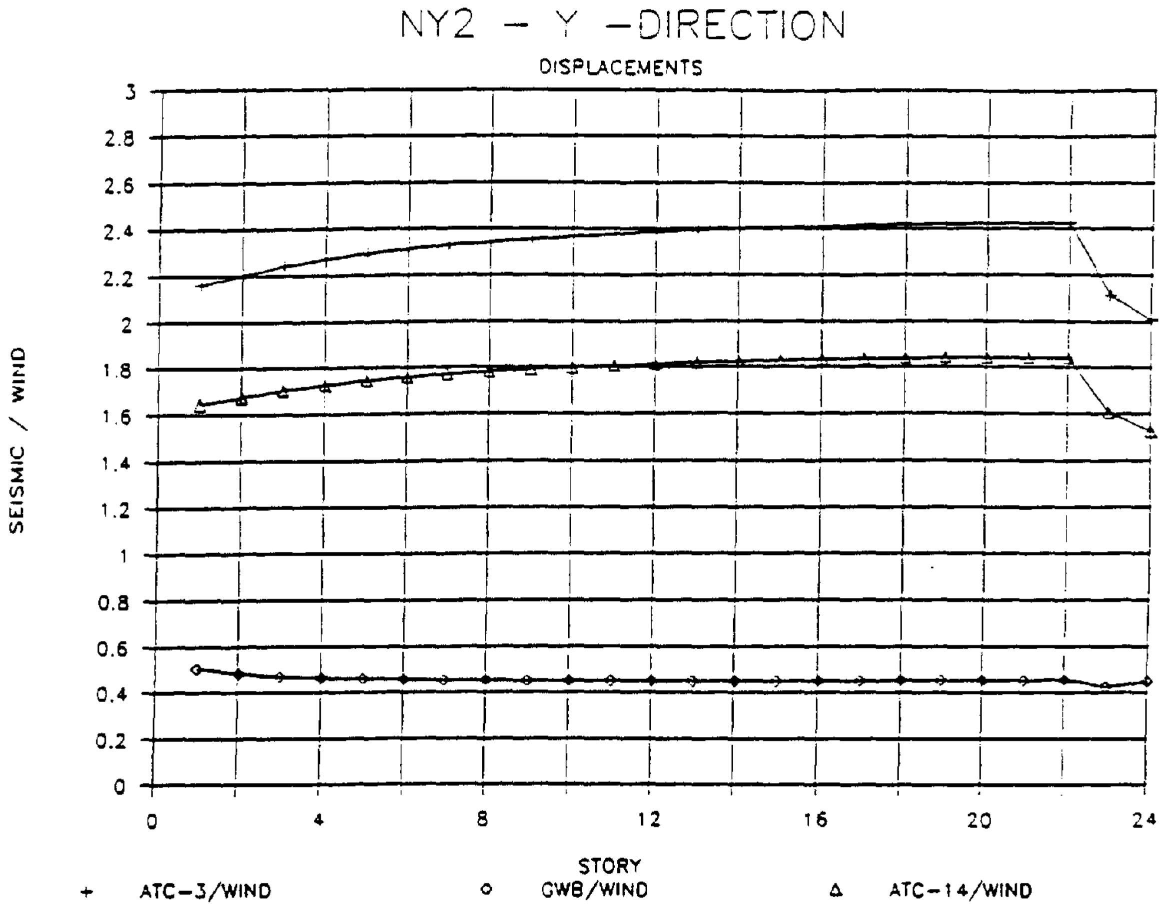


Fig. 2-16 Normalized Displacements, NY2, Y-Direction

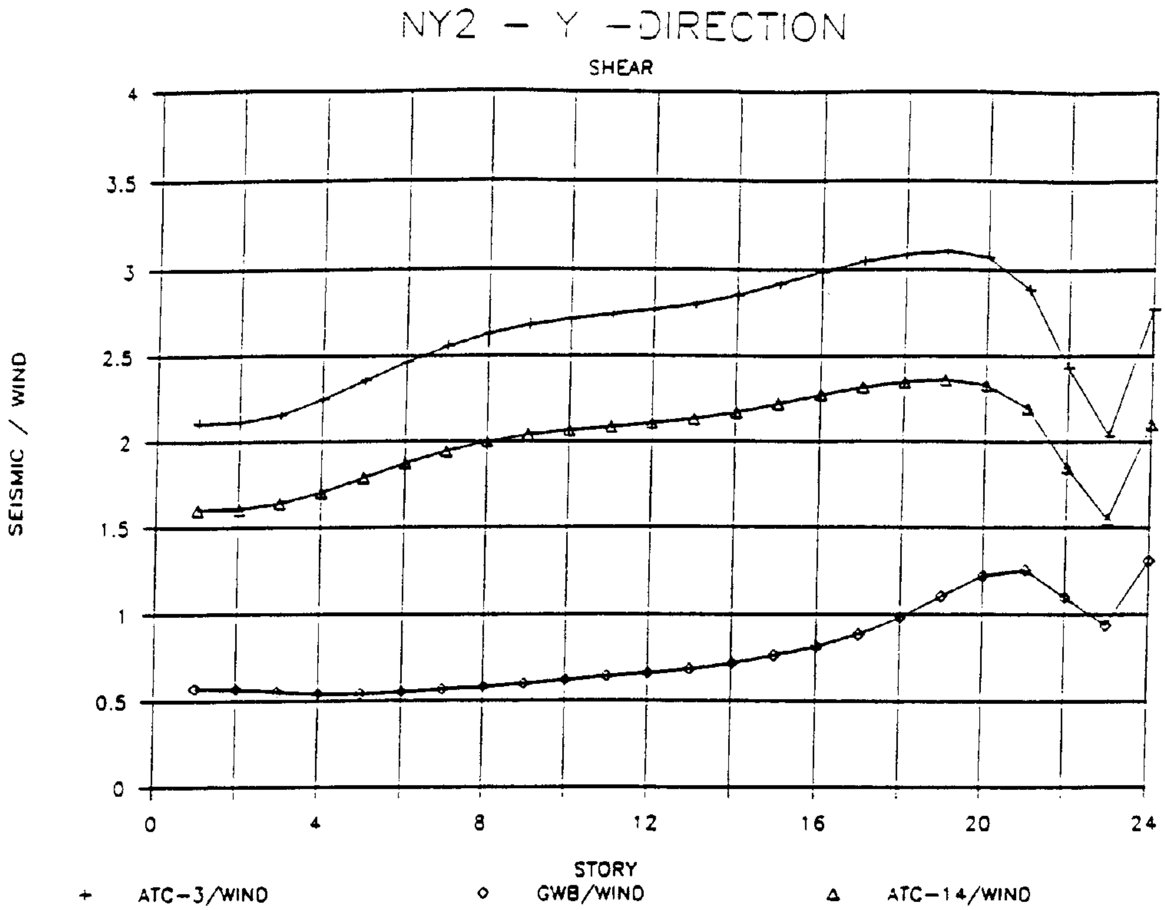


Fig. 2-17 Normalized Shear, NY2, Y-Direction

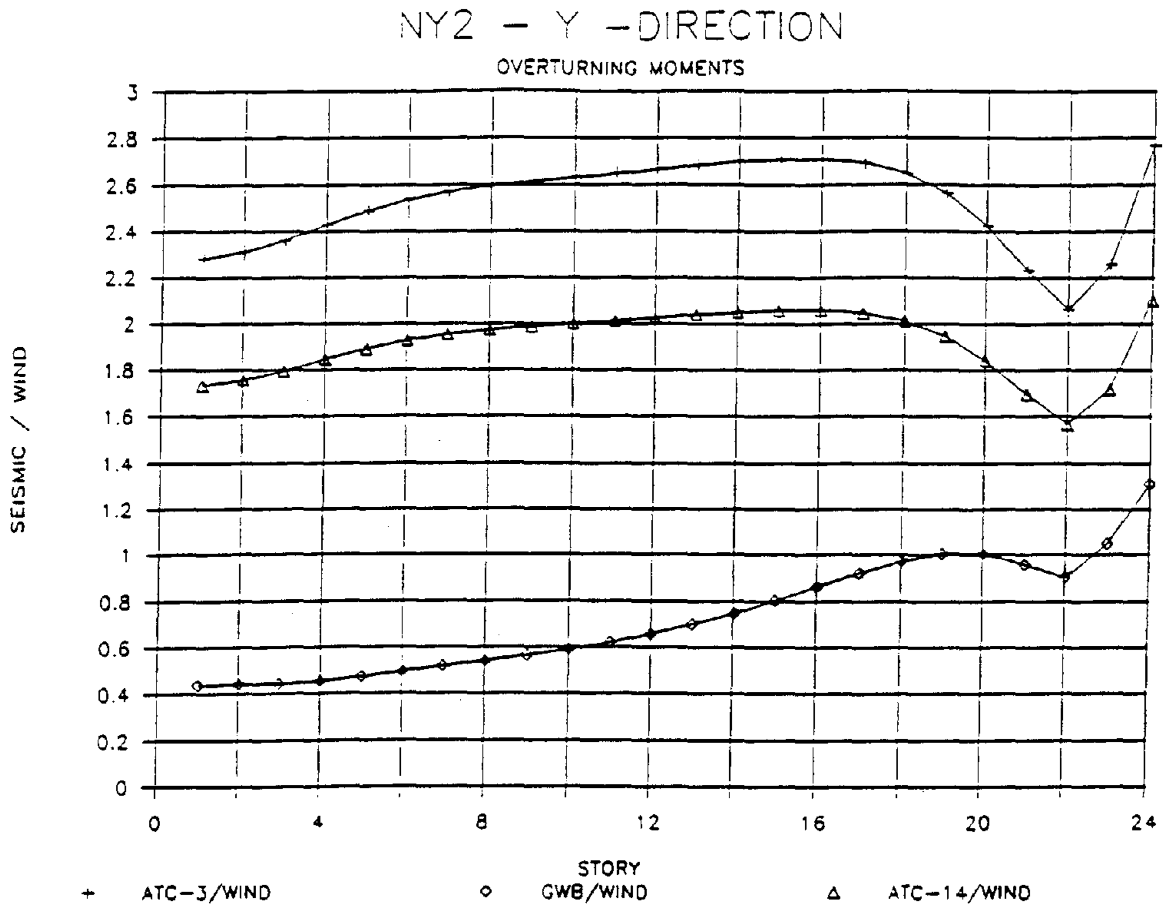


Fig. 2-18 Normalized OTM, NY2, Y-Direction

SECTION 3
AS-BUILT CAPACITY OF BUILDINGS

The nominal ultimate capacities required to resist seismic induced forces of the as-built two buildings (NY1 and NY2) were calculated. Two types of internal forces were considered, namely floor shears and overturning moments. The nominal ultimate capacities were computed for both buildings at all floor levels, using the American Concrete Institute design code (ACI 318-83).

The process of calculating the nominal ultimate capacity of different structural elements using the ACI code is dependent on the loading condition under consideration. The nominal shear capacity of any beam/column (or wall) is dependent on the internal design shear, axial force and bending moments of that particular element. The nominal bending moment of any structural element is a function of the level of internal design axial force of that element. The various lateral load combinations were calculated, as shown in table 3-I. The same table also shows the different load factors which were used in the design. These are the recommended load factors by [1].

Note that in table 3-I, DL is the dead load, LL is the live load, W is the wind load and E is the seismic load. The calculations were performed in the two main directions (X and Y) of both buildings. First, the nominal shear $(V_n)_{ijk}$, nominal axial force $(N_n)_{ijk}$ and nominal bending moment $(M_n)_{ijk}$ of each vertical element (column or shear wall; i^{th} element, j^{th} floor, k^{th} loading combination) were calculated. Then the resultant nominal floor shear capacity, $(V_n)_{jk}$, and overturning moment capacity, $(M_n)_{jk}$, were calculated as follows.

$$(V_n)_{jk} = \sum_{i=1}^{M}(V_n)_{ijk} \tag{3-1}$$

$$(M_n)_{jk} = \sum_{i=1}^{M}((M_n)_{ijk} + (N_n)_{ijk}R_i) \tag{3-2}$$

$$M = \text{Total number of vertical structural elements in the } j^{th}\text{floor}$$

where R_i is the normal distance from the i^{th} element of interest to the axis about which the overturning moment is being considered.

The required ultimate equivalent design shear, $(V_u)_{jk}$ and overturning moment,

TABLE 3-I Loading Combinations
for Lateral Loads

Counter (k)	Type of Lateral Force	Load Factor for Dead Load (DL)	Load Factor for Live Load (LL)	Load Factor for Wind Load (W)	Load Factor for Seismic (E)
1 - 2	Wind	0.75x1.4	0.75x1.7	+/- 0.75x1.7	0.00
3 - 4	Wind	0.90	0.00	+/- 1.3	0.00
5 - 6	Seismic	0.75x1.4	0.75x1.7	0.00	+/- 0.75x1.7x1.1
7 - 8	Seismic	0.90	0.00	0.00	+/- 1.3x1.1

$(M_u)_{jk}$ were calculated as follows.

$$(V_u)_{jk} = \sum_{i=1}^{M}(V_u)_{ijk} \tag{3-3}$$

$$(M_u)_{jk} = \sum_{i=1}^{M}((M_u)_{ijk} + (N_u)_{ijk}R_i) \tag{3-4}$$

where $(V_u)_{ijk}$,$(M_u)_{ijk}$ and $(N_u)_{ijk}$ are the ultimate design shear, bending moment and axial force, respectively.

Finally, the capacity ratios for shear and overturning moment were calculated as:

$$\text{capacity ratio} = \frac{(V_u)_{jk}}{(V_n)_{jk}} \qquad \text{for shear} \tag{3-5}$$

$$= \frac{(M_u)_{jk}}{(M_n)_{jk}} \qquad \text{for overturning moment} \tag{3-6}$$

A capacity ratio greater than unity corresponds to an over stressed structure, while a capacity ratio less than unity corresponds to an over design with reserve capacity for the specified loading combination and floor.

3-1 Results

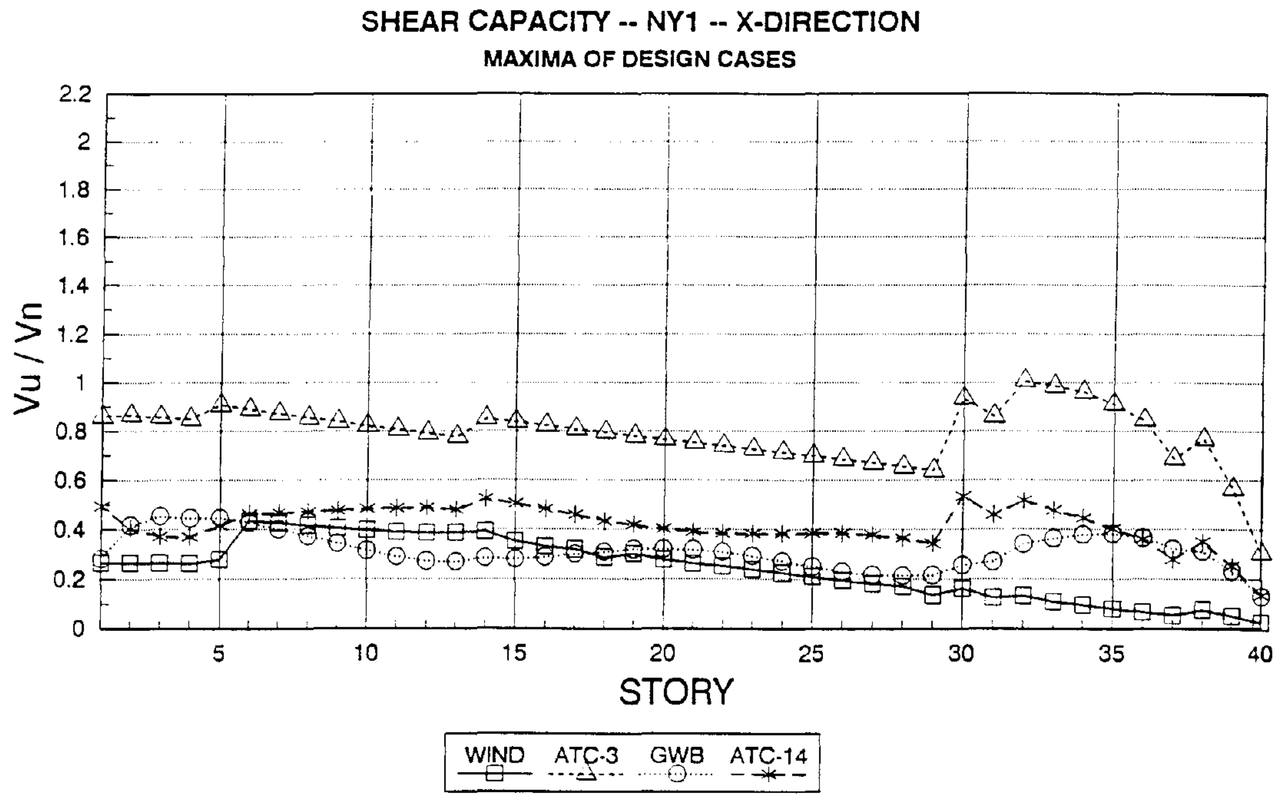
The two types of capacity ratios (shear and overturning moment) of the two buildings were evaluated for the loading combinations of table 3-I. The three representative input seismic response spectra were used, the ATC-3 spectrum [3], the ATC-14 spectrum [4] and the site dependent spectrum (GWB, [15]). For each lateral loading condition, the maxima of the capacity ratios of different loading combinations were chosen.

Figure 3-1a shows the shear capacity ratio for NY1 in the X-direction, while figure 3-2b shows the shear capacity ratio for NY1 in the Y-direction. Figure 3-2 shows the shear capacity ratios of NY2 in the X- and Y-directions, respectively. Figures 3-3 through 3-4 show the overturning moment capacity ratios for the two buildings in the X- and Y-directions.

The shear capacity ratios for wind loading show that there is a considerable reserve strength in the design of both buildings in both directions. This is due to the fact that the lateral stiffness (drift limitation) rather than the strength (capacity) was the governing design parameter for wind conditions. This strength reserve was helpful when the buildings were exposed to different seismic spectra. The site dependent spectrum (GWB) required even less strength than wind. While the strength of the two buildings were adequate for ATC-14, except near the base of NY1, it seems that the strength requirements of ATC-3 exceeded almost the whole height of NY1 and some floors of NY2. Similar observations can be made for the overturning moments results.

These results show that the capacity ratios are sensitive to, among other factors, the input spectra. While the national ACT-3 spectra showed unconservative building designs, the existing structures spectrum, ATC-14, showed just adequate designs, and the site dependent spectrum showed safe seismic responses. This shows the need for a more accurate definition of the design spectra, or any other form of seismic requirements for New York City area in particular, and the eastern United States in general.

The results of figures 3-1 through 3-4 also raise another set of questions. How much ductility requirements are needed when the design requirements exceed the strength capacity of these buildings?. How sensitive are these ductilities to the properties of these structures? In order to answer these questions, a nonlinear time history analysis of the buildings is needed. This is performed in the next section.



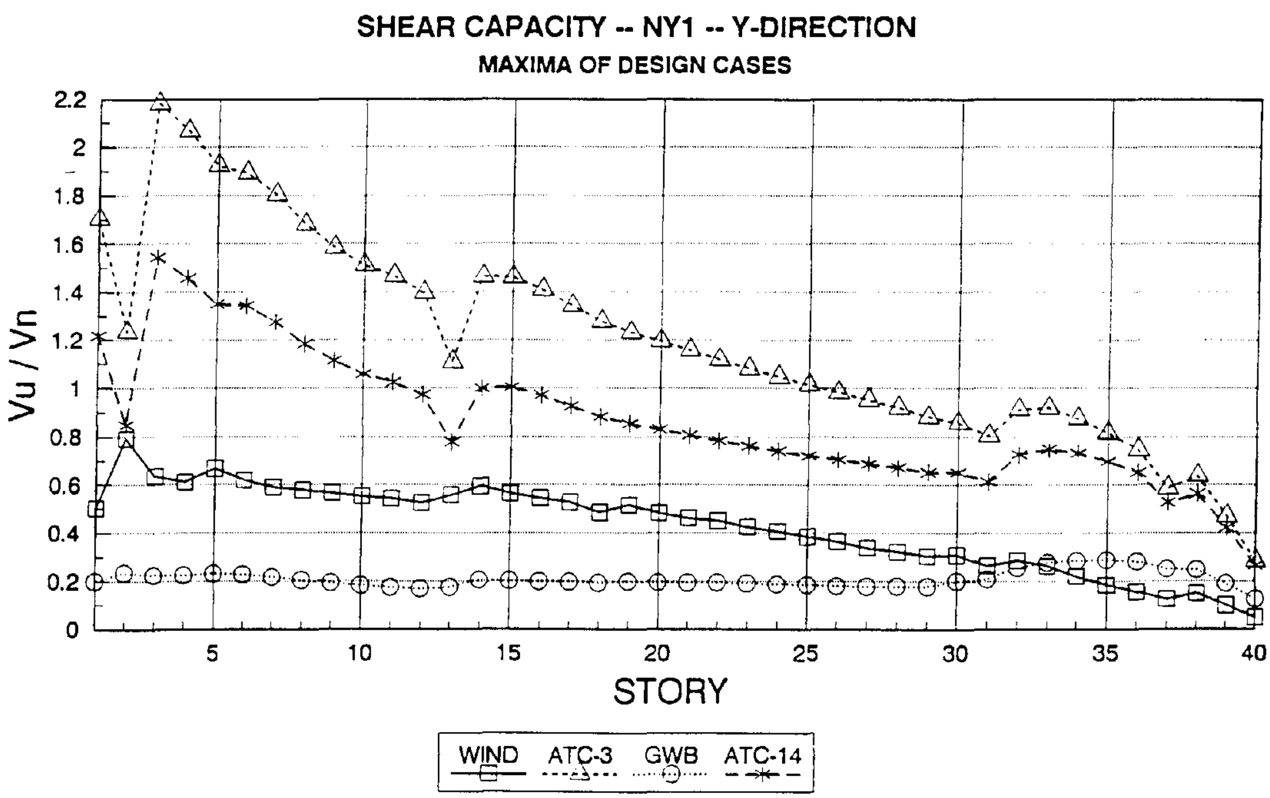


Fig. 3-1 Shear Capacity Ratio, NY1

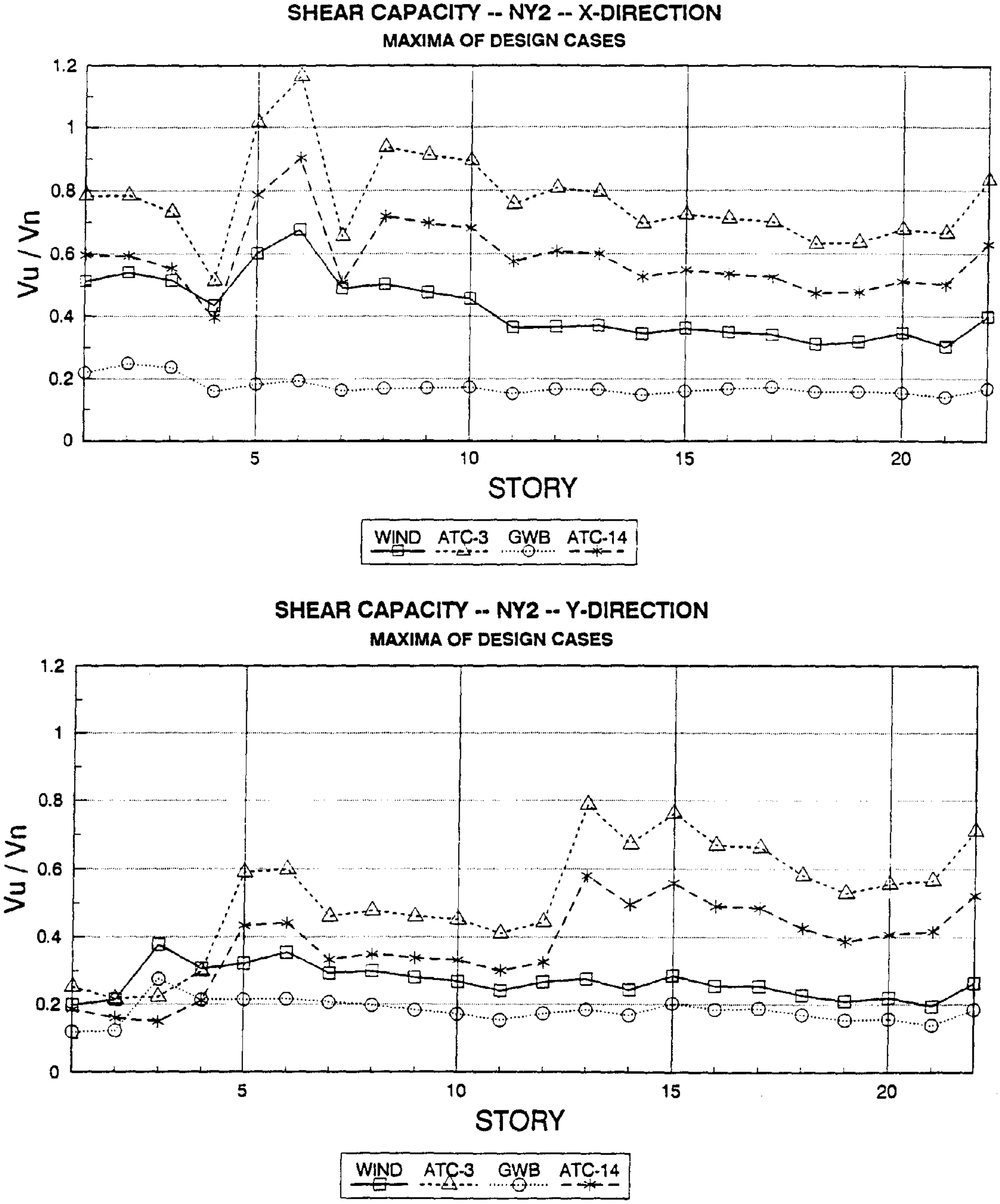
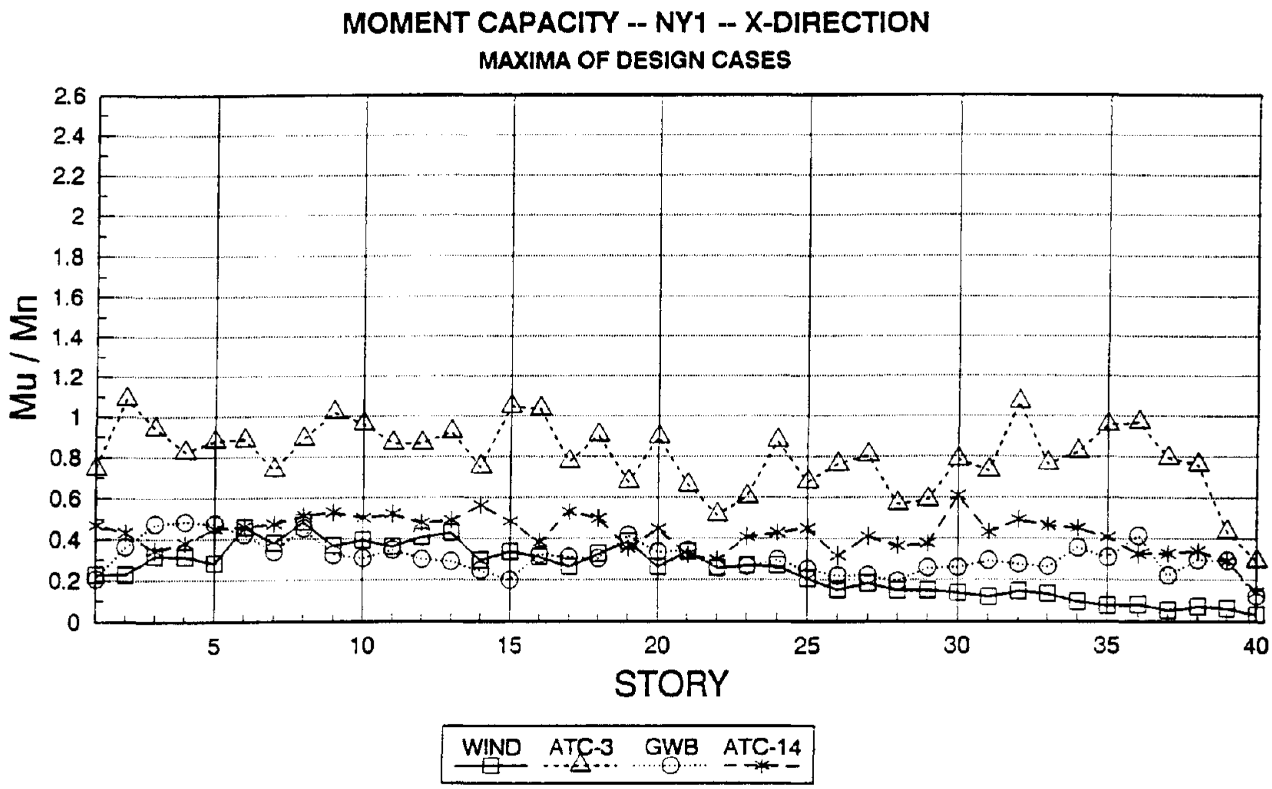


Fig. 3-2 Shear Capacity Ratio, NY2



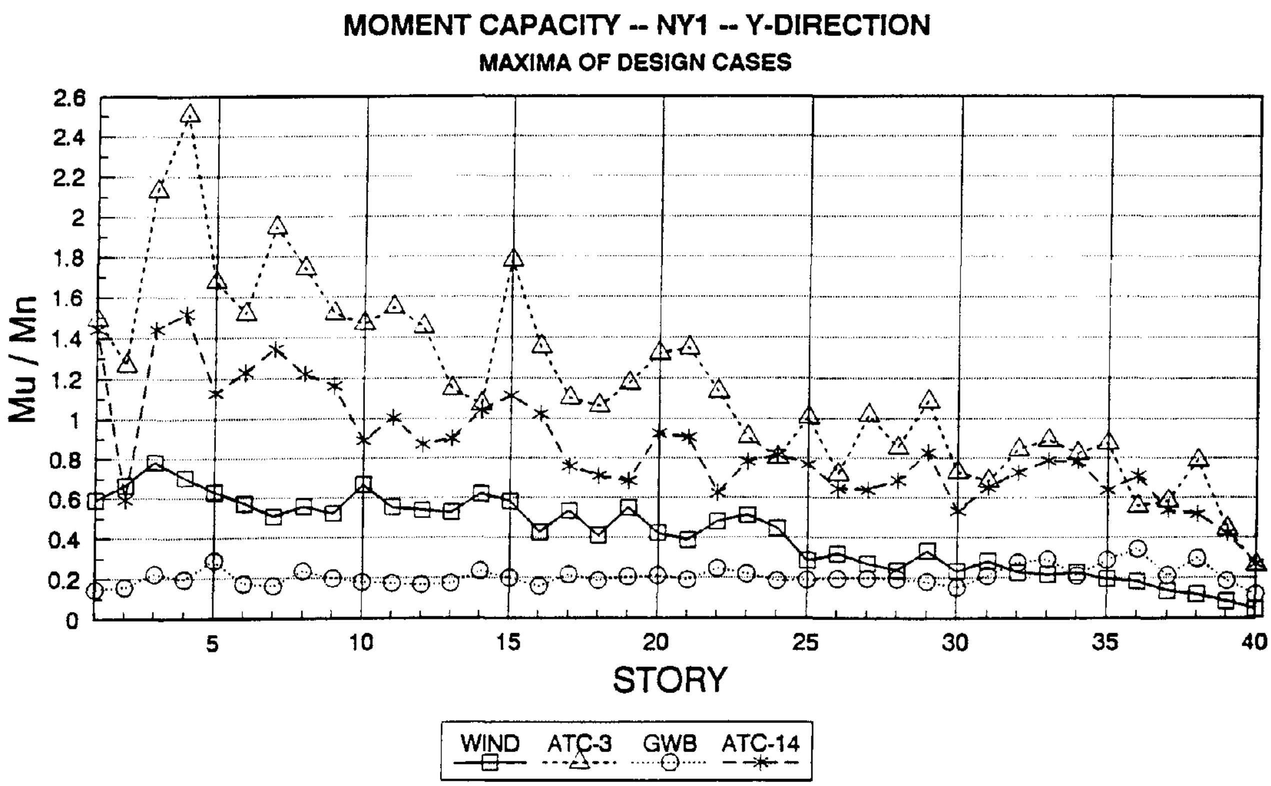
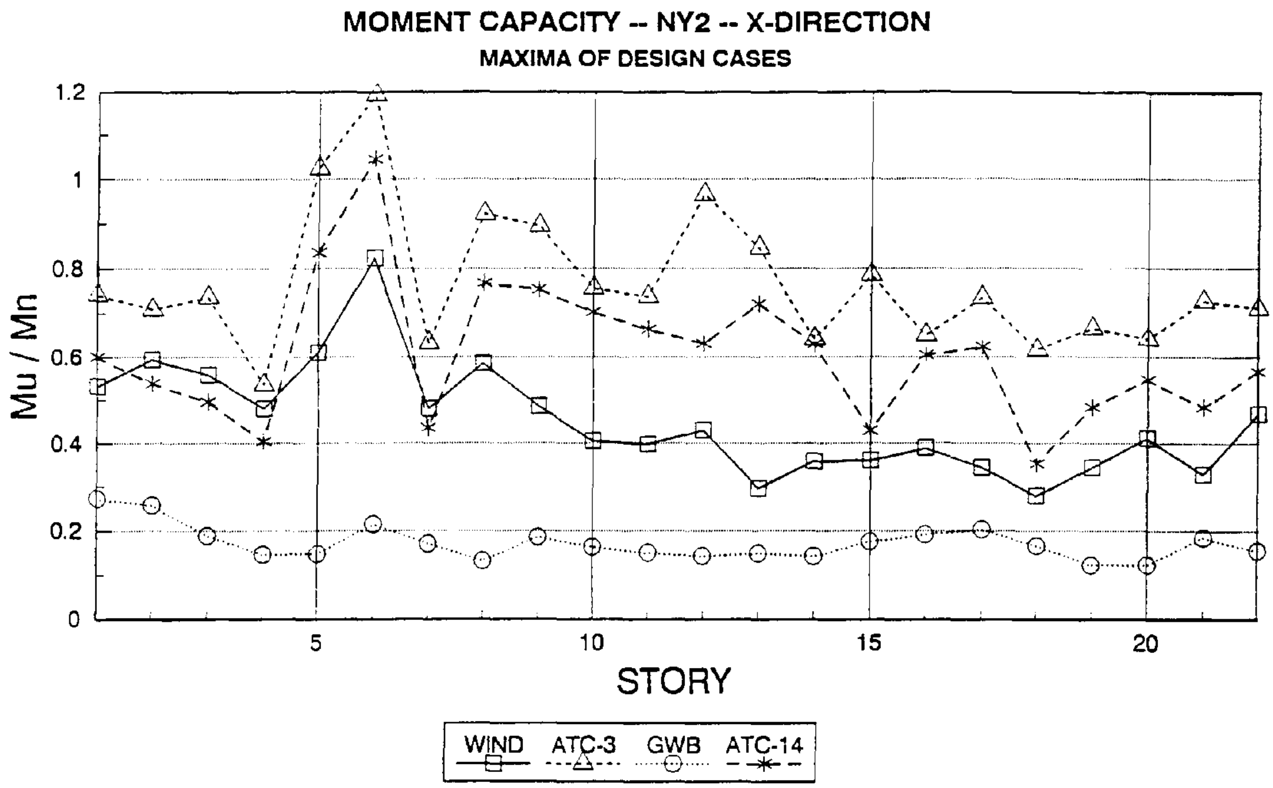


Fig. 3-3 OTM Capacity Ratio, NY1



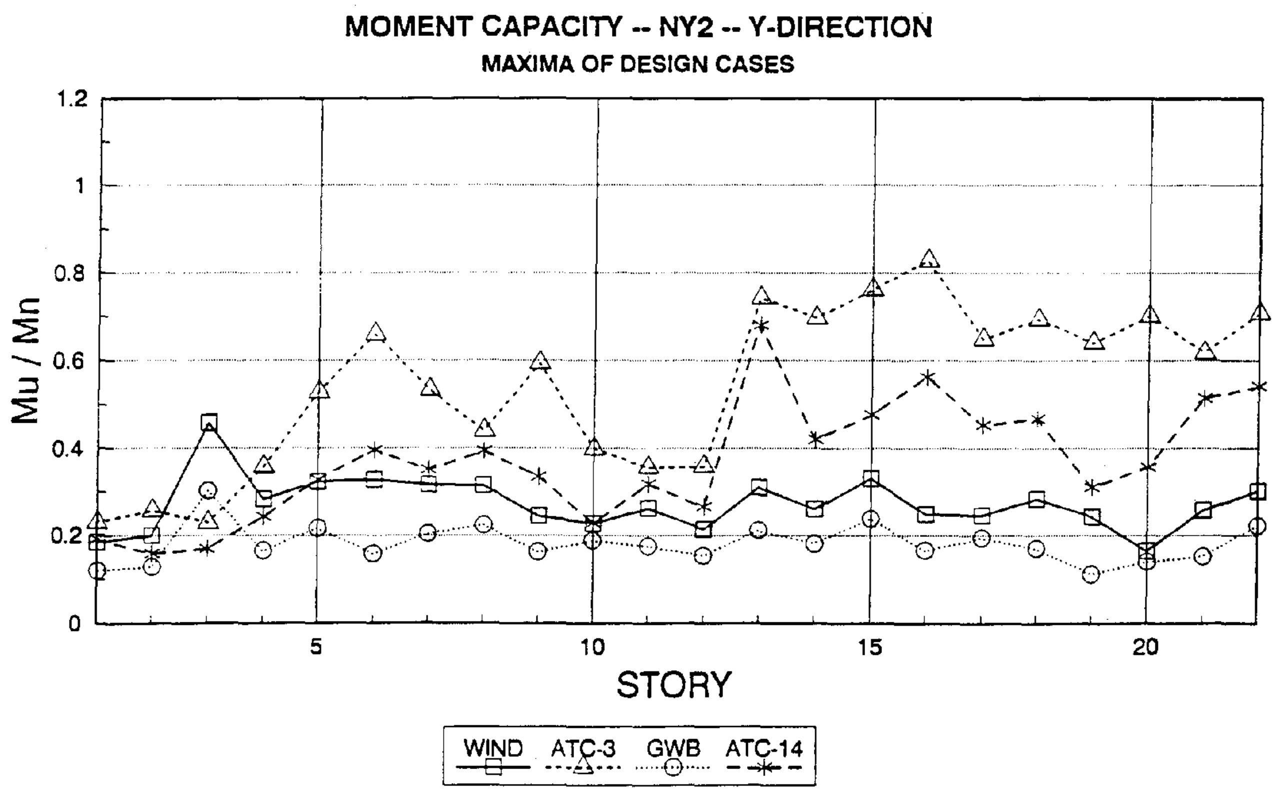


Fig. 3-4 OTM Capacity Ratio, NY2

SECTION 4
NONLINEAR SEISMIC BEHAVIOR

The elastic behavior of the two buildings were studied in the previous sections. The nonlinear behavior of the the buildings are studied in this section. A brief description of the mathematical model used in this analysis and the input time history motions is described first. The results of the analysis are then discussed.

4-1 Nonlinear Mathematical Model

The results in the previous sections showed that there is no coupling between linear and torsional motions during seismic motions for both buildings. This means that each of the buildings behaves essentially as a two-dimensional system. It was decided, for the purpose of the nonlinear analysis of this section, to use a two dimensional mathematical model for simplicity and speed without any appreciable loss in accuracy.

4-1-1 Dynamic Equations of Motion

The basic nonlinear mathematical model used in this study is shown in figure 4-1. It is a lumped mass model with nonlinear shear springs connecting different masses. Each mass of the system has one degree of freedom (horizontal) and represents one floor in the building under consideration. The inter-story shearing forces are represented by nonlinear springs connecting the different floors. In order to model the loss of energy due to nonstructural elements in the buildings, viscous dashpots connecting different floors are added to the model. All the springs and the dashpots in the model are close-coupled. The input seismic motion is prescribed at the base.

The system equilibrium equation can then be written as:

$$\underline{M}\underline{\ddot{U}} + \underline{C}\underline{\dot{U}} + \underline{F} = \underline{M}\underline{I}\ddot{u}_g \tag{4-1}$$

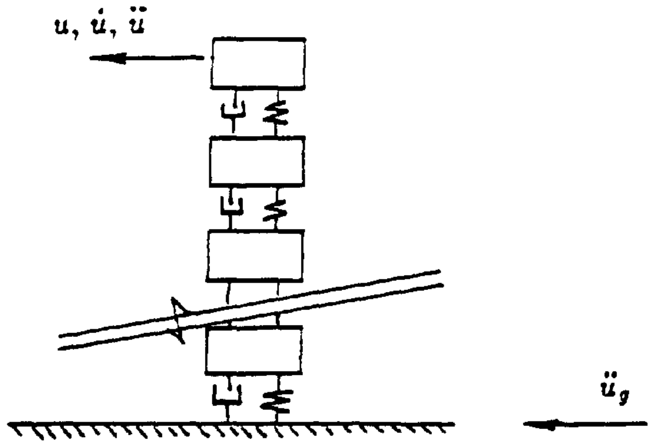


Fig. 4-1 2-D Mathematical Model

where

$$\underline{M} = \text{Diagonal mass matrix}$$
$$\underline{C} = \text{Viscous damping matrix}$$
$$\ddot{\underline{U}} = \text{Floor accelerations}$$
$$\dot{\underline{U}} = \text{Floor velocities}$$
$$\underline{F} = \text{Inter story forces}$$
$$\underline{I} = \text{Identity vector}$$
$$\ddot{u}_g = \text{Input seismic accelerations}$$

Equation 4-1 was solved using a step by step explicit time integration method (central difference scheme), [2].

4-1-2 Inter-Story Constitutive Model

The components of the inter-story force vector $\underline{F}$ were evaluated at every time step using the relation:

$$f_i = k_i^t(u_{i+1} - u_i) \tag{4-2}$$

where

$$f_i = i^{th} \text{ inter story force}$$
$$k_i^t = \text{Tangent stiffness}$$
$$u_i = \text{Displacement of the } i^{th}\text{floor}$$

The tangent stiffness of each floor can be obtained using the simple nonlinear force-deformation model shown in figure 4-2. It is an elastic-perfectly plastic model which can be described using only two parameters, namely the elastic stiffness k_i^e and the yield strength f_i^y. No strain hardening, stiffness or strength degradation were assumed for this model, in order to keep the model as simple as possible.

The inter story yield strength (shear capacity) was calculated for each floor in each horizontal direction for the two buildings using the actual as-built dimensions and reinforcements of the shear walls and columns. The applicable procedures of the ACI-83 [1] code were used for this task. Two inter story strength were calculated. They are:

a. Shear strength of the whole floor, including columns which are connected directly to the flat slabs.

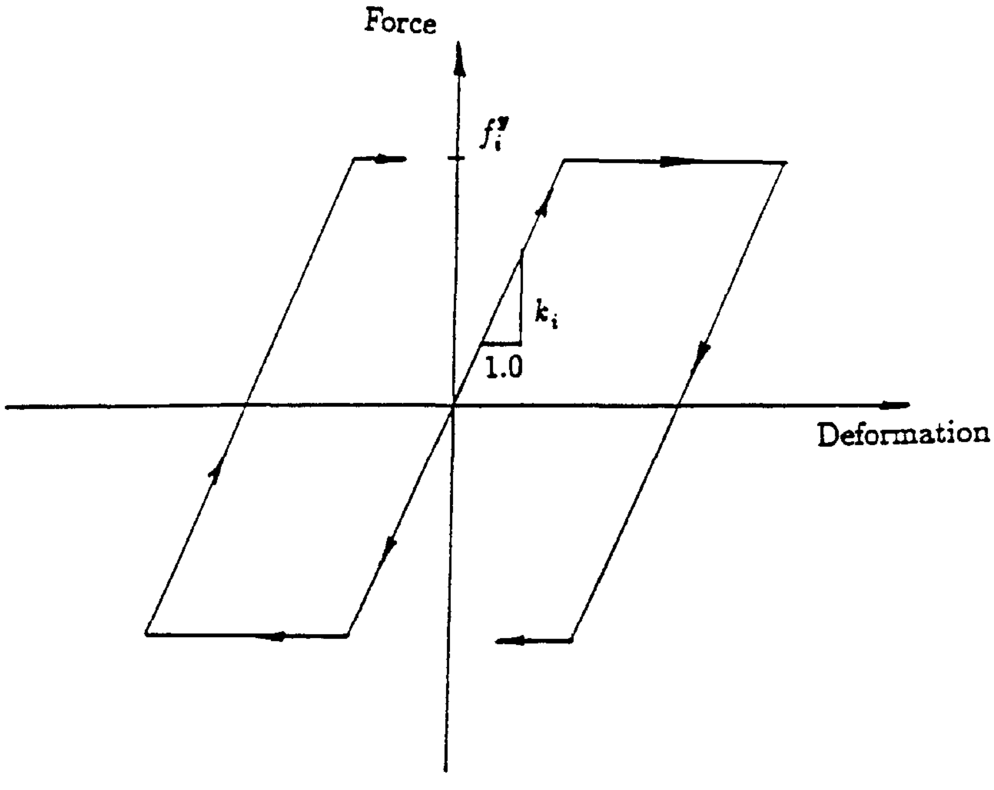


Fig. 4-2 Nonlinear Force-Deformation Model

b. Shear strength of the main shear elements only, not accounting for columns which are connected directly to the flat slabs.

The ratio of the shear strengths of case 'b' to case 'a' was always found to be in the range of 90% to 95%. Only case 'b' was studied.

The inter-story shear stiffnesses were calculated for all cases using elementary beam theory [9]. No rotations of top or bottom joints of the columns or the shear walls were allowed. Also, steel reinforcements and the cracking of the cross sections were not considered.

4-1-3 Viscous Damping Effects

Most of the loss of energy in the buildings under consideration during seismic events is due to the plastic dissipation of the structural elements. This effect has been accounted for in the model by the interstory nonlinear force-deformation relation. However, there is another source of energy loss for this class of structures during seismic events, namely the nonstructural elements such as partitions, windows, etc. In order to account for this energy loss, a viscous damping term, $\underline{C}$ was added to equation 4-1. The damping matrix was calculated for each case using the Rayleigh damping relation [8]:

$$\underline{C} = 2\beta\omega\underline{M} \tag{4-3}$$

where

$$\beta = \text{Damping ratio}$$
$$\omega = \text{First natural frequency}$$

The damping ratio β was chosen for all cases to be 5.0%. This is an arbitrary choice which was kept to a minimum to be on the conservative side.

4-1-4 Discussion of the Mathematical Model

The nonlinear mathematical model used has many inherent approximations. These approximations result in some inaccuracy in the results. However, since the purpose of this report is to study the general behavior of this class of buildings, rather than the detailed behavior of different components of the structures, it was felt that the proposed model is adequate enough. For more detailed study of this class of structures, a more detailed nonlinear model such as the model described in [7] may be required.

4-1-5 Ductility Requirements

Evaluation of damage due to seismic motions in reinforced concrete structures have been studied by several authors. Chung, et al, 1987 [7], presented a fine state-of-the-art review, as well as an advanced damage prediction model. Using such an advanced damage model is beyond the scope of this report. It was decided, however, to use the old popular measure, namely the ductility measure [2], for this study. Here, the inter-story ductility requirement is evaluated as the ratio between the maximum inter-story deformation during the seismic event to the yield deformation of that story, on the other hand, the displacement ductility is calculated as the ratio between the maximum displacements of the non-linear structure during the seismic event to the maximum displacements of the same structure during the same event assuming that its inter-story shear strengths are infinite (linear structure case).

Three cases are studied for each structure. They are:

i. Structure # I , as-built strength. Note that the areas of steel reinforcements in the shear resisting members, especially the shear walls, are usually the minimum required by the ACI-83. This is due to the fact that the original governing design requirement for wind is stiffness not strength (see previous section).

ii. Structure # II, same as structure # I except that the areas of steel reinforcements of all shear resisting members are twice those of structure # I.

iii. Structure # III, same as structure # I except that the areas of steel reinforcements of the shear resisting members are now four times those of structure # I.

Note that structures # II and # III are hypothetical buildings with larger steel reinforcement ratios. They were studied to investigate the effects of increasing the ductile properties (reinforcement ratios) and the strength of the shear resisting members on the nonlinear behavior of the buildings.

4-2 Input Seismic Motion

One of the most difficult decisions for non-linear seismic analysis of buildings is the choice of input time history. What makes this decision even more difficult for this study is the fact that most of available time histories are not applicable to the New York City site conditions. Although there are studies underway to produce such time histories, none were available in time for this report.

The time history which was chosen for this study is shown in figure 4-3 [10]. The whole time history was scaled so as to have a maximum accelaration of 0.24 g, which is consistent with the New York City site.

The choice of this particular time history is not, of course, a perfect choice, since it does not represent the probable wave form that may occure in New York City. However, in the absence of site specific wave form, this choice of is as adequate as any other wave form.

4-3 Results

Figure 4-4 shows the inter-story ductility for NY1 in the X- and Y- directions, respectively. Three curves are shown in each figure, representing structures # I, # II and # III. Figure 4-5 shows similar results for NY2. The results show, on the average, an interstory ductility requirement of about 4 to 5 for both as-built buildings (structures # I). In general, the inter-story ductility requirements decrease as the inter-story shear strength increases, as expected (structures # II and # III).

Figures 4-6 shows the displacement ductility for NY1 in the X- and Y- directions, respectively. Three curves are also shown in each figure, representing structures # I, # II and # III. Figures 4-7 shows similar results for NY2. It is clear that in all the cases studied, displacement ductility can be considered unity, for all practical purposes. This means that the ductility factor method, as presented by Clough and Penzien, 1975, can be used for the practical seismic design of these two buildings, and for buildings which exhibit similar properties. This will help in reducing the cost of non-linear seismic analysis of this type of structures.

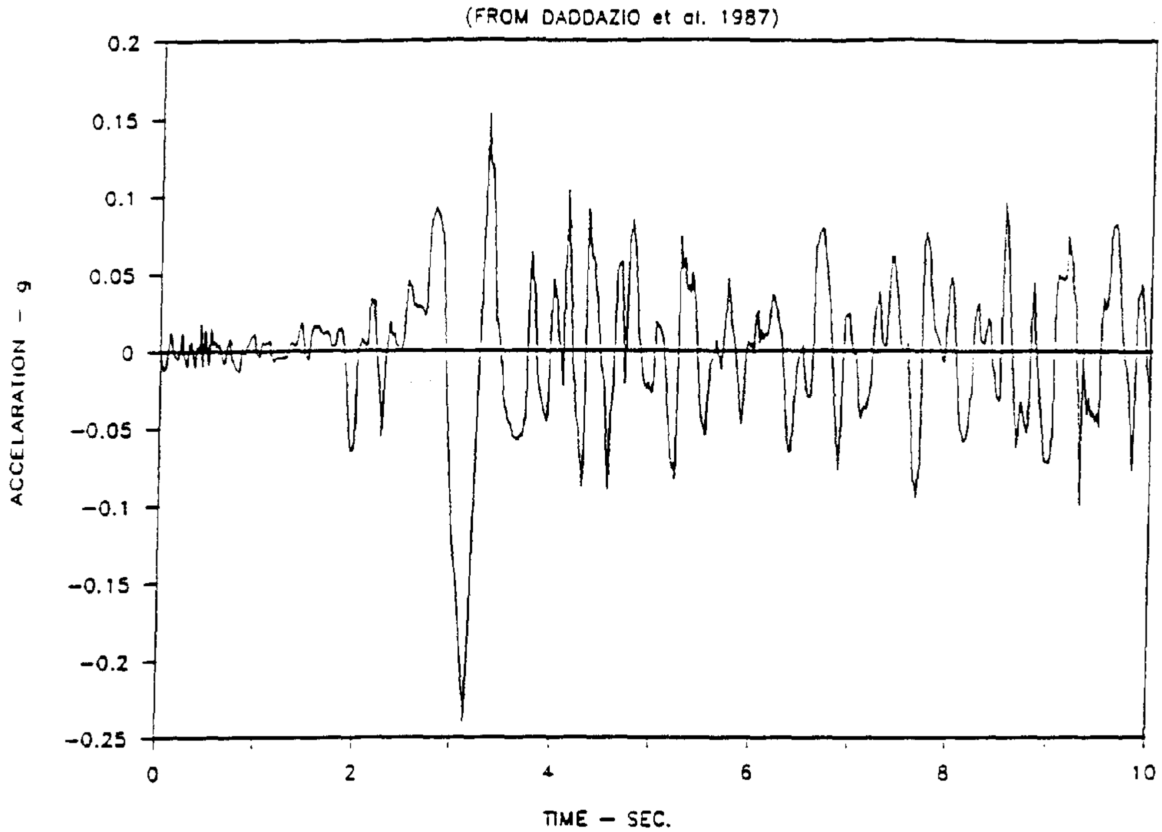
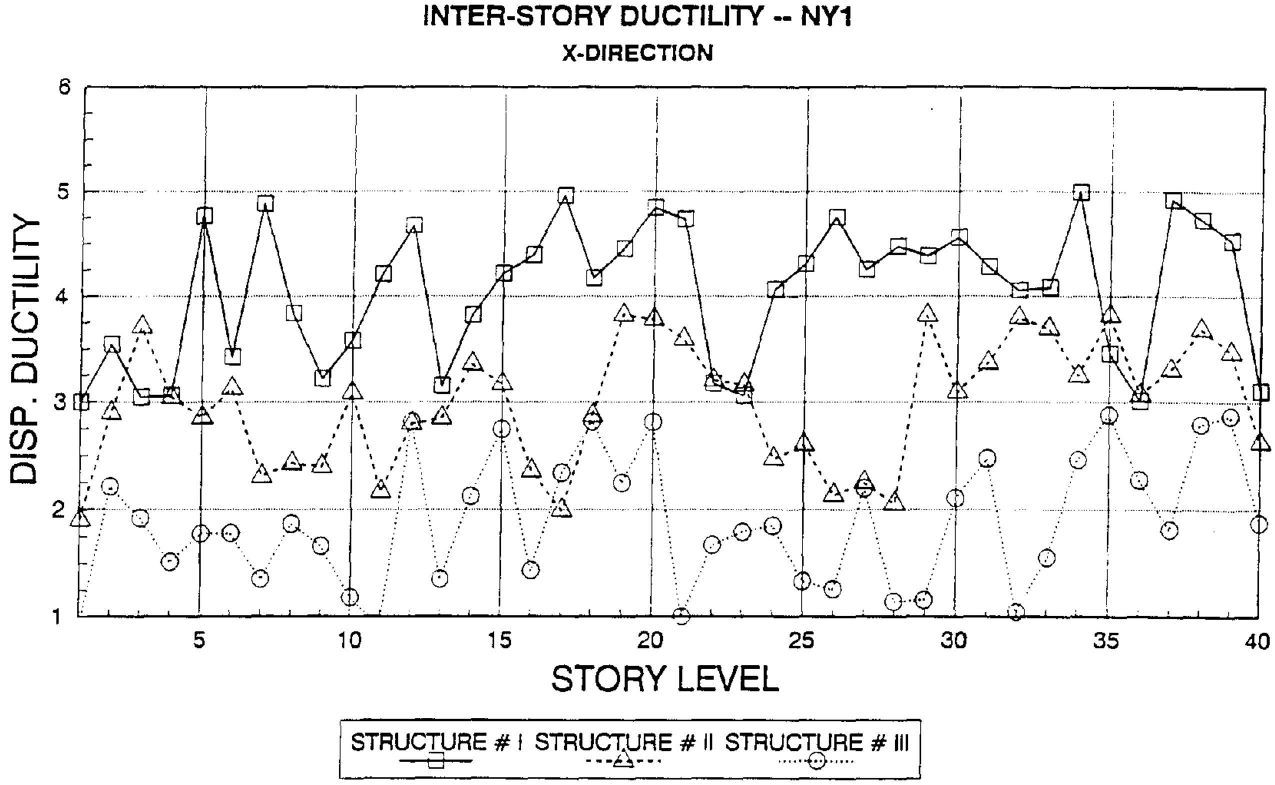


Fig. 4-3 Ground Acceleration Time History



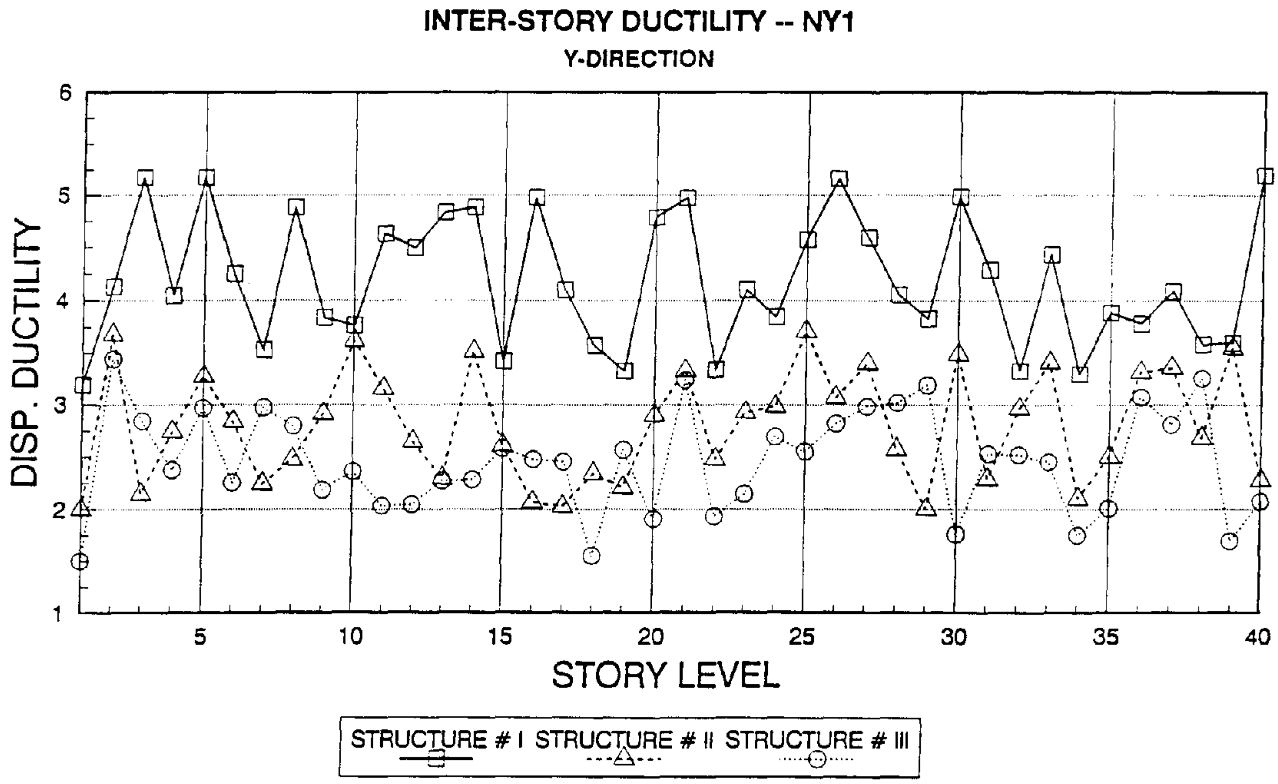
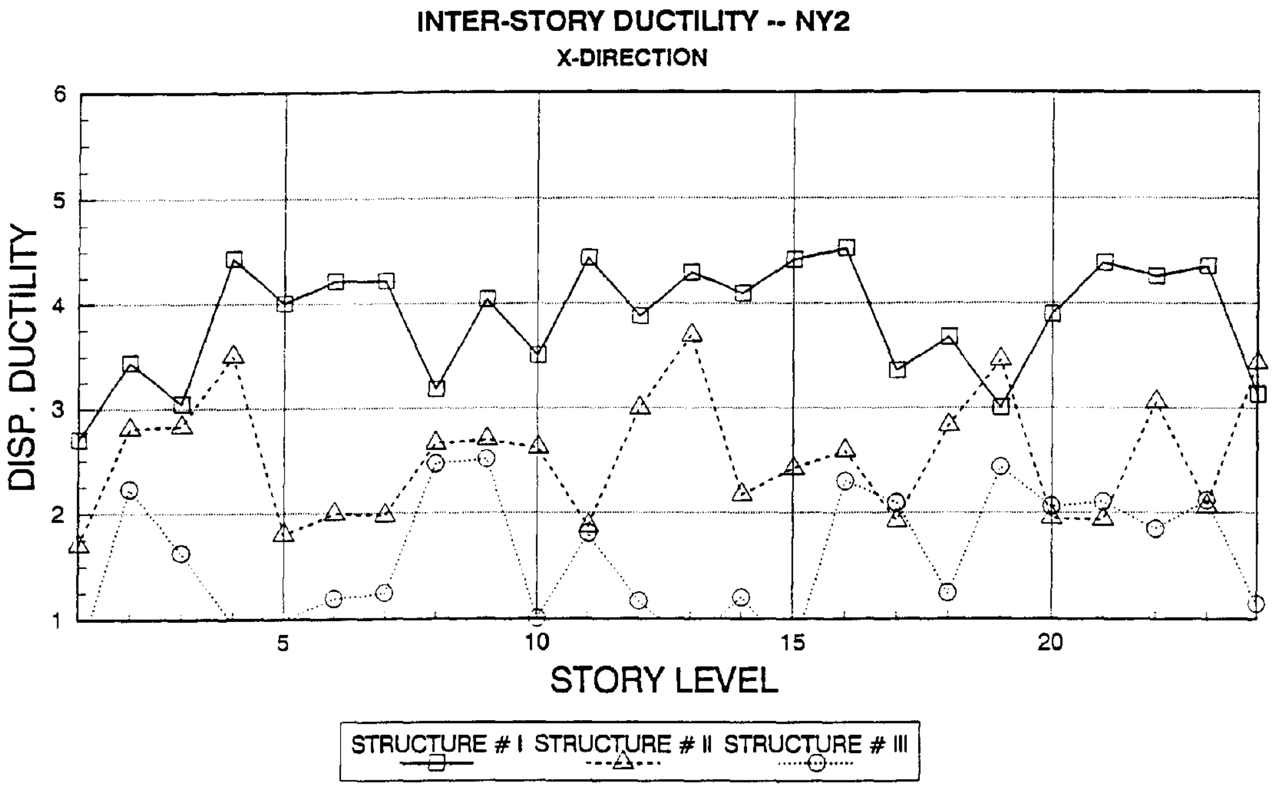


Fig. 4-4 Inter-Story Ductility, NY1



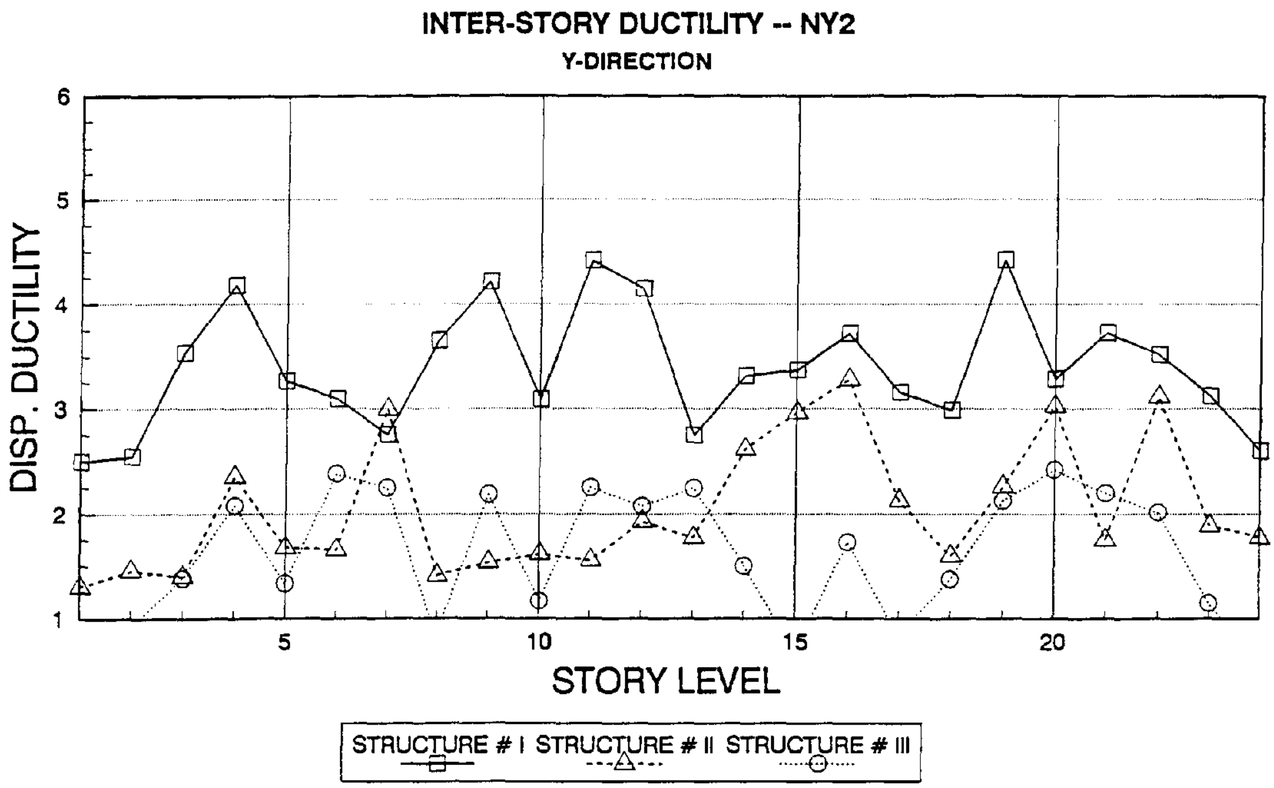
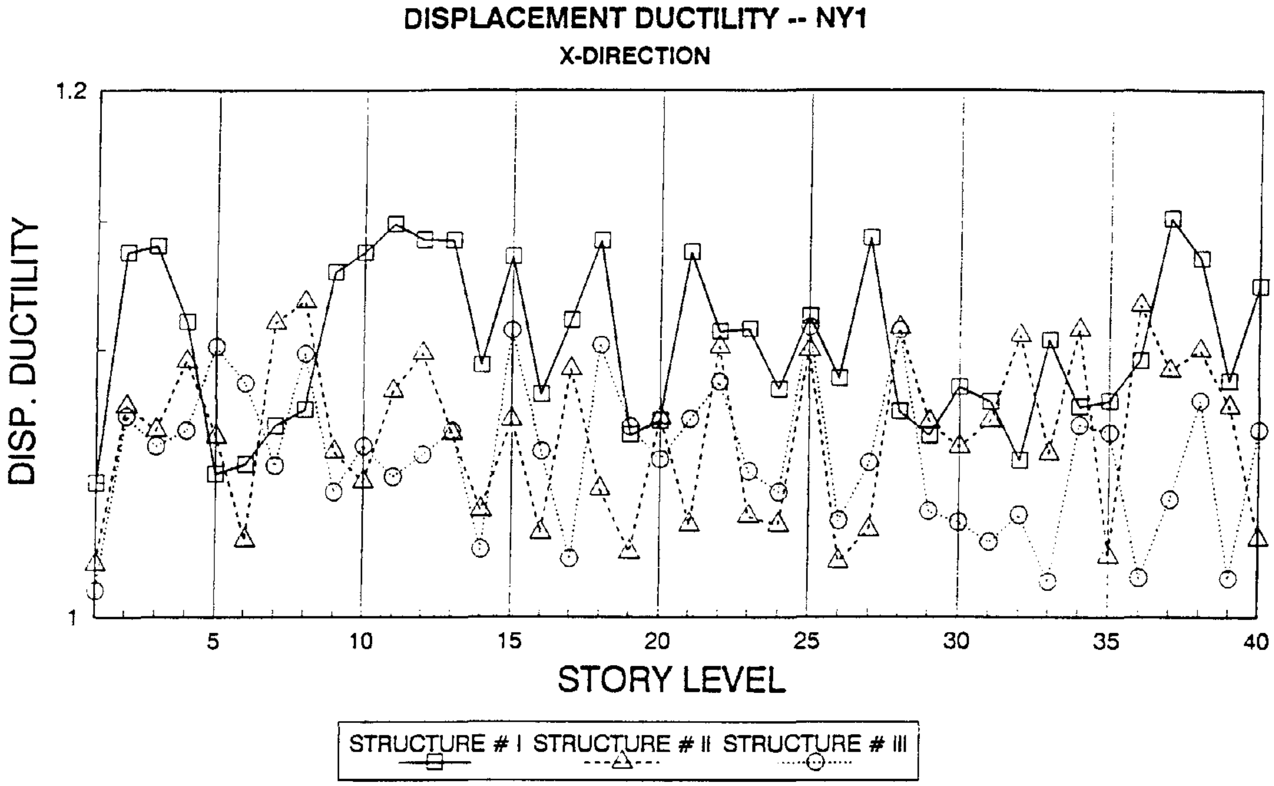


Fig. 4-5 Inter-Story Ductility, NY2



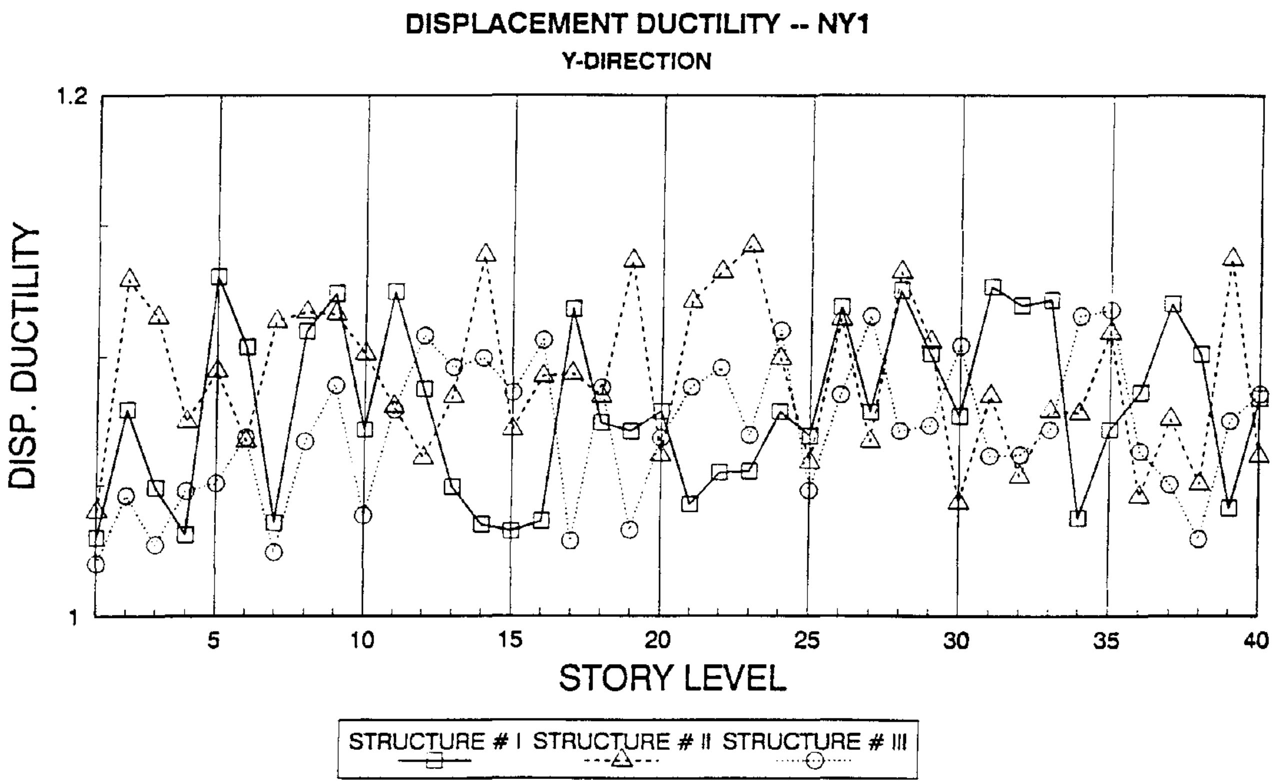
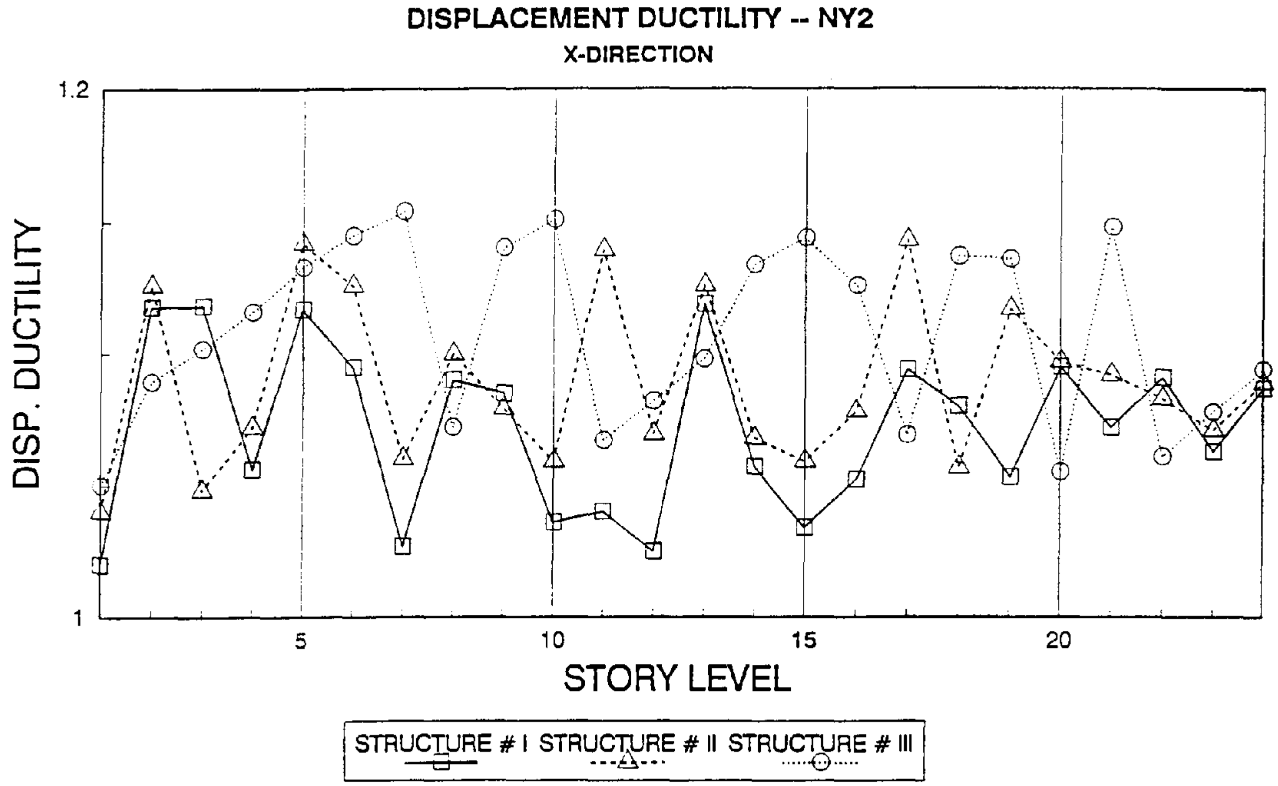


Fig. 4-6 Displacement Ductility, NY1



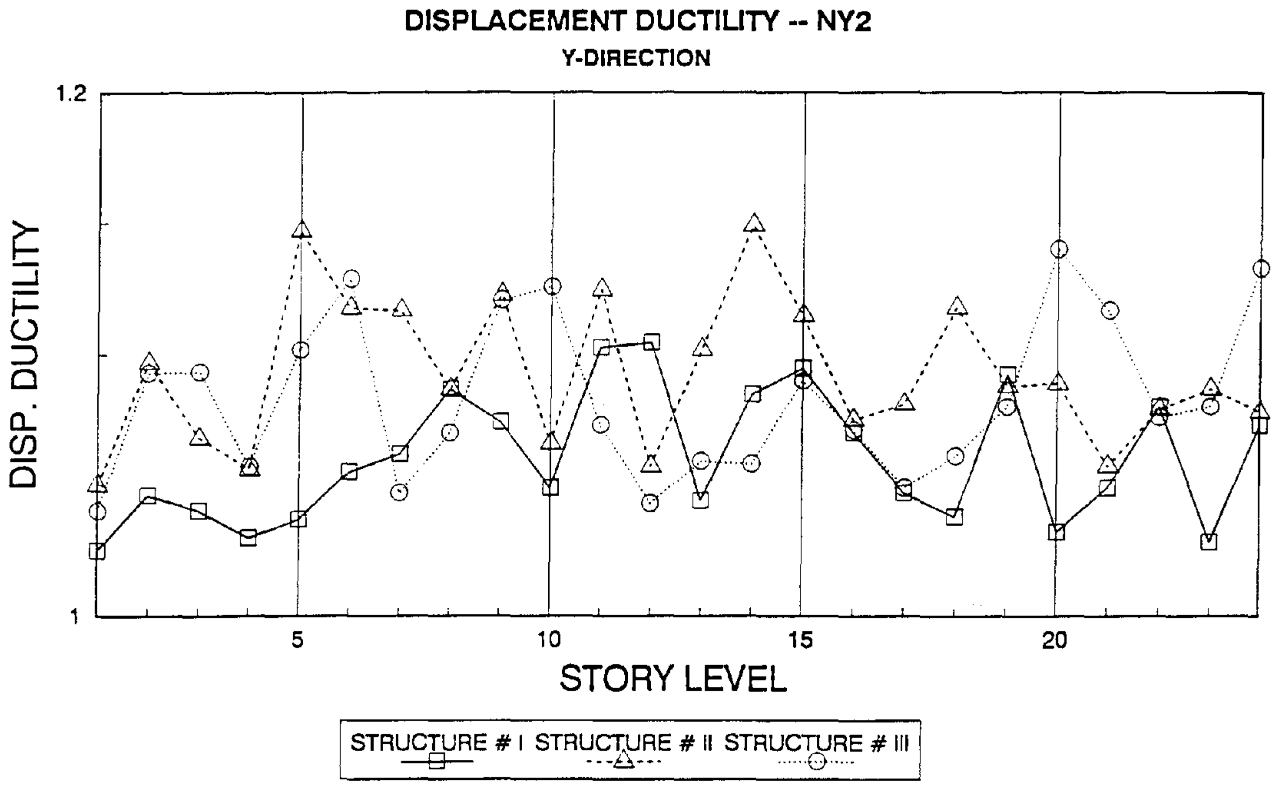


Fig. 4-7 Displacement Ductility, NY2

SECTION 5
CONCLUSIONS AND RECOMMENDATIONS

5-1 Conclusions

Based on the results given in the previous sections, conclusions can be summarized as follows:

i. Direct wind vs. seismic responses for the class of buildings under consideration show that the seismic responses are considerably higher than wind responses.

ii. As-built structural capacities to resist lateral wind loads are considerably greater than the code-required wind internal forces. This reserve margin can be of great help for seismic case.

iii. Seismic capacity ratios of buildings were found to be sensitive to the choice of input seismic spectra.

iv. Inter-story ductility requirements are in the range of 4.0 to 5.0. These ductility ratios tend to get smaller for larger reinforcement ratios in the shear resisting members of the buildings under consideration.

v. General displacement ductilities were found to be of order of unity. This indicates that the ductility factor method can be used for this class of structures.

5-2 Recommendations

i. The general behavior of the two buildings during seismic events seems to be encouraging. However, before making any general conclusions, more studies should be undertaken. Some of the subjects which require in-depth investigations are:

   a. The sensitivity of the seismic response of flat slab (plate) structures to different structural elements (number of floors,reinforcement ratios etc.).

   b. The linear and nonlinear seismic behavior of individual column and slab-column joints. Although these structural elements are not utilized in lateral seismic resistance, their integrity during seismic events must be assured.

   c. The applicability of the ductility factor method to this class of structures, through the use of additional seismic time histories should be applied to other types of buildings.

d. Development of a design input (time history or response spectra) which is more suitable for New York City site conditions.

The investigation of these items should provide a greater understanding of the seismic behavior of this important class of buildings. This understanding will then help in producing design and analysis guidelines for practicing engineers.

SECTION 6
REFERENCES

1. ACI, Building Code Requirements for Reinforced Concrete, Including Commentary, American Concrete Institute Report ACI 318-83, Detroit, Michigan, 1983.

2. Anagnostopoulos, S. A., "Nonlinear Dynamic Analysis and Ductility Requirements of Building Structures Subjected to Earthquakes," Research Report Number R72-54, Civil Engineering Dept., Massachusetts Institute of Technology, Cambridge, Mass., 1972.

3. ATC-3, Tentative Provisions for the Developement of Seismic Regulations for Buildings, Applied Technology Council Report ATC-3-06, Palo Alto, California, 1978.

4. ATC-14, Evaluating the Seismic Resistance of Existing Buildings Applied Technology Council Report ATC-3- 06, Palo Alto, California, 1987.

5. Atkinson, G. M., "implications of Eastern Ground Motion Characteristics for Seismic Hazard Assesment in Eastern North America," Proceedings, Conference on Earthquake Hazards and the Design of Constructed Facilities in the Eastern United States, sponsored by National Center of Earthquake Engineering Research (NCEER) and New York Academy of Sciences (NYAS), Feb. 1988.

6. BOCA Building Code, Building Officials and Code Administrators International, Inc., 1984.

7. Chung, Y. S., Meyer, C., and Shinozuka, M., "Seismic Damage Assesment of Reinforced Concrete Members," Technical Report NCEER-87-0022, National Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, NY, 1987.

8. Clough, R. W. and Penzien, J., "Dynamics of Structures," McGraw Hill, Inc., NY, 1975.

9. Connor, J. J., "Analysis of Structural Member Systems," The Ronald Press Company, NY, 1976.

10. Daddazio, R., et al, "Nonlinear Dynamic Slope Stability Analysis," Journal of Geotechnical Engineering, ASCE, April, 1987.

11. New York City Building Code.

12. Seeber, L., "Seismicity and Techtonics in the Eastern US," Proceedings, Conference on Earthquake Hazards and the Design of Constructed Facilities in the Eastern United States, sponsored by National Center of Earthquake Engineering Research (NCEER) and New York Academy of Sciences (NYAS), Feb. 1988.

13. Somerville, P., "Earthquake Source and Ground Motion Characteristics in Eastern North America,"Proceedings, Conference on Earthquake Hazards and the Design of Constructed Facilities in the Eastern United States, sponsored by National Center of Earthquake Engineering Research (NCEER) and New York Academy of Sciences (NYAS), Feb. 1988.

14. Uniform Building Code, International Conference of Building Officials, Whitter, California, 1985. (Proposed revision, 1988).

15. Weidlinger Assoc., "George Washington Bridge Seismic Study, Phase I: Response Spectrum and Seismic Hazard Evaluation", Report, Weidlinger Assoc., NYC, NY, 1988.

16. Wilson, E. L., "Three dimensional analysis of building systems (extended version)," Earthquake Engineering Research Center Report No. EERC 75-13, University of California at Berkeley, April 1975.

NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
LIST OF PUBLISHED 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).

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). This report is available only through NTIS (see address given above).

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).

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).

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).

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).

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, to be published.

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).

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).

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.