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Design of Connections for Precast Prestressed Concrete Buildings for the Effects of Earthquake

By D.P. Clough

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Buildings designed in conformance with typical building code criteria will yield during a significant earthquake. Conventional design procedures do not, however, deal explicitly with the resulting inelastic deformations. In some types of precast concrete construction, inelastic deformations concentrate in the connections between precast elements and careful detailing is required to guard against brittle failure. In the present work, a design methodology has been developed which takes into account quantitative estimates of plastic deformations and enables a more rational approach to connection detailing. A simple concept, involving internal strain energy associated with the structure's seismic response, is used to estimate the maximum inelastic displacement at roof level. Kinematic principles are used to transform this "global" displacement into plastic deformation estimates for individual joints and connectors. Results from other investigators are applied to estimate the number of reversed inelastic loading cycles which must be sustained during the response to a damaging earthquake. The design methodology is demonstrated with two example structures typical of precast construction for zones of moderate seismicity in the United States. Strength and deformation requirements derived by this approach are presented for selected joints to facilitate the experimental validation of specific connector designs. Guidelines are presented which enable quantitative comparison of strength, plastic elongation, and cyclic load reversal demands on connectors within similar structures sited in zones of higher seismicity.

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DESIGN OF CONNECTIONS FOR PRECAST PRESTRESSED CONCRETE BUILDINGS FOR THE EFFECTS OF EARTHQUAKE

Phase 2 of a Three-Phase Program

by D.P. Clough

A Research Investigation

by

ABAM Engineers Inc.
500 South 336th Street, Suite 200
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The bearing wall apartment building which served as the basis for the second example was developed by W.G. Lischka, J.M. Becker, I.J. Speyer, S. Firnkas, T.J. Ecsedi, M. Fintel, A. Aswad, F.J. Jacques, N. Ladduwahetty, and D.P. Clough, in association with the PCI Committee on Precast Bearing Wall Buildings, chaired by Alexander Popoff. This structure was the focal point for a PCI design seminar presented by the committee in October 1983. The committee's permission to use its building as an example in the present research is gratefully acknowledged.

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CHAPTER 1
INTRODUCTION

1.1 SEISMIC-RESISTANT PRECAST CONSTRUCTION IN THE UNITED STATES -- AN HISTORICAL PERSPECTIVE

Precast concrete buildings have been most widely accepted in regions of low to moderate seismicity, where they compete effectively with buildings of cast-in-place concrete and other construction materials.

While precast can provide aesthetic value and materials quality superior to cast-in-place concrete, economic considerations probably give precast construction its most important competitive advantage. To gain the greatest competitive edge, precast manufacturers in the buildings market have evolved sophisticated modular systems which make maximum repetitive use of standard components and connection details. Precast building design has become highly process oriented, involving production, storage, transportation, handling, and erection of preengineered components which are joined at the site with a minimum of field labor.

Precast building systems most widely used in the United States evolved at a time and in regions where seismic resistance was not a significant design objective. Connection details, standard precast components, production fixtures, and the basic framing concepts of many precast building systems reflect this history to some degree. The present state of the art reflects a substantial investment in design and production tooling for modular systems not ideally suited to earthquake resistance.

Retooling costs can discourage large-scale experimentation with new design approaches; significant up-front expenses are involved when production facilities must be modified and these place the precaster at a disadvantage with respect to his competitors using cast-in-place construction.

Yet there are indications that change will be required in the precast industry's approach to earthquake-resistant design. Precasters express a growing interest in expanding their market into more seismically active regions, and seismic risk assessments for regions traditionally viewed as earthquake-free are being revised upward as seismologists continue to compile and evaluate geological and historical data.

The ability to adapt to these new conditions may be important for the continued vitality of the precast buildings industry. While continued growth into seismic regions may provide an opportunity for new profit, the prospect of increased seismic requirements in existing markets is the more urgent motivation to quantify seismic demands and to develop cost-effective adaptations of existing technology.

There is a need for experimental and analytical investigations into the dynamic behavior of "jointed" structures, in which gaps between precast elements open and close during the response to ground shaking. Rational seismic design procedures must be developed which address the specific requirements of precast concrete buildings. The nature and magnitude of changes required in existing precast connection details and modular framing systems must be determined. While the attainment of these objectives will take time, some of the results can be anticipated.

As stated by Martin and Korkosz [1], issues of economy and suitability for service in seismic regions are intertwined.

> Much of the economy of precast prestressed concrete structural framing lies in its simplicity. It is best used in simple span beams and deck members. The absence of continuity and redundancy has caused some designers to question stability under high lateral loads.

Due to lack of continuity, the precast systems most readily constructed tend to be less rugged or forgiving of extreme overload than typical cast-in-place structures. To make sure that precast structures intended for service in seismic regions deliver all necessary capabilities as well as reduced cost (in other words, true economy), considerations of redundancy and ductility must be introduced in the design process. To the extent that these qualities are "standard equipment" with cast-in-place and "extra-cost options" with precast, the adoption of more rigorous design procedures could soften the competitive stance of precast concrete in zones of higher seismicity.

On the other hand, rational procedures which enable prediction of ductility demand will facilitate the development and experimental validation of suitable connection details. Rather than emulating cast-in-place construction, designers of precast buildings for service in earthquake regions will be able to develop competent lateral resisting systems which retain the economic and aesthetic benefits of precast concrete. Considerable research and development work will be required, but these steps appear essential to the greater acceptance of precast concrete construction in seismic regions.

1.2 TOWARD IMPROVED BUILDING CODE PROVISIONS

Building code provisions for precast concrete, as for other construction materials, evolve through the calibration of design, analysis, and production techniques against the knowledge obtained by experimentation and practical application. Thus, progress depends upon effective communication between design practitioners, researchers, precast producers, and code authorities.

Improvements in seismic code provisions for cast-in-place concrete during recent decades, for example, reflect major advances in seismology, structural dynamics, and empirical knowledge of the reversed cyclic, inelastic behavior of reinforced concrete structures. This progress attests to the productivity of the research community, the vitality of code bodies, the dedication of design practitioners, and the effectiveness of communication among the three groups.

The research facilities and technical competence for producing similar advancements in the seismic-resistant design of precast concrete exist. Unfortunately, it appears that much of the current body of experimental data regarding connections for precast structures has been obtained through privately funded research and is held as proprietary. Much of the private money spent for connection testing seems to have been directed toward specific short-term goals of the sponsor. If improved design procedures and code provisions for precast structures are desired, improved communication among researchers, producers, designers, and code officials will likely accelerate their development.

An important communications resource is the significant number of researchers and academicians among the membership of the Prestressed Concrete Institute (PCI). The Institute was founded in 1954 for the purpose of advancing the design, manufacture, and use of prestressed, precast concrete, and represents the prestressed concrete industry in the United States and Canada. Because all segments of the profession engaged in seismic-resistant design of precast concrete structures are represented in its membership, this organization is a logical forum for developmental efforts.

PCI traditionally has engaged in activities that promote communication among members of the structural design and construction community. These include documenting the evolution of the "state of the art," disseminating research findings to design practitioners, developing design standards and codes of recommended practice, and participating in the development of building code recommendations.

1.3 BACKGROUND OF THE PRESENT RESEARCH EFFORT

This report was produced during Phase 2 of a three-part program, conceived by PCI and intended to advance the state of the art of connection design for seismic-resistant precast concrete structures. The Phase 1 report [1], funded by the National Science Foundation (NSF) and prepared by the Consulting Engineers Group, Inc., presents an overview of the current state of the art.

Included in the Phase 1 report are a discussion of basic seismic response concepts, a review of research findings on seismic ductility demand in cast-in-place concrete structures, design formulas for computing primary-load reinforcement at the connections of precast structures, and a compilation of the most commonly used connection details.

The connections were evaluated by the precast industry with regard to simplicity, durability, and volume change accommodation. It is revealing of the current state of the art that strength and ductility were not among the rating criteria. Although seismic ductility demand is a topic of current research, specific steps to assess ductility requirements and to ensure that they are satisfied in a particular structure have not thus far been incorporated in the design methodologies available to building engineers.

In Phase 2, also funded by NSF, a rational methodology for the derivation of connector strength and ductility requirements has been developed, suitable connection details for representative example structures have been conceived, guidelines for adapting existing technology to regions of higher seismicity have been formulated, and recommendations for physical testing of selected connection details have been prepared. This report presents these developments. Physical testing is to be conducted by others in Phase 3.

1.4 OVERVIEW OF THE PROPOSED SEISMIC DESIGN METHODOLOGY

Satisfactory seismic performance of a precast structure depends to a great extent on fundamental issues of building configuration and framing scheme. Accordingly, as shown in Fig. 1.1, the design methodology which has been developed in the present research begins with the selection of a suitable lateral force resisting system, proceeds through the determination of design loads and the estimation of global inelastic displacements during a damaging earthquake, and concludes with an interpretation of kinematic properties of the yielded lateral resisting system in terms of the deformational requirements for connectors in specific joints.

Another important feature of the proposed technique is that it enables a range of design strategies with varying degrees of reliance on inelastic action. Hence, in the methodology developed here, the designer is offered a choice among possibilities which range from producing a ductile structure with yield strength significantly smaller than the "elastic strength demand" to producing a high-strength structure which will experience reduced ductility demands during the largest earthquake anticipated at the site. Such an approach could be advantageous for some types of construction (for example, low panelized buildings in regions of moderate seismicity) because it is sometimes easier, more reliable, and less expensive to provide extra connection strength than to provide a degree of ductility consistent with the reduced design force levels prescribed by typical building codes.

In addition, the proposed approach enables the designer to compute rational estimates of connection ductility and deformational compliance demands in critical regions throughout the structure. This additional information can facilitate the appropriate selection of structural layout, jointing strategy, and connection details if the designer compares results obtained from alternative structural configurations.

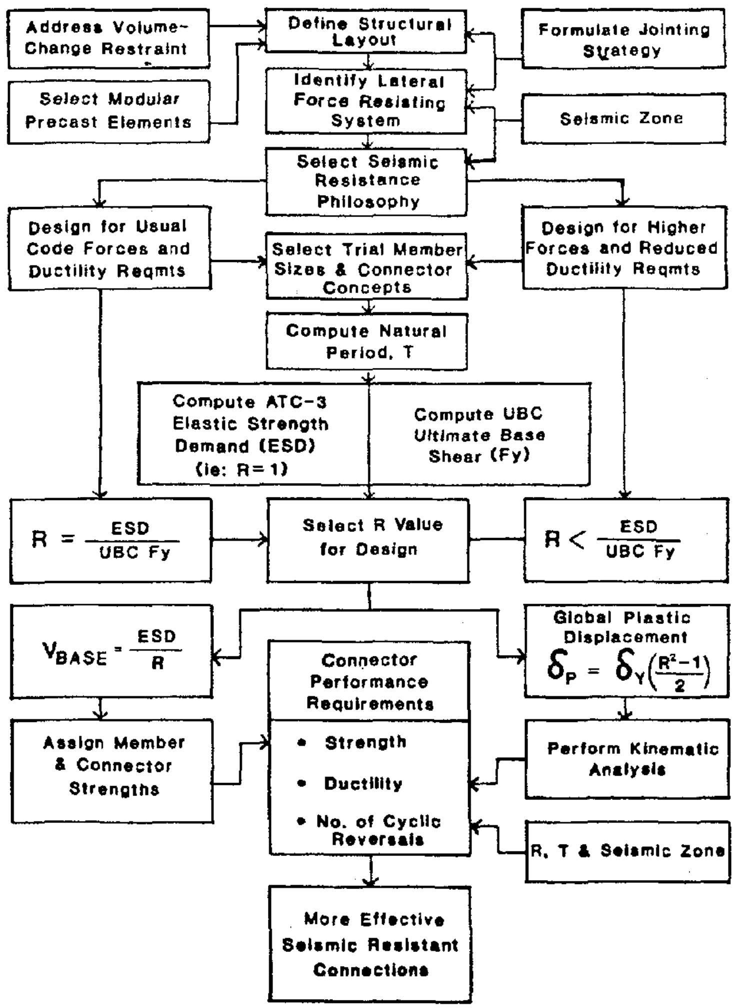


FIGURE 1.1 - DESIGN METHODOLOGY FOR SEISMIC RESISTANT PRECAST CONCRETE BUILDINGS

1.5 OBJECTIVES OF THE PHASE 2 REPORT

Beyond the presentation of an improved design methodology, the Phase 2 report is intended to explain seismic response of buildings in a way that is useful to design practitioners having no formal training in earthquake engineering or structural dynamics. This is the subject of Chapter 2.

Another objective is to apply the methodology to representative examples of precast construction in the United States, illustrating the appropriate degree of analytical refinement and identifying inadequacies of present knowledge concerning the behavior of jointed precast assemblages. Based on the example structures, connection details and associated seismic performance criteria have been proposed for physical testing by others in Phase 3. Chapters 4 and 5 contain detailed applications of the proposed methodology presented in Chapter 3.

A final objective is to draw general conclusions for structures similar to those treated in the examples, to serve as interim guidelines for seismic-resistant connection detailing before completion of the Phase 3 tests. Accordingly, a summary and conclusions are provided in Chapter 6.

1.6 TECHNICAL INPUT GROUP

To ensure that the Phase 2 study addressed the topics of greatest practical concern across the country, the work was planned in cooperation with a Technical Input Group, composed of members of the Connection Details, Technical Activities, and Seismic committees of the Prestressed Concrete Institute.

The Technical Input Group was asked to propose examples for study. It was agreed that a candidate structure should represent a type of construction which provides large public exposure to structural precast concrete, considering building size, occupancy, and frequency of occurrence across the country. Further, the structure should be of a type which offers significant market potential for precast concrete in seismically active regions if present design, performance, and acceptance limitations are overcome.

Using these criteria, a seven-story precast concrete parking structure based on the Metro-Space building system and a hypothetical 17-story bearing wall apartment building designed by the PCI Bearing Wall Committee were selected.

1.7 INDUSTRY AWARENESS AND PARTICIPATION ARE VITAL

Some of the most important information concerning the adequacy of seismic design and analysis methods is gained by surveying the performance of structures which have been subjected to earthquakes. Damaging earthquakes, however, are few and far between, so this information accumulates slowly. Further, these "experiments" performed in the "real-world laboratory" seldom are planned to maximize the value of the information gained. Seismic design details tend to receive attention in proportion to a structure's size, cost, importance, and seismic zone. Because the number of "minor" structures greatly exceeds the number of "major" structures constructed of precast concrete, the typical earthquake victim tends to be a "minor" structure not designed for seismic resistance. Although precast structures seem to have fared well in earthquakes in the past, these observations suggest that the seismic behavior of precast structures in future earthquakes may convey little information on the adequacy of state-of-the-art technology for earthquake-resistant connection design.

By disseminating results of the present research, the precast concrete industry can promote uniformity of approach and attention to seismic-resistant detailing in the design of precast concrete structures, large and small. This will increase the opportunities for observing and evaluating state-of-the-art connection performance, enhance the credibility of precast in seismic regions, and accelerate the evolution of connection design technology.

These steps are appropriate given the current state of knowledge, but it is important to see the present effort as a waypoint on a longer journey. Many important questions remain unanswered:

- What is the effect of elastic nonlinearities, due to joints which open and close during an earthquake, on the seismic response amplitude of a jointed precast structure? Do traditional seismic loading criteria, developed for monolithic structures, apply?
- When assessing the fundamental period of a jointed precast structure for computing code seismic forces, is it necessary to model

joint flexibility realistically, or is it sufficient to approximate the connections as either fixed or pinned? If realistic modeling is essential, experimental data or reliable predictive techniques not presently available must be obtained.

- Current seismic design codes presume monolithic behavior and energy dissipation involving large volumes of material. Is it valid to use code forces for jointed precast construction in which relatively small volumes of material yield?

Precast buildings are being constructed in earthquake zones and there is immediate need for seismic design guidelines. This means, for the time being, that answers to some important questions must be assumed. New questions will arise as today's questions are answered but, in time, an improved understanding will evolve.

Technological evolution is iterative, involving hypothesis, experimentation, evaluation, and deduction. In the case of seismic-resistant design, evolution is the product of effective communication between industry, government, the research community, and private consulting practice, motivated by a shared commitment to earthquake safety. This report is offered as evidence of technological evolution in progress.

CHAPTER 2
PRECAST CONCRETE CONSTRUCTION VERSUS EARTHQUAKES

2.1 INTRODUCTION

While the building design profession shows a growing awareness of the need for an effective and practical response to seismic hazards, engineering seismology and structural dynamics are beyond the scope of most undergraduate civil engineering programs; the majority of engineers are first exposed to the questions of earthquake resistance only after entering professional practice.

On the other hand, all engineers are familiar with the basic principles of mathematics and physics. With these tools, a solid conceptual understanding of building code seismic provisions and the structural requirements for earthquake safety can readily be achieved. Such understanding will significantly help many engineers responsible for the design of regular structures in regions of low to moderate seismicity who want to know that their interpretations of code requirements are correct, and who seek greater confidence in their ability to select the design alternatives best suited to earthquake resistance.

This chapter is intended to provide a conceptual understanding of the sources of earthquake ground motions and the earthquake behavior of buildings. Building code seismic provisions are explained. Important differences in the seismic behavior of "jointed" and "monolithic" structures are described. Specific considerations in the planning, analysis, and detailing of seismic-resistant precast concrete structures are presented in the design methodology of Chapter 3, which is then applied to practical examples in Chapters 4 and 5.

Though the material presented here can improve the effectiveness of building designers who lack academic training and a depth of experience in earthquake engineering, it is neither precise nor complete. Formally trained specialists should be consulted in the planning and design of large, irregular, or unusual seismic-resisting structures.

2.2 THE ORIGINATION AND PROPAGATION OF SEISMIC GROUND MOTIONS

Earthquakes and their capacity to inflict damage can best be described in terms of energy, its changes from one physical manifestation to another, and its propagation through time and space. Energy may be considered to occur in either of two conditions, active or passive.

Energy in its passive state can be difficult to recognize. A frozen blanket of snow in the high mountains bears no outward resemblance to the sun's heat which drives the evaporative process and the storms, causing an upward migration of water against the gravitational field. Yet, a snowfield high above sea level can be regarded as a passive reservoir of stored solar energy. When this energy bank is unlocked by warmer temperatures in the spring, the energy is transformed to the active state; the waters rush downward, moving rocks and boulders in the streams, eroding river banks, and spinning the turbines in hydroelectric generating plants.

Energy in the passive state is referred to as potential energy, or the stored capacity to perform work. The coiled spring in a clock, the altitude of an airplane above the ground, and the fuel in its tanks are examples of potential energy.

Energy in its active state is easily observed and is manifested as movement or flow. Kinetic energy associated with mass in motion, as the water flowing in a stream; electromagnetic energy, such as visible light and heat; and sound waves traveling through air are all examples of energy in the active state.

The staggering gravitational potential energy of the earth's upthrust landmasses demonstrates the enormous energy flux associated with the earth's internal workings. With continued activity of the geological processes which shaped the continents, earthquakes are the active manifestation of energy once stored in a passive state beneath the earth's surface as elastic strains.

Elastic strain energy released during an earthquake radiates from the location of fault rupture, somewhat like ripples on a pond expanding from the point where a fish has jumped. A large earthquake sets the entire world into vibration; the Chilean earthquake of May 21, 1960, started oscillations which continued for nearly two weeks [2].

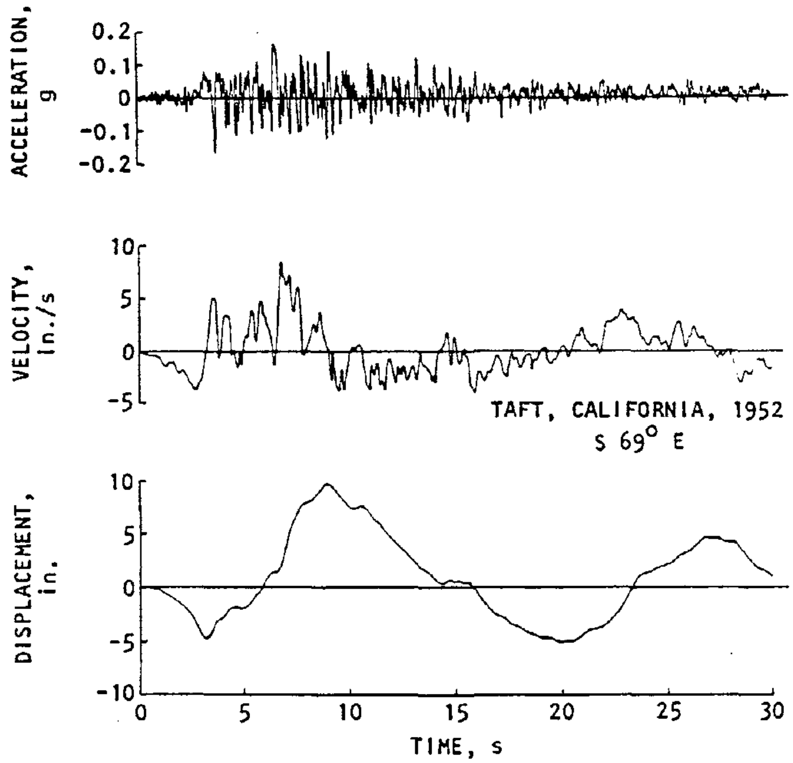
Though of interest to geophysicists and seismologists, these barely detectible oscillations persisting for many days are not important to structural engineers. The ground motions which can damage buildings are of enormously greater amplitude and shorter duration, and occur in a more limited geographic region around the epicenter, the point on the earth's surface directly above the point of first fault rupture.

To record earthquake ground motions, structural engineers use instruments called strong-motion accelerographs, strategically located in anticipation of future seismic events. These devices automatically store ground acceleration values as a function of time, and yield the information of greatest importance with regard to seismic resistant design: amplitude, frequency content, and duration of the strong ground shaking portion of the earthquake at the site where the record was made.

Fig. 2.1 presents the S69E component of horizontal ground shaking at Taft, California, during the Tehachapi earthquake of 21 July 1952. This record was made by an accelograph located about 25 miles from the causative fault and depicts moderately strong shaking. Typically, plots of ground velocity and ground displacement are obtained by integrating the acceleration record.

Neglecting effects of nonuniform geological structure and local soil conditions, the amplitude and duration of ground shaking at a given distance from the epicenter depend on the amount of seismic energy released. The greater the energy, the larger the geographical area within which high-amplitude ground motions will occur. Additionally, however, the earth acts as a filter; high-frequency, jolting motions which tend to predominate near the epicenter die out more rapidly with distance than lower frequency motions, which can propagate over considerably greater distances at significant amplitude.

Due to this filtering, earthquake records obtained at different distances from the epicenter can exhibit widely differing frequency content.



EARTHQUAKE GROUND MOTION

FIGURE 2.1 – GROUND MOTIONS DURING A MODERATELY STRONG EARTHQUAKE (FROM REFERENCE 3)

Apartment dwellers are familiar with this effect: the bass from a too-loud stereo can be heard down the hall, while the melody is inaudible until one is within striking distance of the offending neighbor's door.

Earthquakes, then, are seen as energy in motion, bearing a message of destruction from bursting strata, propagating at high speed through the ground, and seeking expression in the sympathetic vibrations of any structural system, natural or man-made, which will resonate at the appropriate frequencies.

2.3 EARTHQUAKE RESPONSE OF BUILDINGS

A city is like a forest of tuning forks standing on top of a piano. Though some of the tuning forks will vibrate sympathetically, depending on what chord is played, the energy they absorb does not perceptibly diminish the piano's volume. Similarly, man-made structures absorb an insignificant fraction of the energy released during an earthquake.

Thus, while ground shaking is in progress, the vibrating earth appears to a building as an infinite energy source. What, then, prevents all man-made structures in the vicinity of the epicenter from absorbing enormous quantities of seismic energy and being destroyed? The answer to this question lies in the mechanics of seismic energy transfer between a building and the shaking ground beneath it.

2.3.1 Energy as a Measure of Seismic Response

The term "seismic response" usually evokes images of forces and displacements. This is natural because these are the parameters treated in the building codes and are quantities directly applicable in design. To develop a deeper understanding of seismic response, however, it is useful to think in terms of energy.

Fig. 2.2 presents an equation describing the action by which energy is imparted to a structure during an earthquake. Recall that energy can be defined as a capacity for doing work; both work and energy have units of (force) x (distance). The seismic energy absorbed by a structure equals the net work done on it by ground motions during the earthquake.

Equation 2.1

$$\underbrace{W(t_1)}_{\text{Work}} = \int_0^{t_1} \underbrace{\overset{\text{Base Shear}}{\downarrow}\atop V(t)}_{\text{Force}}\ \underbrace{\overset{\text{Ground Velocity}}{\downarrow}\atop \dot{x}(t)\ \ \overset{\text{Time}}{\downarrow}\atop dt}_{\text{Displacement}}$$

FIGURE 2.2 – WORK PERFORMED ON STRUCTURE BY SEISMIC GROUND MOTIONS

Equation 1 states that after a length of time "t" from the beginning of ground shaking, the net work done on a structure equals the integral of the base shear multiplied by the ground velocity. Mathematics aside, the concept is as simple as pushing a swing.

As shown in Fig. 2.3, pushing in the direction of motion increases the swing's response; pushing in opposition to the swing's motion decreases its response. In the context of Equation 1, force and velocity with like sign produce positive work; with opposite sign, negative work.

As the swing oscillates, energy is constantly being transformed from the active state to the passive and back again. When the swing is stopped at the top of its arc, all of its energy is passive; the velocity is zero and there is no kinetic energy. The mass of the swing and rider are at their highest point so the gravitational potential energy of the system is at its maximum.

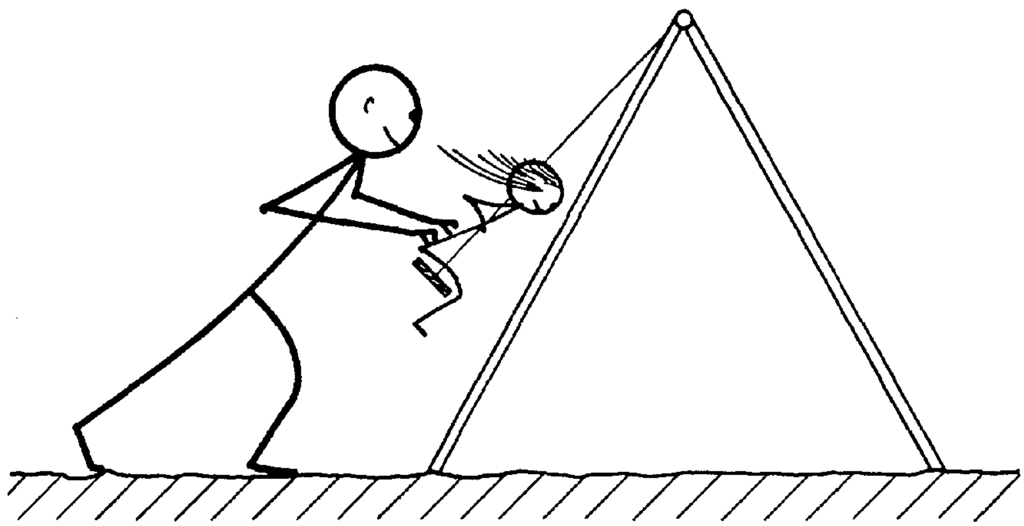
Conversely, when the swing is at the bottom of its arc, the potential energy is at a minimum and, because the velocity is greatest at this point, the kinetic energy is at a maximum. Thus, as the swing travels along the arc, there is a continual flow of energy between the passive and active states; in other words, a continual tradeoff between potential and kinetic energy. This tradeoff occurs with absolute regularity. The time required for one complete cycle is called the natural vibration period; its reciprocal is the natural frequency.

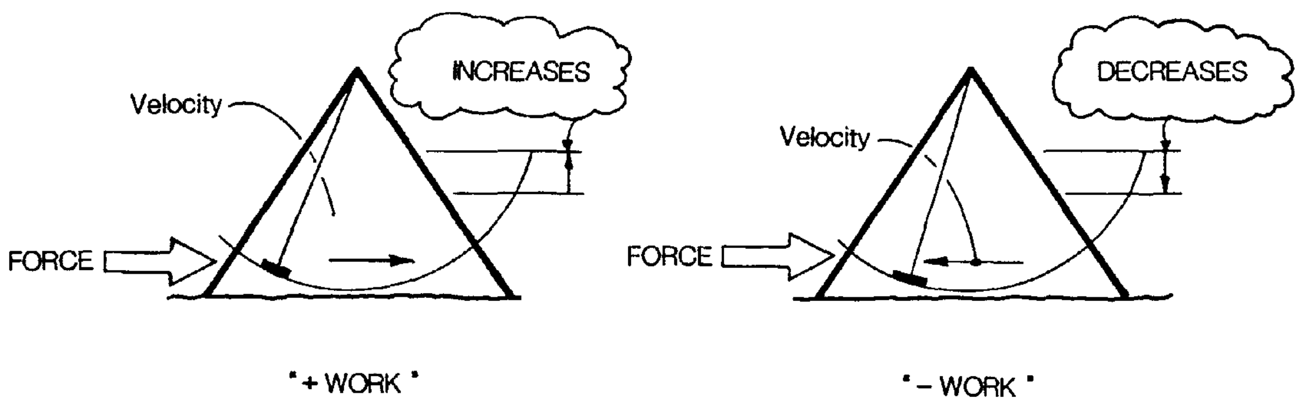
In the case of a swing undergoing "small" displacements or a linearly elastic spring/mass system, the natural vibration period is unaffected by the amplitude of motion. The vibration amplitude is directly related to the amount of energy which has been absorbed.

2.3.2 How Buildings Absorb Seismic Energy

Oscillating external forces timed to coincide with a dynamic system's natural frequency are the most effective in feeding kinetic energy into the mass. Consider the base shear and ground velocity terms in Equation 2.1.

Figure 2.4 shows the time history of base shear for a flexible, single-degree-of-freedom (SDOF) structure vibrating freely. For flexible structures, the base shear response during an earthquake tends to alternate regularly at the building's natural vibration frequency, and looks somewhat

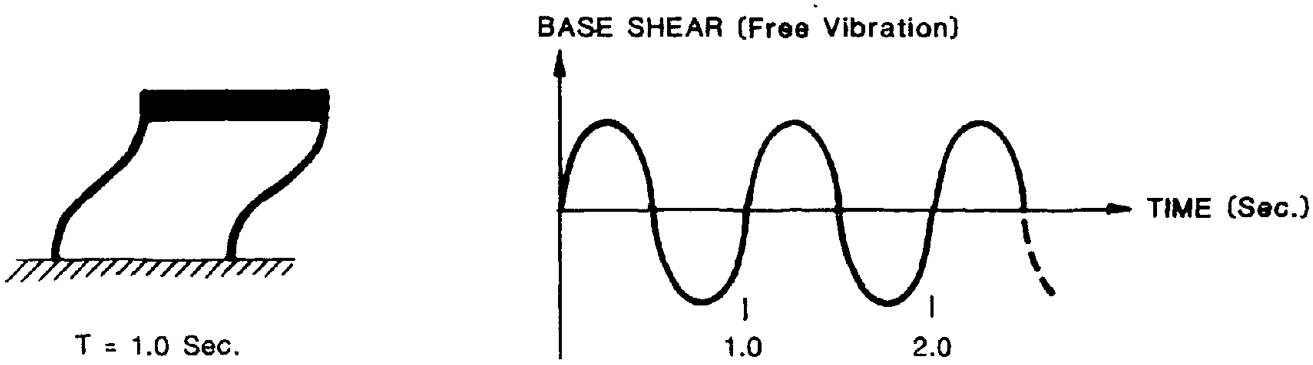




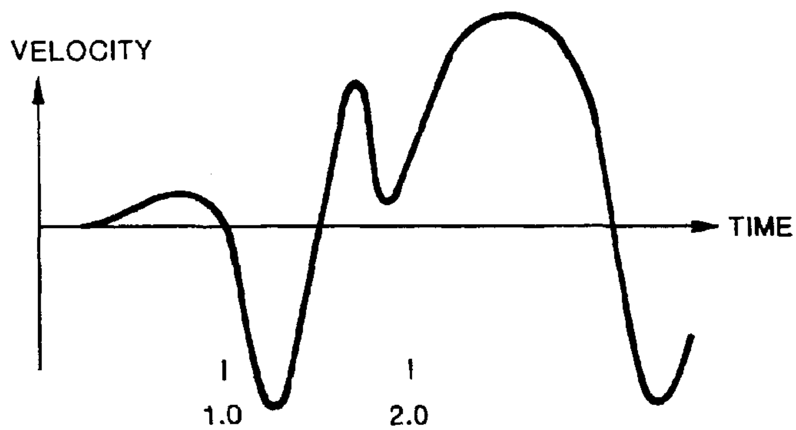
a. Force and Velocity in same direction produce Positive Work - RESPONSE INCREASES

b. Force and Velocity in opposite directions produce Negitive Work - RESPONSE DECREASES

FIGURE 2.3 – RESPONSE OF SWING, ILLUSTRATING POSITIVE AND NEGATIVE WORK



A. UNDAMPED FREE-VIBRATION RESPONSE



B. SEISMIC GROUND VELOCITY

FIGURE 2.4 - COMPARISON OF BASE SHEAR AND GROUND VELOCITY TIME HISTORIES

like Fig. 2.4.a. Fig. 2.4.b represents the time history of ground velocity during an earthquake. This plot varies erratically. The ground velocity may change sign several times for every change in sign of the base shear.

The erratic variation in ground velocity results from the superposition of a large number of regularly varying components, each with a different amplitude and frequency, as illustrated in Fig. 2.5. Ground velocity components in the neighborhood of the structure's natural vibration frequency induce a progressive, resonance-like buildup of energy, similar to someone pushing a swing. Excitation at higher frequencies can be likened to a second person running alongside and shaking the swing. This erratic excitation causes energy to flow into or out from the structure in sudden pulses, according to the instantaneous match, or mismatch, in signs of force and velocity.

2.3.3 Factors Tending to Limit the Absorption of Seismic Energy

With this background, what does prevent all structures in the vicinity of the epicenter from taking on vast quantities of energy and being destroyed?

First, just as in pushing a swing, it takes time for the energy level in a structure to build up. So, one limiting factor is the finite duration of ground shaking.

Second, only the ground motion components nearly matching the structure's natural vibration period are effective in producing a resonant response buildup. Accordingly, on firm soils near the zone of fault rupture, stiff structures tend to be more strongly excited than flexible structures while, at great distance from the epicenter, flexible structures tend to be the more strongly excited.

Third, characteristics of a structure's lateral force-displacement relationship affect its receptivity to energy input from ground shaking. The most receptive structures are those with linearly elastic stiffness properties, and with natural vibration frequencies similar to the frequencies of strongest ground shaking. Structures such as guyed towers, walls on rocking bases, and frames allowed to lift off their foundations in response to lateral loads

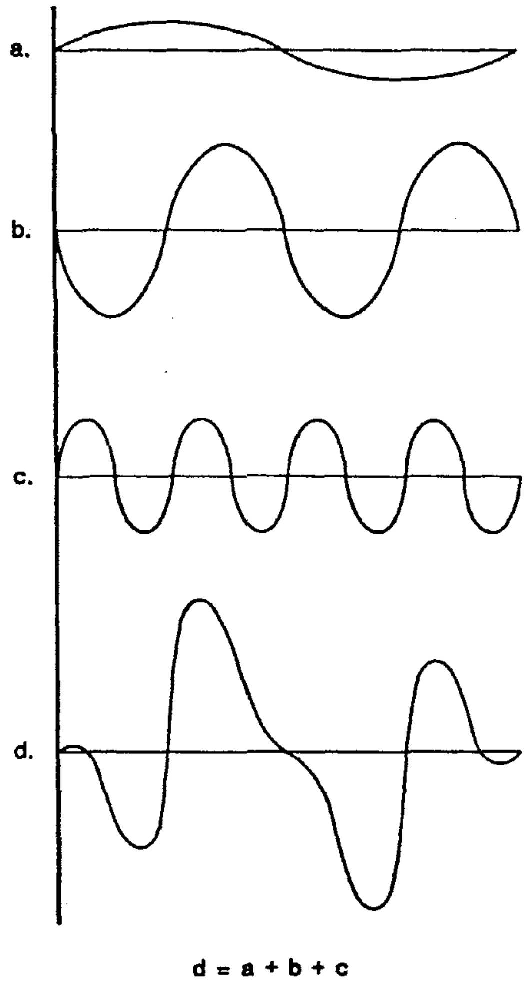


FIGURE 2.5 – "IRREGULAR" WAVEFORM AS THE SUM OF REGULARLY VARYING COMPONENTS

have nonlinear force-displacement characteristics, even with stresses below the elastic limit.

Nonlinear structures do not exhibit uniquely defined natural vibration periods. As they begin to respond to a harmonic load, their effective period of vibration changes. Thus, nonlinear structures tend to detune themselves from the excitation and reject a further energy buildup. Reference [4] reports some interesting shaking table studies on a large-scale, nine-story steel moment frame with columns free to lift off the foundation. The intentionally induced uplift nonlinearity was demonstrated to be very effective in reducing seismic forces and ductility demand.

Fourth, some of the energy fed into a structure is dissipated as a consequence of deformations within the "elastic" range. Microcracking of reinforced concrete, working of joints, wracking of partitions, and inelastic behavior of the soil/foundation interface are energy-dissipating mechanisms that do not involve structural damage. Often these effects are lumped together and modeled analytically as linear viscous damping.

Finally, if the response builds far enough, the elastic limit will be surpassed. Nonlinearities of the force-displacement relationship after the elastic limit is exceeded tend to inhibit a further response buildup; if the structure is brittle, however, or if its deformations have grown to the point of instability, collapse is imminent. On the other hand, if ductile construction materials and connections have been used, inelastic member deformations will dissipate energy, tending to maintain response values of tolerable amplitude.

Even with a ductile structure, however, there must be a limit to the seismic abuse it can sustain. Returning to the swing analogy one last time, imagine a white-knuckled rider attempting to stop, heels gouging desperate furrows in the sand, as the neighborhood bully rockets the swing into another sky-kicking arc. A designer's hope is that the bully will get called home to supper before the victim runs out of sand to gouge.

This is the essence of the seismic design problem. First, the amplitude, duration, and frequency content of ground shaking due to the most severe earthquake anticipated in the structure's lifetime must be deduced from geological observations and historical seismicity of the site. Second, recognizing the considerable uncertainties of these loading parameters, the designer must provide sufficient strength, ductility, and stability that the structure, though damaged beyond possibility of economical repair, will survive that once-in-a-lifetime earthquake without collapse.

2.3.4 Storage Tank Analogy for the Absorption and Dissipation of Seismic Energy

An earthquake represents a virtual ocean of energy, of which a structure can absorb a certain amount elastically. The rate at which energy flows into the structure depends on the degree of matching between frequencies of the structure and the excitation, on the intensity of the excitation, and on the degree to which the relationship between force and displacement is linear.

In Fig. 2.6, the structure is represented by a storage tank. The height of the overflow nozzle above the base depicts the structure's elastic strength. The smaller tank to the left represents the structure's ability to dissipate energy through plastic deformation. The small nozzle at the base of the main tank represents the effect of damping, which also dissipates energy that has flowed into the structure.

Two large pipes conduct "energy" into and away from the main tank. The inlet pipe is fitted with a valve. For linear elastic structures, this valve is wide open; for nonlinear (elastic or inelastic) structures, it is partially closed.

Let's see how it works. As the ground displaces under a flexible structure, the first-story columns deflect, inducing a base shear. As the ground motion continues, the product of base shear and ground velocity integrates as described by Equation 2.1; energy flows into the structure during time intervals when the signs of base shear and ground velocity agree, and flows out when the signs disagree.

Thus, in the conceptual analogy, the "energy" level in the tank rises and falls under the erratic ground shaking. If the ground motion intensity is slight or the duration short, the overflow level may never be reached; the structure would survive the earthquake undamaged. On the other hand, if the excitation is intense enough or the duration is long enough, response will build until the structure's elastic limit has been attained. Further energy

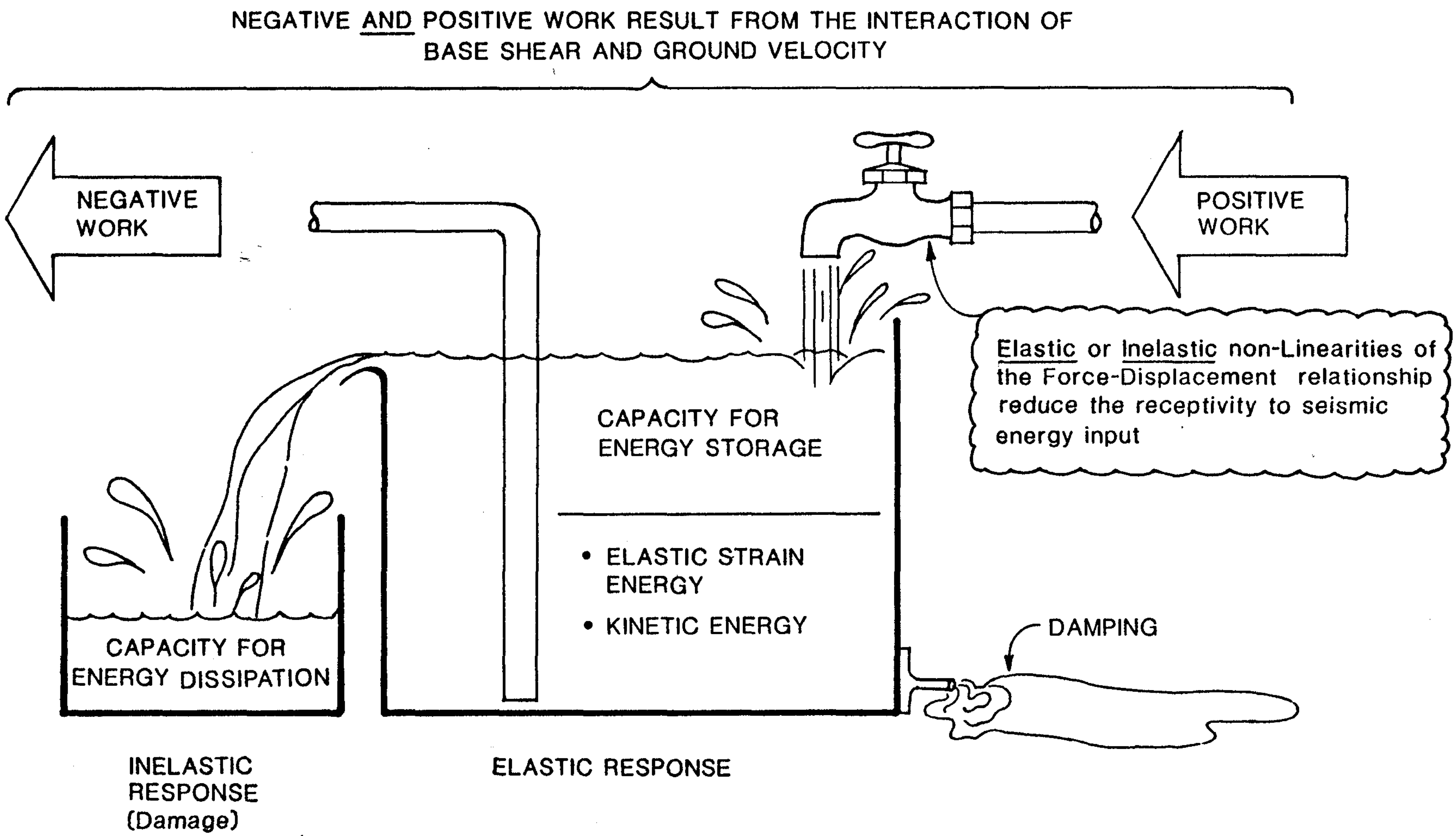


FIGURE 2.6 – STORAGE TANK ANALOGY FOR THE SEISMIC RESPONSE OF BUILDINGS

input will produce damage; additional "energy" flowing into the main tank will spill over into the smaller reservoir.

In general, a structure will experience several episodes of inelastic deformation as its "energy reservoir" repeatedly fills, overflows, and is drawn down during a damaging earthquake. If the structure's capacity for plastic energy dissipation is sufficient, the earthquake will subside before it fails. However, if the structure is relatively brittle, depicted by a small overflow tank, failure may occur soon after the elastic limit is reached.

When the ground shaking stops, energy no longer flows into the structure. Elastic strain energy and kinetic energy, represented by the contents of the large tank, are dissipated by damping and the structure comes to rest. The contents of the overflow tank represent plastic work done on the structure and this usually implies structural damage. These concepts are summarized in Fig. 2.7.

2.3.5 Design Implications

Interaction of the base shear with the ground motions imparts energy to a structure during an earthquake. For a family of linearly elastic structures of differing weight but with the same natural vibration period, seismic base shear magnitude (hence the amount of energy absorbed) during a given earthquake varies directly with mass.

For a family of structures with constant weight, but with a range of vibration periods, the amount of energy absorbed depends on the amplitude of ground shaking at a given structure's frequency. Near the epicenter, higher frequency components predominate, while lower frequency motions characterize the ground shaking observed at distant sites.

Design decisions have a direct impact on the amount of energy which will be absorbed. In general, for a linearly elastic structure, seismic energy absorption will be mitigated by reduced mass and lengthened vibration period. These parameters can be manipulated through the choice of construction materials and framing scheme.

The net energy input must be dissipated, either through damping or through inelastic action. While energy absorption is related to stiffness and mass, inelastic energy dissipation is related to strength and ductility. Brittle

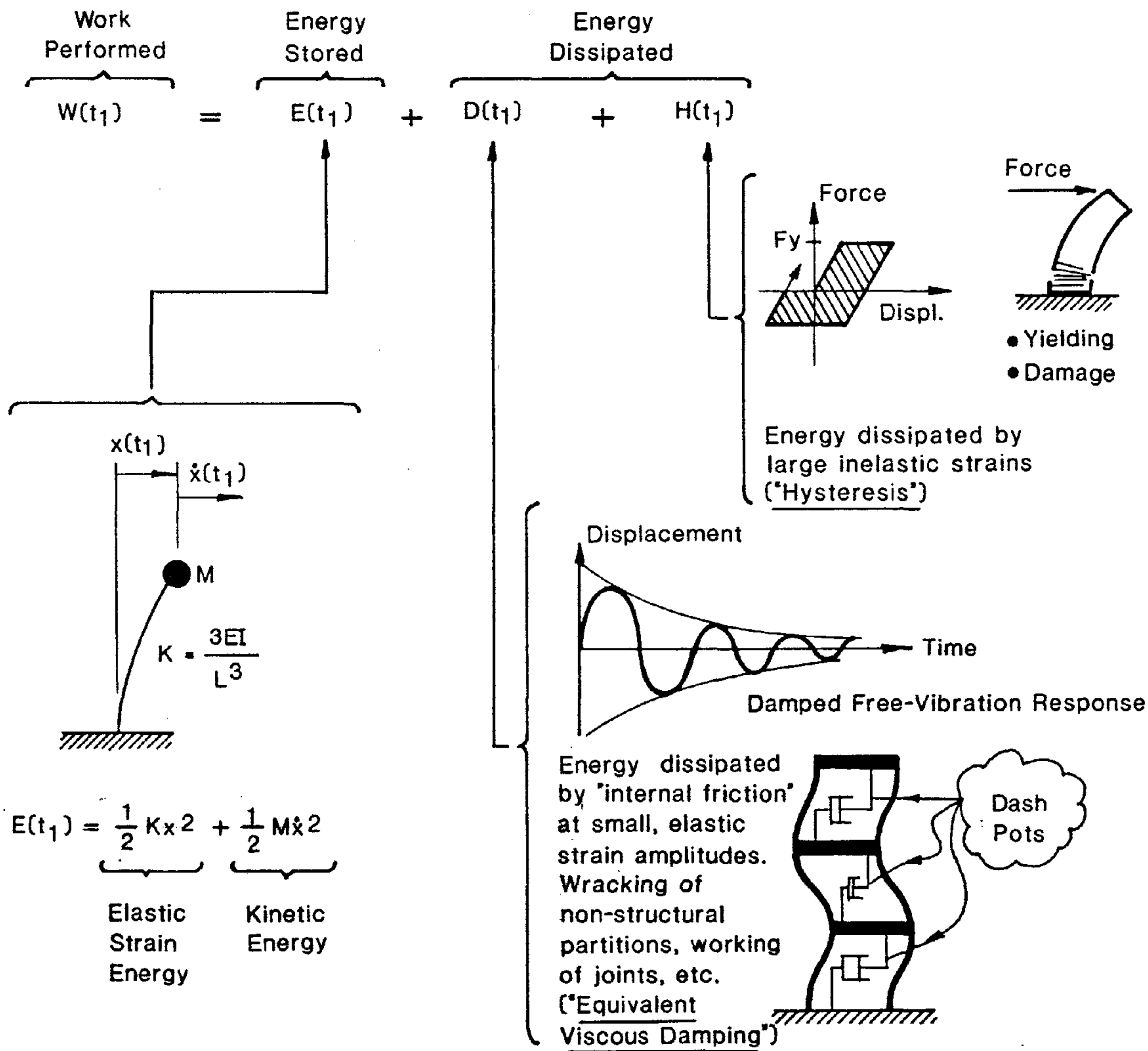


FIGURE 2.7 - ENERGY BALANCE

structures must be designed to contain most of the absorbed energy elastically. For a given mass and period, this means that the design strength of a brittle structure must be higher than that of a ductile structure, which can dissipate much of the absorbed energy through inelastic action.

A design alternative receiving increasing attention is seismic base isolation [5, 6]. Special elastomeric bearings or other compliant devices are used to reduce the seismic base shear, thus directly inhibiting the mechanism by which energy is fed into a structure by ground shaking. Reduced energy absorption produces a direct reduction in the requirements for strength and energy dissipation.

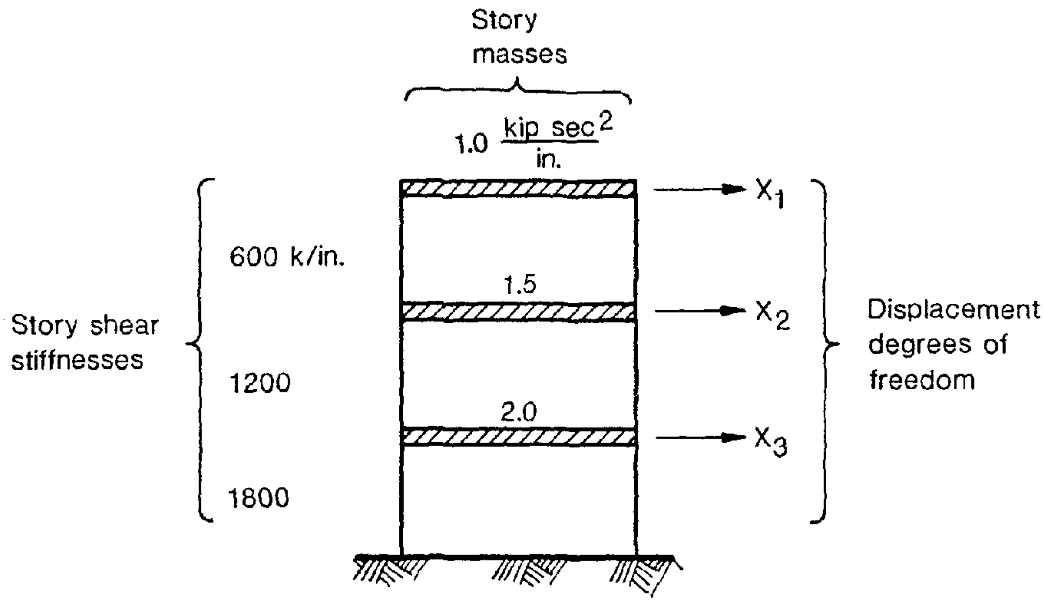
2.4 MULTI-DEGREE-OF-FREEDOM SYSTEMS

Earthquake response of buildings has been explained by analogy with the motions of a swing. Because it has mass, a swing can absorb (active) kinetic energy; due to the gravitational field, displacements of the swing away from its equilibrium position absorb (passive) potential energy. Similarly, a building's mass can absorb kinetic energy, while its elastic deformations absorb (passive) strain energy. For both, free vibration involves a continual transfer of energy back and forth between the active and passive states. In this sense, the two systems are analogous.

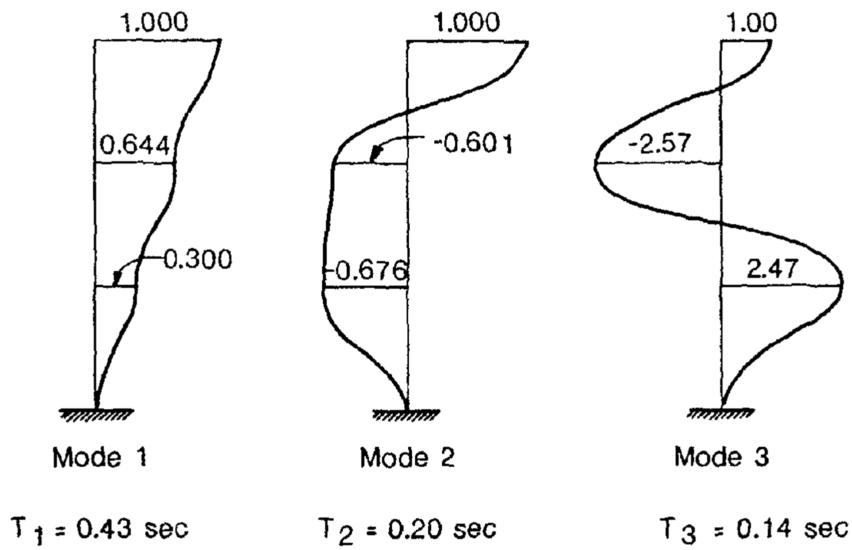
2.4.1 Limitations of the Swing Analogy

With regard to the distribution of mass and the possible patterns of deflection, however, important differences can exist. The swing's mass is concentrated at a single point; the displacement and velocity of that point fully describe the swing's deflected position, its potential energy, and its kinetic energy. Because the motions of this single point provide a complete description of its dynamic state, a swing is said to be a "single-degree-of-freedom" system.

In contrast, information about many points may be required to describe the deflections and internal energy of a building during an earthquake. For example, consider the three-story plane frame shown in Fig. 2.8.a. The lateral displacement at each of the three floors must be



a. Structural Properties



b. Mode Shapes and Vibration Periods

FIGURE 2.8 - HYPOTHETICAL 3 - DOF SYSTEM (FROM REFERENCE 9)

known to describe its deflected shape and strain energy, while velocities of the three floors are required to describe its kinetic energy. This structure is a three-degree-of-freedom system, and its dynamic motions are too complicated to be described by analogy with a swing.

2.4.2 Mode Shapes

Although most practical structures have many degrees of freedom, it is not necessary to consider each degree of freedom separately. The dynamic motions of many systems involve a predictable pattern of displacements. If the deformation pattern is known, knowledge of the displacement at any given point enables the calculation of displacements at all other points by proportionality.

The characteristic deformation patterns useful in dynamic analysis are called "mode shapes." While mathematical description of mode shapes and their properties is beyond the scope of this work, there is nothing complicated about the concept.

Dynamic systems have a number of mode shapes equal to their number of degrees of freedom. Mode shapes have the property of "orthogonality." This means that the set of mode shapes is like the primary colors; no given mode shape can be constructed as a combination of the others. Yet any possible deformation of a structure can be described as a combination of its mode shapes, each magnified by a scale factor which can be determined mathematically.

Each of a structure's mode shapes describes a pattern of deformations in which free vibration is possible. With each mode shape is associated a natural vibration frequency. Mode shapes of the three-story frame described above are presented in Fig. 2.8.b.

2.4.3 Modal Response to Dynamic Loads

When subjected to simple harmonic excitation (in other words, a sinusoidal motion with constant amplitude and frequency), it is possible for a multi-degree-of-freedom structure to experience resonant response in a single mode. This means that the pattern of the structure's displacements is described by the mode shape, while the response amplitude can be measured as the displacement at a single point, such as the roof. The excitation frequency determines which of the modes responds.

When subjected to earthquake ground shaking, which contains energy at many frequencies, many modes may be excited at once. However, for buildings of moderate height, with a rectangular footprint and uniform distributions of mass and stiffness, the greatest response usually occurs in the two or three modes with the smallest natural frequencies.

Thus, the seismic response of a building with many degrees of freedom often can be estimated with reasonable accuracy using only two or three displacment quantities: the response amplitudes of the first two or three vibration modes. For the example structures treated in Chapters 4 and 5, the significant seismic response occurs in the first mode and the single-degree-of-freedom swing analogy provides an adequate representation of their behavior.

2.5 PRACTICAL SEISMIC DESIGN APPROACH

Structural parameters affecting seismic response, such as mass, stiffness, damping, and strength, can be estimated with reasonable accuracy. Because the significant parameters of earthquake ground motions cannot be forecast with similar reliability, however, the prediction of a structure's seismic forces and deformations is uncertain.

Seismic design can be likened to the task of sizing a reservoir system to accommodate an uncertain volume of water. Two approaches could be considered. In the first, a single tank would be provided, which would need sufficient capacity to hold the largest anticipated volume. In the second approach, a primary tank would be sized to accommodate the most likely storage demand, while a less costly secondary reservoir would be provided to catch any excess flow after the main tank had been filled.

Designing a structure to remain elastic during an earthquake is analogous to the single-tank approach. The strength provided would need to match the largest possible seismic forces. Usually the expense to ensure elastic behavior would not be justified because well-detailed structures possess ductility and an inherent capacity for energy dissipation, giving them, in effect, an "overflow reservoir" at no extra cost.

Accordingly, in the approach taken by modern building codes, the structure is designed to yield under the most severe ground shaking anticipated at the site. Code requirements for ductile construction details ensure sufficient energy-dissipating capacity and the toughness required to sustain significant damage without collapse. On the other hand, the codes require sufficient strength to ensure elastic behavior under minor ground shaking.

The important design issue is one of selecting the most favorable combination of elastic energy storage capacity and inelastic energy-dissipation capacity; in other words, finding the best combination of strength and ductility for the intended construction materials and framing scheme.

Although the concept is straightforward, there is no analytical method for deriving seismic design criteria and the development of code provisions depends heavily on experience and judgment. For traditional construction materials and framing systems, appropriate combinations of strength and ductility have evolved and are implied in the lateral force and detailing provisions of the applicable building codes.

2.6 CODE METHODS FOR DETERMINATION OF DESIGN LATERAL FORCE MAGNITUDE

Building code requirements for cast-in-place concrete are sufficiently advanced to assure satisfactory performance under usual circumstances. In contrast, knowledge of seismic behavior for some categories of precast construction is not sufficiently complete to support the formulation of seismic code provisions.

Without the guidance which adequate design codes would provide, engineers of earthquake-resistant precast concrete buildings must understand both the basis of existing code provisions and the significant differences which can exist between precast and cast-in-place concrete structures in order to achieve the desired safety against collapse.

Current seismic design codes refer to two practical techniques for the determination of lateral seismic loads: equivalent static analysis and mode-superposition dynamic analysis. Equivalent static analysis is appropriate for simple structures of the type discussed in the examples of Chapters 4 and 5. The essential seismic response of these structures can be described with a single deformation pattern. The swing analogy presented earlier suitably illustrates their behavior. Instructions for performing such analyses are contained in the seismic design codes in which formulas specify the magnitude and vertical distribution of seismic loads as a function of straightforward parameters that are easy to evaluate by hand or with the aid of a computer.

The mode-superposition technique is capable of response prediction in structures with complicated deformation patterns, where the total response contains significant contributions from several vibration modes. Mode-superposition analysis should be used for structures with complex plan geometry or discontinuities in the distribution of mass and stiffness over the building's height.

In the mode-superposition technique, dynamic properties of the structure are determined from a mathematical model. Practical application of this approach requires the use of a computer and a dynamic analysis program, such as STRUDL/DYNAL, available on the McAuto timesharing service, or GTSTRUDL, available through Control Data Corporation.

Seismic Analysis by Computer [7], from the SEAOC Committee on Electronic Computation, discusses many of the issues involved in formulating a dynamic analysis model. Seismic analysis of large or complicated structures requires experience, judgment, and a thorough background in the fundamentals of dynamics and design for earthquake loads.

Dynamics of Structures, a Primer, by A.K. Chopra [8], presents the basic concepts and knowledge needed to understand the response of structures subjected to ground shaking. For a formal, detailed, and comprehensive treatment of the subject, refer to Dynamics of Structures, by R.W. Clough and J. Penzien [9].

Two approaches to the specification of equivalent static lateral forces for seismic design of buildings are described below. Probably the most widely used in the United States is the approach of the Uniform Building Code (UBC) [10]. Currently under evaluation by the design profession is a somewhat more refined approach, proposed by the Applied Technology Council (ATC) [11].

2.6.1 Uniform Building Code (UBC) Approach

The UBC equivalent static analysis computes seismic base shear as a function of six parameters:

$$V = Z * I * K * C * S * W$$

where

V = Seismic base shear (kips)

Z = Seismic zone coefficient

I = Importance factor

K = Framing factor

C = Coefficient related to vibration period

S = Site-structure resonance coefficient

W = Weight of structure (kips)

The terms Z, C, and S can be thought of as quantifying the design seismic force amplitude considering, respectively, seismicity of the site, dynamic characteristics of the structure, and the potential for dynamic interaction between the structure and the underlying soil. C is computed as a function of the building's fundamental period, which is determined either from empirical formulas or by the dynamic analysis described above.

Note that the calculated numerical coefficient for site-structure resonance, S, varies between 1.0 and 1.5, depending on the ratio of the structure's fundamental period, T, to the characteristic site period, T_S. Unless the value of T_S has been established by properly substantiated geotechnical data, S = 1.5 must be used. Thus, a geotechnical investigation of the site could justify reductions of up to 50 percent in the design base shear.

The term I increases the design force for essential facilities, such as hospitals, that must be serviceable following an earthquake, or for large meeting halls where the primary occupancy is by more than 300 persons in one room. Implicit in this approach is the notion that increased safety can be achieved by increasing the design force amplitude. As will be seen subsequently, UBC and ATC provisions differ in this regard; rather than increasing the design loads, ATC imposes more stringent detailing requirements for essential facilities.

K increases or decreases the design force in consideration of the relative ductility inherent in the framing system and materials of construction.

As emphasized previously, the computed design load is smaller than the forces the structure would experience if it were to remain elastic during the "design" earthquake. By prescribing the use of ductile construction details and materials proportioned to withstand the prescribed static lateral force, the Uniform Building Code aims to ensure satisfactory inelastic behavior of the structure during a major earthquake. In many framed structures with uniform distributions of mass and stiffness, the fundamental vibration mode is characterized by a linear variation of lateral displacement with height above the base. For moderately tall buildings, the most significant seismic response occurs in this mode. Accordingly, the UBC approach distributes the computed lateral force, V, linearly over the height of the structure, except that the lateral force in the top story is increased to account for the "whiplash" effect of response in higher modes. Story shears and overturning moments for design are computed from the lateral forces using the equations of statics.

2.6.2 Applied Technology Council (ATC-3) Approach

In ATC-3's equivalent static analysis approach, seismic base shear is expressed as a function of seven parameters:

$$V = C_s * W$$

where

V = Seismic base shear for design (kips)
C_s = Seismic design coefficient
W = Total gravity load of building (kips), including partitions and permanent equipment, and operating contents

and

C_s = Function of (A_a or A_v, S, R, and T)
A_a = Coefficient representing effective peak acceleration
A_v = Coefficient representing effective peak velocity-related acceleration
S = Soil profile coefficient
R = Response modification factor
T = Fundamental period of the building (seconds)

The coefficients A_a and A_v are site dependent and are obtained from seismicity maps contained in the ATC-3 document. The base shear coefficient, C_s, depends on A_a and A_v and varies as a function of building period, T. T is computed from empirical formulas or determined from a dynamic analysis.

C_s quantifies the effects of seismicity, dynamic characteristics of the building, and properties of the underlying soil on expected base shear. For very stiff buildings, C_s is directly proportional to A_a. This reflects the fact that the base shear in a stiff building is proportional to ground acceleration. For flexible buildings, C_s is directly proportional to the product (A_v) (S), and inversely proportional to T raised to the two-thirds power. This expresses the fact that the base shear in a more flexible structure depends not only on the ground motions, but also on the dynamic amplification or attenuation of those motions in the building's response.

A major difference between the UBC and ATC-3 approaches concerns the manner in which credit is taken for the structure's inherent ductility when specifying lateral design forces. In the UBC, the basic design force magnitude is computed for a structure of "average" ductility, then scaled upward or downward by use of the K-factor for structures with either lesser or greater energy-dissipation capacity. No mention is made of the force magnitudes that would occur if the structure remained elastic or of the deformation magnitudes that will occur when the structure is damaged by an earthquake.

In the ATC-3 procedure, the magnitude of reduction between the forces due to fully elastic response and the forces to be used in design is presented explicitly, in the response modification factor, R. As can be seen in Table 2.1, structures with relatively little redundancy and ductility qualify for relatively small force reductions. For example, the recommended R for a partially reinforced masonry shear wall is 1.25. Structures with significant redundancy and ductility qualify for greater force reductions. For example, the recommended R for a specially detailed reinforced concrete moment frame is 7.

Similarly, the ATC-3 approach enables the designer to estimate the total inelastic displacement under earthquake loading. Elastic displacements under the design loads are scaled up by a drift coefficient, C_d, to estimate maximum displacements of the damaged structure.

Type of Structural System	Vertical Seismic Resisting System	Coefficients R[7]	C_d[8]
BEARING WALL SYSTEM: A structural system with bearing walls providing support for all, or major portions of, the vertical loads. Seismic force resistance is provided by shear walls or braced frames.	Light framed walls with shear panels	6½	4
	Shear walls		
	Reinforced concrete	4½	4
	Reinforced masonry	3½	3
	Braced frames	4	3½
	Unreinforced and partially reinforced masonry shear walls[6]	1¼	1¼
BUILDING FRAME SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.	Light framed walls with Shear panels	7	4½
	Shear walls		
	Reinforced concrete	5½	5
	Reinforced masonry	4½	4
	Braced frames	5	4½
	Unreinforced and partially reinforced masonry shear walls[6]	1½	1½
MOMENT RESISTING FRAME SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. Seismic force resistance is provided by Ordinary or Special Moment Frames capable of resisting the total prescribed forces.	Special moment frames		
	Steel[3]	8	5½
	Reinforced concrete[4]	7	6
	Ordinary moment frames		
	Steel[2]	4½	4
	Reinforced concrete[5]	2	2
DUAL SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. A Special Moment Frame shall be provided which shall be capable of resisting at least 25 percent of the prescribed seismic forces. The total seismic force resistance is provided by the combination of the Special Moment Frame and shear walls or braced frames in proportion to their relative rigidities.	Shear walls		
	Reinforced concrete	8	6½
	Reinforced masonry	6½	5½
	Wood sheathed shear panels	8	5
	Braced frames	6	5
INVERTED PENDULUM STRUCTURES. Structures where the framing resisting the total prescribed seismic forces acts essentially as isolated cantilevers and provides support for vertical load.	Special Moment Frames		
	Structural steel[3]	2½	2½
	Reinforced concrete[4]	2½	2½
	Ordinary Moment Frames		
	Structural steel[2]	1¼	1¼

[1]These values are based on best judgement and data available at time of writing and need to be reviewed periodically.
[2]As defined in Sec. 10.4.1.
[3]As defined in Sec. 10.6
[4]As defined in Sec. 11.7.
[5]As defined in Sec. 11.4.1.
[6]Unreinforced masonry is not permitted for portions of buildings assigned to Category B. Unreinforced or partially reinforced masonry is not permitted for buildings assigned to Categories C and D; see Chapter 12.
[7]Coefficient for use in Formula 4-2, 4-3, and 5-3.
[8]Coefficient for use in Formula 4-9.

TABLE 2.1 - ATC-3 RESPONSE MODIFICATION COEFFICIENTS (FROM REFERENCE 11)

The response modification factor, R, serves somewhat the same purpose as K in the UBC approach; it accounts for differences in the inherent redundancy and ductility of various framing systems and materials. The ATC-3 approach differs from the UBC approach in an important point of philosophy, however: while UBC intends to improve the safety of critical structures by increasing their design lateral force magnitude (I-factor), ATC-3 prescribes the same loads but requires more ductile reinforcement and connection details for critical structures.

In ATC-3, the distribution of lateral forces over the height of a structure is based on an empirical formula that links the distribution pattern to the building period. The distribution is linear (like UBC) for buildings with period of 0.5 second or less, and progresses toward a parabola (vertical at the base) for structures with fundamental period longer than 2.5 seconds. Design story shears are computed from the lateral forces by statics. However, because the design shears are maximum values resulting from response in several modes, and therefore are not fully in phase over the structure's height, code-specified overturning moments are reduced from the statical values by as much as 20 percent at the base of a 20-story building.

It should be noted that the ATC-3 equivalent static forces are "ultimate" quantities, while the UBC values are "working" quantities that must be factored up for use in ultimate-strength design formulas. Detailing guidelines and strength reduction "ϕ-factors," as well as the lateral force magnitudes, differ between UBC and ATC-3. It would be misleading, therefore, to draw conclusions regarding the relative conservatism of the two approaches based on lateral force magnitudes alone.

Because the ATC-3 recommendations have not yet been incorporated by any building code, the UBC approach is used to specify design strength in the examples presented here. However, ATC criteria are used in the examples to predict the forces which would occur in structures with infinite elastic strength. This "elastic strength demand" is then used in a procedure by which the inelastic displacement magnitudes of precast concrete structures may be estimated.

2.7 CATEGORIES OF PRECAST CONCRETE CONSTRUCTION

Precast concrete buildings are of two types, "jointed" and "monolithic," with widely differing structural properties. In monolithic construction, precast elements are joined by well-reinforced connections possessing continuity of stiffness, strength, and ductility comparable to well-designed cast-in-place concrete. Jointed construction describes all means of connecting precast components in which the interelement boundaries behave as zones of significantly reduced stiffness, strength, or ductility under the ultimate design loads and deformations. Many buildings employing "wet connections" and the majority of buildings employing "dry connections" (welded or bolted inserts, dry-pack grout, etc.) belong in this category.

Usually, jointed construction involves fewer field operations and is less expensive. With its greater redundancy, monolithic construction seems to be employed most frequently where strong earthquakes are anticipated and increased resistance to ground shaking is needed.

2.8 DIFFERENCES IN THE SEISMIC BEHAVIOR OF JOINTED AND MONOLITHIC PRECAST STRUCTURES

Aside from possible foundation uplift effects, conventional cast-in-place structures tend to respond linearly to seismic excitation up to the damage threshold, where cracking of concrete and yielding of reinforcement begin. In contrast, due to effects of construction joinery, some precast concrete structures may exhibit nonlinear, "energy-rejecting" behavior at response amplitudes below their damage threshold.

Such nonlinear elastic properties have been reported for a concrete shear wall constructed of stacked precast panels with weak horizontal joints [12]. As dead load stresses are overcome by tensile flexural stresses, the horizontal joints begin to open up. The wall exhibits softening, nonlinear force-displacement behavior even though concrete and steel stresses are within the elastic range.

Thus, a precast structure can exhibit both a lower initial lateral stiffness and early onset of nonlinear behavior with stresses below the elastic limit. These effects can result in lower base shear and reduced seismic energy input, thereby reducing the amount of energy available for producing damage.

On the other hand, if sufficient energy is input to the structure to open up the joints, connections between precast elements may yield. Thus, connectors may be called upon to function as localized sites for energy dissipation through cyclic plastic deformation. Because the connectors are weaker than the joined elements and yielding is confined to relatively small volumes of material, however, the energy-dissipating capacity of a typical jointed precast assemblage is significantly lower than that of a monolithic structure detailed as conventional cast-in-place concrete.

To summarize, jointed and monolithic structures differ in two regards. On one hand, the nonlinear stiffness properties of jointed construction can result in reduced seismic energy input. On the other hand, reduced capacity for energy dissipation means jointed structures tend to be less rugged than their conventionally detailed monolithic counterparts.

2.9 TOWARD RATIONAL DESIGN OF SEISMIC-RESISTANT PRECAST CONCRETE STRUCTURES

Seismicity maps of the United States, such as Fig. 2.9, show a likelihood of major earthquake damage along much of the west coast, on isolated stretches of the east coast, and within small regions of the east and west central states. Over the greatest geographic area, however, the expected intensity of seismic damage is moderate or less. Because this environment has provided the largest market for precast concrete buildings, jointed construction is considerably more common than monolithic.

While strength and ductility requirements for cast-in-place concrete can be employed in the design of monolithic precast structures, current building code provisions are virtually silent on jointed construction. The economic and functional success of a jointed structure depends to a great degree on discrete connections. Designing connections that are easily fabricated, speedily erected, stable, strong, and ductile is a demanding task.

Research is needed to establish seismic strength requirements consistent with the available ductility of jointed precast structures. For the interested reader, Mueller [13] presents a detailed review of analytical and

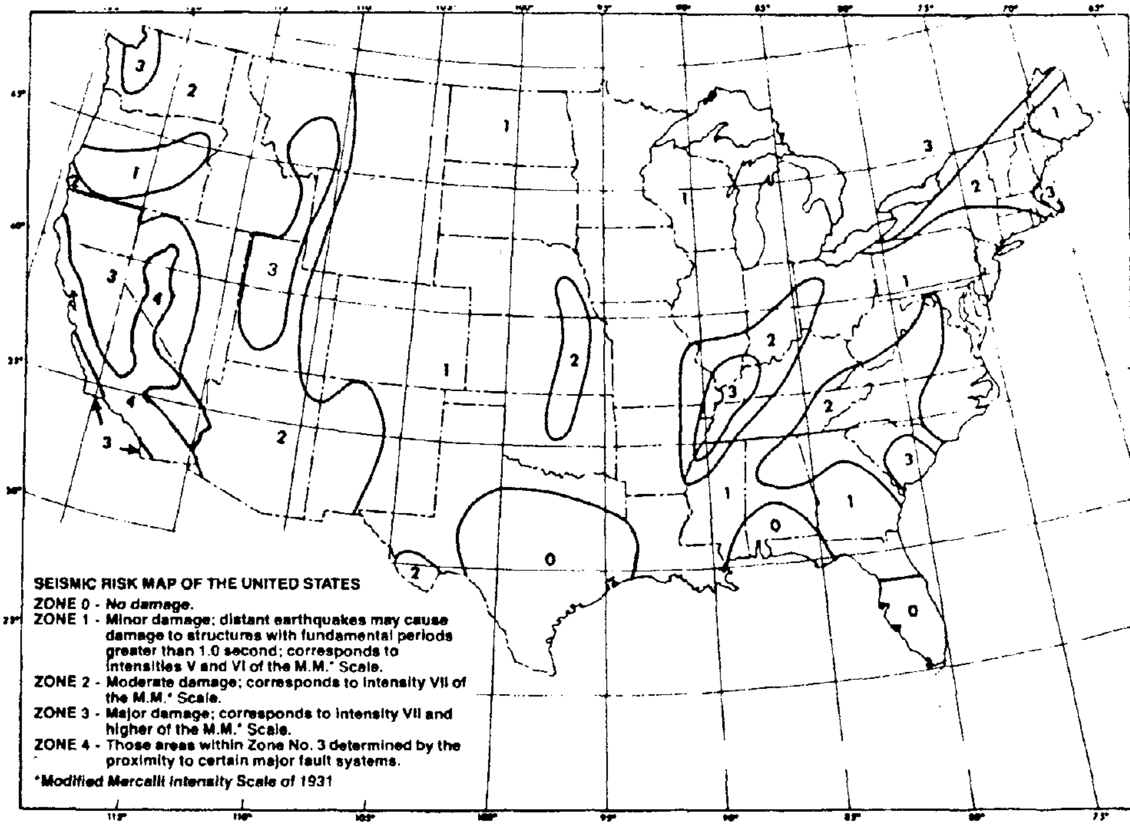


FIGURE 2.9 – SEISMIC ZONE MAP OF THE UNITED STATES (FROM REFERENCE 10)

experimental research into the behavior of jointed precast walls, with reference to code provisions, design options, and the role of connections.

In this report, the seismic behavior of jointed structures has been contrasted with that of monolithic construction. Is the apparent disadvantage of reduced energy-dissipating capacity offset by the apparent advantage of reduced seismic energy input? This question challenges the validity of traditional design procedures which proportion jointed construction according to lateral force specifications for monolithic concrete without ensuring a corresponding capacity for inelastic energy dissipation.

Because there is no alternative at present, lateral force provisions of existing codes intended for monolithic concrete will continue to be employed in the design of jointed structures. It seems essential therefore that extended design procedures, accounting explicitly for inelastic deformation demands in the detailing of discrete connectors, be adopted for jointed precast construction. One such procedure is described in the following chapter.

CHAPTER 3
A RATIONAL METHODOLOGY FOR THE DERIVATION OF CONNECTOR PERFORMANCE REQUIREMENTS

3.1 INTRODUCTION

A comprehensive seismic design procedure is proposed in this chapter. Guidance is provided in the selection of framing schemes which offer favorable patterns of inelastic deformation. A simple concept involving internal strain energy is introduced for estimating the maximum inelastic displacement of a structure during an earthquake. Kinematic principles are then used to transform this "global" displacement into deformations of individual joints and connectors. Design strengths are established by accepted United States building code criteria.

The proposed methodology is suitable for regular structures of moderate height, and is illustrated in later chapters by application with structures typifying precast construction in UBC Seismic Zones 1 and 2. This selection of examples in no way reflects on the possibility of properly designed jointed construction performing satisfactorily in UBC Seismic Zones 3 and 4. Zones 1 and 2 are considered simply because they constitute the arena of greatest current experience. A discussion at the end of this chapter compares strength, inelastic deformation, cyclic load reversal, and plastic energy dissipation requirements of connections for identical precast structures located in regions of moderate and major seismicity, thus indicating the nature of changes that would be required if connection details originally intended for use in moderate seismic zones were to be adapted for use in zones of greater earthquake hazard.

Thus, while the methodology is not completely general, it can nevertheless be applied in many practical cases beyond the seven-story frame/shear wall structure and 17-story bearing wall structure presented in Chapters 4 and 5.

3.2 DETAILED TECHNICAL OVERVIEW

Taken together, the items in the block diagram, Fig. 3.1, form a comprehensive design methodology for earthquake-resistant, jointed precast concrete structures. Major components of the methodology are described in detail in the following pages and are summarized in the "Designer's Checklist" presented at the end of the chapter.

3.2.1 Definition of Structural System

Building configuration usually is based on functional requirements and site considerations over which the structural engineer has little choice. Within these constraints, however, the engineer does have some freedom in locating, orienting, and proportioning the structural elements.

At a minimum, the engineer needs to define load paths for forces due to gravity, wind, and earthquake. Final selection of a framing scheme will involve considerations of constructability, serviceability, volume change accommodation and, of particular interest here, intended behavior under seismic loads. It is important that seismic considerations be formally addressed early in the design process and that a list of design objectives and an array of possible solutions be prepared and systematically evaluated.

Engineers approaching earthquake-resistant design for the first time may find the clear, nonmathematical presentation of seismic behavior and design issues in Building Configuration and Seismic Design [14], by Arnold and Reitherman, informative and useful.

3.2.2 Concept for Lateral Load Resistance

Selection of a framing scheme with favorable inelastic properties is an important step in controlling the patterns of inelastic deformation and the magnitudes of inelastic strain which will occur during a damaging earthquake.

Specifically, the lateral force resisting system should be proportioned so as to form a suitable single-degree-of-freedom yield mechanism (with pre-determined plastic hinge locations) when loaded beyond its elastic strength.

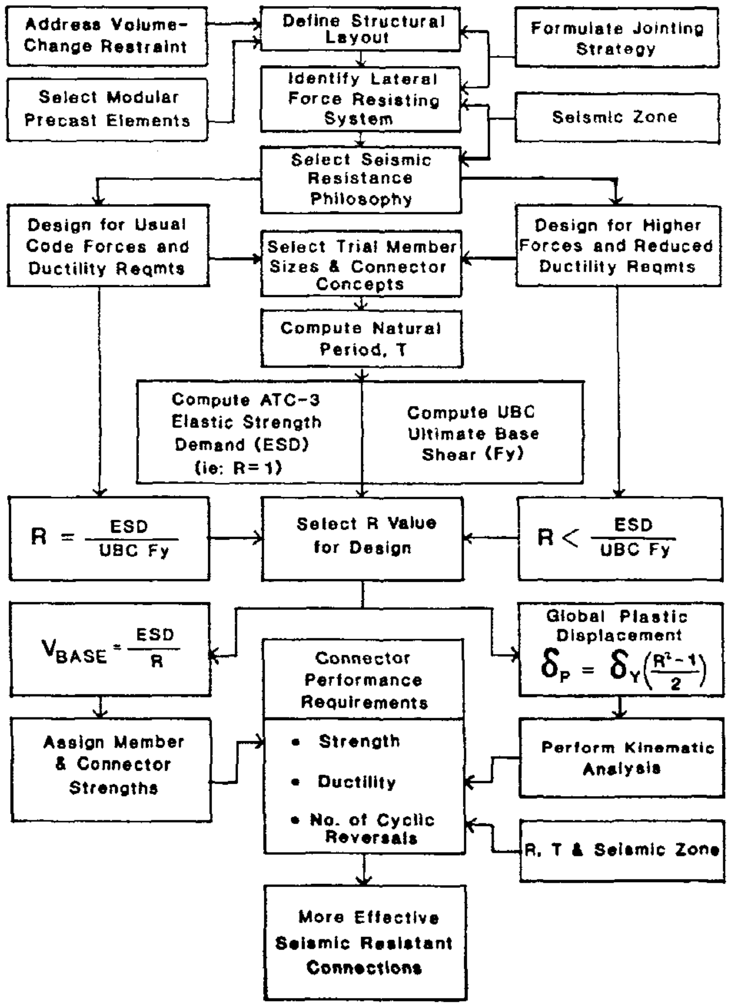


FIGURE 3.1 - DESIGN METHODOLOGY FOR SEISMIC RESISTANT PRECAST CONCRETE BUILDINGS

Examples of suitable mechanisms are cantilever walls which yield in flexure near the foundation and coupled shear walls detailed for ductility.

When properly designed systems of such configuration are used for lateral load resistance in low- to medium-rise structures alone or in combination with frames to carry the gravity loads, inelastic roof-level displacements of the magnitude produced by moderate earthquakes can be accommodated without a loss of stability.

The important feature of these framing arrangements is the great stiffness of the vertical elements when compared to the foundations and horizontal elements. This relationship can improve the predictability of seismic response in medium-rise buildings, because it tends to increase the frequencies of the higher vibration modes beyond the range of significant earthquake excitation, thereby reducing their contribution to the structure's overall seismic response. Further, due to foundation flexibility, the shape of the first mode is well approximated as a rigid-body rotation of the vertical elements about their respective bases, producing a linear variation of lateral displacement with height.

For moderately tall structures, this means that the predominant seismic response will occur in the first mode and the shape of the first mode will agree well with the static deflected shape produced by the code-specified equivalent static earthquake loads. Accordingly, force distributions predicted by the static analysis will closely approximate the true dynamic responses, and strengths proportioned according to the static results will produce the desired single-degree-of-freedom yield mechanism.

Thus, the design approach strives for simplicity and predictability of dynamic behavior through appropriate selection of relative stiffnesses and strengths of the vertical and horizontal structural elements.

3.2.3 Elastic Analyses for Undamaged Condition

As discussed in Chapter 2, the magnitude of seismic forces experienced by a structure is a function of its mass, stiffness, damping, and strength. The distribution of these forces in elevation and in plan among the structure's lateral resisting elements depends on the patterns of deformation which develop as the structure vibrates and on the relative stiffnesses of the frames and walls that resist the lateral loads.

Thus, following conceptual formulation and preliminary sizing of one or more possible lateral resisting systems, there is need for two types of analysis. The first is to determine the vibrational characteristics of the structure which govern the magnitude of seismic forces and how they are distributed over the structure's height. The second is to determine the distribution of seismic loads among the lateral resisting elements.

These analyses can be performed by hand calculations or by computer as dictated by the complexity of the structure and the resources available. The increasing variety and decreasing cost of computer hardware and software have made this the preferred option in many design offices, large and small.

In planning a computer model, bear in mind that the results must be interpreted before they can be applied; the real work begins after the model has been set up and run. Some computer programs contain graphical post-processing features which are extremely helpful in reviewing force and deformation results. The most important labor-saving concept, however, is to keep the model as simple as possible. Increasing a model's complexity almost always increases the cost of input preparation, program execution, and output interpretation. In addition, the uncertainties of loading and foundation stiffness and the numerous assumptions involved in the calculation of structural properties mean that increased complexity is not guaranteed to increase the realism of the analysis or the quality of the insight it provides on the structure's actual behavior.

It is recommended that member properties of the analytical model be derived from uncracked, gross concrete areas. Effects of variations in connector and foundation flexibilities on overall behavior of the structure should be studied before final values of modeling parameters are selected. In many cases, it will be efficient to use the same model for period determination and elastic load analysis, as was the case for the two examples treated here.

Both available strength and required capacity can vary directly with size of the resisting members. Thus, proportioning relative strengths and stiffnesses among lateral resisting elements often requires a few iterative cycles. The process can be speeded up if relative stiffness effects (governing load distribution among members) and absolute stiffness effects (governing total seismic load on the structure) are considered separately. A convenient approach is to work initially with an equivalent static load case of arbitrary "unit" magnitude. Typically, this would be a set of forces distributed over the structure's height in accordance with the applicable building code provisions and producing a 1000-kip base shear. Once relative stiffnesses have been adjusted to produce the desired load distribution among the resisting elements, giving preliminary sizes for all members of the lateral force resisting system, the structure's fundamental period can be determined.

Results of the unit load analysis are scaled up according to the code-specified base shear for the predicted fundamental period and are used in defining member and connection strength requirements. If the preliminary member sizes are inappropriate, changes are made and the process is repeated until strength requirements are met by available capacities. The unit-load lateral deflection is used subsequently in the procedure for estimating maximum inelastic displacement magnitude.

3.2.4 Specification of Seismic Loads

Two loading cases are considered in this approach: the usual load condition describing the structure's required ultimate capacity and an auxiliary load condition describing the maximum seismic forces the structure would experience if it had infinite strength, referred to here as the "elastic strength demand."

Neither the Uniform Building Code (UBC) nor ATC-3 includes design strength requirements for jointed precast construction. For the examples presented in Chapters 4 and 5, it was decided to use UBC strength criteria for monolithic concrete to define the required ultimate capacity. This seems to be the approach followed by most designers; the ATC-3 provisions have not yet been adopted by code bodies.

Loads defining the elastic strength demand are derived from ATC-3 formulas using an R-value of 1.0 (refer to Chapter 2). The ratio of elastic strength demand to ultimate capacity is used to compute the estimated inelastic displacement at roof level, as is described presently.

In both the UBC and ATC-3, seismic design loads depend on the structure's fundamental period. Empirical formulas for estimating vibration periods (as found in the UBC and ATC-3) should not be relied upon blindly. They depend only on external building dimensions and do not reflect the structural layout.

Accordingly, three-dimensional computer models were used to determine periods of the example structures. Parametric studies performed in the course of this work, however, revealed that predicted periods are sensitive to flexibilities of the foundation and the connectors between precast elements. Because these values can never be predicted with certainty, it is clear that the analytical approach also has its limitations.

Empirical formulas can be useful for assessing the reasonableness of period predictions from refined analyses in cases where reliable experimental data substantiating foundation stiffnesses and connector flexibilities incorporated in the models are not available. Still, in many practical situations, this aspect of the seismic design process must rely heavily on engineering judgment.

3.2.5 Assignment of Member Strengths

Consistent with the chosen mechanism of inelastic behavior, selected regions within the lateral force resisting system are intended to yield under application of the factored design loads. In contrast, horizontal elements which distribute forces to the lateral resisting system, such as floor and roof diaphragms, are intended to remain elastic at these force levels insofar as their action in restraining relative horizontal movements of the walls or frames is concerned.

Thus, the strength of a given member or connection must be assigned with the intention either of allowing or preventing inelastic action under code lateral forces. Particular features of a given structure will dictate the measures to be taken to ensure the desired relative strengths. Again, this is an issue for engineering judgment. Its practical application is illustrated by specific details of the examples.

3.2.6 Prediction of Global Inelastic Displacement Magnitude

It must be emphasized that the methodology presented here for predicting inelastic deformation magnitudes is valid only for structures in which the significant seismic response occurs in the first lateral vibration mode. As illustrated by the examples in Chapters 4 and 5, only regular structures with moderate height and with lateral resisting systems proportioned as described above are likely to satisfy this requirement.

For structures with nonuniform mass and stiffness distributions, or tall structures with frame action providing the principal means of lateral resistance, significant response in other vibration modes must be anticipated. For such cases, the methodology presented here is overly simplified and a more sophisticated approach must be taken. If there is any doubt as to the suitability of this procedure for the structure at hand, a dynamic analysis should be performed to determine its mode shapes and vibration frequencies. Design should proceed by this approach only if the results rule out the possibility of significant response in torsion or higher modes of lateral vibration.

In Chapter 2, seismic response of buildings was described in terms of energy. For a simple linear system (such as a swing undergoing small displacements or a spring-mass oscillator) vibrating freely, it was explained that a direct relationship exists between the maximum displacement amplitude and the amount of stored energy. Similarly, for a building with infinite strength, the lateral displacement at roof level produced by the elastic demand forces (i.e., corresponding to the elastic strength demand) is related to the maximum internal energy of the building during the strongest ground shaking anticipated at the site.

In Fig. 3.2, the lateral force-displacement relationship for a hypothetical building with infinite strength is plotted. As shown, displacement varies linearly with applied load. When the lateral force equals the elastic strength demand, the shaded triangular area under the curve represents the maximum elastic strain energy which will be absorbed by the building during the code-specified earthquake.

Because the lateral force resisting system has been designed to form a single-degree-of-freedom (SDOF) yield mechanism when subjected to a

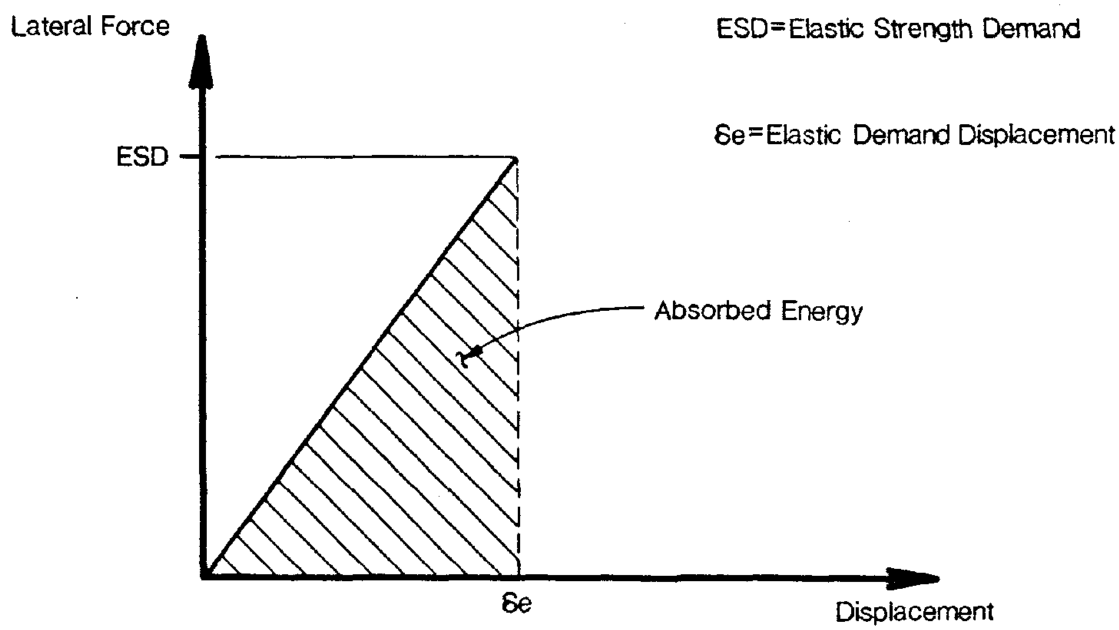


FIGURE 3.2 – LATERAL FORCE – DISPLACEMENT RELATIONSHIP FOR STRUCTURE WITH INFINITE STRENGTH

damaging earthquake, the seismic response theories of elastoplastic SDOF systems developed by Newmark and Hall and others may be applied.

For structures with fundamental period in the range 1/8 to 1/2 second, Newmark and Hall [15] have shown that peak internal strain energies of elastic and elastoplastic SDOF systems subjected to seismic excitation are approximately equal. This fact can be used to estimate the maximum inelastic displacement of an elastoplastic SDOF system, provided that its yield strength, yield displacement, and elastic strength demand are known.

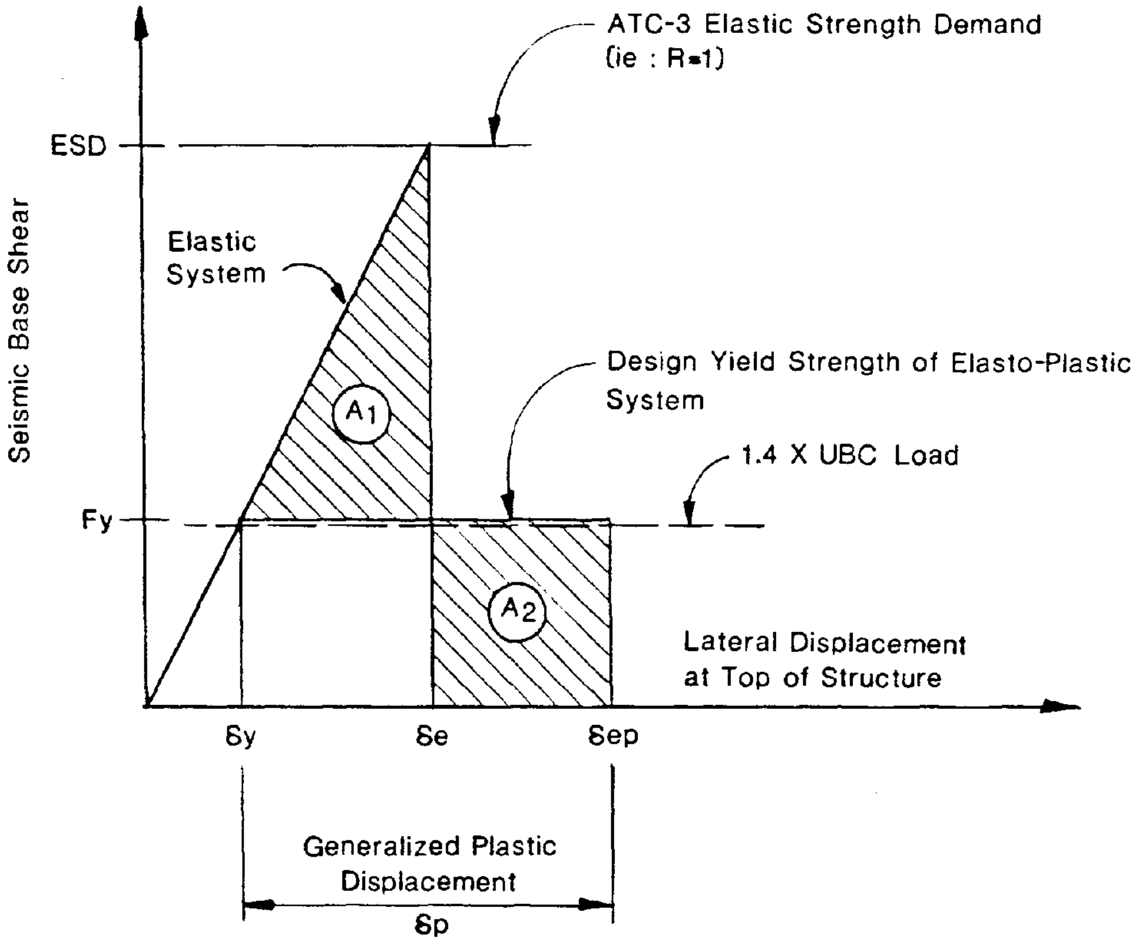
To see how this is done, refer to Fig. 3.3, which shows force-displacement relationships for elastic and elastoplastic SDOF systems with lateral force plotted on the vertical axis and lateral displacement at roof level plotted on the horizontal axis. For either the elastic or elastoplastic system, the area under the force-displacement curve bordered on the right-hand edge by a vertical line through a given displacement value represents the strain energy absorbed by the system as it is forced to displace laterally through the given distance.

In the figure, the elastic system is shown under action of a lateral force equal to the elastic strength demand. The elastoplastic system is shown displaced by an amount δep, the maximum inelastic displacement this system would experience during the design earthquake, assuming its strength is less than the elastic strength demand. By the "equal energy" concept of Newmark and Hall, δep is that value which gives a trapezoidal area equal to the triangular area of the elastic system displaced to δe.

δep consists of an elastic component (the displacement which occurs before yield) and a plastic component (which occurs after yield). The plastic displacement increment, δp, which is needed for input to the kinematic analysis described below, is the difference between δep and the yield displacement, δy. This quantity is easily determined, since δy is available from the elastic analysis results.

Another response quantity often used in describing the performance of elastoplastic SDOF systems is described in Fig. 3.3. This is the global ductility factor, μ, which is defined as the ratio of δep to δy.

For structures with fundamental period greater than 1/2 second, Newmark and Hall have concluded that the maximum inelastic displacement, δep, equals δe. Other researchers [16] have shown, however, that the



<u>Response Modification Factor</u>

$$R = \frac{ESD}{Fy}$$

<u>Ductility Factor</u>

$$\mu = \frac{\delta ep}{\delta y}$$

$$A_1 = A_2$$

$$\boxed{\mu = \frac{R^2 + 1}{2} \qquad \delta p = \delta y \left(\frac{R^2 - 1}{2}\right)}$$

FIGURE 3.3 - EQUAL-ENERGY PRINCIPLE FOR ESTIMATING MAXIMUM SEISMIC DISPLACEMENTS OF AN ELASTO-PLASTIC SYSTEM

maximum inelastic displacement is sensitive to the ground motion record; while the equal-displacement assumption is conservative on the average, numerically computed inelastic displacement maxima for individual accelerograms representing the same statistical "family" of earthquake ground motions differ widely and often exceed the elastic demand displacement, δe, by a significant margin. Accordingly, the "equal energy" hypothesis is here proposed for application with all structures of period greater than 1/8 second; while it is more conservative, the predicted displacements are not so large as to be unmanageable for the types of structures covered by this design methodology.

Extremely stiff structures, with period less than 1/8 second, require special consideration. On one hand, capacities reduced below the elastic strength demand can result in ductility requirements which greatly exceed values predicted by the equal energy approach and which may be difficult to achieve in practice. On the other hand, some building code loading provisions can significantly overestimate the elastic strength demand for structures in this period range. Such cases are not covered by the methodology presented here.

Fig. 3.3 also demonstrates the inverse relationship which exists between yield strength and required ductility. In theory, when the strength of an elastoplastic system equals the elastic demand (as given by ATC-3 with an R-value of 1, for example), it will not yield during the design earthquake and inelastic displacements will not occur. As described in Chapter 2, however, building codes prescribe design forces considerably smaller than the elastic strength demand. Reducing the strength toward typical design values (as specified by the Uniform Building Code, for example) increases the magnitude of inelastic displacements.

Thus, the proposed methodology gives designers a degree of control over seismic ductility demands. For some precast structures in the zones of low seismicity, it could be economically attractive to design for higher loads than required by the code in return for reduced ductility requirements. It must be recognized, however, that the predicted elastic strength demand is only an estimate; the actual elastic forces experienced during a strong earthquake could be significantly higher. Accordingly, any structure subjected to seismic ground shaking must possess a measure of ductility, even if designed for the full elastic demand loads predicted by ATC-3. On the other hand, reduced design forces specified by the building codes should be regarded as defining the limits of minimum allowable yield strength or, equivalently, maximum allowable ductility demand.

Fig. 3.3 completely describes the proposed method of estimating inelastic displacements at the structure's roof level. This technique has been applied in the examples of Chapters 4 and 5.

3.2.7 Kinematic Analysis of Post-Yield Behavior

Using the predicted inelastic displacement at the top of the structure, a kinematic analysis is performed to determine the corresponding deformations at individual joints. By the design methodology proposed above, the lateral force resisting system has been proportioned with stiff, strong vertical elements. Consequently, displacement patterns of the inelastic mechanism which forms during a damaging earthquake are simple to describe mathematically.

In this kinematic analysis, elastic deformations are insignificant when compared with the structure's inelastic movements. Thus, motions of the vertical elements (such as walls, elevator cores, or similar stiff members) can be approximated as rigid-body rotations about their respective foundations, with concentrated hinge points or hinge lines where they intersect the floor and roof systems.

Assumed locations of base rotation axes depend on the dead load and on the aspect ratio of a given vertical element. For a heavily loaded, slender elevator core, rotations may be assumed to occur about the geometric centroid of the foundation. On the other hand, uplift should be anticipated for a lightly loaded wall of considerable lateral dimension and rotations should be assumed to occur about a point near the compression end of the foundation. Note that the rotation point will migrate from one end of the foundation to the other as the structure sways back and forth.

Working with a scaled sketch of the displaced inelastic mechanism, as shown in Fig. 3.4, simple trigonometric relationships can be applied to calculate hinge rotations and vertical and lateral joint displacements which correspond to a unit lateral movement at the top of the structure. Unit kinematic analyses of this type can be used to compare hinge rotation and joint displacement magnitudes for alternative framing schemes.

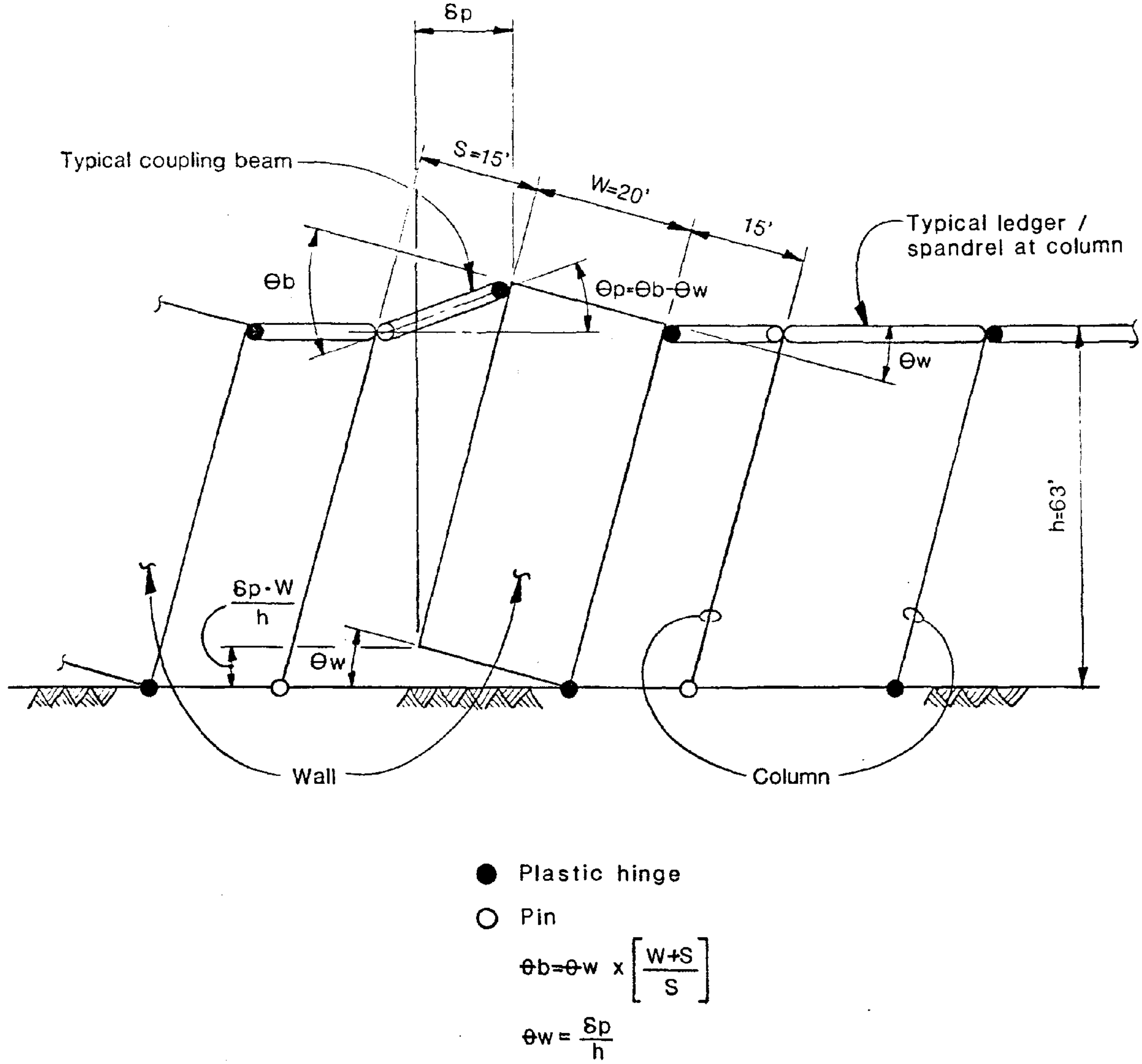


FIGURE 3.4 - KINEMATIC MODEL OF FRAME-SHEAR WALL SYSTEM SUBJECTED TO PLASTIC DISPLACEMENT, δp

To estimate relative displacements and hinge rotations due to a damaging earthquake, results of a unit kinematic analysis are scaled up to agree with the roof-level inelastic displacement predicted by the "equal energy" principle already discussed. Note that this analysis deals with plastic displacements which are additive with elastic deformations occurring before the structure yields.

3.2.8 Number of Cyclic Load Reversals to be Sustained

To reflect the oscillatory nature of the structure's dynamic response, it is necessary to prescribe a certain number of reversed cycles of loading and deformation to be sustained by the connections. The number of response cycles varies directly with the intensity and duration of ground shaking. In addition, studies by others [16] have shown that the number of displacement reversals in the response of an elastoplastic SDOF system depends on specific features of the ground motion. In general, however, the number of inelastic cycles varies inversely with the fundamental structural period and directly with the response modification factor, R. For severe ground motions recorded on firm soils at moderate epicentral distances, the influence of these parameters on the expected number of reversed loading cycles is characterized by the data in Table 3.1, which were obtained from [16].

To account approximately for the variation of ground shaking intensity and duration in areas of different seismicity, the data of Table 3.1 have been scaled using the UBC seismic zone coefficient, Z. Assuming that the results from [16] represent Zone 3 conditions, values for Zones 1 through 4 have been projected and are presented in Table 3.2. Based on the information in Table 3.2, it is proposed that connectors be designed to sustain a number of reversed loading cycles consistent with seismicity of the site, the structural period, and the chosen ratio of elastic strength demand to design yield strength, R.

3.2.9 Specification of Connector Performance Criteria

Having selected a lateral force resisting system with suitable elastic and inelastic properties, and having determined the magnitude of internal

TABLE 3.1. INFLUENCE OF PERIOD AND RESPONSE MODIFICATION FACTOR ON NUMBER OF REVERSED LOADING CYCLES

Period (seconds)	Response Modification Factor, R: 2	4
0.5	4-7	15-18
1.0	3-6	9-10

TABLE 3.2. EXTRAPOLATED VALUES FOR ALL ZONES INFLUENCE OF PERIOD AND RESPONSE MODIFICATION FACTOR ON NUMBER OF REVERSED LOADING CYCLES

	Period (seconds)	Response Modification Factor, R: 2	4
Zone 4 (Z = 1)	0.5	6-9	20-24
	1.0	4-8	12-13
Zone 3 (Z = 3/4)	0.5	4-7	15-18
	1.0	3-6	9-10
Zone 2 (Z = 3/8)	0.5	3-4	8- 9
	1.0	2-3	5- 6
Zone 1 (Z = 3/16)	0.5	1-2	4- 5
	1.0	1-2	2- 3

joint and member forces and the magnitude of joint deformations which must be sutained during a damaging earthquake, it is possible to describe quantitatively the strengths and deformational capacities required of connections between the structure's precast elements.

These strength, deformation, and load reversal requirements constitute a set of rational seismic performance criteria. Obtained through analysis and the application of engineering judgment, they provide the building engineer with a comprehensive view of the conditions to be sustained during the largest earthquake anticipated during the structure's lifetime. This in turn enables a more effective detailing of connections for jointed precast construction than is possible when strength requirements alone are considered.

More important in the long term, widespread adherence to a design approach that gives insight into connector deformational requirements would facilitate the use of physical testing to assess the adequacy of connection details intended for a particular application. Laboratory results and performance observations of precast structures during earthquakes could then be used to refine detailing concepts and analytical predictive techniques, thus advancing the state of the art of earthquake-resistant design for jointed precast concrete buildings.

3.2.10 Beyond Traditional Bounds

Precasters express a growing interest in expanding their market into more seismically active regions, and seismic risk assessments for regions traditionally viewed as earthquake-free are being revised upward as seismologists continue to compile and evaluate geological and historical data. Experimental and analytical investigations are required to guide the development of improved framing systems and earthquake-resistant connections. While the pursuit of these objectives will take time, the rational methodology presented here can give some indication of the changes that will be required in adapting traditional details of jointed precast construction for service in regions of higher seismicity.

Consider a jointed precast building with fundamental period of 1 second sited in Las Vegas (UBC Zone 2). What changes in interelement connections would be required to adapt the design to the stronger ground

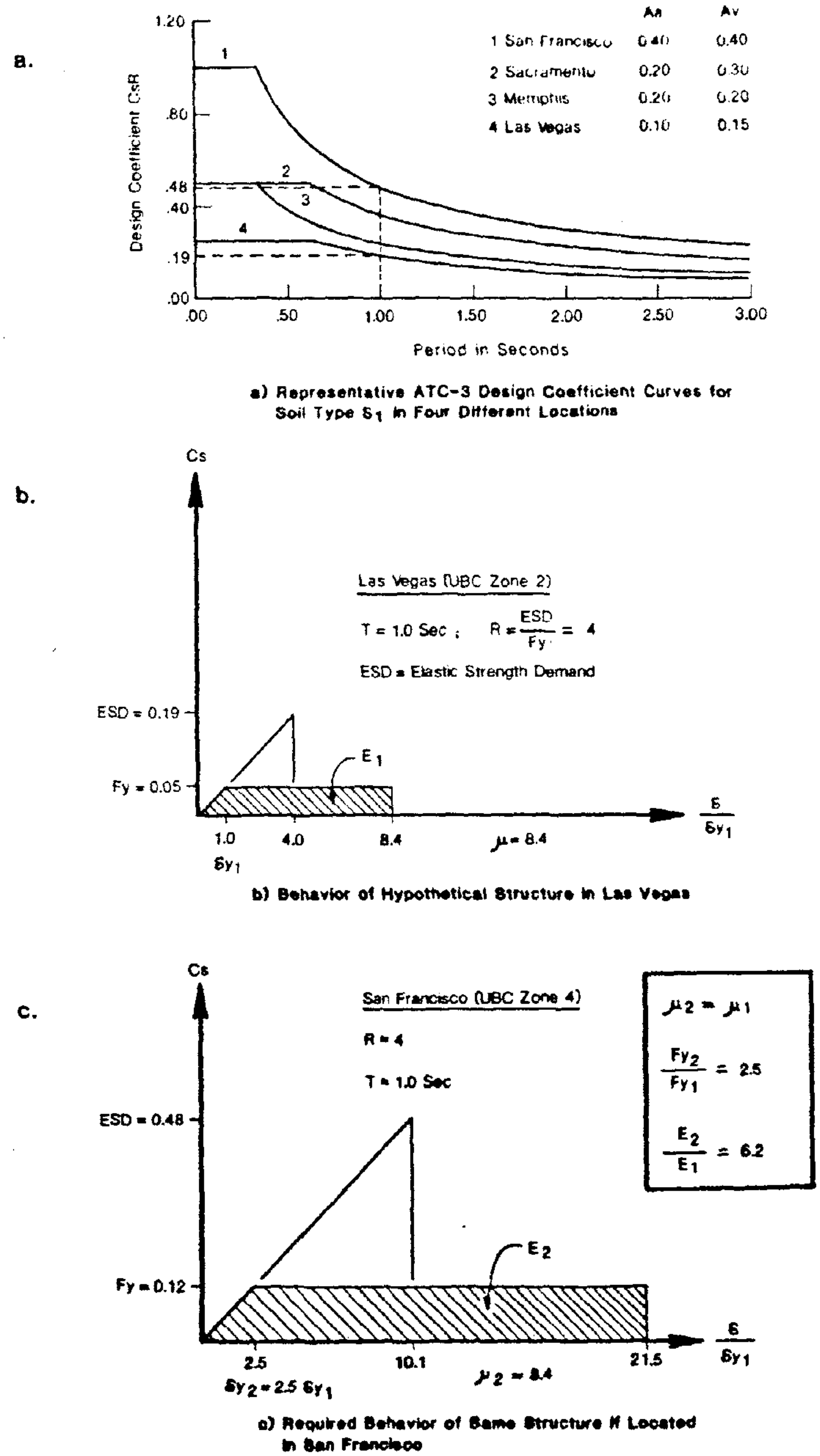


FIGURE 3.5 – COMPARISON OF PLASTIC ENERGY DISSIPATION REQUIREMENTS IN REGIONS OF MODERATE AND HIGH SEISMICITY

shaking anticipated in San Francisco (UBC Zone 4)? Reference to Fig. 3.5.a shows that seismic base shear increases by the ratio 0.48/0.19, indicating that connector strengths would have to increase by a factor of 2.5.

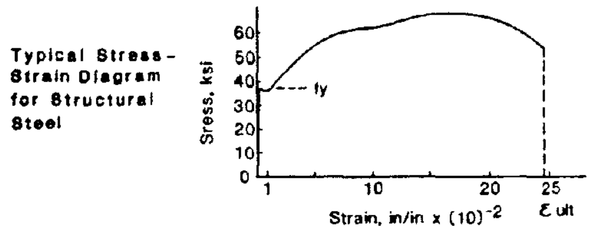
But increased strength is not the only requirement. In Fig. 3.5.b, the equal energy principle is applied to estimate the inelastic deformation magnitude of the Zone 2 building, assuming a response modification factor, R, of 4.0. Adapting to Zone 4 conditions, as shown in Fig. 3.5.c, requires that inelastic displacements increase by the same factor as the base shear. Further, comparing shaded areas of Figs. 3.5.b and c, it is seen that the required capacity for inelastic energy dissipation increases by the square of the base shear ratio. Thus, the plastic energy dissipating capacity of the San Francisco structure must exceed that of the Las Vegas structure by a factor greater than 6.

If the connectors are made of steel with stress/strain properties as shown in Fig. 3.6.a, Fig. 3.6.b shows that, for a given strain magnitude, the energy absorption capacity varies directly with the volume of material undergoing plastic deformation. Thus, the volume of connector material mobilized for energy dissipation must increase with the square of the base shear ratio, as shown in Fig. 3.6.c.

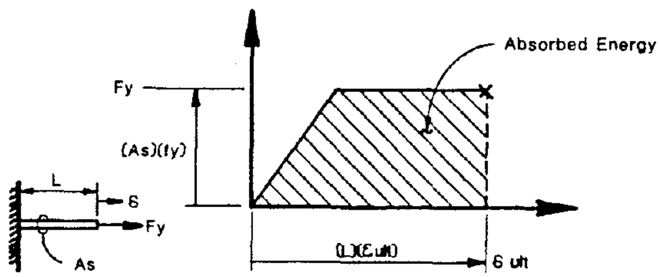
If this analysis is repeated for a structure with 0.5-second period, Fig. 3.5.a shows that the base shear ratio is about 3. This means that earthquake-resisting connectors in the San Francisco structure would need three times the strength and would have to mobilize nine times the material volume as connectors in the Las Vegas structure.

Further, the data in Table 3.2 indicate that while the building with a 1-second period will undergo 5 or 6 inelastic loading cycles in Las Vegas, it will be subjected to 12 or 13 cycles in San Francisco. The stiffer structure will experience 8 or 9 inelastic cycles in Las Vegas and 20 to 24 cycles in San Francisco. Thus, the potential for low-cycle fatigue failure of connections is significantly higher for structures in the higher seismic zones.

Reflecting framing schemes and jointing strategies typical of precast construction in the United States, the examples of Chapters 4 and 5 characterize the forces and deformations anticipated for jointed precast buildings subjected to moderate earthquakes. While consideration of jointed structures in regions of major seismic hazard is beyond the scope of this study, the a.



b.



Amount of energy absorbed depends on (As)(L) = <u>Volume of material</u>

c. Let $\frac{V2}{V1} \equiv S$ ("Base Shear Ratio")

Then, as shown in Fig. 3.5

$$\frac{Fy2}{Fy1} = S \quad \text{and} \quad \frac{\delta 2}{\delta 1} = S$$

$$\Downarrow \qquad\qquad \Downarrow$$

$$\frac{As2}{As1} = S \qquad\qquad \frac{L2}{L1} = S$$

$$\Downarrow$$

$$\boxed{\text{So} \quad \frac{\text{Vol. 2}}{\text{Vol. 1}} = \frac{As2}{As1}\,\frac{L2}{L1} = S^2}$$

FIGURE 3.6 – REQUIRED VOLUME OF MATERIAL MOBILIZED FOR PLASTIC ENERGY DISSIPATION AS A FUNCTION OF SEISMIC BASE SHEAR

rational methodology presented here gives some insight on the changes required to adapt the example buildings for service in higher seismic zones.

RATIONAL PREDICTION OF SEISMIC PERFORMANCE REQUIREMENTS FOR CONNECTORS IN JOINTED PRECAST CONCRETE STRUCTURES

-- DESIGNER'S CHECKLIST --

1. Define Structural System

   o Identify gravity load paths.

   o Identify lateral load paths.

   o Avoid problems of volume change due to creep and shrinkage.

   o Describe intended structural behavior under largest anticipated earthquake.

2. Evolve Concept for Lateral Load Resistance

   o Design the lateral force resisting system to form a suitable single-degree-of-freedom yield mechanism with predetermined plastic hinge locations when loaded beyond the elastic limit. Two characteristics of a "suitable" mechanism are (a) that the structure remains stable under the largest inelastic deformation produced by anticipated ground shaking at the site, and (b) that the mechanism mobilizes inelastic action in regions of the structure which can readily be detailed for strength and ductility.

   o Plan interface between vertical elements of the lateral load resisting system (e.g., walls or frames) and horizontal elements of the lateral load distributing system (e.g., roof and floor diaphragms).

   o Select joints or interior regions of precast members within which inelastic actions are to concentrate.

   o Identify joints or regions within precast members intended to remain elastic during a damaging earthquake.

3. Perform "Unit" Elastic Analysis for Undamaged Condition

   o Compute member properties based on uncracked section.

   o Include foundation flexibilities.

   o Distribute lateral loads over height of structure per UBC formulas, 1000-kip base shear.

   o Note distribution of forces among lateral resisting elements.

- o Note lateral deflection at top of structure when unit loads are applied.

- o Does structure behave as desired? If not, modify and repeat unit analysis.

4. Specify Seismic Loads

- o Determine fundamental period, using same model as for unit elastic analysis.

- o Obtain design base shear using building code provisions for reinforced concrete.

- o Obtain "elastic strength demand" using ATC-3 procedure.

5. Assign Member Strengths

- o Scale unit elastic analysis results to correspond with design base shear.

- o Assign strength requirements to connections or regions within elements of lateral force resisting system intended to yield under application of factored base shear.

- o Assign strength requirements to horizontal distributing elements (such as floor and roof diaphragms) so as to ensure elastic behavior under in-plane loads due to the factored ultimate base shear.

6. Predict Global Inelastic Displacement Magnitude

The following approach is justified because the framing concept for the lateral force resisting system was conceived to develop a single-degree-of-freedom mechanism during inelastic response.

- o Apply the "equal energy" principle of Newmark and Hall, using a strength reduction factor, R, equal to the ATC-3 elastic strength demand divided by the factored design base shear, and an elastic displacement demand, δe, equal to the roof-level displacement under a base shear corresponding to the elastic strength demand.

- o Use predicted inelastic displacement at roof level as input for kinematic analysis.

7. Perform Kinematic Analysis for Post-Yield Condition

CHAPTER 4
EXAMPLE 1 -- FRAME/SHEAR WALL PARKING STRUCTURE

4.1 DESCRIPTION OF BASIC STRUCTURAL SYSTEM

The parking garage treated in this example is a ramp-type structure with seven levels above grade. Based on the Metro-Space parking system illustrated in Fig. 4.1, it was included at the suggestion of the PCI Technical Input Group.

As shown in Fig. 4.2, plan dimensions are 200 feet in the east/west direction by 250 feet in the north/south direction; the height is 63 feet. Across the east/west dimension are four 50-foot bays spanned by precast double-T's. The double-T's are supported on precast ledger beams of 25-foot simple span. Thus, columns are spaced on 25-foot centers along the north/south column lines, which are 50 feet apart.

The ledger beams also serve as guard rails at the perimeter, along the ramp sides, and around openings in the decks. To enable use of the same 25-foot ledger spans in all locations, columns along the east/west perimeter walls are located on 25-foot centers.

4.2 CONCEPT FOR LATERAL LOAD RESISTANCE

As laid out by Metro-Space, the structure is intended for UBC Seismic Zone 0; wind loads are resisted by combined action of the elevator towers, the ramps, and the frames formed by columns, ledgers, and spandrels. In this example, modifications to the basic structure are developed which give satisfactory seismic performance in Zones 1 and 2.

The principal alteration which has been made is the addition of ductile "outrigger walls" in the east/west perimeter frames and at the outboard edges of the ramps in the north/south direction. As shown in Fig. 4.3, an outrigger wall is formed by stacking precast "infill panels" and

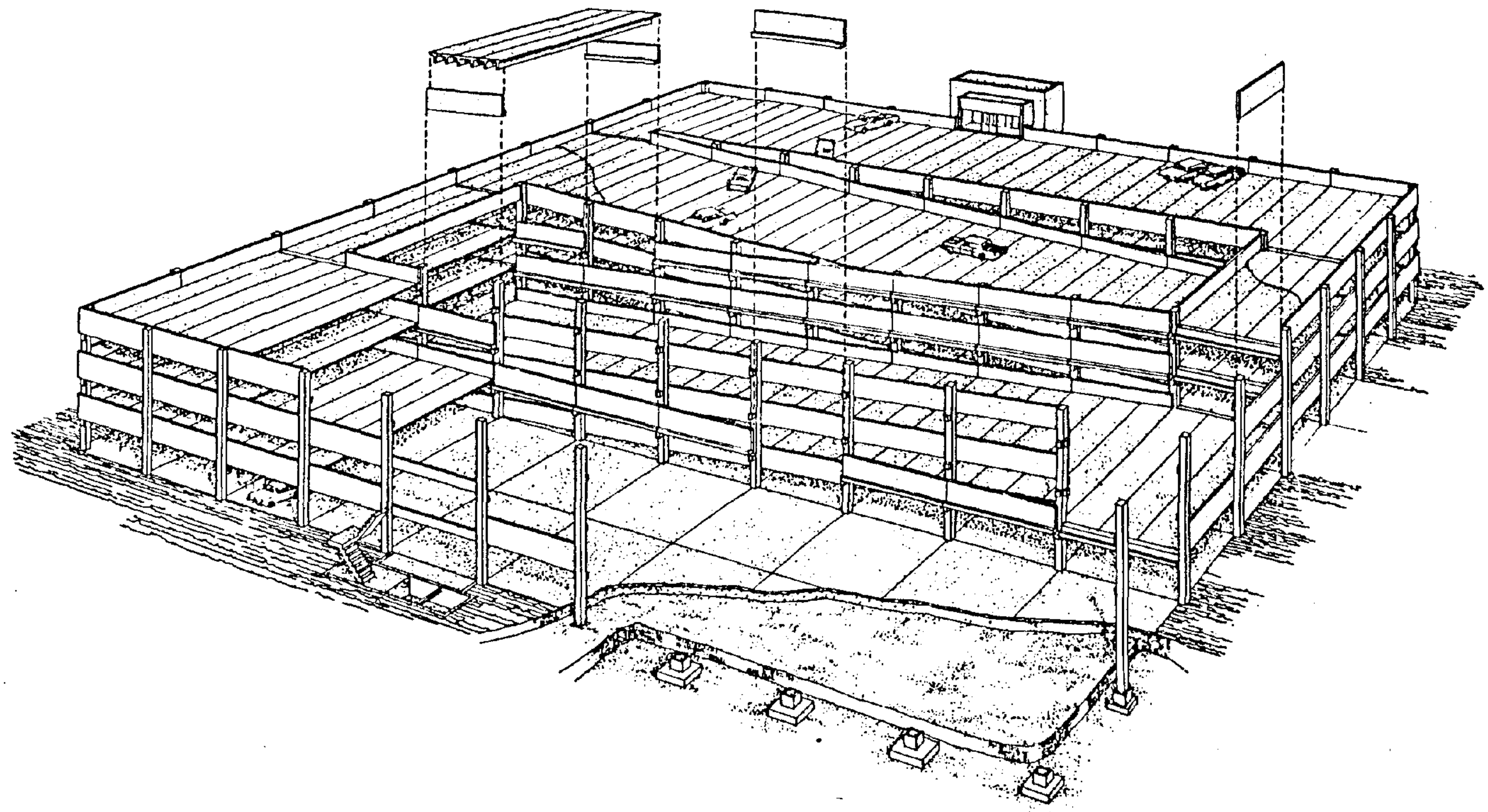


FIGURE 4.1-REPRESENTATIVE STRUCTURE BASED ON METRO-SPACE PARKING SYSTEM

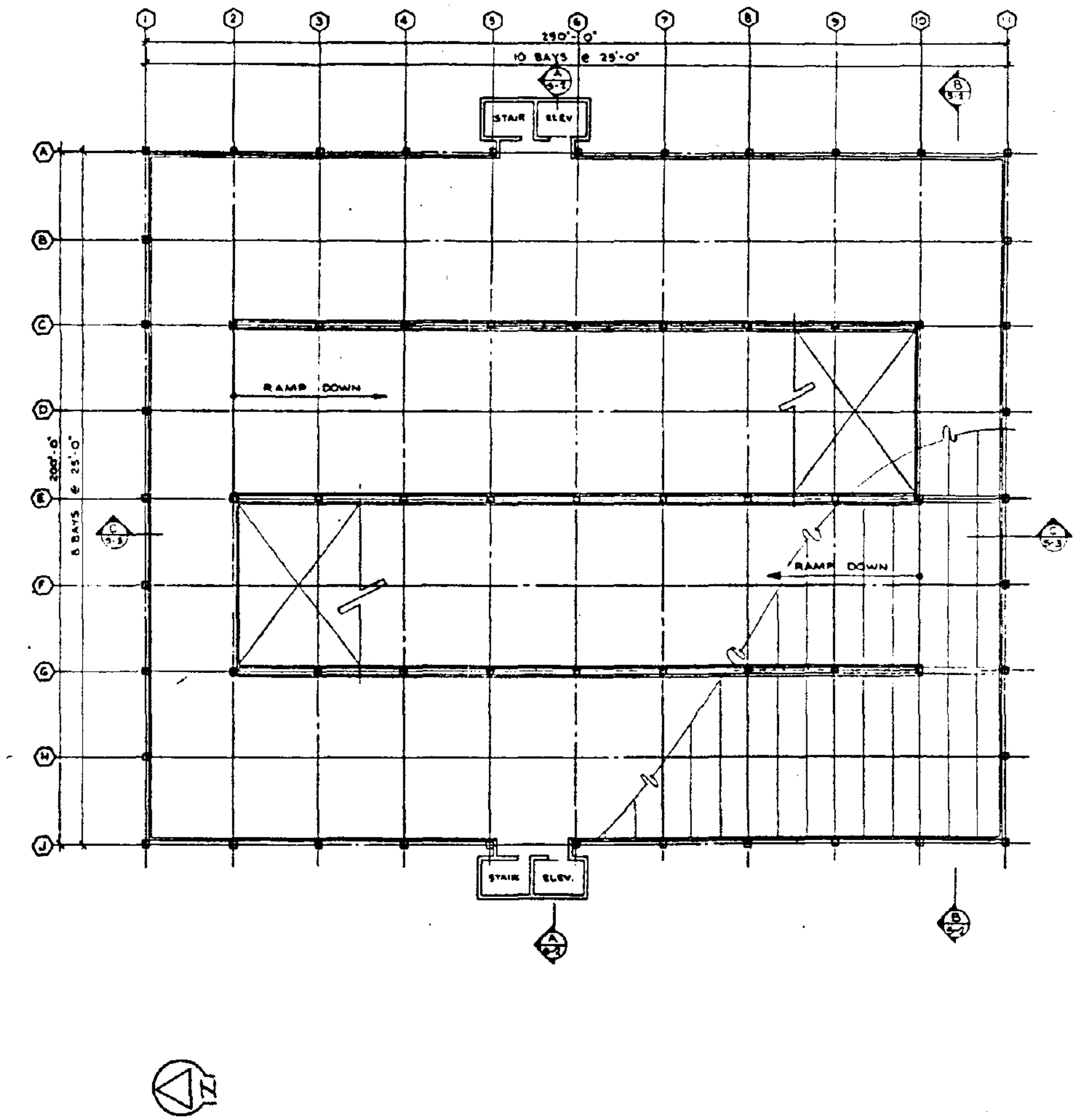


FIGURE 4.2 – TYPICAL FLOOR FRAMING PLAN

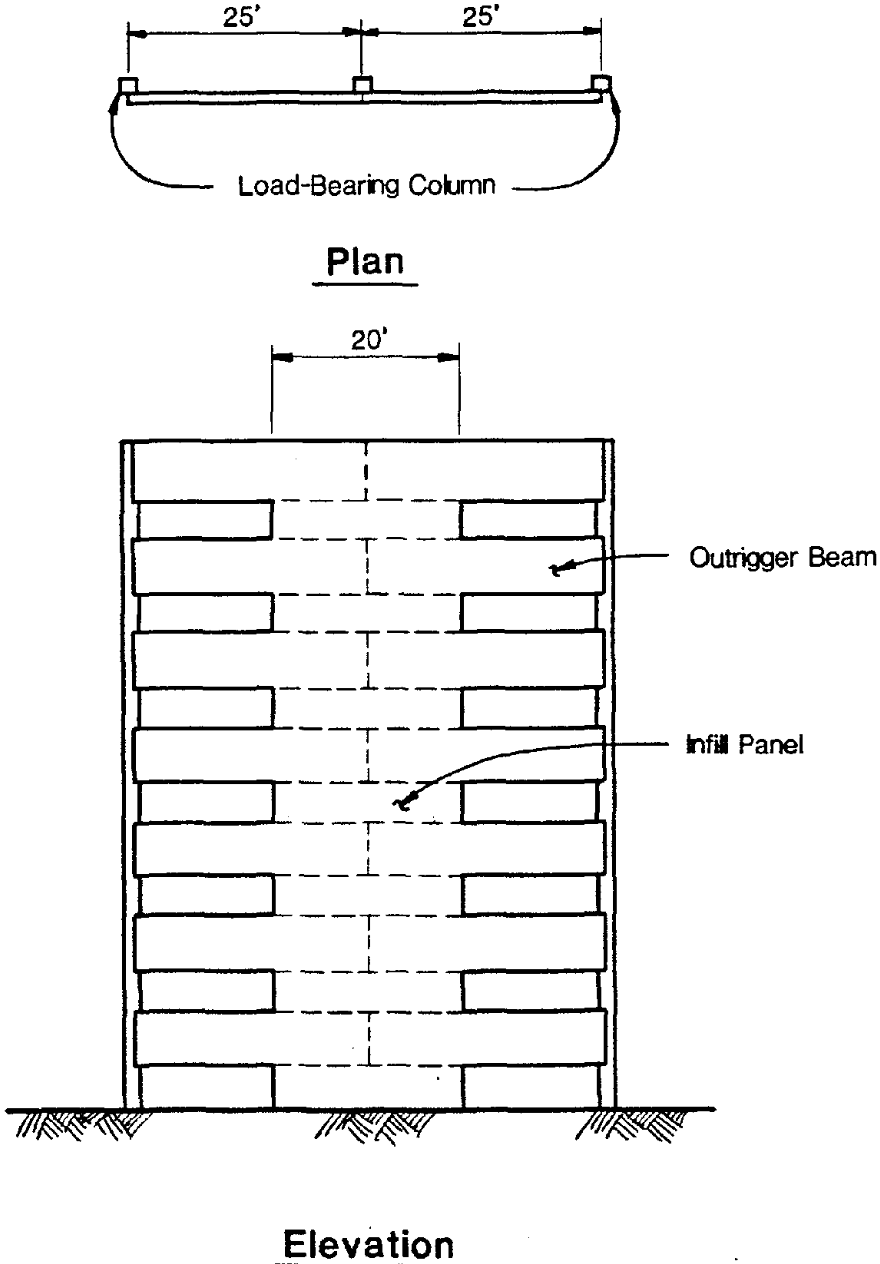


FIGURE 4.3–TYPICAL OUTRIGGER WALL ASSEMBLY

"outrigger beams," which have the same concrete outlines as standard ledger beam or spandrel shapes but contain special, ductile reinforcement details.

Stacks of outrigger beams and infill panels which form an outrigger wall are post-tensioned together after erection to form a monolithic unit. The infill panels are 20 feet long and, in the east/west frames, are centered on non-load-bearing columns. Thus, a given 25-foot-long outrigger beam is clamped between infill panels over a 10-foot length at one end, and is pin-connected to a load-bearing column at the opposite end. In the north/south frames, all columns carry dead load.

As shown in Fig. 4.4, the outrigger systems behave similarly to coupled shear walls. Ultimate lateral load capacity of the outrigger assemblages is governed by flexural yielding of the beams. Dead load and prestress resist opening of the horizontal joints between outrigger beams and infill panels, while the vertical prestressing steel is sized to prevent flexural yielding of the wall under ultimate seismic forces. Continued elastic response of the stiff vertical elements after yielding of the outrigger beams results in a variation of lateral displacements over the structure's height which is practically linear. The walls behave essentially as rigid bodies, rotating about their bases as the outrigger beams yield. This satisfies the objective stated in the design methodology of selecting a structural system with simple and predictable inelastic behavior.

The outrigger wall concept for lateral load resistance is developed below. Analytical models and loads are described. Strength and deformational requirements of the outrigger walls and their attachments to the floor system are presented. Seismic performance requirements for connections across selected joints are described, and the associated connection details proposed for testing in Phase 3 are illustrated.

4.3 ELASTIC ANALYSIS FOR UNDAMAGED CONDITION

Important conclusions regarding the structure's behavior under lateral loads were drawn from unit elastic analyses, initiated with three-dimensional computer models representing the original Metro-Space system. Models of Zone 1 and 2 designs were established, the distinction between them having to do with construction features of the horizontal decks which affect

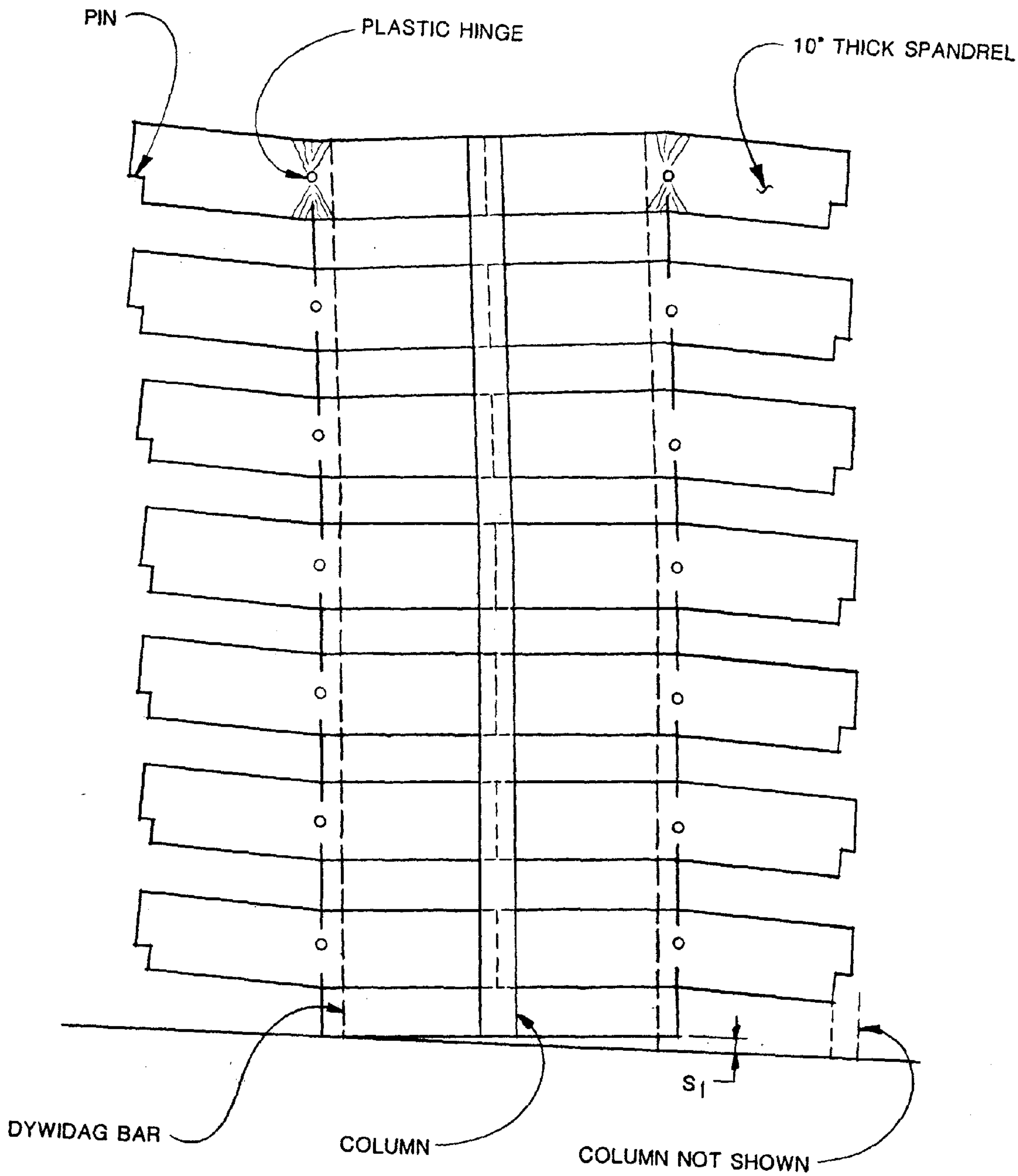


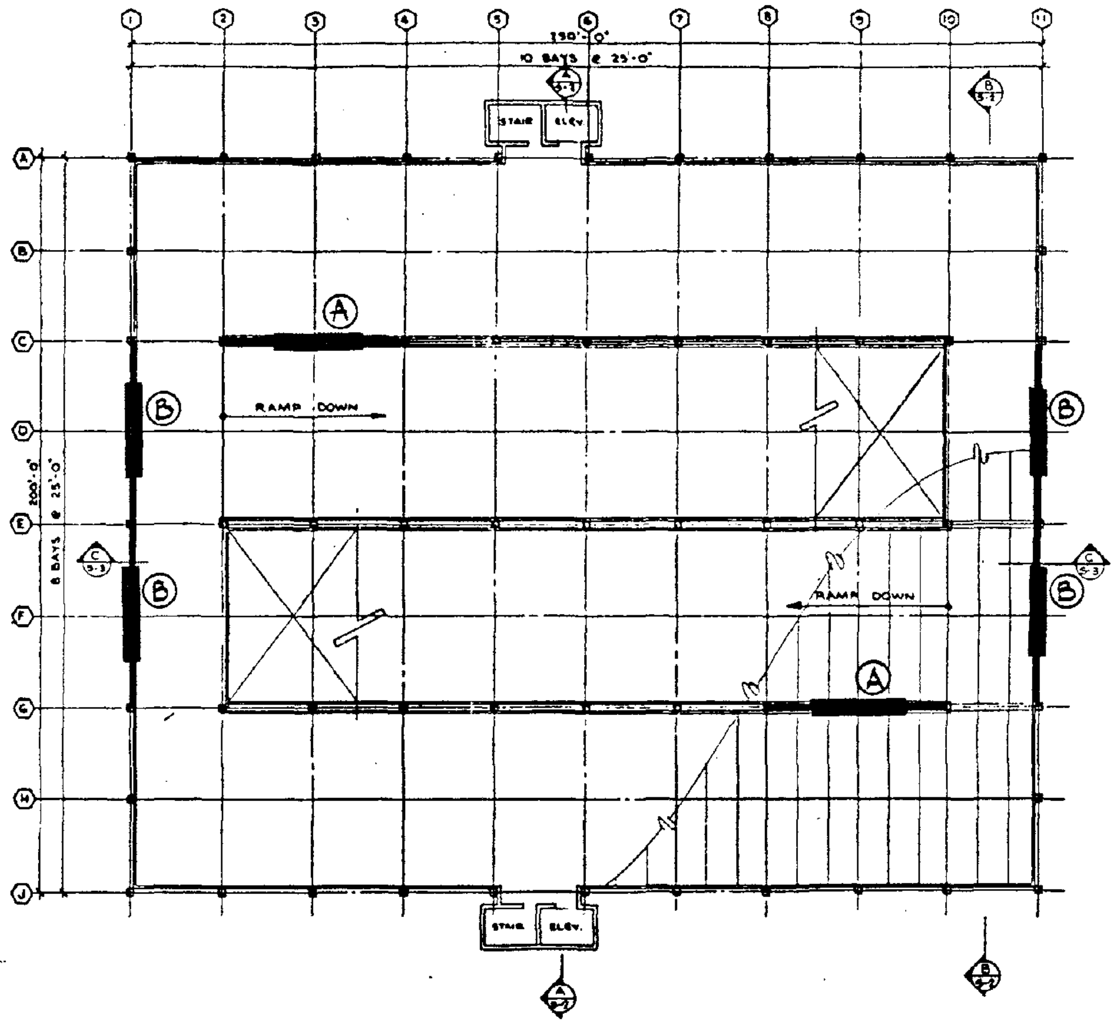
FIGURE 4.4 - INELASTIC DEFORMATIONS OF OUTRIGGER WALL

their in-plane stiffnesses. For Zone 1, the decks were assumed to be untopped, and stiffnesses were derived from assumed properties of the flange-to-flange connectors between double-T's. For Zone 2, the decks were assumed to be surfaced with a 2-inch reinforced, cast-in-place concrete topping and flexural properties were derived from the composite section. For both zones, the ramps were conceived as untopped and their in-plane stiffness was derived from assumed properties of flange-to-flange connectors between the double-T's. Column-to-spandrel connections were treated as pinned, and columns and elevator towers were elastically supported.

An early objective was to explore lateral resisting capacities of the elevator towers and ramps. Due to foundation flexibility and low gravitational overturing resistance, it was found that the elevator towers offer little potential for development as major elements of the lateral force resisting system.

Other considerations weighed against using the ramps for lateral resistance. When they are mobilized for truss action, large axial forces accumulate in the ramps level by level from the top downward in response to north/south loads. Resolving ramp thrusts into deck shears and back again down the spiral load path to the foundation presents some unusually demanding performance requirements for the discrete connections used in typical jointed construction. Further, unconventional details would be required to achieve the necessary ductility in such a system.

Thus, it was decided to rely totally on outrigger walls for seismic resistance in both directions. The computer models were modified by the inclusion of outrigger wall systems at the locations shown in Fig. 4.5, and the unit lateral analyses were repeated. Results for both the Zone 1 and Zone 2 models are presented in Table 4.1, which shows that for initial, undamaged elastic stiffnesses, seismic loads are resisted primarily by the outrigger walls with minor participation by the ramps and elevator towers. Member force results of the unit analyses were saved to be scaled later in accordance with the design base shear value.



Ⓐ Two outrigger walls "back to back"

Ⓑ Two outrigger walls "side by side"

FIGURE 4.5 - TYPICAL FLOOR FRAMING PLAN SHOWING LOCATION OF OUTRIGGER WALLS

TABLE 4.1. ELASTIC ANALYSIS RESULTS
DISTRIBUTION OF BASE SHEAR AMONG
LATERAL RESISTING SYSTEMS

	Zone 1[1]		Zone 2[2]	
System	North/South	East/West	North/South	East/West
Towers	12%	6%	11%	9%
Ramps	12%	----	16%	----
Outrigger walls	62%	74%	54%	81%
Frames[3]	14%	20%	19%	10%
Σ	100%	100%	100%	100%

[1] Decks and ramps untopped. Column bases elastically restrained. Column-spandrel and column-ledger connections pinned. Intended for Zone 1.

[2] Same as Note 1 except horizontal decks topped. Intended for Zone 2.

[3] Frame action due to elastic restraint of column bases.

4.4 SPECIFICATION OF SEISMIC LOADS

As described in Chapter 3, both "elastic demand" and "design ultimate" seismic loading conditions are considered in the proposed design approach. These quantities depend on periods of the fundamental vibration modes in the two principal plan directions. Periods of the Zone 1 and Zone 2 structures were computed using the three-dimensional computer models described above. These are presented in Table 4.2, and theoretical results are compared with empirical values from the Uniform Building Code.

The large discrepancy between analytical and code values requires comment. As explained in Chapter 3, code empirical formulas depend entirely on external dimensions and neglect configuration, proportions, and orientation of the lateral resisting elements. Due to the structure's low height-to-width ratio, the empirical formulas predict relatively short periods in both directions. Design forces computed from these values would exceed the loads resulting from the more rational, analytical periods by 20 to 30 percent. Seismic base shears for the elastic demand and design ultimate load cases have been computed based on the analytical periods and are presented in Table 4.3.

4.5 ASSIGNMENT OF MEMBER STRENGTHS

Refer again to the elastic distributions of base shear for the topped and untopped structures (Zones 1 and 2, respectively) presented earlier in Table 4.1. It can be seen that, when the structure is in the elastic range, lateral resistance is contributed by the columns, ramps, and towers in addition to the outrigger wall systems. Under the most intense ground shaking anticipated for either the Zone 1 or Zone 2 site, lateral resisting elements of the respective structures are intended to yield. The manner in which strength and ductility are apportioned among these elements determines the distribution of resisting forces which will exist when the structure is loaded beyond the yield point.

Often, member strengths are assigned in strict agreement with the elastic force distributions. However, it is feasible (and often desirable) to concentrate a structure's inelastic resistance among a smaller number of

TABLE 4.2. FUNDAMENTAL VIBRATION PERIODS
(seconds)

Configuration	North/South Computed	North/South Empirical*	East/West Computed	East/West Empirical*
Zone 1	0.74	0.20	0.88	0.22
Zone 2	0.64	0.20	0.84	0.22

* $T = \dfrac{0.05h}{\sqrt{D}}$

h = Height (feet)

D = Horizontal dimension in direction parallel to motion

TABLE 4.3. SEISMIC BASE SHEARS
(kips)

Configuration	Case	North/South	East/West
Zone 1	Elastic demand[1]	4,487	4,021
	Design ultimate[2]	1,246	1,148
Zone 2	Elastic demand[3]	10,241	10,241
	Design ultimate[2]	2,685	2,353

[1] ATC-3 base shear for R = 1.0; $A_a = A_v = 0.05$; S = 1.5

[2] 1.4 x UBC base shear

[3] ATC-3 base shear for R = 1.0; $A_a = 0.10$, $A_v = 0.15$; S = 1.5

members. This can be done if sufficient strength and ductility can be developed in those elements, provided that the integrity of gravity load paths and lateral stability of all portions of the structure are maintained, and if the integrity of load paths to the chosen ductile members is assured in the design of floor and roof systems which distribute forces horizontally among the lateral resisting elements.

In the present example, it was decided to rely entirely on the outrigger wall systems (which could be detailed readily for strength and ductility) to resist the lateral earthquake forces. This meant that strengths would not be apportioned in strict agreement with the elastic force distributions; also, the other members shown by elastic analysis to participate in resisting lateral loads would need to be detailed to ride along with the deformations of the outrigger walls, either elastically or through ductile action.

Except in the outrigger wall systems, the only frame action in the computer models was produced by elastic restraint at the column bases; all other connections between the columns and horizontal members are pinned. It was decided to detail the column-base connection to sustain moments due to erection loads but to neglect the predicted seismic moments in these members. The column-base connection would thus need to be detailed to accommodate rotations consistent with predicted inelastic deformations of the outrigger walls.

Similarly, it was decided not to rely on cantilever action of the elevator towers for seismic resistance. In Zone 1, predicted moments at the tower bases under the UBC design lateral force are 4500 and 980 kip-feet in the north/south and east/west directions, respectively. Due to their relatively small dead load, the towers will begin to uplift at base moments of 4300 and 1600 kip-feet in the two directions, respectively. While the towers could be used for lateral resistance in Zone 1, their lateral stiffness would begin decreasing due to uplift well before the Zone 2 design load was reached.

Accordingly, it was decided that connections between the towers and the main structure would be designed strong enough to overcome the gravity overturning resistance of the towers and ductile enough to sustain the relative movements which will occur when the towers rock during strong ground shaking. Thus, the main structure will, in effect, be used to keep the towers from falling over during an earthquake.

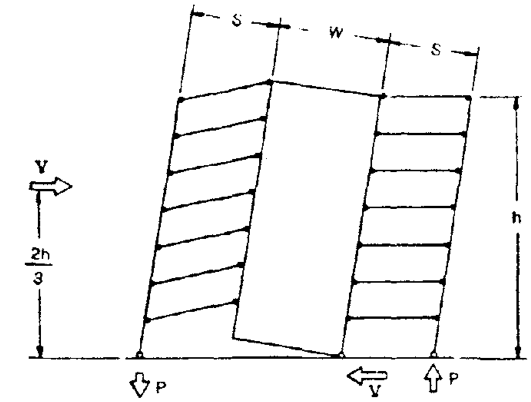
Member strengths were assigned in accordance with this design philosophy. Fig.4.6.a shows an outrigger wall at ultimate load. Note that plastic hinges have formed in all the outrigger beams and that the wall remains elastic. For reasons of production economy, all beams incorporated in outrigger systems will be detailed in the same manner; thus, their plastic moment capacities will be equal. By statics, a simple relationship can be derived expressing required ultimate moment capacity of the outrigger beams as a function of the design ultimate base shear to be resisted by the outrigger system. Similarly, the required elastic strength of the wall in shear and flexure can be computed by statics. These calculations are carried out in Fig. 4.6.b.

4.6 PREDICTION OF GLOBAL INELASTIC DISPLACEMENT MAGNITUDE

To enable rational detailing of connections between the outrigger walls and the horizontal decks, the magnitude of lateral wall displacements and rotations under ultimate seismic loading must be estimated. For this purpose, the ratio of elastic strength demand to ultimate strength, R, must be computed. As shown in Table 4.3, the ratio computed from ATC-3 elastic demand forces and factored UBC design loads is about 3.8.

Thus, as shown in Figs. 4.7 and 4.8, the "equal energy principle" described in Chapter 3 gives predicted plastic lateral displacements at roof level of about 2-1/2 inches for Zone 1 and 4-1/2 inches for Zone 2. Although the calculations result in different values for north/south and east/west response, as shown in Fig. 4.8, the differences are probably smaller than the uncertainty of the values themselves. Accordingly, for each zone the larger of the displacements predicted for the two directions has been taken to characterize both north/south and east/west response, as is reported in Fig. 4.9.

a.



- Plastic hinge of moment capacity Mp (kip ft)
- o Pin

Seismic Loads for Design

V (Kips)*	Zone 1	Zone 2
N/S	312	671
E/W	287	588

$* \ \dfrac{1.4 \times \text{UBC Base Shear}}{4 \text{ Walls}}$

b.

Plastic Analysis of Outrigger Beams

$$P = \frac{\text{Overturning Moment}}{\text{Arm}} = \left(\frac{2Vh}{3}\right)\left(\frac{1}{W+2S}\right)$$

$$Vp = \text{Plastic Shear in Beam} = \frac{P}{N}$$

$$Mp = \text{Required Plastic Moment in Beam} = Vp \cdot S$$

Beam Design Forces

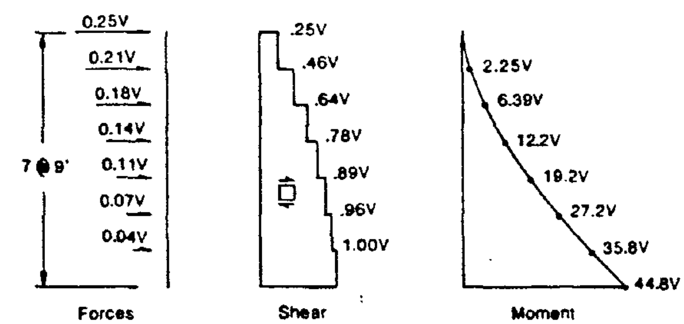
W = 20ft h = 63ft N = 7

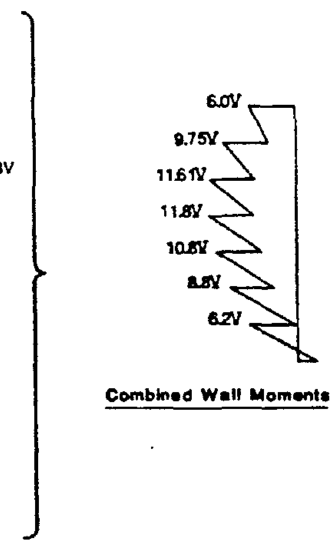
S = 15ft P = 0.84V (Kips)

		Required Capacity
Vertical Shear.	Vp (kips)	0.12 V
Plastic Moment.	Mp(kips ft.)	1.80 V

Wall Shear and Moments

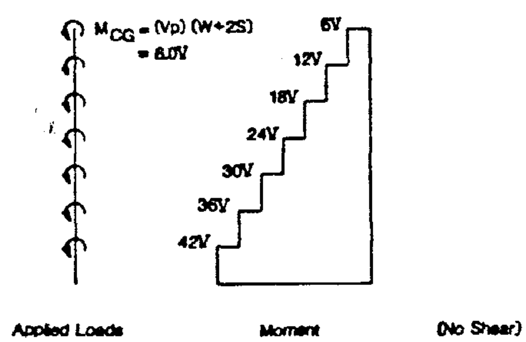
Due to Lateral Loads





Combined Wall Moments

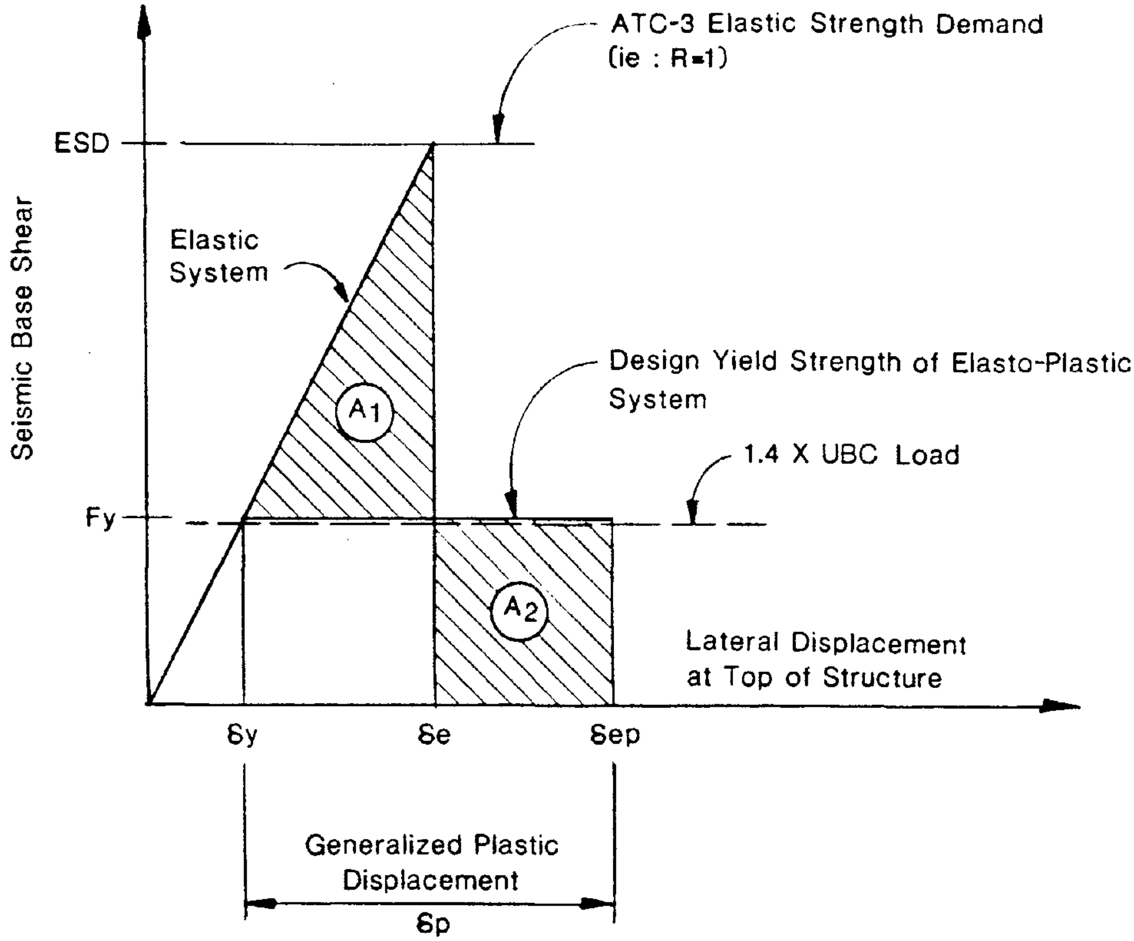
Due to Coupling Beam Shears



Wall Design Forces

	Maximum	At
Moment (Kip-Ft)	11.8V	4th Level
Shear (Kips)	V	Base

FIGURE 4.6 - STRENGTH DESIGN OF OUTRIGGER WALL SYSTEM



<u>Response Modification Factor</u>

$$R = \frac{ESD}{Fy}$$

<u>Ductility Factor</u>

$$\mu = \frac{\delta ep}{\delta y}$$

$$A_1 = A_2$$

$$\boxed{\mu = \frac{R^2 + 1}{2} \qquad \delta p = \delta y \left(\frac{R^2 - 1}{2}\right)}$$

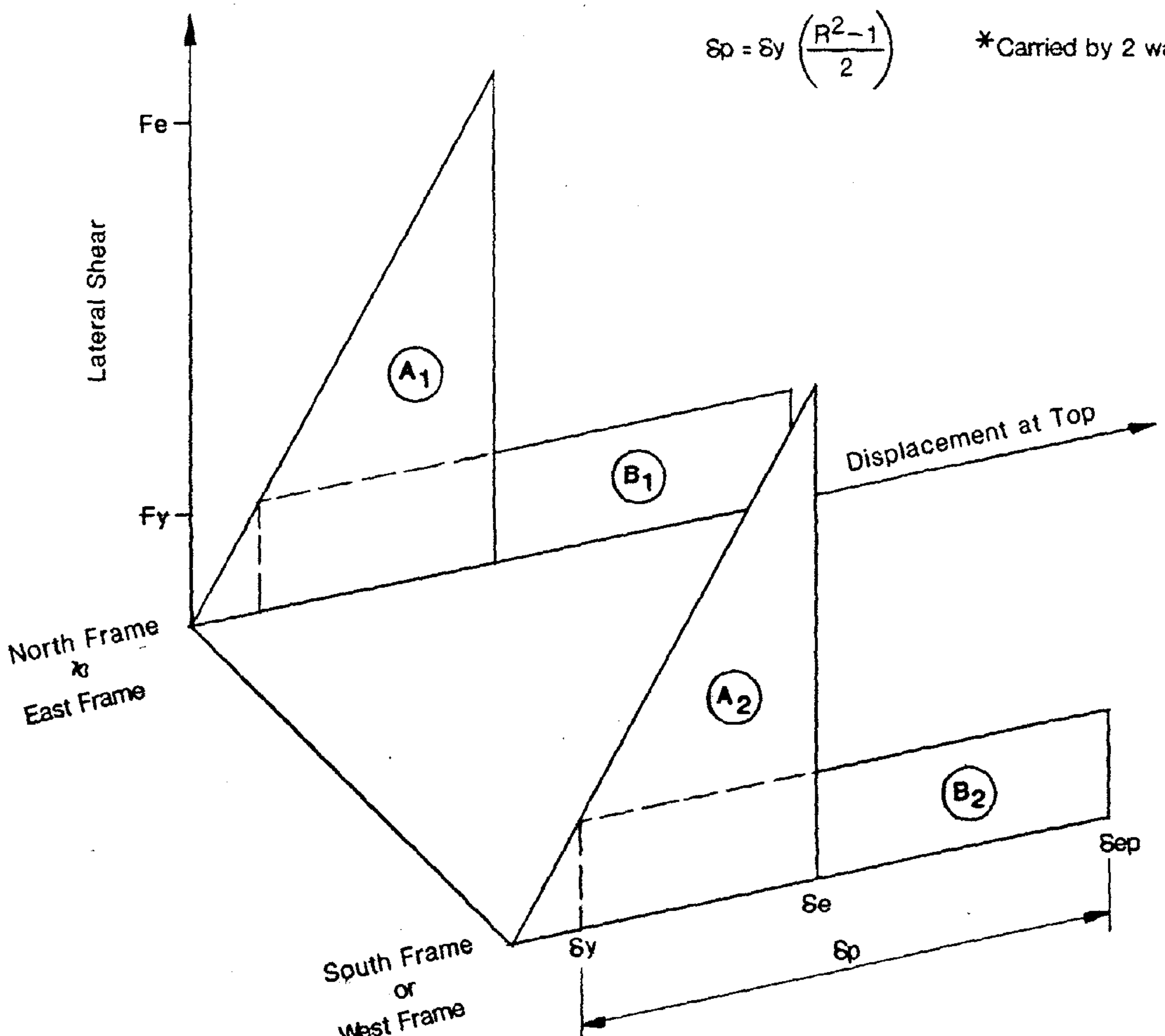
FIGURE 4.7 - EQUAL-ENERGY PRINCIPLE FOR ESTIMATING MAXIMUM SEISMIC DISPLACEMENTS OF AN ELASTO-PLASTIC SYSTEM

PREDICTED PLASTIC DISPLACEMENT MAGNITUDES

Zone	Direction	R = Fe/Fy	Fy (kips)*	δy (in.)	δe (in.)	δp (in.)
1	North/South	3.8	624	0.35	1.33	2.35
	East/West	3.8	574	0.32	1.22	2.15
2	North/South	3.8	1342	0.50	1.90	3.36
	East/West	3.8	1176	0.64	2.43	4.30

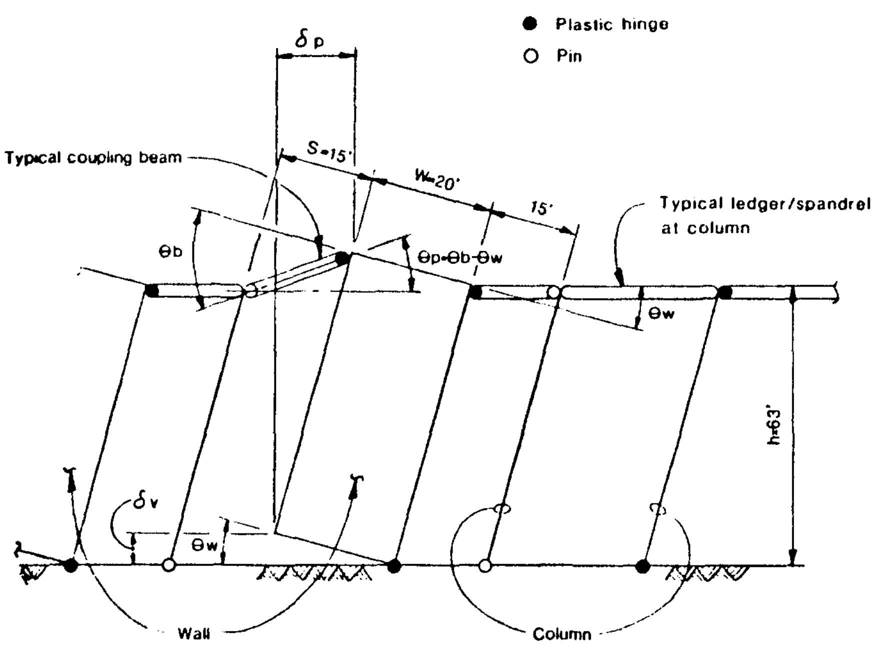
$$\delta p = \delta y \left(\frac{R^2-1}{2}\right)$$

*Carried by 2 walls



By Equal-Energy Principle $\sum_{i=1}^{2} Ai = \sum_{i=1}^{2} Bi$

FIGURE 4.8 – INELASTIC SEISMIC BEHAVIOR OF OUTRIGGER WALL PAIRS



PREDICTED PLASTIC DISPLACEMENTS AND HINGE ROTATIONS

ZONE	δ_p (in)	δ_v (in)	θ_b (deg)	θ_w (deg)
1	2.5	0.8	0.7	0.3
2	4.5	1.5	1.2	0.5

NOTES :
1. Values have been rounded
2. The larger of N/S and E/W values for each zone has been specified for both directions

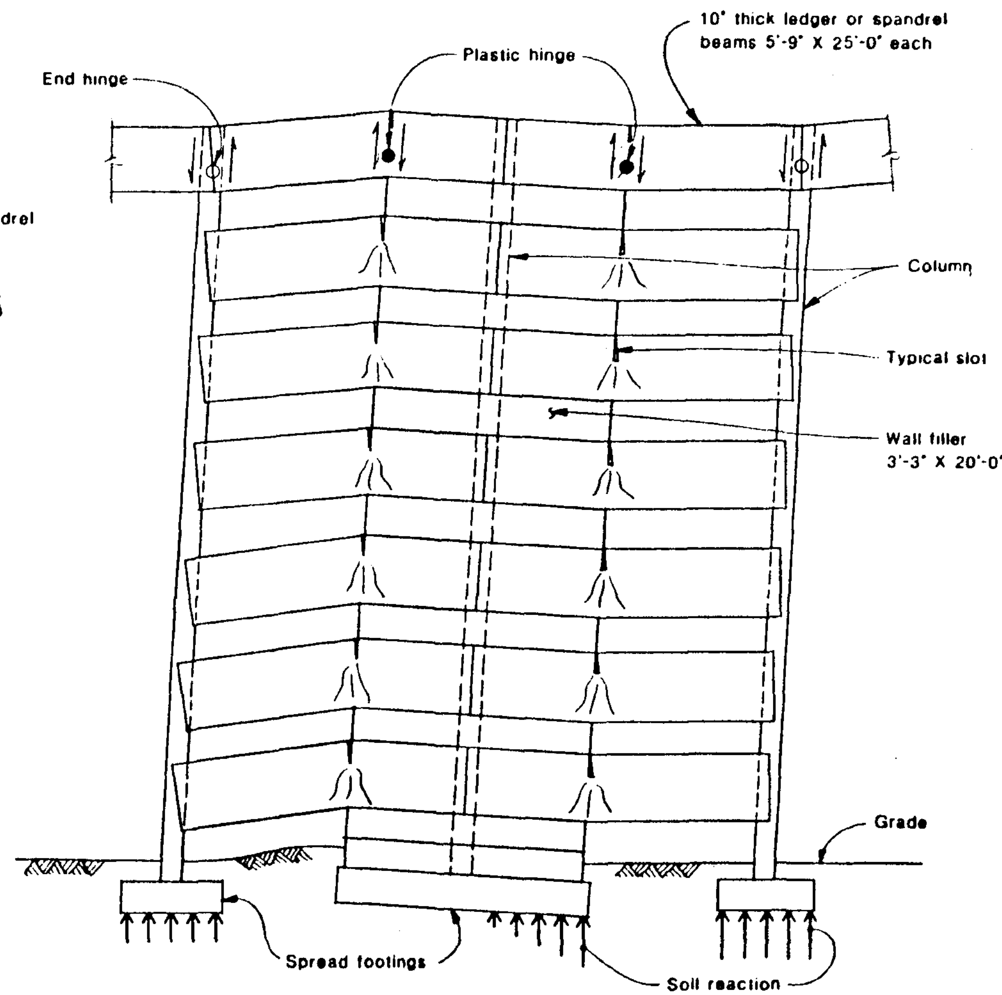


FIGURE 4.9 – KINEMATICS OF ISOLATED OUTRIGGER WALL UNDERGOING PLASTIC DEFORMATION

4.7 KINEMATIC ANALYSIS OF POST-YIELD BEHAVIOR

Fig. 4.9 shows the kinematic relationships between lateral plastic displacement at the top of the structure and displacements and plastic hinge rotations at selected points within a typical outrigger wall assemblage. Regardless of plan orientation, predicted outrigger beam plastic hinge rotations are about 3/4 degree and 1-1/4 degree for Zones 1 and 2, respectively. Predicted vertical displacements at the end of an outrigger wall are about 5/6-inch and 1-1/2 inches, respectively, for the two zones. Note that these are the magnitudes of plastic deformation, and are additive with the elastic deformations which occur before the loads have reached the yield point.

4.8 NUMBER OF CYCLIC LOAD REVERSALS TO BE SUSTAINED

The expected number of cyclic load reversals in the inelastic range is a function of fundamental vibration period and the ratio of elastic strength demand to yield strength (i.e., the R-value). According to the data provided in Table 3.2, an elastoplastic single-degree-of-freedom system with period of about 0.9 second, designed to an R-value of 3.8, is expected to experience two or three fully reversed cycles of inelastic deformation during a Zone 1 earthquake and five or six cycles in Zone 2. To be conservative, three cycles will be assumed for Zone 1 and six cycles for Zone 2.

4.9 SPECIFICATION OF CONNECTOR PERFORMANCE CRITERIA

Figs. 4.10 through 4.19 present connection details for selected joints of the precast parking structure, along with quantitative descriptions of the cyclic forces and deformations which they are to sustain. Connections have been grouped according to their function, either as participating in lateral load resistance or as "riding along" with the lateral resisting system during a damaging earthquake.

For example, connections within the outrigger systems participate directly in resisting lateral forces and are included in the first group. In contrast, connections within the horizontal decks, which are intended to remain elastic as the decks ride through inelastic lateral displacements of the outrigger systems, are included in the second category. Also included in the second group are connections between the decks and the outrigger systems which are intended to remain elastic with respect to in-plane shear transfer, but are also intended to accommodate relative rotations or relative vertical displacements through either elastic or inelastic flexure.

4.9.1 Connections and Special Ductile Members Within Primary Lateral Resisting System

Structural elements within the primary lateral resisting system play a direct role in resisting seismic loads. Designed to code force levels, they are expected to undergo significant inelastic deformations during a damaging earthquake. Plastic action, however, is confined to predetermined locations through judicious assignment of relative strengths and careful detailing of reinforcement.

4.9.1.1 Plastic Hinge Within Ledger Beam (Fig. 4.10)

Outrigger systems in the north/south direction incorporate ledger beams specially detailed for ductility. As shown in Fig. 4.10, outrigger walls with a north/south orientation are placed back-to-back on opposite sides of a load-bearing column. Inelastic action of the outrigger mechanism requires the formation of a plastic hinge at the face of the outrigger wall about the strong axis of the ledger (Fig. 4.11). A vertical diaphragm (Fig. 4.12) inserted between paired ledgers on opposite sides of the column resolves the opposing torques due to double-T reactions, which are eccentric with respect to the ledger shear centers, and eliminates the need to carry torsion through the plastic hinge region.

Vertical shears of 38 and 81 kips (including 1.40 load factor) occur under the ultimate seismic loadings of Zones 1 and 2, respectively. These are additive with factored shears of 35 kips due to dead load and 23 kips due to live load. Ultimate moments of 570 and 1215 kip-feet (including 1.40 load factor) are generated in the plastic hinge region under Zone 1 and 2 seismic loadings, respectively.

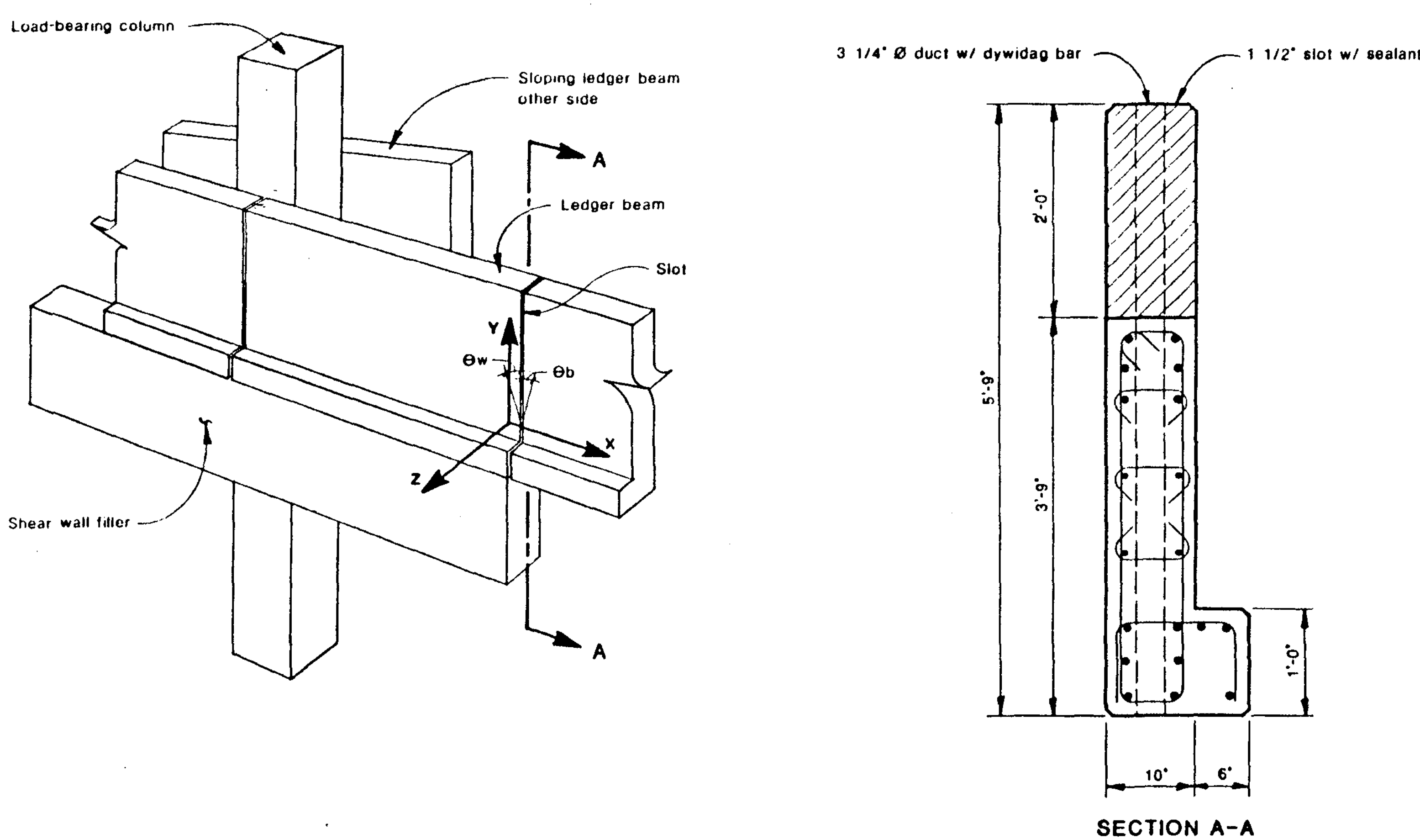


FIGURE 4.10 - PLASTIC HINGE REGION WITHIN LEDGER BEAM

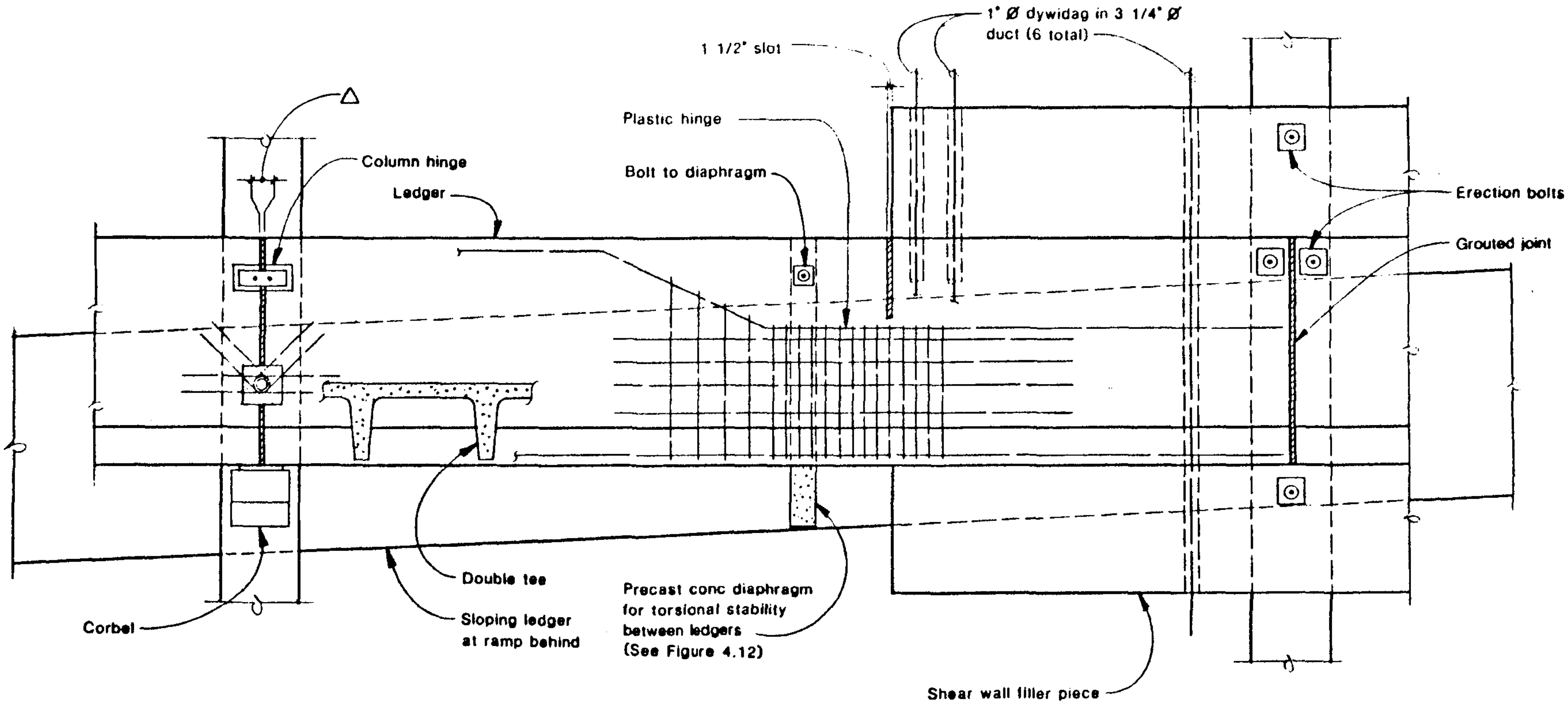


FIGURE 4.11 – OUTRIGGER BEAM – LEDGER DETAILS
(SPANDREL DETAILS SIMILAR)

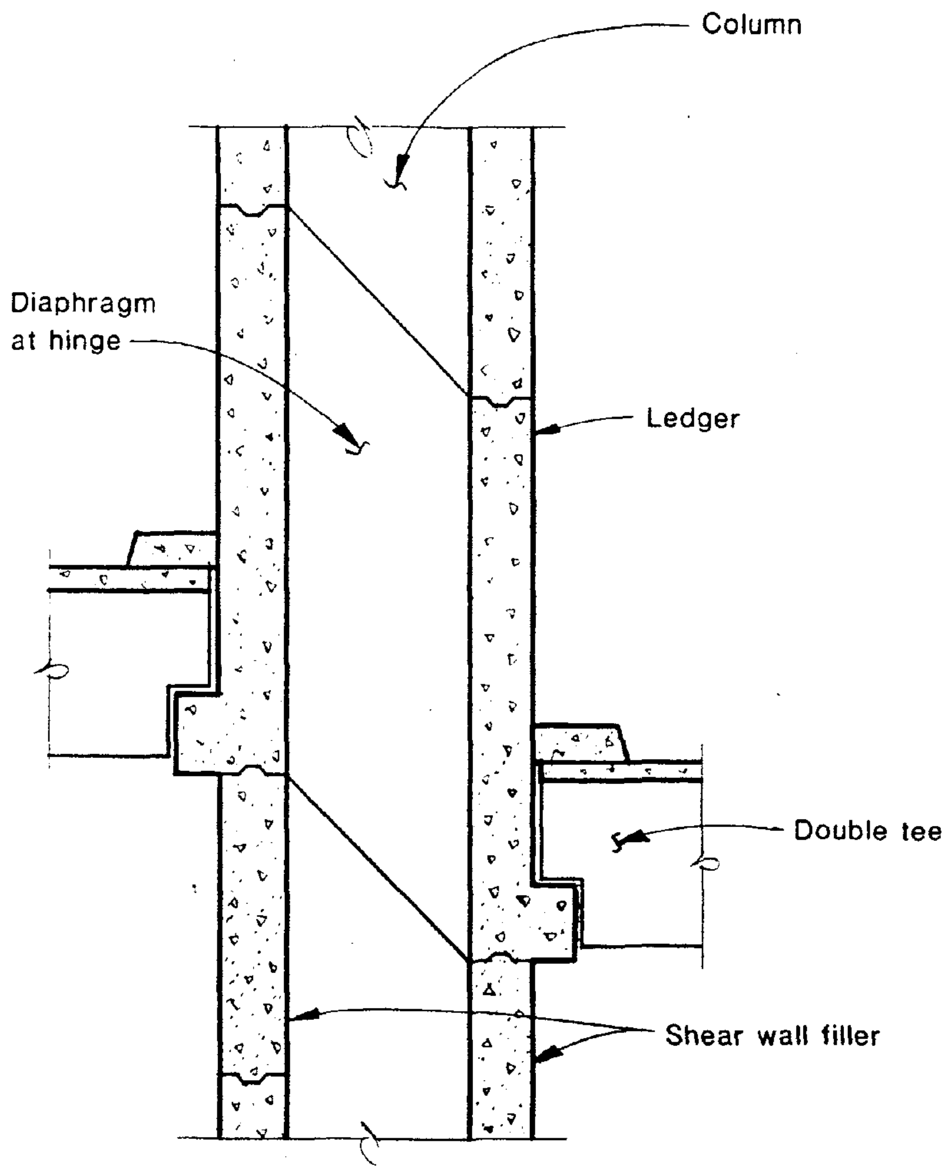


FIGURE 4.12 – INTER-LEDGER DIAPHRAGM

As shown in Figure 4.9, the wall rocks about the compression end of the footing and the rotation point migrates from one side to the other as the structure sways. This produces a difference in predicted magnitudes of "positive" and "negative" hinge rotation. For Zone 1, plastic hinge rotations are on the order of negative 3/4-degree (producing tension in the top fibers) and positive 1/4-degree (producing compression in the top fibers). For Zone 2, the values are on the order of negative 1-1/4 degrees and positive 1/2-degree. Note that these are the magnitudes of plastic rotation occurring after the structure has been loaded to the yield point.

4.9.1.2 Plastic Hinge Within Spandrel Beam (Fig. 4.13)

Outrigger systems in the east/west direction incorporate spandrel beams specially detailed for ductility. As shown in Fig. 4.5, outrigger walls with this orientation are placed side by side in the plane of perimeter frames at north and south ends of the structure. Inelastic action of the outrigger mechanism requires the formation of a plastic hinge at the face of the outrigger wall about the strong axis of the spandrel.

Vertical shears of 38 and 81 kips (including 1.40 load factor) occur under the ultimate seismic loadings of Zones 1 and 2, respectively. The spandrels span in the same direction as the double-T deck members and carry no superimposed dead or live load. Ultimate moments of 570 and 1215 kip-feet (including 1.40 load factor) are generated in the plastic hinge region under Zone 1 and 2 seismic loadings, respectively.

Plastic hinge deformations are the same as predicted for the ledger beam, described above.

4.9.1.3 Connection of Ledger or Spandrel to Column (Figs. 4.14 and 4.15)

The ledger- or spandrel-to-column connection is critical to the outrigger systems. Plastic hinge moments of the outrigger beams (either ledgers or spandrels) at the face of the outrigger walls are generated by vertical forces at the beam-to-column connection. Calculated magnitudes of these forces are 38 and 81 kips for Zones 1 and 2, respectively (including a load

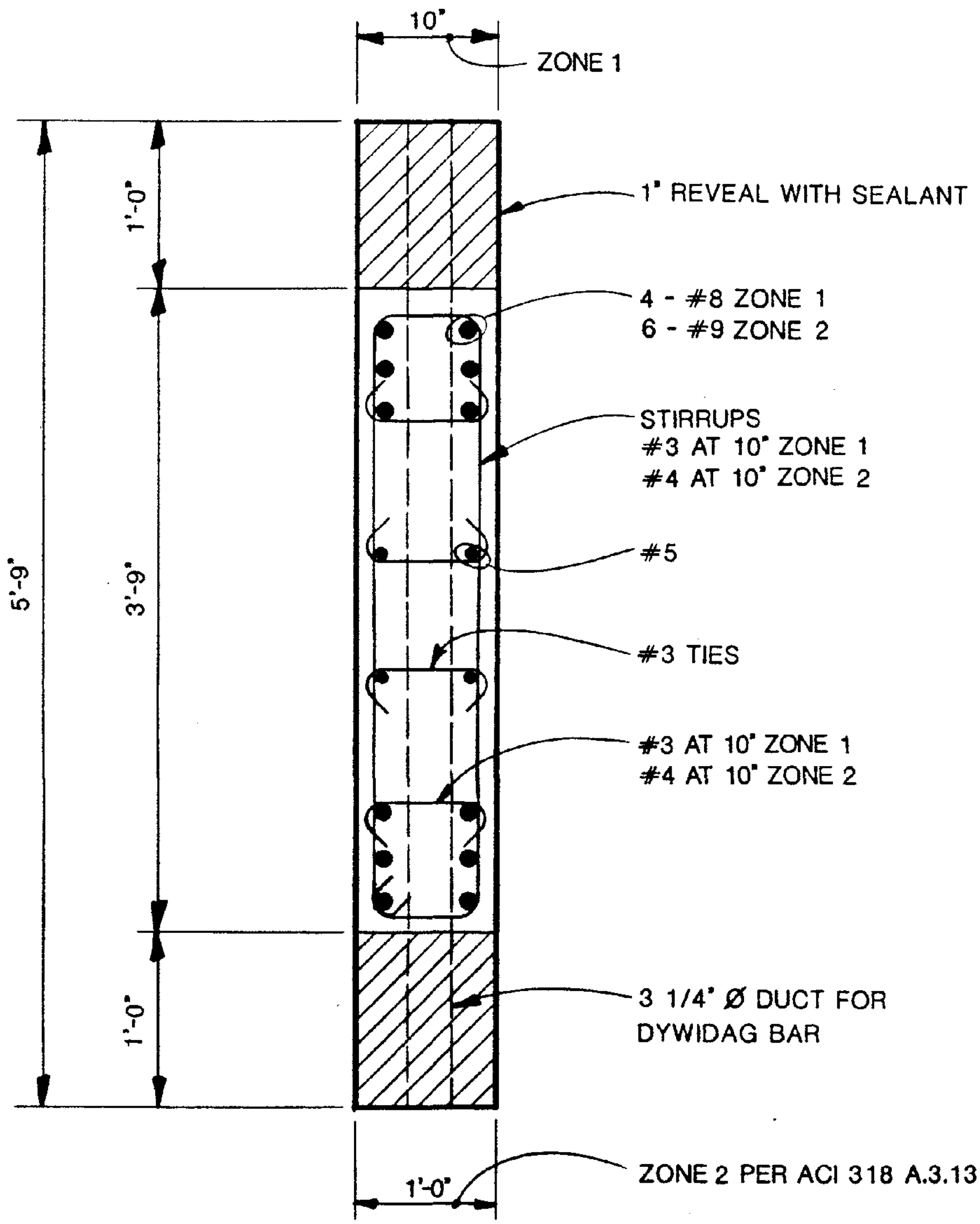
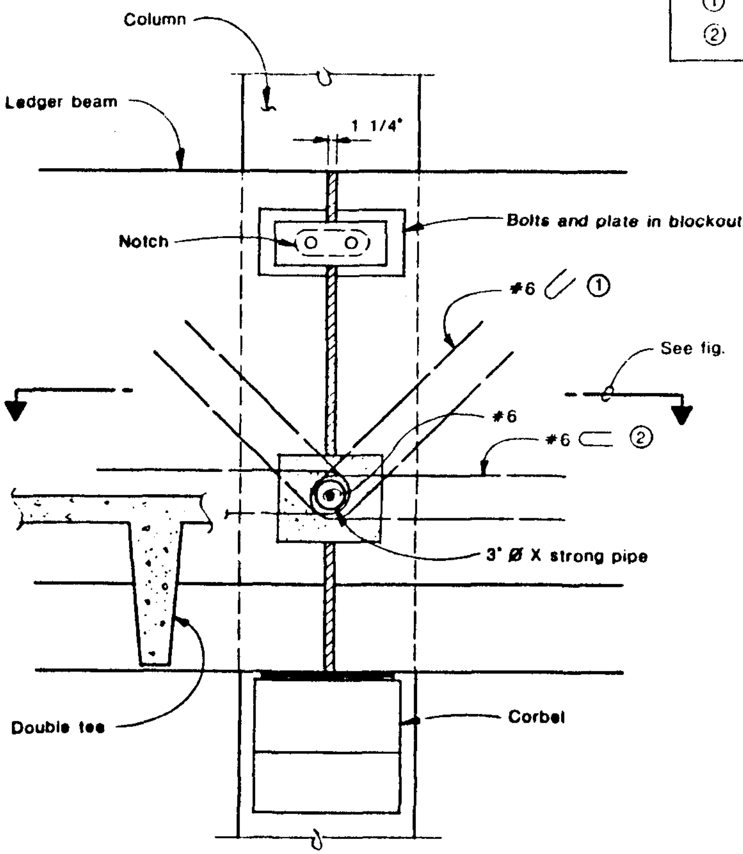


FIGURE 4.13 - SECTION THRU SPANDREL HINGE
(LEDGER SIMILAR)

MARK	ZONE 1	ZONE 2
①	1-#6 ea side	2-#6 ea side
②	1-#6 ea side	2-#6 ea side



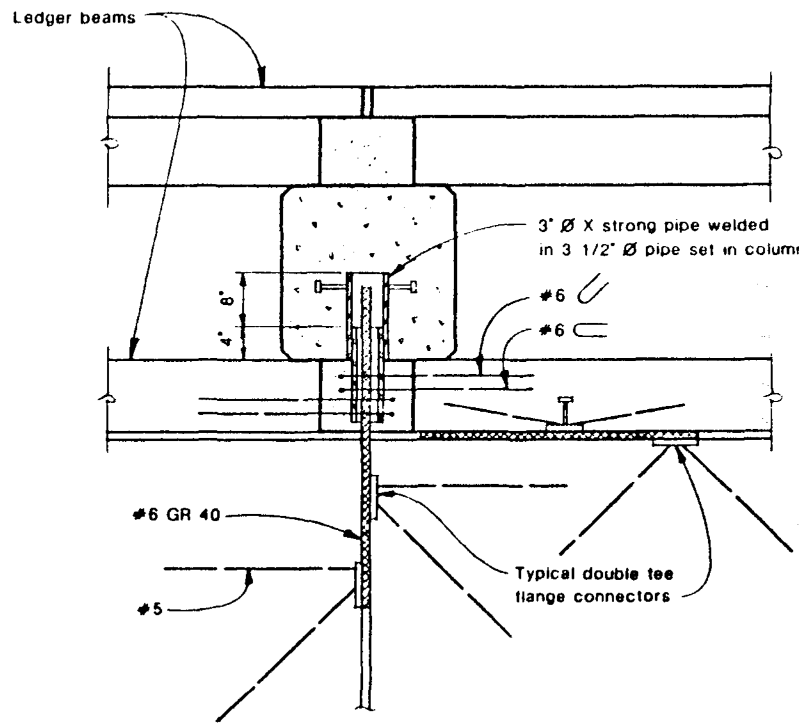
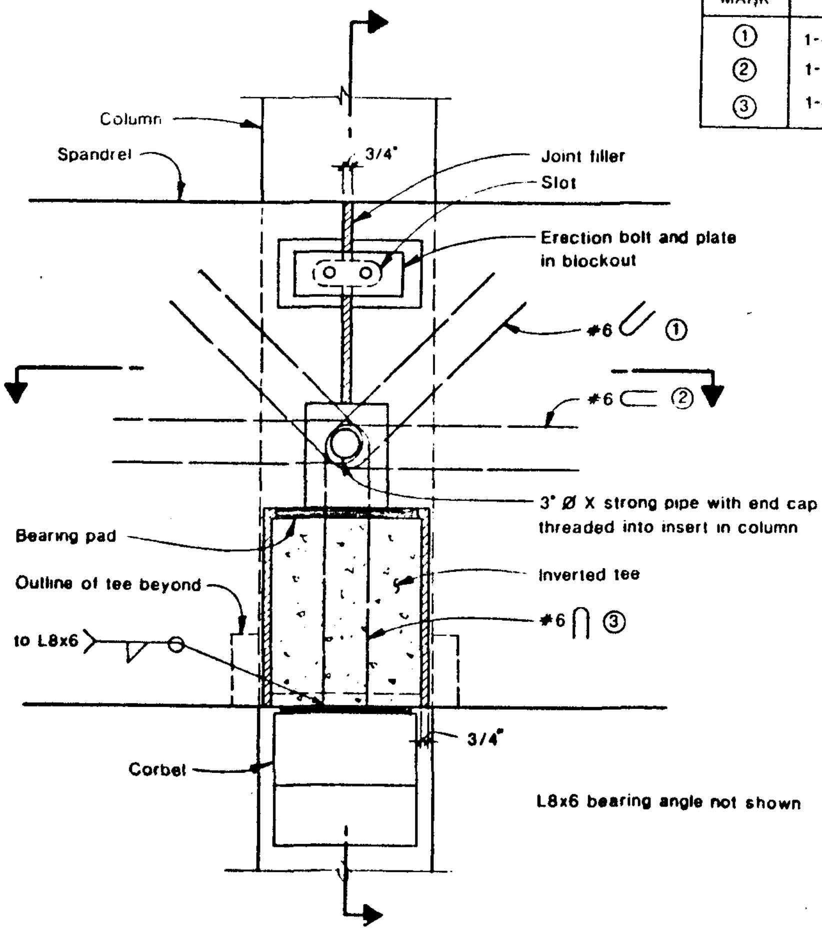


FIGURE 4.14 – LEDGER TO LEDGER/COLUMN



SPANDREL

MARK	ZONE 1	ZONE 2
①	1-#6 ea side	2-#6 ea side
②	1-#6 ea side	1-#6 ea side
③	1-#6	2-#6

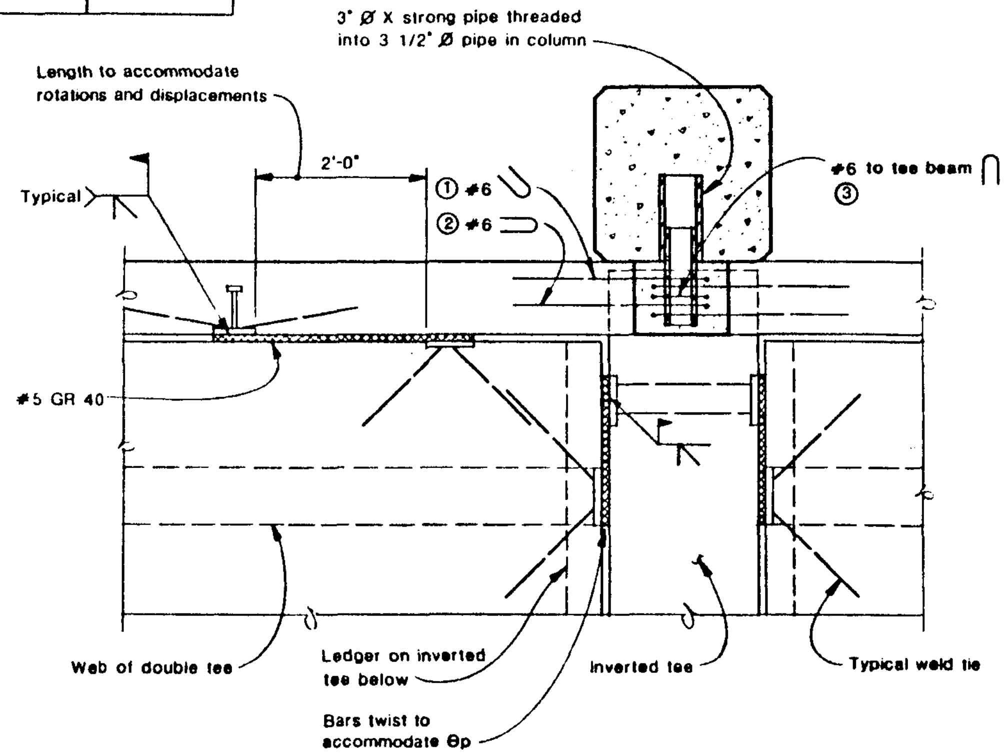


FIGURE 4.15 - SPANDREL TO SPANDREL/COLUMN/INVERTED TEE

factor of 1.40). To account for possible overstrength or strain-hardening of flexural reinforcement at the plastic hinge, however, the beam-column connection should be designed to sustain forces at least 25 percent in excess of these values.

Accompanying the vertical shears are rotations of the beams with respect to the column, and of the beam ends with respect to each other. As with the ledger and spandrel rotations, results presented here reflect the structure's plastic displacements and are additive with elastic movements which occur as the structure is loaded to the yield point.

Rounding values to the nearest 1/8 degree, predicted beam-to-column rotations are negative 1/4 degree (tension on top) and positive 3/4 degree (compression on top) for Zone 1; they are negative 1/2 degree and positive 1-1/4 degree for Zone 2. Differences in magnitudes of the positive and negative values are caused by vertical displacements at the plastic hinge due to wall rocking and occur only when the wall sways away from the beam-column connection in question.

Corresponding beam-to-beam end rotations are positive 3/8 degree for Zone 1 and positive 3/4 degree for Zone 2 (compression on top). Note that these rotations always close the top of the beam end-to-end gap and occur only when the wall sways away from the beam-column connection in question.

Typical connections (i.e., not part of the outrigger system) must accommodate equal positive and negative beam-to-column rotations of 1/4 and 1/2 degree for Zones 1 and 2, respectively, even though the beam-to-beam end rotation does not occur at these locations.

4.9.1.4 Connections Across Horizontal Joints in Outrigger Wall (Fig. 4.16)

As shown in Fig. 4.6.b, horizontal shears are transmitted to the wall by its connections with the parking decks. Due to foundation flexibility, the moment at the base of the wall is negligible while maximum moment occurs at the fourth level above grade. The wall acts primarily to deliver shear to the foundation.

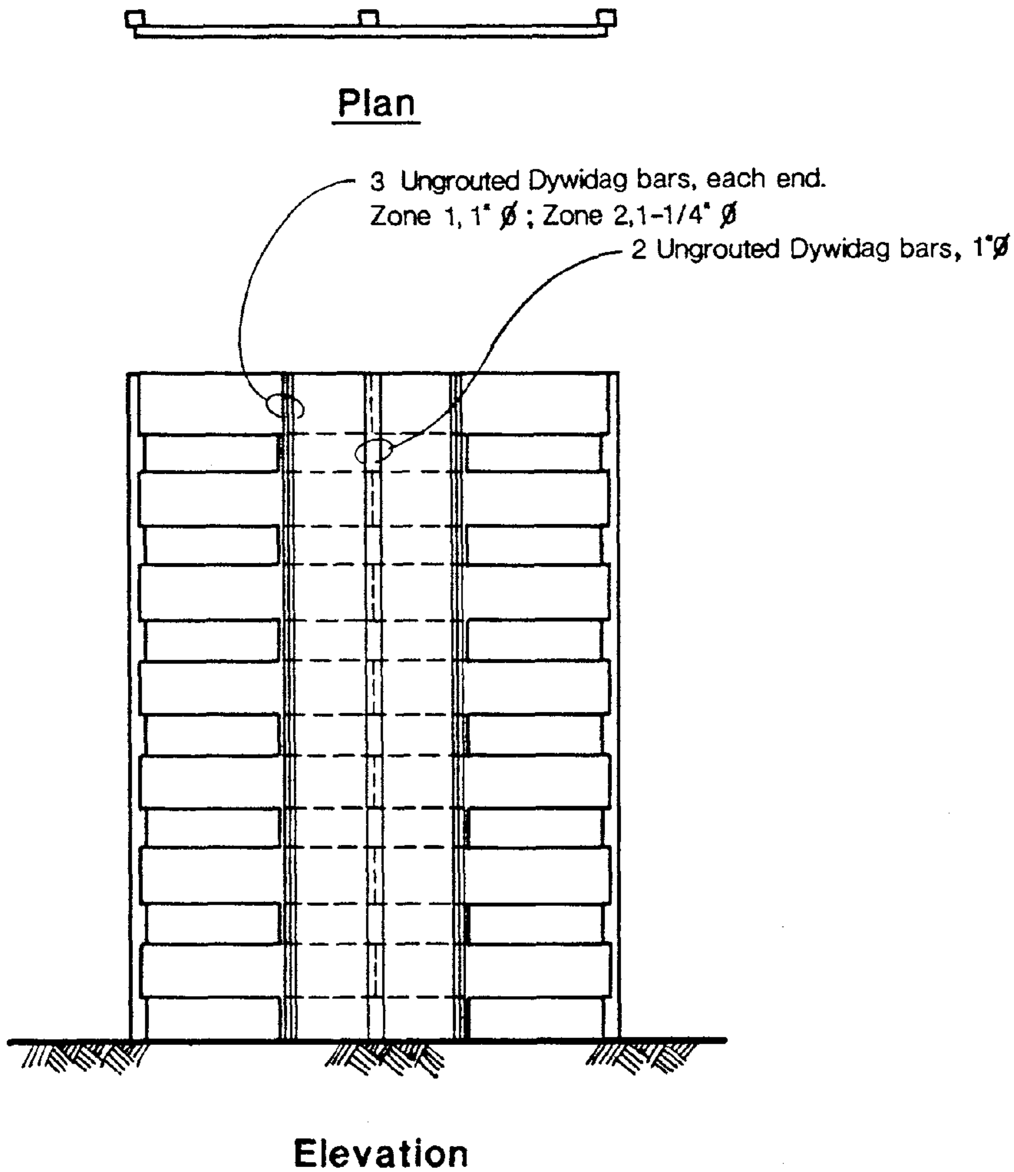


FIGURE 4.16 - OUTRIGGER WALL POST-TENSIONING

Overturning resistance of the outrigger wall systems is provided almost entirely by cantilever flexural action of the outrigger beams, which react against load-bearing columns to either side of the wall. Thus, the magnitude of wall moments is limited by flexural capacities of the outriggers.

Sufficient vertical prestress must be provided to avoid shear slippage along horizontal joints. At the same time, a sufficient steel area must be provided to avoid yielding across horizontal joints under flexural loads in the plane of the wall. Design calculations for a typical north/south wall show that six 1-inch-diameter Dywidag bars, three at each end of the wall, are required in Zone 1, while six 1-1/4-inch bars are required in Zone 2. The bars are 160-ksi grade, ungrouted, and stressed to 70 percent of ultimate.

Under ultimate seismic forces, joint precompression due to dead load and prestress will be overcome by flexural stresses. Horizontal joints will open on the tension end of the wall. Axial stress in the ungrouted Dywidag bars will increase to about 80 percent of ultimate, allowing roughly a 25-percent overload due to strain-hardening of flexural reinforcement in the outrigger beam plastic hinges, before the ultimate capacity of the prestressing steel is exceeded.

Under the design ultimate loading, total elastic elongation of the Dywidags beyond onset of joint opening is about 1/2 inch. Thus, assuming all the strain accumulated over a single horizontal joint, the joint opening would amount to a maximum of 1/2 inch. Because the Dywidags remain elastic under this condition, the joint would close and the original precompression would remain upon removal of the flexural load.

Though not required for strength, two additional bars 2 feet apart and symmetrical with the wall centerline are provided to maintain positive alignment of the stacked outrigger elements and infill panels. While loads on east/west walls are somewhat lower in either zone, it was decided to specify the same steel area and post-tensioning force in these as for north/ south walls for the sake of construction simplicity.

4.9.2 Connections Intended to Ride Along with Deformations of the Lateral Force Resisting System

Connections between precast elements which form the parking decks and between the decks and the outrigger walls play an important role in the structure's seismic response, transmitting inertia forces to the primary lateral resisting elements. Also, because the decks tie together the outrigger systems oriented in the north/south and east/west directions, these connections are essential to the structure's overall stability.

Calculated diaphragm forces in the parking decks are relatively low. Consequently, the connections discussed in this section can be sized to remain elastic under in-plane loads of the ultimate limit state. Careful detailing is required, however, because in addition to resisting the in-plane forces, the connections must accommodate relative vertical motions or longitudinal rotations across the joints associated with the structure's inelastic lateral displacements.

Performance criteria presented below are stated in terms of the joints between selected precast elements. In each case, a connection concept appropriate to the required strengths and deformational capacities is suggested. The emphasis, however, is on characterizing the joint actions independently of connector configuration, orientation, or distribution; thus, the competence of any proposed connection detail can be evaluated by physical tests in Phase 3.

4.9.2.1 Diaphragm Chord (Fig. 4.17)

A reinforced, cast-in-place curb has been incorporated to provide a continuous diaphragm chord for both topped and untopped decks. Calculations show maximum ultimate diaphragm chord forces of 30 kips for Zone 1 and 60 kips for Zone 2 (including a 1.40 load factor). To ensure that the diaphragm remains elastic under this load, it is suggested that chord reinforcement be sized for loads of 45 and 90 kips in the two zones, respectively. The connection of the chord to the diaphragm can be accomplished as shown in Fig. 4.17.

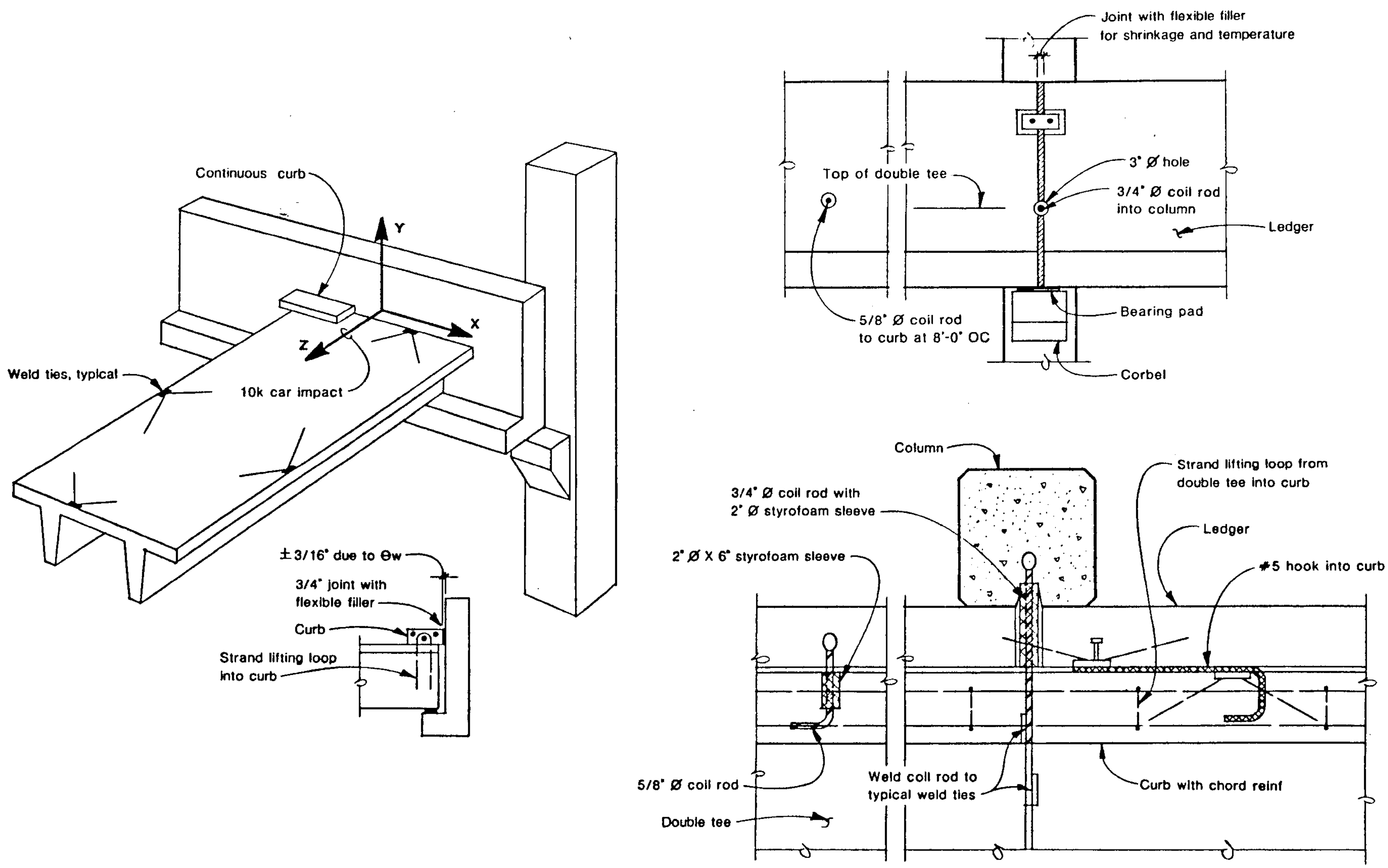


FIGURE 4.17 - DIAPHRAGM CHORD; CAST-IN-PLACE CURB INTEGRATES DOUBLE-TEES AND LEDGERS

4.9.2.2 Double-T to Double-T Joints within Horizontal Decks (Fig. 4.18)

For the Zone 2 structure, parking decks are topped and the nominal reinforcement required for service loads, temperature, and shrinkage, together with the chord reinforcement discussed earlier, provides sufficient strength for seismic loads. For the Zone 1 structure, however, adjacent double-T's are joined only by discrete flange connectors and special attention must be given to their seismic performance requirements.

Maximum panel-to-panel forces in the double-T's occur at the sixth level above grade. Including a 1.40 load factor, ultimate in-plane panel-to-panel shears of 34 kips (Zone 1) in the long direction (i.e., 710 pounds per lineal foot along a 48-foot panel) must be transferred while accommodating relative panel-to-panel rotations about the joint longitudinal axis.

Panel-to-panel rotations are induced by north/south sidesway at locations where the ledger beams, which support the double-T's, are incorporated into the north/south outrigger system. For Zone 1, plastic hinge rotations are on the order of negative 3/4 degree (producing tension in the top fibers) and positive 1/4 degree (producing compression in the top fibers). Note that these are the magnitudes of plastic rotation occurring after the structure has been loaded to the yield point.

4.9.2.3 Double-T to Ledger Joint (Fig. 4.17)

To accommodate the maximum in-plane forces which occur at the sixth level above grade, the double-T to ledger connection is required to transfer in-plane shears of approximately 100 and 200 pounds per lineal foot in Zones 1 and 2, respectively. To maintain diaphragm integrity, in-plane shears should be carried elastically. Thus, it is suggested that the double-T to ledger connections be designed for forces of 150 and 300 pounds per lineal foot in the two zones. In addition, a nominal 10-kip tie is required to sustain vehicle impact normal to the ledger.

Both the shear connection and the impact tie must accommodate end displacements and rotations of the double-T's due to thermal gradient, shrinkage, and live load; they must also accommodate rotations of the ledgers

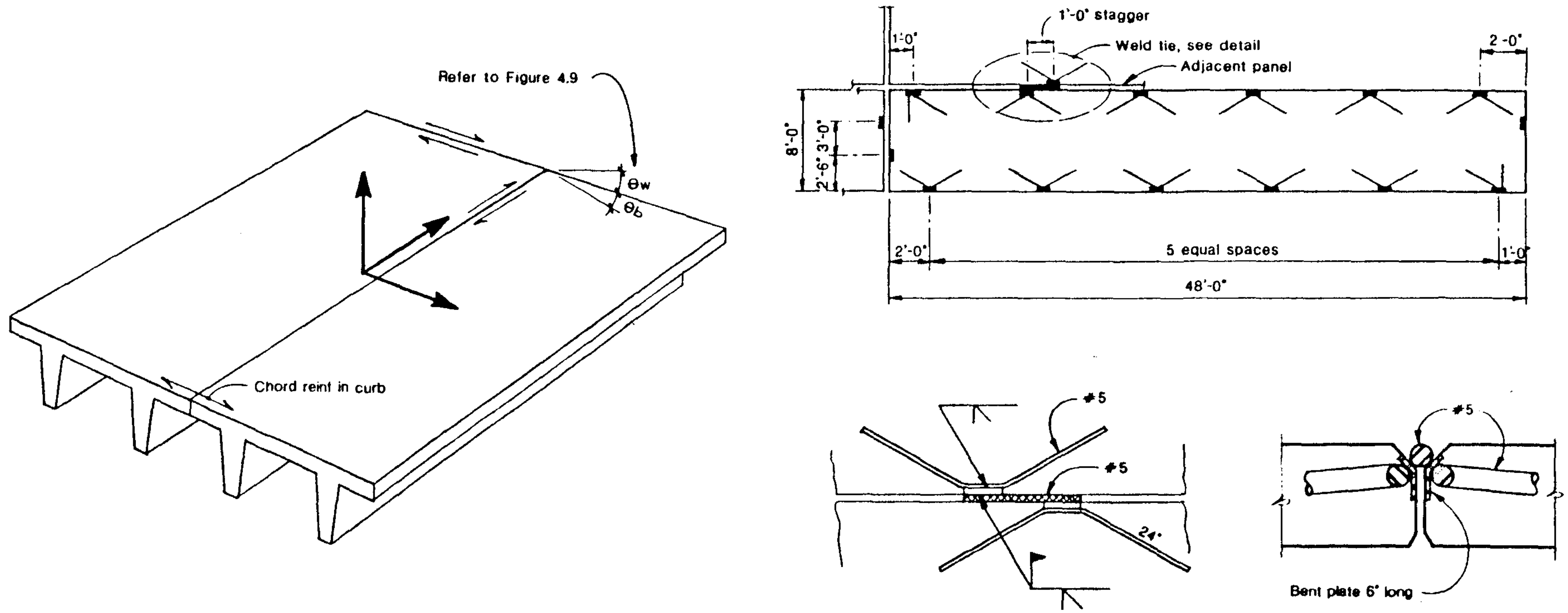


FIGURE 4.18 - DOUBLE-TEE FLANGE-TO-FLANGE CONNECTORS

about their longitudinal axes which occur when the structure sways under east/west excitation. Ledger rotations associated with the structure's plastic displacement during seismic response are estimated as ±0.3 degree for Zone 1 and ±0.5 degree for Zone 2. This produces relative lateral movements across the joint of ±1/8 inch and ±1/4 inch over the 2-foot height of the double-T's for the two zones, respectively.

A suitable detail for the double-T to ledger connection is suggested in Fig. 4.17.

4.9.2.4 Double-T to Spandrel Joint (Fig. 4.19)

Connections across this joint are required to transfer an ultimate in-plane shear of 36 kips per span for Zone 1 or 72 kips per span for Zone 2 from the double-T to the spandrels in the plane of the east/west outrigger walls. To ensure elastic performance, double-T-to-spandrel connections should be sized for loads 50 percent in excess of these values. To resist accidental vehicle impact, a 10-kip tension tie is required between the spandrel and the double-T.

In addition, due to rocking of the outrigger walls, relative vertical displacements between the double-T and the spandrel of about 3/4 inch for Zone 1 and 1-1/2 inches for Zone 2 must be accommodated in those bays where spandrels are mobilized as outrigger beams.

Sidesway under north/south excitation causes the vertical elements to "lean over," producing a rotation of the spandrel with respect to the double-T about the longitudinal axis. Rotation magnitudes are roughly ±1/4 and ±1/2 degree for Zones 1 and 2, respectively. In Fig. 4.19, a connection detail suitable for the double-T to spandrel joint is suggested.

4.9.3 Other Locations

In the examples above, seismic performance criteria for some important interelement connections have been presented. By systematic consideration of the structure's seismic forces and deformations, performance requirements for connections across other joints of interest can be deduced.

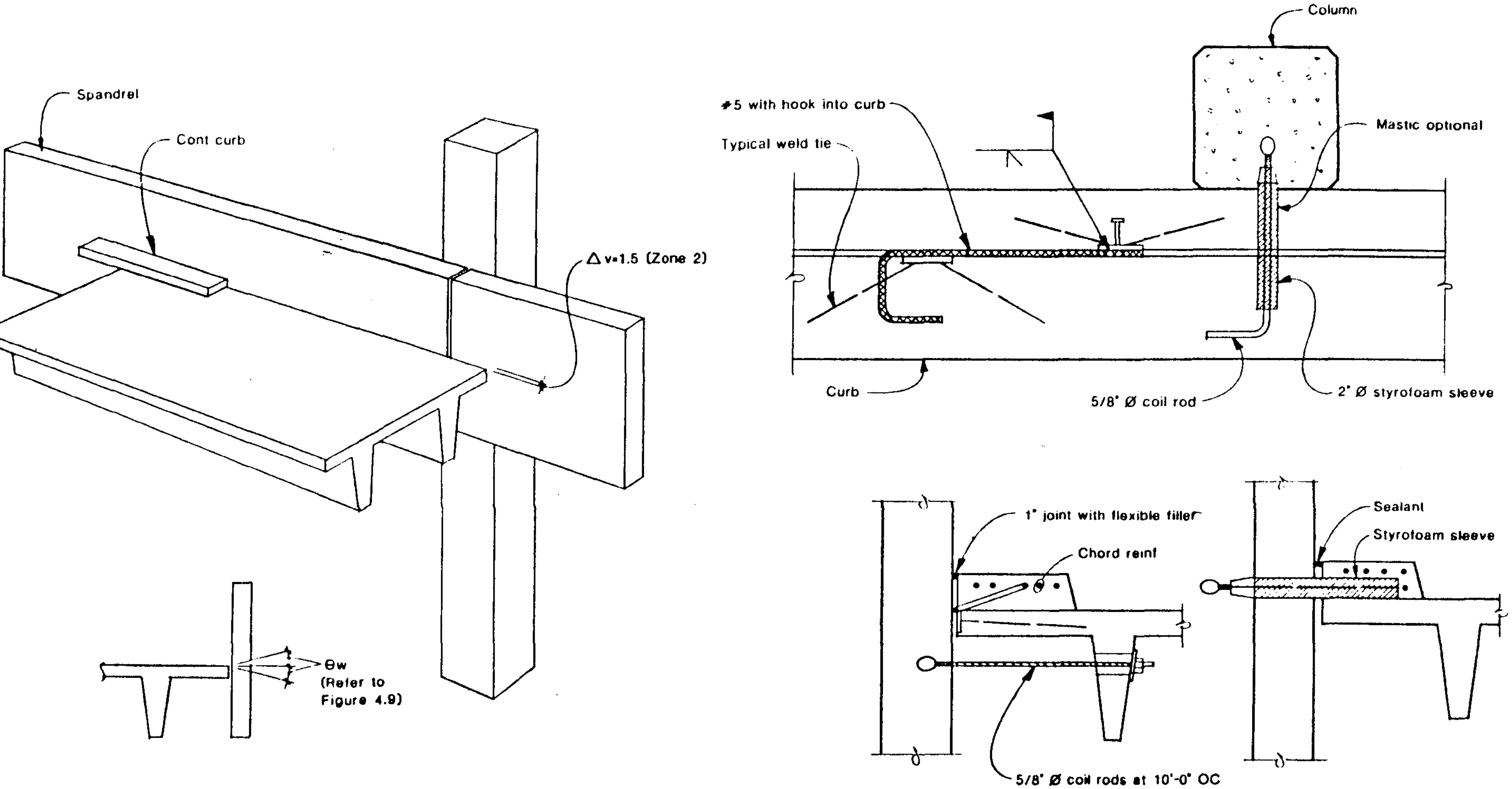


FIGURE 4.19 - SUGGESTED DETAILS FOR -TEE TO SPANDREL CONNECTION

4.10 SUMMARY

The rational methodology for deriving connector performance requirements described in Chapter 3 has been illustrated by application to a precast parking structure with seven levels above grade. Beginning with an investigation of lateral load paths in the original Metro-Space structural system (not intended for use in seismic regions), it was concluded that earthquake behavior could most readily be enhanced by deemphasizing the participation of the ramps and elevator towers in lateral load resistance. An "outrigger wall" system was then developed which satisfies the objective stated in the design methodology of providing a lateral resisting system with simple and predictable inelastic deformational properties while at the same time preserving the simplicity and regularity of the Metro-Space precast building system.

Based on predicted vibration periods, lateral forces for the design ultimate and elastic demand limit states were computed using provisions of the Uniform Building Code and ATC-3, respectively. Then, using the ratio of elastic strength demand to design ultimate strength (i.e., the R-value), the equal energy principle was applied to estimate the maximum plastic displacement at the top of the structure during the code-specified, once-in-a-lifetime earthquake. According to the information presented in Table 3.2, the lateral vibration period of 0.9 second and the R-value of 3.8 indicate that the structure must sustain three fully reversed cycles of loading and deformation for Zone 1 and six cycles for Zone 2 ground shaking.

Kinematic principles were applied to translate the gross lateral movement at the top of the structure into displacements and rotations across individual joints. These values, combined with predicted forces and moments obtained in the lateral load analyses, were used to quantify the seismic performance requirements for some of the important connections in the outrigger walls which form the primary lateral force resisting system, for connections within the parking decks, and for connections between the decks and the outrigger walls.

Conceptual sketches of appropriate connection details were presented for each of the joints. With an emphasis on the presentation of quantitative seismic performance criteria, any connection detail proposed for a given joint can be evaluated by physical testing in Phase 3.

CHAPTER 5

EXAMPLE 2 -- BEARING WALL APARTMENT BUILDING

5.1 DESCRIPTION OF STRUCTURE

The bearing wall structure selected for study was developed by the PCI Committee on Bearing Wall Buildings and is described in detail in Planning and Design of a Precast Concrete Bearing Wall Building [17]. Dubbed the "Chameau Condominiums," the building is 17 stories high with overall plan dimensions of 213 feet 4 inches by 82 feet 8 inches. First-story height is 10 feet 0 inch, while the typical story height is 8 feet 8 inches.

As shown in Fig. 5.1, the structure is of "cross-wall" configuration; load bearing walls are parallel to the shorter plan dimension with hollow-core floor planks spanning between them. Roof and floors are untopped except for a 3/4-inch layer of nonstructural gypcrete leveling compound.

Precast wall panels are one story high and vertical joints occur only where wall panels intersect at right angles, as shown in Fig. 5.2. Intersecting walls are joined by strong connections stiff enough to provide stability during erection but without the capacity of developing full composite action across the vertical joints.

5.2 CONCEPT FOR LATERAL LOAD RESISTANCE

Seismic response of a bearing wall building involves flexural yielding of the walls aligned with the direction of ground shaking. To minimize nonstructural damage and to assure overall integrity of the structure during a strong earthquake, roof and floor diaphragms are intended to remain elastic. This means they must accommodate both the factored ultimate in-plane seismic forces and the corresponding out-of-plane displacements resulting from inelastic action in the walls. Since the diaphragm forces are small, this produces no serious cost penalty.

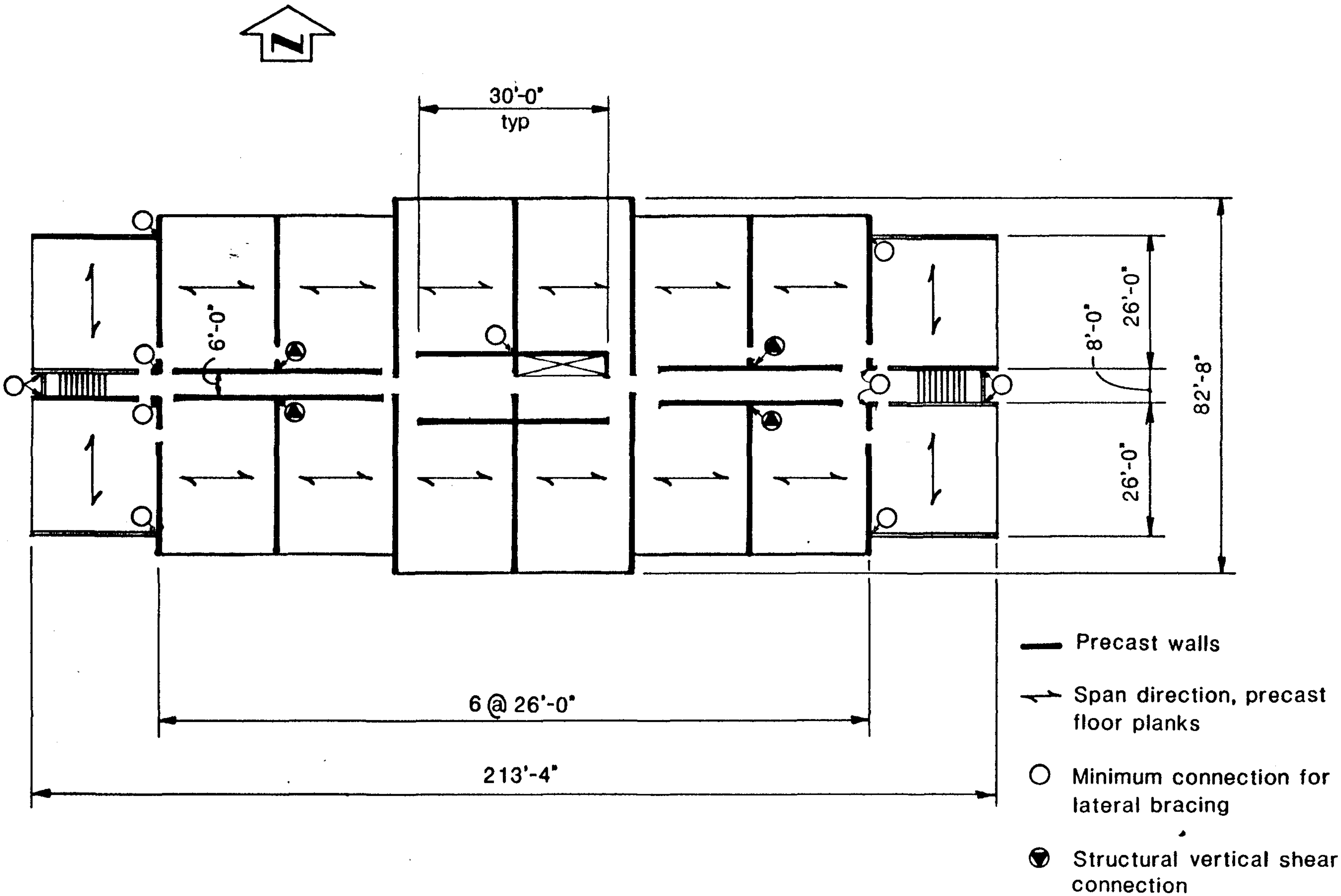


FIGURE 5.1 – BUILDING LAYOUT

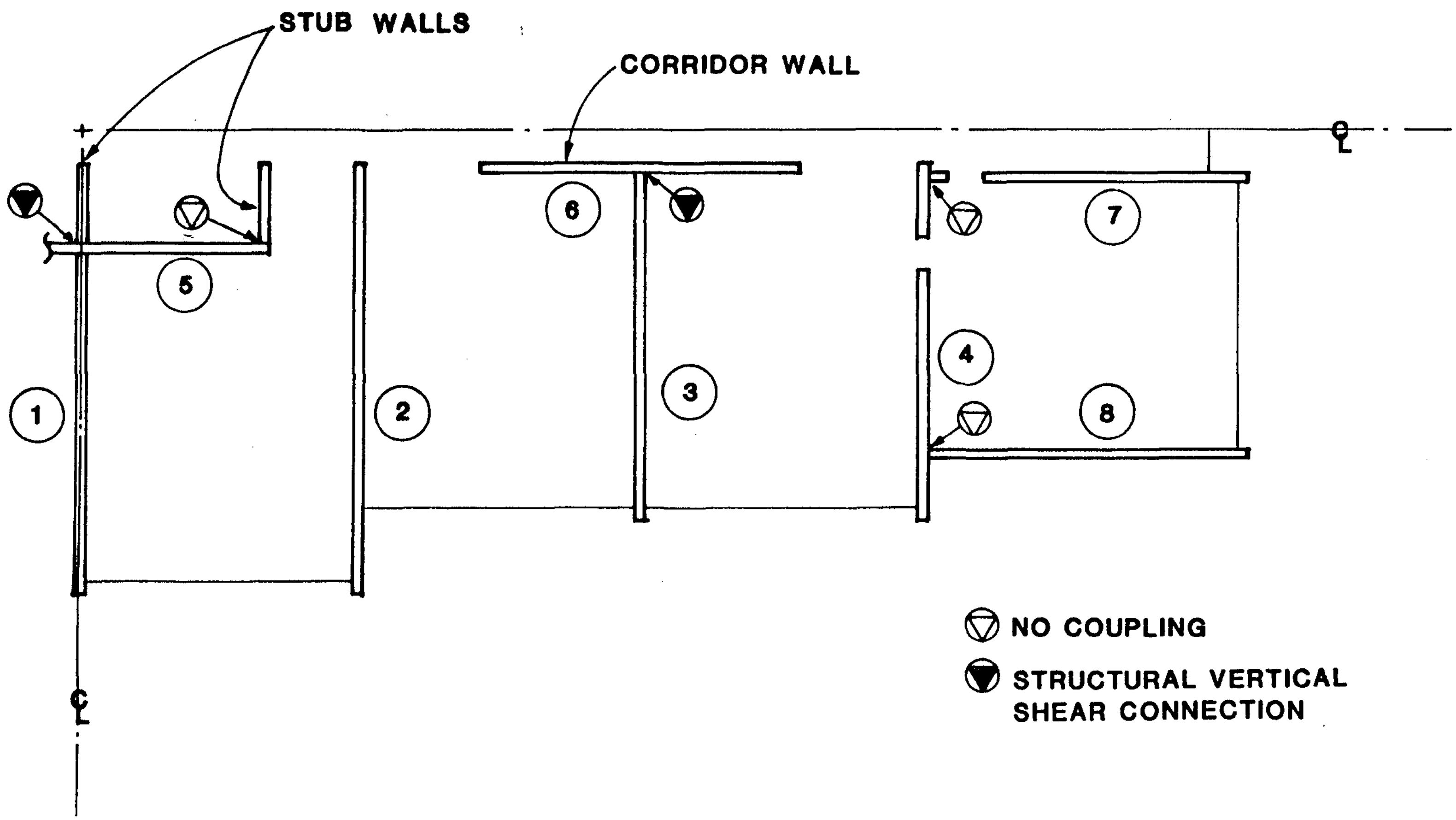


FIGURE 5.2 - IDENTIFICATION OF WALLS FOR PRESENTATION OF SEISMIC ANALYSIS RESULTS

5.3 ELASTIC ANALYSES FOR UNDAMAGED CONDITION

A three-dimensional computer model was used for lateral force analyses. The structure was treated as doubly symmetric in plan and only the southeast quadrant, shown in Fig. 5.3, was modeled explicitly. Effects of the structure's unmodeled portions were incorporated by the following boundary conditions:

Gravity and volumetric loads	Symmetry across X-Y and Y-Z planes
North/south earthquake	Symmetry across Y-Z plane Antisymmetry across X-Y plane
East/west earthquake	Symmetry across X-Y plane Antisymmetry across Y-Z plane

5.3.1 Member Properties

Gross flexibilities and deformation patterns of the post-tensioned structural walls were modeled by three-dimensional beam elements, deformable in bending and shear. Their elastic properties were based on uncracked, gross concrete sections.

A concern in modeling the diaphragms was to obtain a reasonable stiffness value considering cracking effects. The initial, "pre-earthquake" diaphragm stiffness is a function of the degree of cracking due to service loads and shrinkage. During a significant earthquake, additional cracking is likely to occur and the stiffness will decrease. A model based on uncracked properties would overestimate the in-plane forces which arise when compatibility of north/south wall displacements is enforced by the diaphragms.

Accordingly, in-plane stiffness of the untopped diaphragms with respect to relative north/south displacements of the cross walls was calculated assuming diaphragm planks are rigidly clamped against in-plane rotation and longitudinal shear slippage at the walls, but that longitudinal shearing displacements between adjacent planks occur without restraint between the walls.

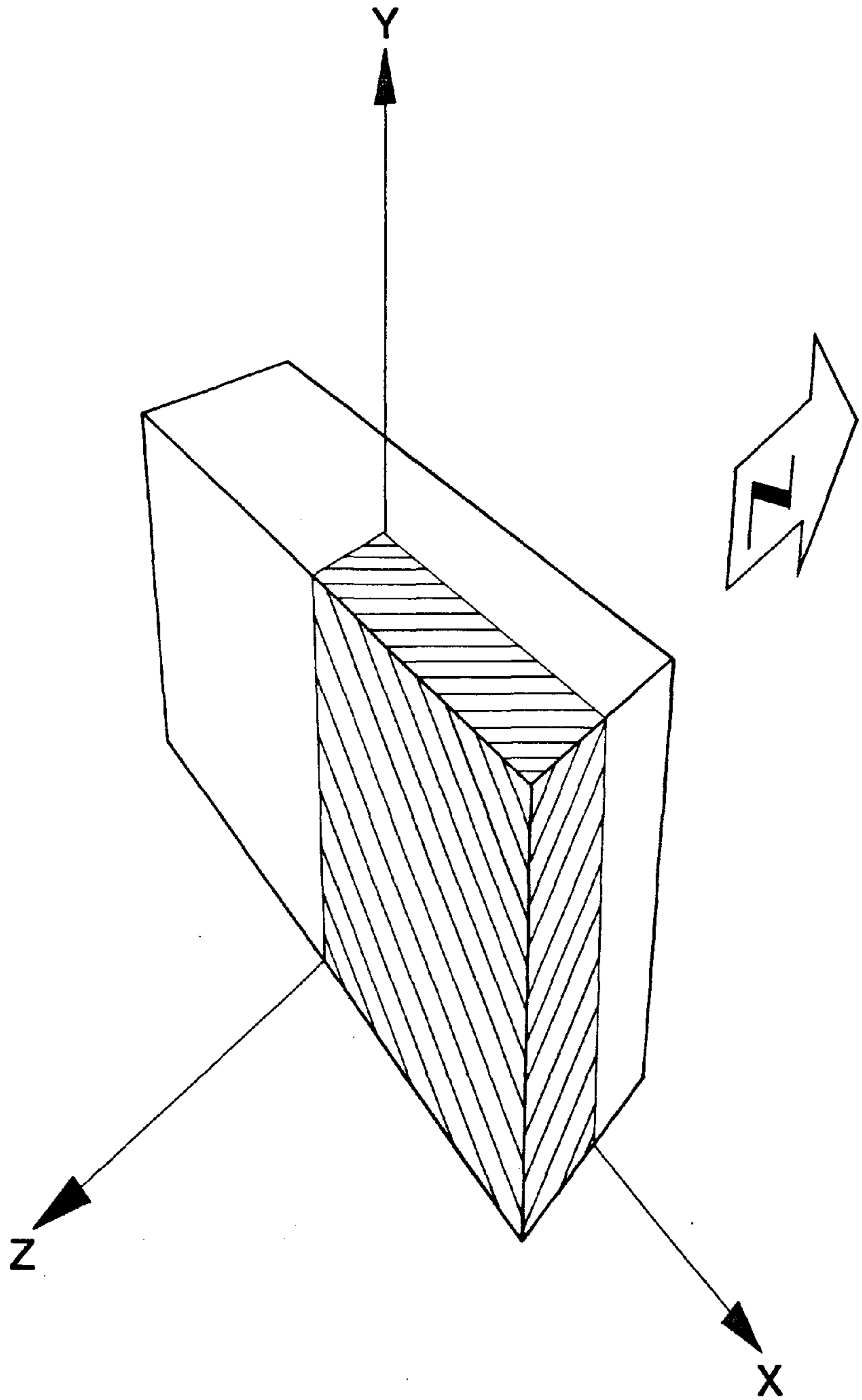


FIGURE 5.3 - MODELED QUADRANT OF STRUCTURE

Thus, the diaphragm in a given bay was assumed to behave like a vierendeel truss in the horizontal plane, with rigid girders where the diaphragm is supported by cross walls, and with each plank acting as a horizontal "column." This gives the diaphragm an in-plane shear stiffness of about one fifth the value corresponding to fully composite behavior of the planks (i.e., no longitudinal shear slippage).

5.3.2 Foundation Properties

Walls were assumed to be supported on spread footings, as shown in Fig. 5.4, and wall bases in the computer model were elastically restrained. Foundation properties were computed using procedures described in [18].

5.3.3 Elastic Analysis Results

Moments and shears in walls and diaphragms for a 1000-kip lateral load distributed over the structure's height in accordance with UBC criteria are presented in Figs. 5.5 and 5.6. When results for both rigid and flexible foundations are examined, it is apparent that effects of foundation flexibility have significant impact on the distribution of loads among the lateral resisting elements. Flexible foundation results have been used here.

5.4 SPECIFICATION OF SEISMIC LOADS

Fundamental vibration periods were determined using the three-dimensional computer model described above. Coupling of north/south cross walls due to out-of-plane stiffness in the floor diaphragms, illustrated in Fig. 5.7, was found to exert a strong influence on predicted period. Because reliable data on out-of-plane stiffness are not available, a period based on an intermediate coupling value was used for seismic load computations. However, because the coupling stiffness is expected to degrade as the building responds to strong ground shaking, effects of out-of-plane diaphragm stiffness have been neglected in all analyses except the elastic period computations. Periods used for seismic load computation were 0.52 second in the north/south direction and 0.80 second in the east/west direction.

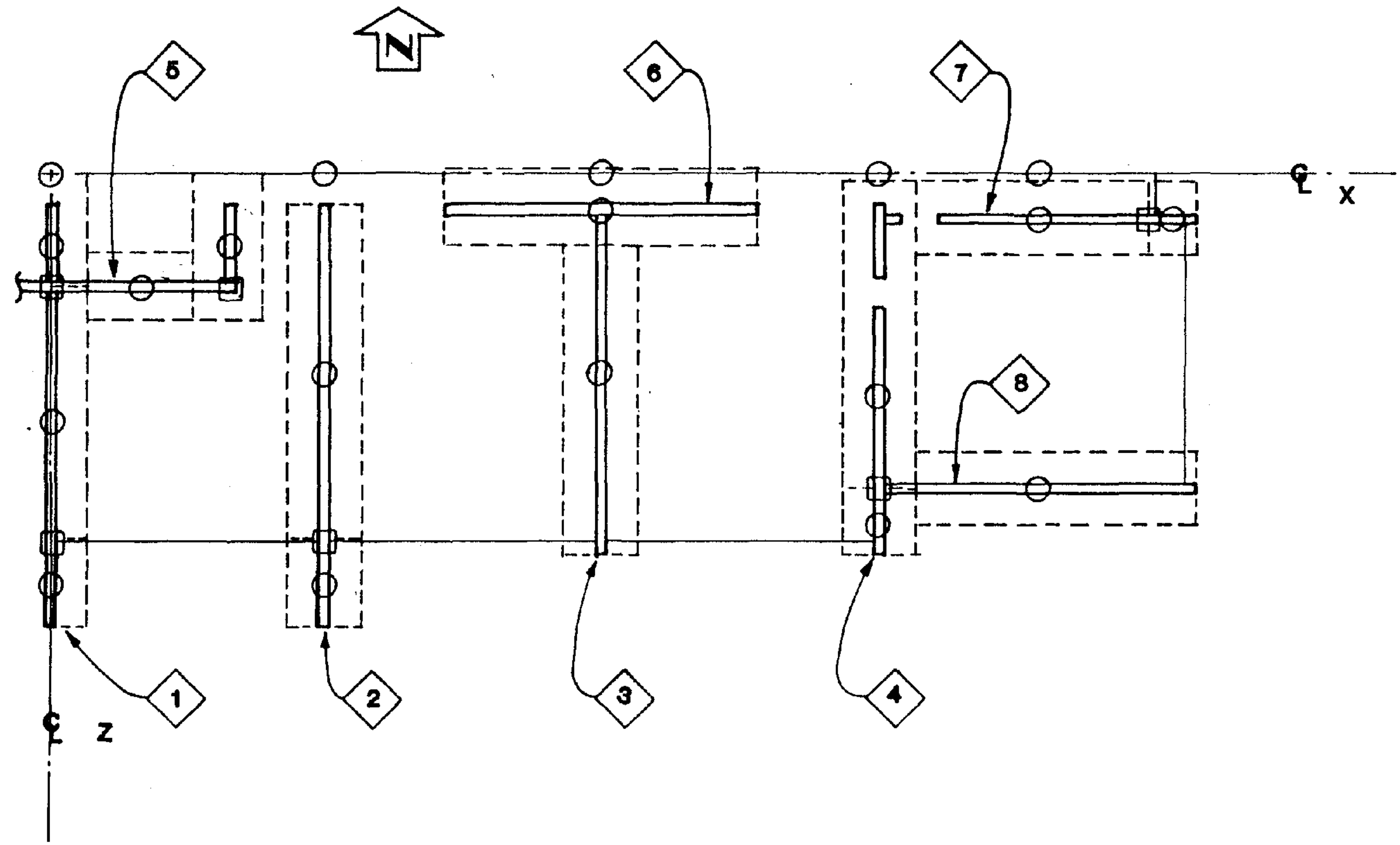


FIGURE 5.4 - ASSUMED FOUNDATION LAYOUT FOR CALCULATION OF FOUNDATION STIFFNESSES

◇ WALL NUMBER

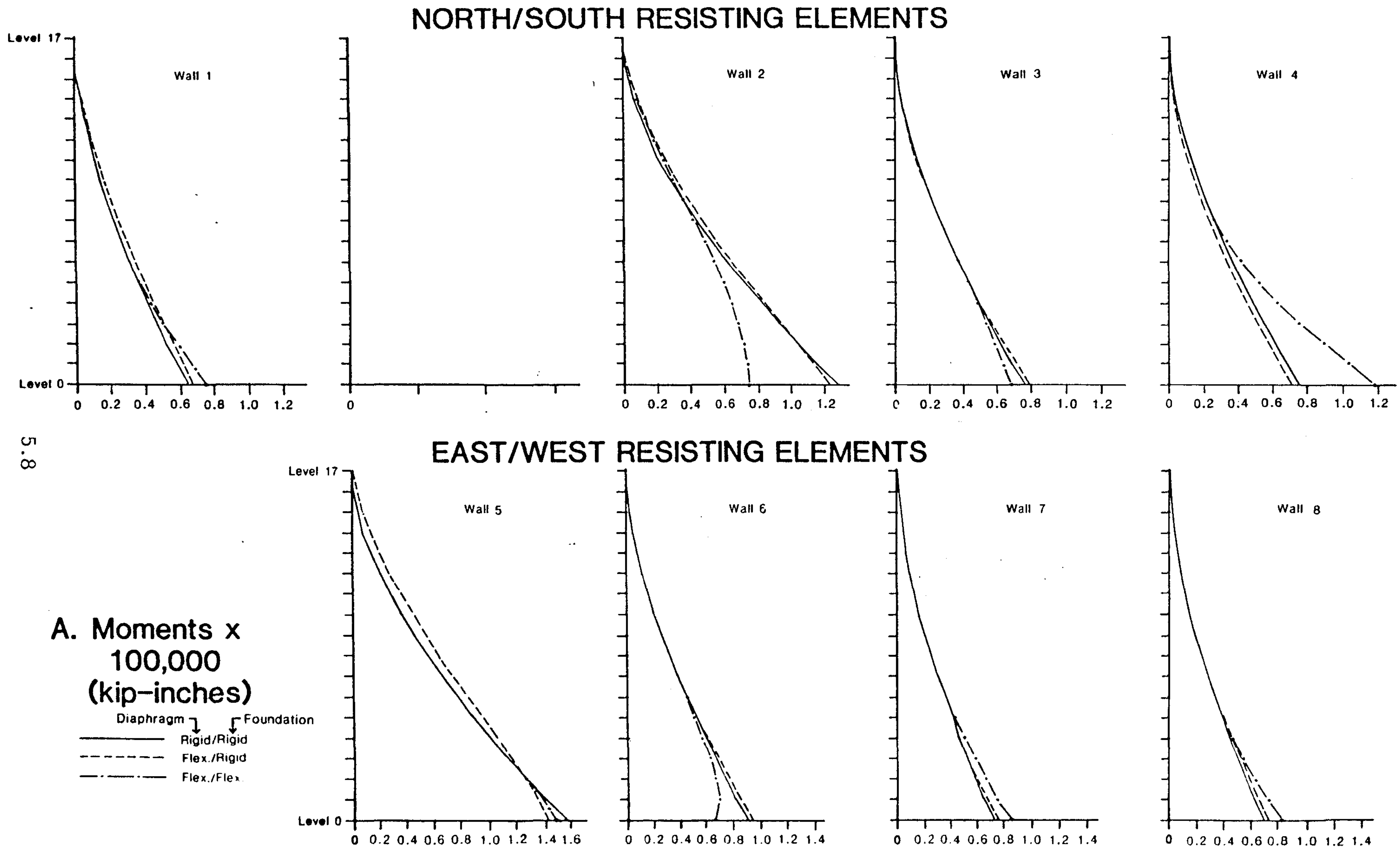


FIGURE 5.5 - ELASTIC ANALYSIS RESULTS FOR 1000 kip BASE SHEAR

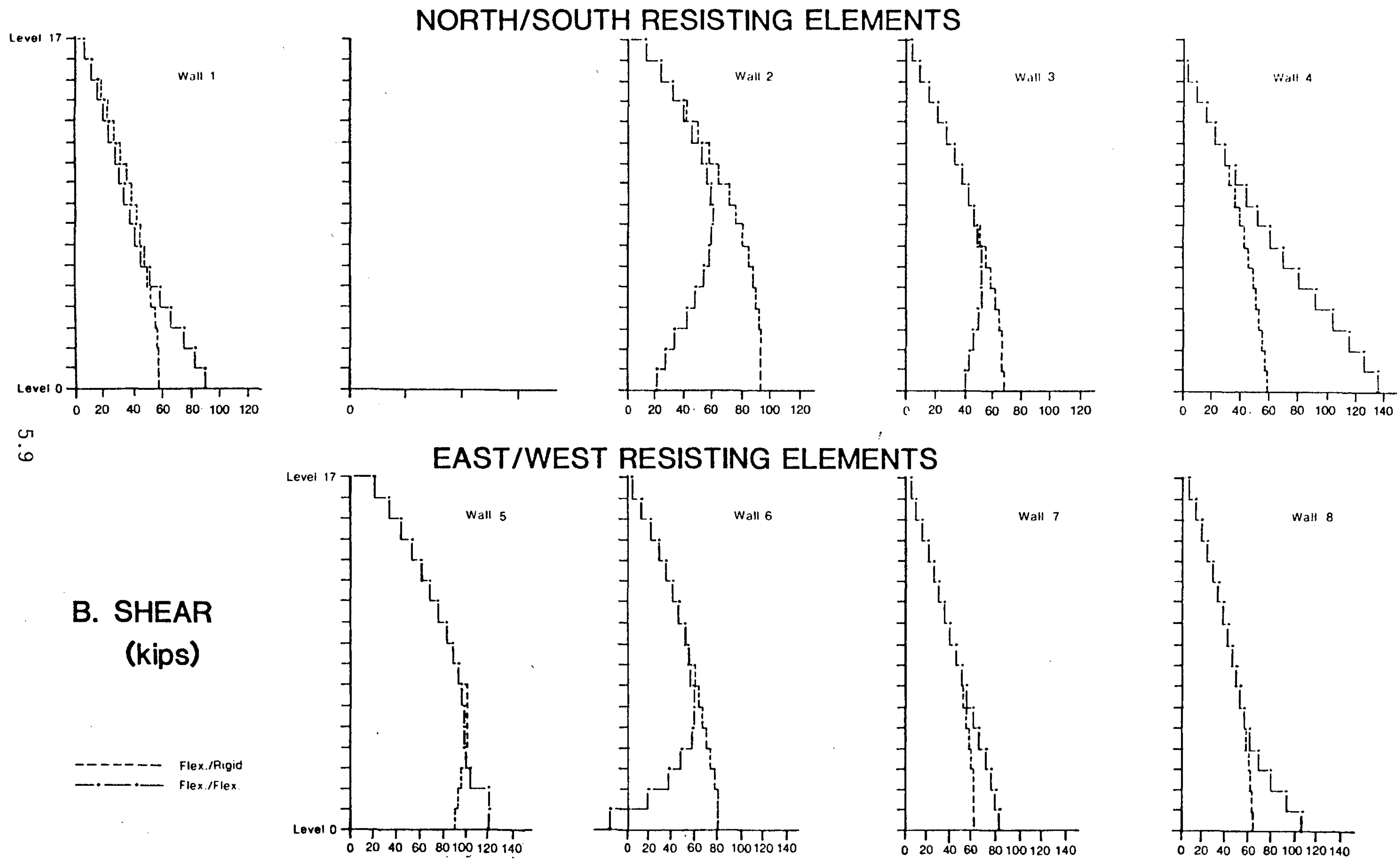


FIGURE 5.5 – ELASTIC ANALYSIS RESULTS FOR 1000 kip BASE SHEAR

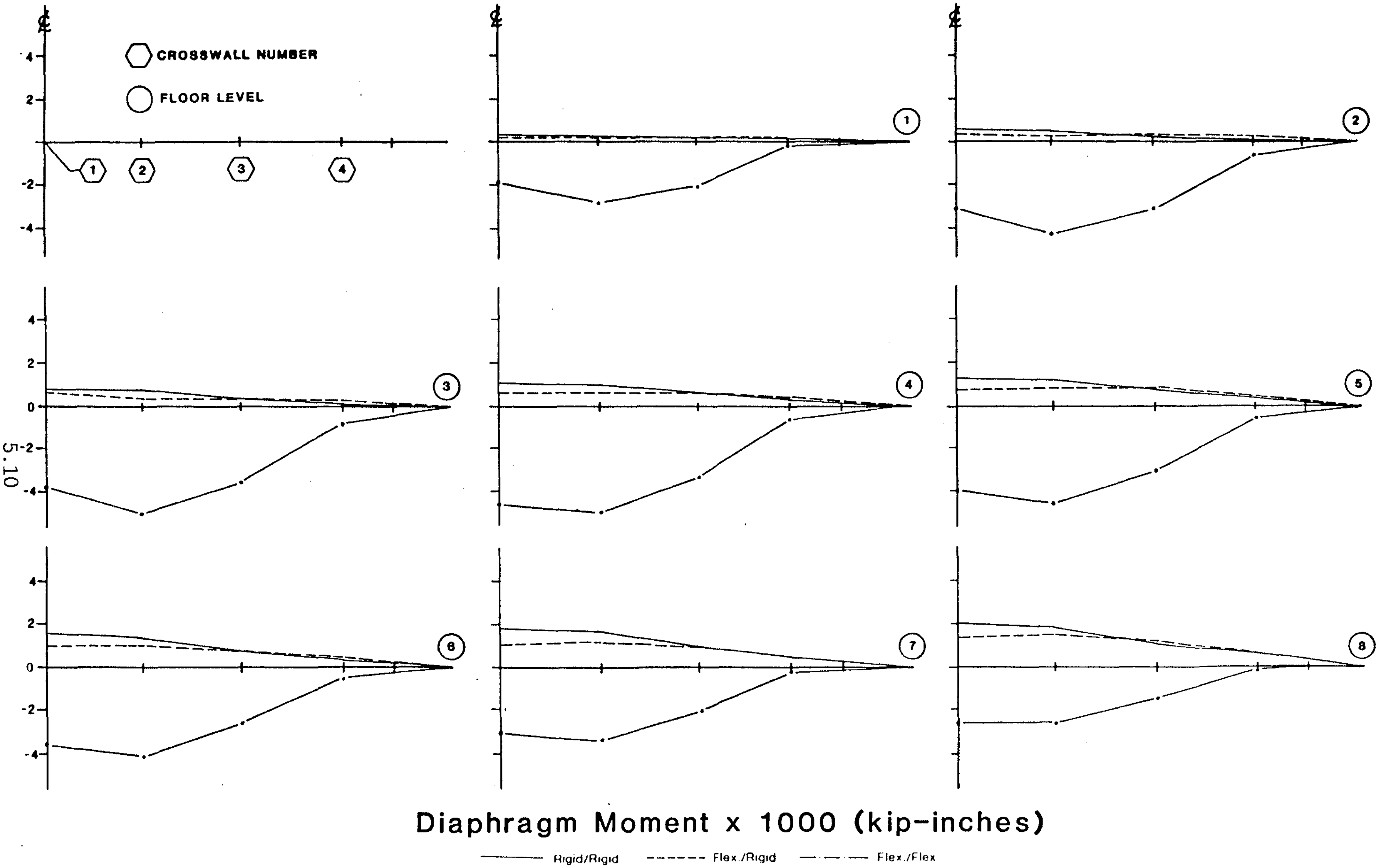


FIGURE 5.6 – ELASTIC ANALYSIS RESULTS FOR 1000 kip BASE SHEAR

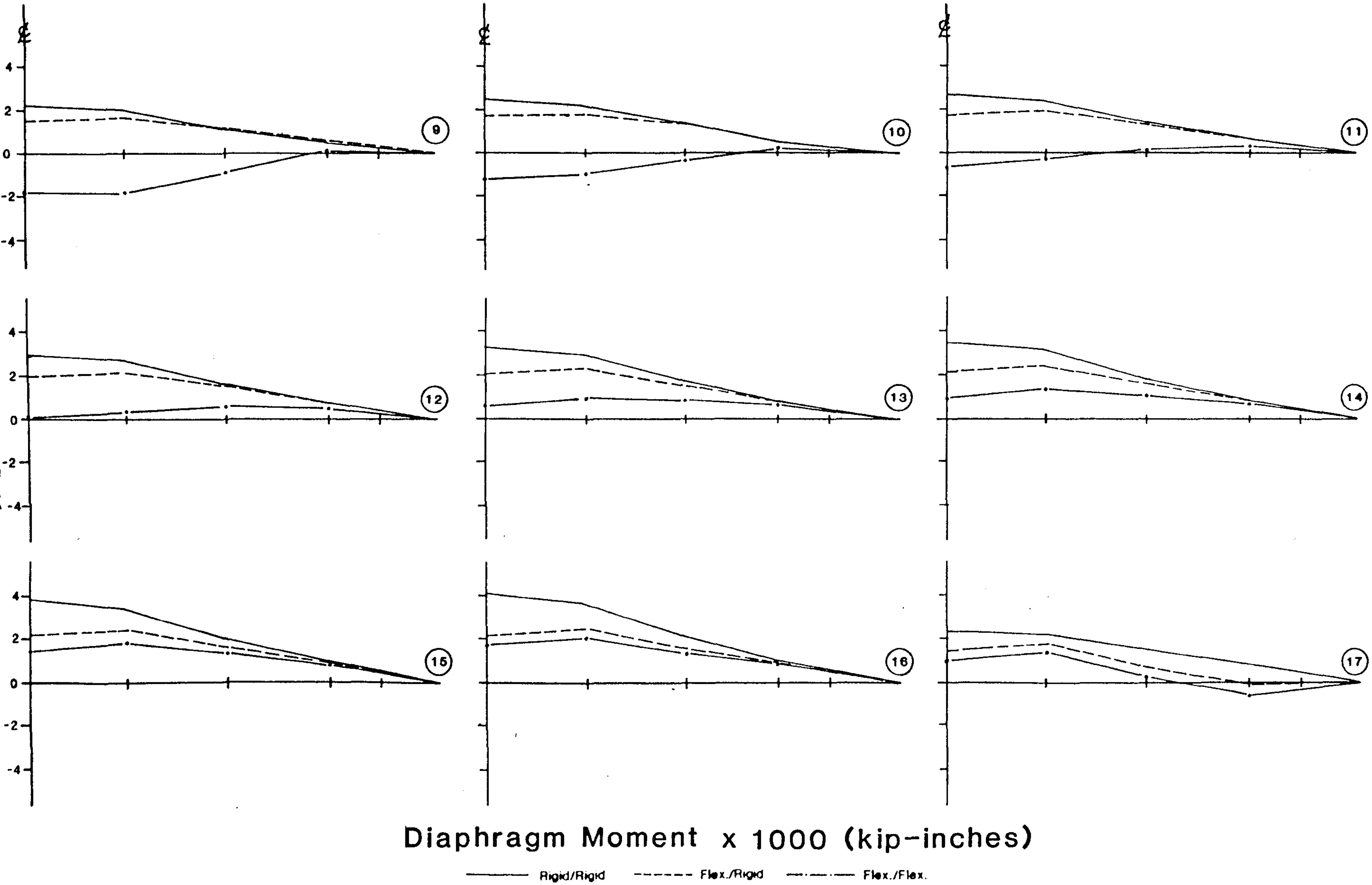


FIGURE 5.6 (cont.)

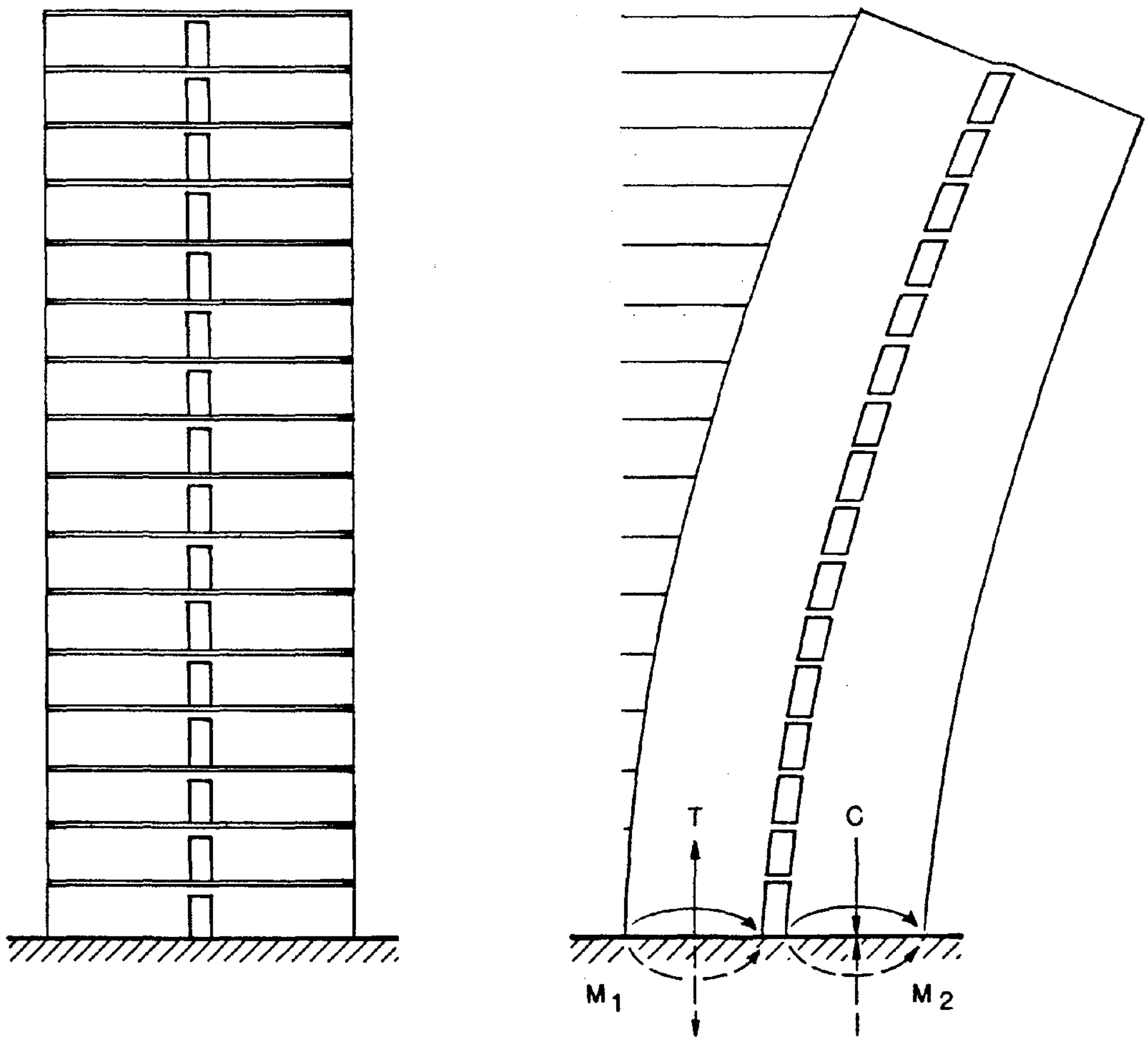


FIGURE 5.7 - LATERAL LOAD EFFECTS IN A COUPLED WALL SYSTEM

It is useful to compare these analytical values with UBC empirical period predictions. The UBC formula depends only on external dimensions of the building, and assumes that the structure is stiffer in the longer (i.e., east/west) plan direction. In the present example, however, the north/south stiffness exceeds the east/west stiffness (due to the building's cross wall configuration). UBC period predictions (0.89 second in the north/south direction and 0.51 second in the east/west direction) are, consequently, in direct conflict with the analytical results.

It was assumed that the structure is located in UBC Seismic Zone 2. ATC-3 excitation parameters are taken as $A_a = 0.10$ and $A_v = 0.15$; the soil type is assumed to be S_3. Factored UBC Zone 2 base shears are presented below, along with ATC-3 elastic strength demands for the two directions:

	Shear (kips)		
	North/South	East/West	
ATC-3 (R=1)	8992	8992	Elastic demand
Factored UBC	3516	2810	Required ultimate inelastic capacity
Ratio	2.6	3.2	R-value

It seems most logical to impose the same ratio of elastic demand to design ultimate strength for north/south and east/west excitation. Thus, although the UBC and ATC-3 values shown above imply different response modification factors in the two directions, UBC forces have been used to define ultimate strength requirements and the strength reduction factor, R, has been taken as 2.6 for computing ultimate inelastic displacement magnitudes in both directions. This implies, in effect, that the elastic strength demand for east/west excitation is 7306 kips instead of the 8992 kips given by ATC.

5.5 ASSIGNMENT OF MEMBER STRENGTHS

Strength requirements have been assigned to the walls based on results of the three-dimensional computer model with flexible foundations and diaphragms. Ultimate flexural capacities of the north/south walls are provided by dead load (80 percent) and post-tensioning (20 percent). Post-tensioning of north/south walls is accomplished by undeformed Dywidag bars, as shown in the table below. Post-tensioning of the east/west walls is similar.

Wall	Ultimate Moment Required	Ultimate Moment Provided	Post-Tensioning	
1	29,494	30,579	Two	1-inch rod each end
2	24,949	24,276	One	1-inch rod each end
3	22,710	23,464	Two	1-inch rod each end
4	28,810	27,973	Three	1-1/4 inch rod each end

Notes

1 Moments are in kip-ft.

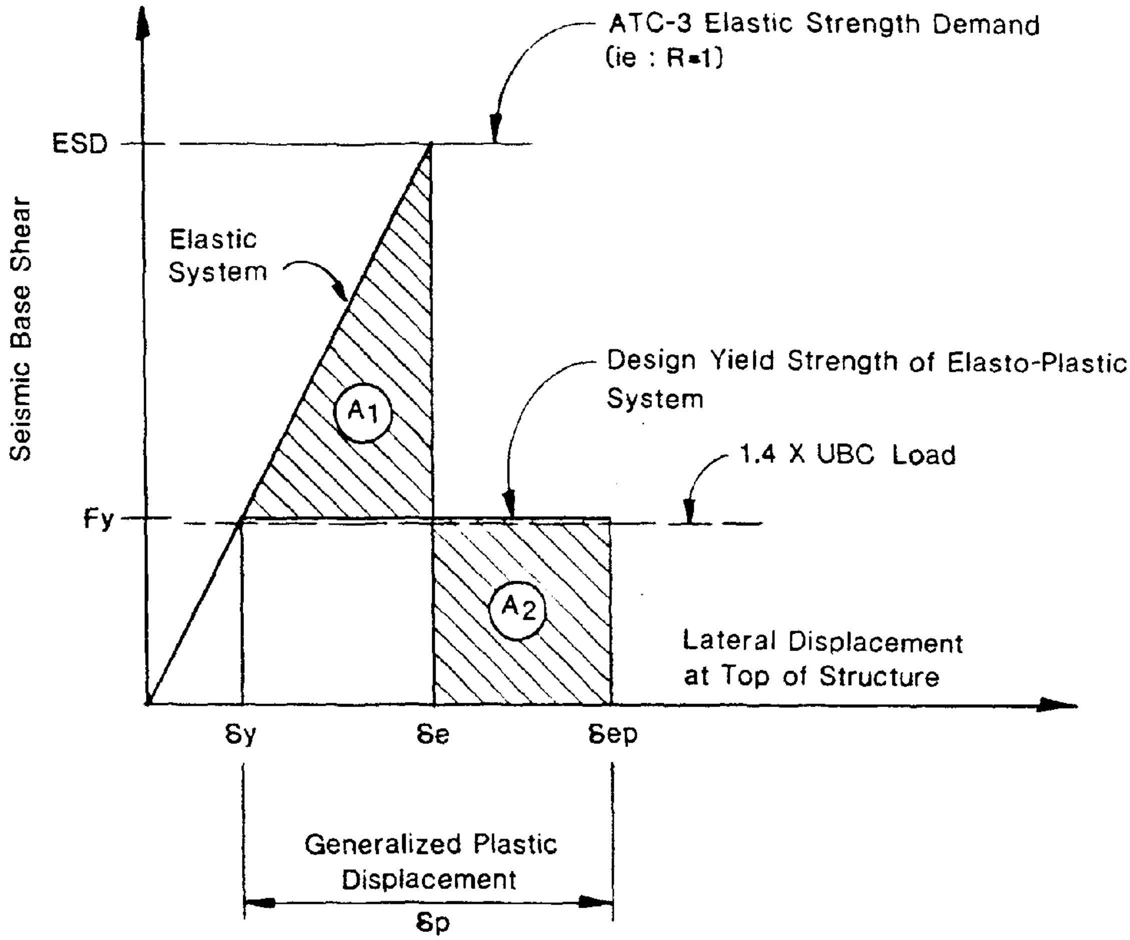
2 Required moments correspond to factored UBC Zone 2 base shear.

3 Provided capacity includes appropriate ϕ-factor.

Note that if a geometrically identical copy of this structure were being considered for Zone 1 instead of Zone 2, UBC seismic design forces would decrease by 50 percent. Because resistance to the opening of horizontal wall joints is provided mainly by dead load, however, the flexural capacity of the walls would decrease only by about 20 percent, even if all the post-tensioning were eliminated.

Thus, while the code-required overturning resistance would be halved, the walls would "automatically" retain about 80 percent of their Zone 2 flexural capacity. With reference to Fig. 5.8, this means that instead of having a yield strength slighly in excess of 1.4 x Zone 1 UBC, as would typically be established by design, unavoidable gravity overturning resistance would produce a "yield" strength of (0.8) x 1.4 x Zone 2 UBC = 1.6 x 1.4 x Zone 1 UBC. In other words, the structure would remain linearly elastic in flexure for lateral loads up to 60 percent higher than the ultimate values specified by the code.

In effect, the design R-value becomes 2.6/1.6 = 1.6 for UBC Zone 1, and required shear capacities of horizontal wall joints are governed by the



Response Modification Factor

$$R = \frac{ESD}{Fy}$$

Ductility Factor

$$\mu = \frac{\delta ep}{\delta y}$$

$$A_1 = A_2$$

$$\boxed{\mu = \frac{R^2 + 1}{2} \qquad \delta p = \delta y \left(\frac{R^2 - 1}{2}\right)}$$

FIGURE 5.8 - EQUAL-ENERGY PRINCIPLE FOR ESTIMATING MAXIMUM SEISMIC DISPLACEMENTS OF AN ELASTO-PLASTIC SYSTEM

inherent, gravity-induced flexural capacity of the walls rather than the code-specified lateral design forces.

Strength requirements for the other major structural elements require further consideration. Because they are essential to the structure's overall stability, it is desirable to avoid damage to the foundations and grade beam system, and to avoid yielding of the diaphragms under in-plane loads. Thus, to increase the likelihood that inelastic action will be confined to the walls, it is suggested that diaphragms and foundations be provided with an elastic strength 25 percent in excess of the analytically predicted requirements.

5.6 PREDICTION OF GLOBAL INELASTIC DISPLACEMENTS

As described in Chapter 3 and illustrated in Fig. 5.8, global inelastic displacements are estimated by applying the "equal energy principle" to the three-dimensional structural model. As shown in Figs. 5.9 and 5.10, roof level plastic displacements of 8-3/4 inches and 12-1/2 inches are predicted for the north/south and east/west directions, respectively.

In this calculation, elastic strain energies are summed for the walls parallel to the direction of loading. Because diaphragm flexibility was included in the computer model, predicted lateral displacements of the walls differ, although by an insignificant amount.

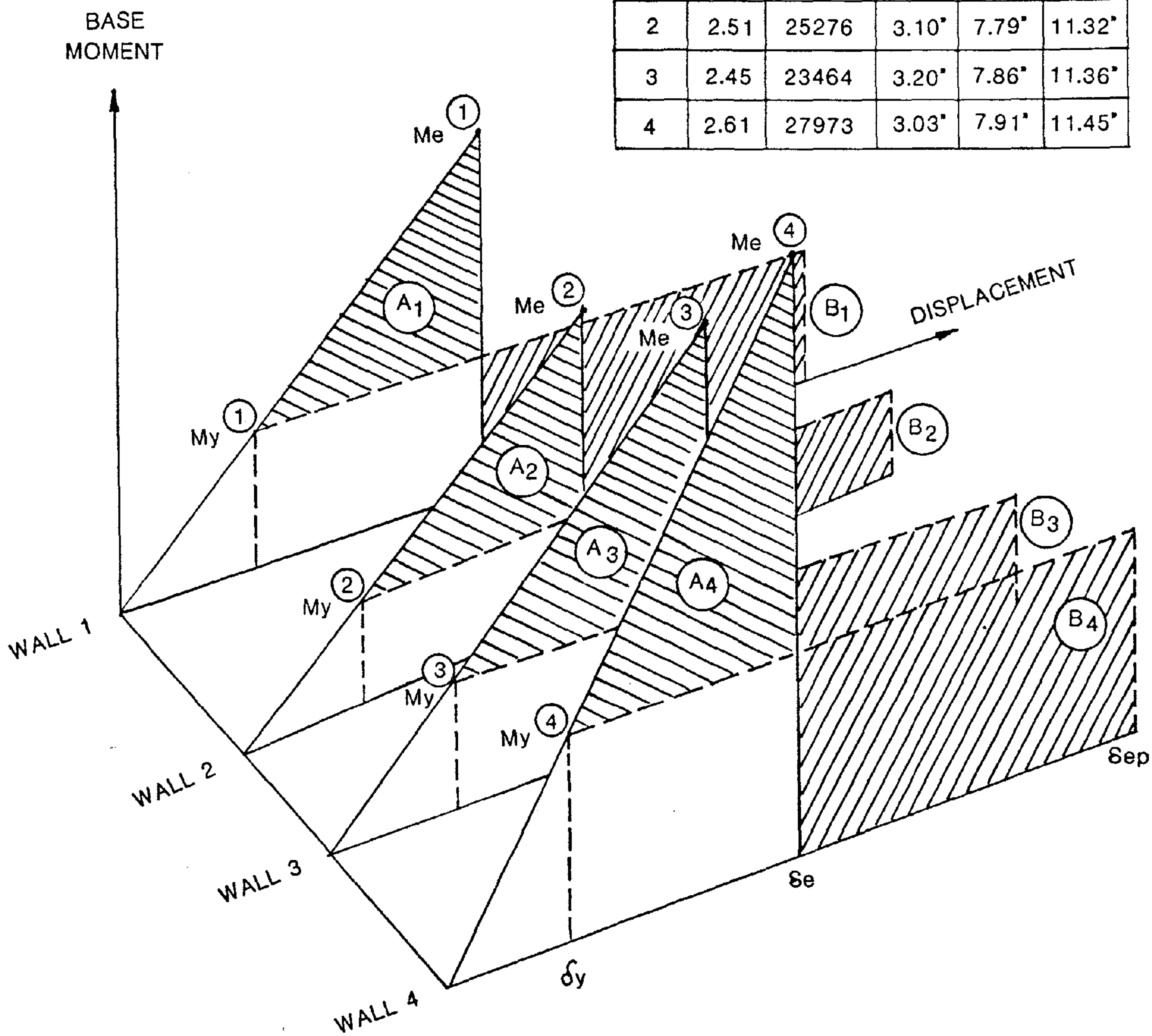
5.7 KINEMATIC ANALYSIS OF POST-YIELD BEHAVIOR

Lateral deflection at the top of the structure was obtained in the calculations described above. Individual joint rotations can now be estimated, based on rational assumptions concerning the distribution of cracking among horizontal joints over the height of the walls.

The total elastoplastic deflection at the wall top includes elastic flexural deformations of the uncracked regions, effects of elastic foundation rotation, and rotations of the cracked joints. Joint rotations include elastic and inelastic components. Consistent with the assumption of elastic/perfectly plastic behavior, the plastic displacement increment at the top of the wall corresponds to plastic strains in steel connections crossing the joint plane.

Wall	R	My(K.ft)	δy	δe	δep
1	2.42	30579	3.22"	7.79"	11.30"
2	2.51	25276	3.10"	7.79"	11.32"
3	2.45	23464	3.20"	7.86"	11.36"
4	2.61	27973	3.03"	7.91"	11.45"

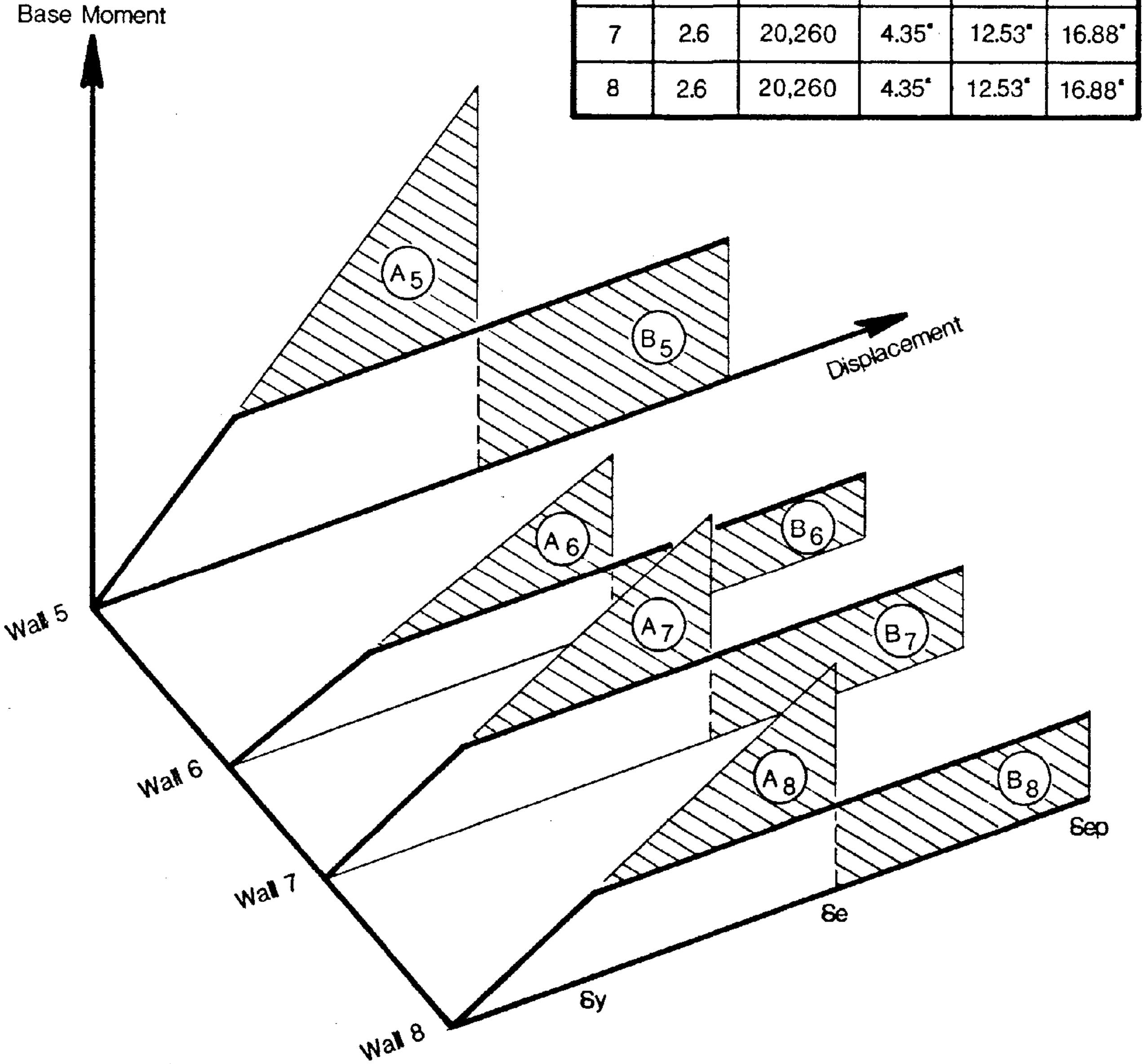
$Me = R * My$



By Equal-Energy Principle, $\sum_{i=1}^{4} Ai = \sum_{i=1}^{4} Bi$

FIGURE 5.9 - INELASTIC SEISMIC BEHAVIOR OF N/S CROSSWALLS

Wall	R	My (K. ft.)	δy	δe	δep
5	2.6	35,000	4.35"	12.53"	16.88"
6	2.6	15,500	4.35"	12.53"	16.88"
7	2.6	20,260	4.35"	12.53"	16.88"
8	2.6	20,260	4.35"	12.53"	16.88"



By Equal - Energy Principle $\sum_{i=5}^{8} Ai = \sum_{i\ 5}^{8} Bi$

FIGURE 5.10 - INELASTIC SEISMIC BEHAVIOR OF E/W WALLS

If the wall is assumed to rotate about its bottom corner at the compression end of the wall-to-footing joint, the crack width at the tension end could be computed by similar triangles, as shown in Fig. 5.11. For a lateral displacement of 8.5 inches at the roof, a vertical displacement of about 2 inches is predicted at the tension end of the wall.

Rather than concentrating at the wall-to-foundation joint, however, the plastic rotation is expected to distribute among the four lowest horizontal joints, as shown in Fig. 5.12. In Table 5.1, extreme fiber plastic elongations are estimated, assuming plastic rotations distribute 40, 30, 20, and 10 percent among joints at the foundation, Levels 1, 2, and 3, respectively. This assumed behavior is generally consistent with analytical results described by Mueller [13].

TABLE 5.1. ESTIMATED PLASTIC ELONGATIONS
(Total Height of Walls = 148.72 Feet)

Assumed Distribution of Plastic Deformation
Over Horizontal Joints in First Three Stories

Wall Number	Length (feet)	Extreme Fiber Plastic Elongation (inches) Level 0	Level 1	Level 2	Level 3
North/South					
1	38.33	0.83	0.62	0.42	0.21
2	38.33	0.85	0.64	0.42	0.21
3	32.33	0.71	0.53	0.35	0.18
4	32.33	0.73	0.55	0.37	0.18
East/West					
5	36.00	1.21	0.91	0.61	0.30
6	30.00	1.01	0.76	0.51	0.25
7	28.67	0.97	0.72	0.48	0.24
8	28.67	0.97	0.72	0.48	0.24

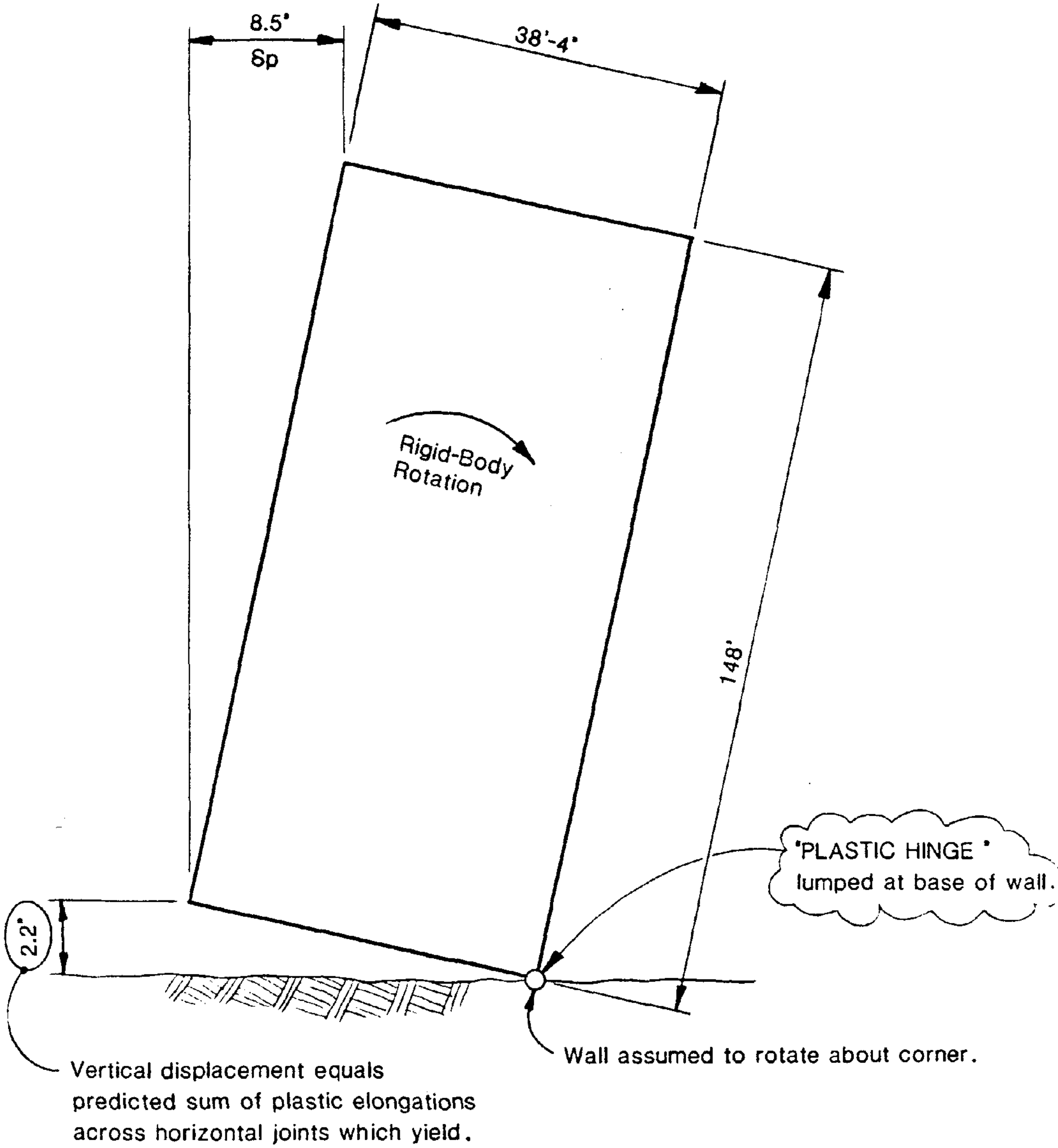
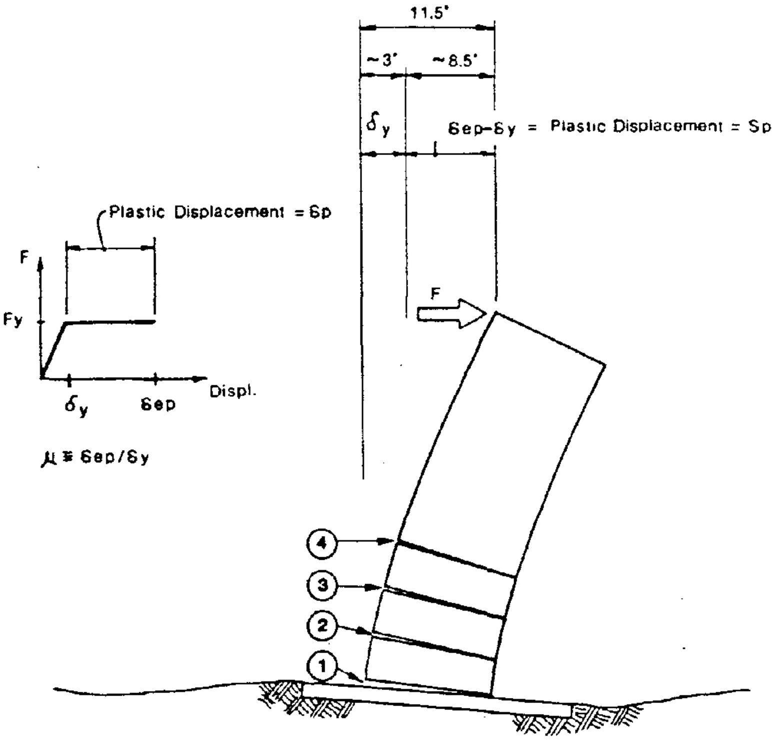


FIGURE 5.11 - KINEMATICS OF ISOLATED WALL UNDERGOING PLASTIC DEFORMATION



Assumed Distribution of Fiber Elongations Among Yielding Horizontal Joints	
4	(2.2) (0.10) = 0.22"
3	(2.2) (0.20) = 0.44"
2	(2.2) (0.30) = 0.66"
1	(2.2) (0.40) = 0.88"
TOTAL	2.20"

FIGURE 5.12 - ELASTO-PLASTIC DEFORMATIONS OF A TYPICAL WALL

5.8 NUMBER OF CYCLIC LOAD REVERSALS TO BE SUSTAINED

Fundamental periods are 0.5 and 0.8 second in the north/south and east/west directions, respectively. Cantilever walls in both directions have been designed to an R-value of 2.6. For these values, Table 3.2 in Chapter 3 indicates that five fully reversed loading cycles must be sustained for Zone 2 ground shaking.

5.9 SPECIFICATION OF CONNECTOR PERFORMANCE CRITERIA

Global behavior of the bearing wall structure has been described in the preceding sections. Forces and deformations of the complete structure will now be interpreted with regard to their implications for connectors in representative joints between precast components.

Joints have been selected for discussion based on relative severity of the consequences stemming from connector failure. Highest importance was placed on joints at which relative member displacements or rotations could endanger overall structural stability. Secondary importance was assigned to joints at which relative member displacements or secondary forces brought about by incidental restraint of relative displacements could result in significant nonstructural damage.

In the sections that follow, selected joints are described in terms of their geometries and the intended functions of the associated connection. Necessary conditions for the successful performance of these functions are specified based on forces and deformations computed in the global structural analyses and based on engineering judgment. These specifictions constitute "rational connector performance criteria."

5.9.1 Load Bearing Wall to Hollow-Core Floor Joint (Fig. 5.13)

5.9.1.1 Description

Probably the most crucial detail in this bearing wall building is the joint at which hollow-core floor planks frame into the load-bearing walls. In this structure, the "platform" joint, typical of bearing wall construction in the United States, is used.

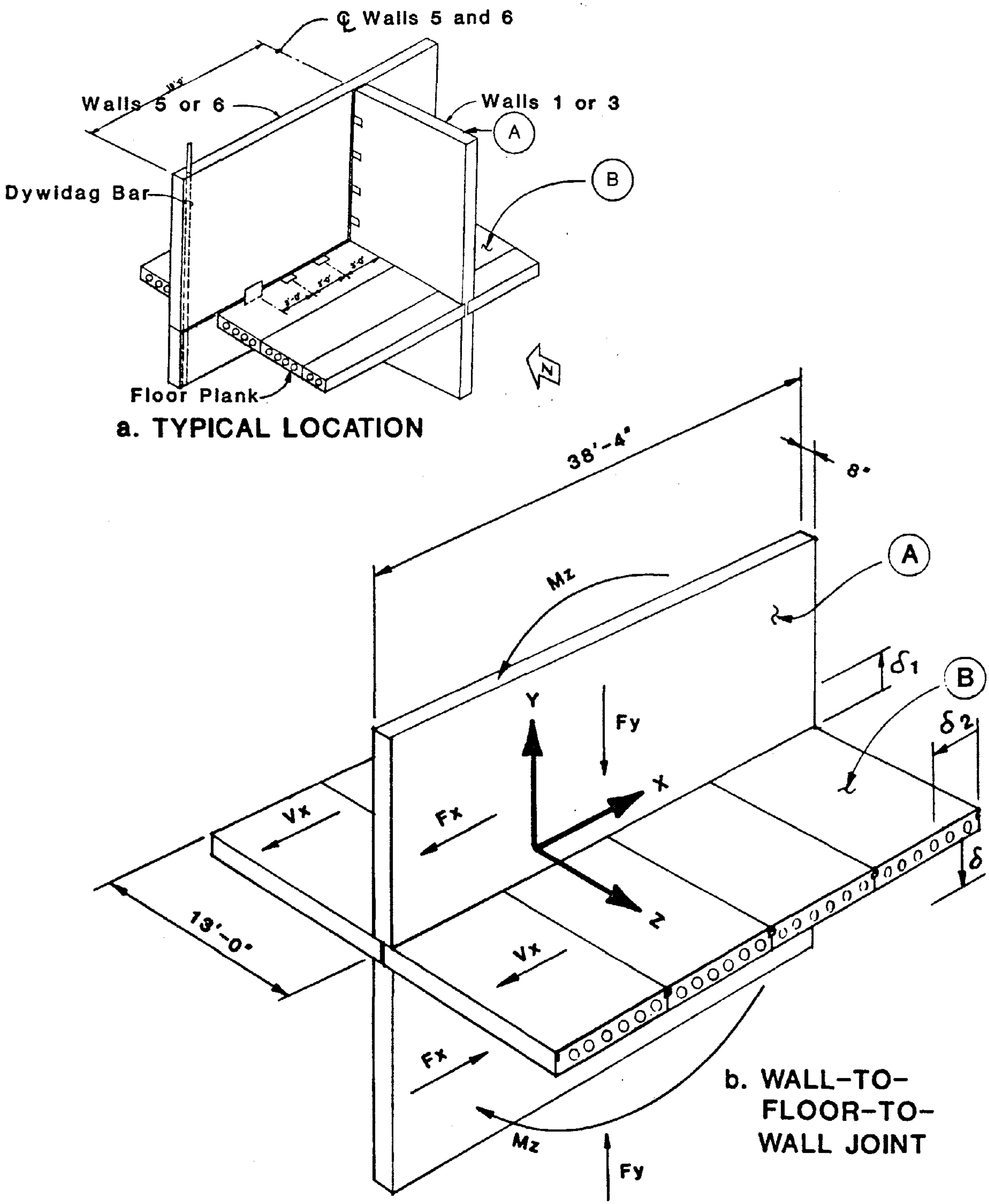


FIGURE 5.13 - LOAD-BEARING WALL TO HOLLOW-CORE FLOOR JOINT

As shown in Fig. 5.13, hollow-core planks bear on top of precast wall units of the story below. Wall units of the next story bear on the planks. Details vary among precast producers but typically, for an 8-inch wall thickness, plank ends overlap the wall edge by 2-1/4 to 3 inches, leaving a 2- to 3-1/2-inch gap between the ends of planks spanning adjacent bays. The gap between plank ends and the volume of cores extending about 2 inches into the span from the wall faces are filled with grout.

Fig. 5.13 depicts a subassemblage that could be tested in a laboratory. One of the north/south cross walls is shown along with a half span length of diaphragm from the bay to either side. Note that in the complete structure pairs of these walls occur side by side, separated by a 6-foot-wide corridor. As shown by δ_1, δ_2, and δ_3 in the sketch, relative member displacements and rotations are described in a manner convenient for test purposes.

5.9.1.2 Joint Actions

Connections within this complex joint are intended to transmit forces in all three coordinate directions shown in Fig. 5.13. In the Y-direction, dead and live load forces must pass through to the wall below. In the X-direction, lateral shears due to wind and seismic loads must be transferred from the floor and wall above into the wall below. Chord forces in the Z-direction, due to diaphragm action, must be transmitted from one span through the plane of the wall to the adjacent span.

In addition to forces, connections within the joint are intended to transmit wall moments about the Z-axis and plank end moments about the Y-axis. The walls are primary elements of the lateral force resisting system and are expected to yield in the lower stories during a major earthquake. Because inelastic action is expected to concentrate in the joints, plastic deformations associated with Z-rotation must be sustained by the connections.

To tie the structure together, the diaphragms are intended to remain elastic during a major earthquake. Thus, Y-moments at the plank ends, in addition to the Z-forces desribed above, should be carried elastically.

5.9.1.3 Seismic Performance Criteria

Two regimes can be identified for platform joints in the Chameau Condominiums: the lower stories, where axial loads, horizontal shears, and in-plane moments in the walls are highest but diaphragm forces are relatively low; and the upper stories, where axial loads, horizontal shears, and in-plane moments in the walls are relatively low but diaphragm forces are highest.

Seismic performance criteria for platform connections in these two regimes are summarized in Tables 5.2.a and 5.2.b. Each of these tables contains two sections, one pertaining to actions of the wall above relative to the wall below, and the other to actions of the diaphragm relative to the wall.

a. Near Base of Structure

Referring to Fig. 5.13 and Table 5.2.a, consider the behavior of a platform joint near the base of the structure during a strong earthquake. Connections within this joint must transmit a horizontal shear, F_x, of about 300 kips from the wall above to the wall below. Simultaneously, an axial load, F_y, of 2200 kips and an in-plane bending moment, M_z, of 30,000 kip-ft must be carried through the joint region.

The ultimate moment, M_z, causes yielding of vertical wall connections through the joint, resulting in a rocking action. Plastic elongation across the joint may be assumed to vary linearly from 7/8 inch at one end of the wall to zero at the other, as described by δ_1 in the figure. Note that this value is the plastic deformation increment and is additive with the joint movement which may occur while vertical wall connections are still in the elastic range.

Now consider actions of the diaphragm with respect to the wall. In the lower stories, diaphragm shears parallel to the wall arise primarily due to compatibility effects. The diaphragm acts to enforce equality of lateral displacements between the transverse walls. Due to differing wall moments and footing stiffnesses, deflected shapes of the walls would differ were the diaphragm not present. It was found in the analyses that relative wall displacements with diaphragms inactive in the first five stories were very small (less than 0.01 inch). However, due to the stiffness of walls and diaphragms, preventing these movements generates significant forces.

TABLE 5.2.a. SEISMIC PERFORMANCE REQUIREMENTS FOR LOAD-BEARING WALL TO HOLLOW-CORE FLOOR JOINT NEAR BASE OF STRUCTURE

	Actions of Wall Above Relative to Wall Below Degree of Freedom					
	Translation			Rotation		
Action	X	Y	Z	X	Y	Z
Load	F_x 300 kips	F_y 2,200 kips				M_z 30,000 kip-ft
Plastic Disp		δ_1 0.88 in.				

	Actions of One Diaphragm Span Relative to Wall Degree of Freedom					
	Translation			Rotation		
Action	X	Y	Z	X	Y	Z
Load	V_x 10 kips	F_y 57 kips	F_z 40 kips	M_x 117 kip-ft	M_y 13.6 kip-ft	
Plastic Disp		δ_2 0. in.	δ_3 0.5 in.			

TABLE 5.2.b. SEISMIC PERFORMANCE REQUIREMENTS FOR LOAD-BEARING WALL TO HOLLOW-CORE FLOOR JOINT NEAR TOP OF STRUCTURE

	Actions of Wall Above Relative to Wall Below Degree of Freedom					
	Translation			Rotation		
Action	X	Y	Z	X	Y	Z
Load	F_x 100 kips	F_y 630 kips				M_z 3,800 kip-ft
Plastic Disp		δ_1 0.00 in.				

	Actions of One Diaphragm Span Relative to Wall Degree of Freedom					
	Translation			Rotation		
Action	X	Y	Z	X	Y	Z
Load	V_x 15 kips	F_y 29 kips	F_z 40 kips	M_x 86 kip-ft	M_y 20.4 kip-ft	
Plastic Disp	δ_2 0. in.	δ_3 0.0 in.				

Thus, if the diaphragm in-plane stiffness degrades, the compatibility forces diminish or disappear altogether, yet wracking displacements of the diaphragms are small because they are limited by the small relative displacements of the transverse walls which flank the bay under consideration.

If the diaphragm maintains its elastic in-plane stiffness, a shear force, V_x, of about 10 kips must be transmitted to the wall from each span. Because the diaphragm is untopped, longitudinal movement of one plank relative to its neighbor is assumed to occur without restraint along the span. However, each plank is assumed to be clamped against rotation about the Y-axis, where it frames into the wall.

Thus, the planks act as isolated cantilevers in response to the horizontal shear, V_x. For a 4-foot plank width, assuming inflection points at midspan, the shear produces a moment, M_y, of 13.6 kip-ft at the end of each plank. Simultaneously, the full 38-foot width of diaphragm subjects the wall to a vertical force, F_y, of 57 kips including plank dead load, superposed dead load, and 40-psf live load. Accompanying the vertical force is a fixed-end moment, M_x, of 117 kip-ft distributed across the wall width due to the 54-psf live load plus superposed dead load.

The tensile force, F_z, of 40 kips in the plane of the diaphragm represents a variety of actions. Longitudinal diaphragm forces of this magnitude are induced by seismic excitation parallel to the planks which may occur simultaneously with transverse excitation. Also, this value approximates chord forces due to in-plane bending of the diaphragm. Because lateral stability of the structure depends on the integrity of load paths between walls in the two orthogonal directions, and because the end-bearing length for planks where they rest on top of the wall units is quite short, tensile yielding of ties through the walls and parallel to the span of the hollow-core planks is not acceptable. Reinforcement should be sized to remain elastic under the predicted loads.

If the diaphragm loses its elastic in-plane shearing stiffness, for example due to opening of the horizontal joint as the wall rocks, the compatibility shear force, V_x, will disappear. Degradation of diaphragm in-plane shear stiffness can be tolerated in the lower stories provided that the "working" of plank ends in the platform joint does not impair the transfer of vertical load and horizontal shear from the wall above to the wall below.

To model the diaphragm shear distortion or "wracking" that would occur due to relative wall movements at opposite ends of the diaphragm span, the planks should be displaced laterally about 0.1 inch at a point 13 feet from the wall, as shown by δ_2 in Fig. 5.13. Note that even this small displacement exaggerates the expected wracking distortion.

If X-rotational support fixity of the hollow-core planks is lost due to wall rocking, the fixed-end moment, M_x, will disappear and simple-span end rotations of about 0.003 radian will occur due to live load and superposed dead load. Again, this action can be tolerated only if it does not impair axial force and horizontal shear capacities of the wall-to-wall connections through the joint region. Inelastic X-rotation of the plank ends can be modeled in the laboratory by displacing the diaphragm downward by about 0.5 inch at a point 13 feet from the wall, as indicated by δ_3 in Fig. 5.13.

b. Near Upper Third Point

In upper levels of the structure, wall moments are low enough that yielding will not occur. On the other hand, diaphragm forces are greater than in the lower stories. While degradation of in-plane shear stiffness may be acceptable near the base, it is intended that the diaphragms maintain their initial stiffness in the upper levels.

Referring to Table 5.2.b, in-plane wall shear, F_x, is about 100 kips and axial load, F_y, is 630 kips. To minimize potential damage in the diaphragm-to-wall connections, thus preserving the diaphragm's in-plane shear stiffness, vertical connectors between wall panels above and below the joint must be designed to remain elastic under the wall moment, M_z, of 3800 kip-ft. Accordingly, the acceptable plastic elongation, δ_1, associated with joint rocking is specified as zero.

As shown in the lower section of Table 5.2.b, lateral shears, V_x, of 15 kips are transmitted to the wall from each adjoining span. A vertical shear, F_y, of 57 kips and a tensile force, F_z, of 40 kips must also be sustained. A fixed-end moment, M_x, of 177 kip-ft and the moment M_y = 20.4 kip-ft at the end of each plank (due to diaphragm wracking) complete the specification of performance requirements. Note that plastic displacements δ_2 and δ_3, related to yielding of the connection between plank ends and the wall, are not acceptable and are specified as zero.

5.9.1.4 Suggested Connection Detail for Testing

Essential features of the suggested platform connection detail are shown in Fig. 5.14. Though its basic configuration is representative of typical bearing wall construction in the United States, several specific features have been incorporated in response to the seismic performance requirements described above. Physical tests in Phase 3, to be performed by others, will be needed to determine how well these suggested details satisfy the predicted strength and deformation requirements.

Two 1-inch-diameter deformed Dywidag bars are required at each end of a typical north/south wall. To accommodate the predicted 7/8-inch plastic elongation, a 24-inch unbonded length of Dywidag is provided immediately above the joint. This is accomplished by means of a plastic sheath which locally prevents adhesion between the grout and the post-tensioning rod.

As shown in Fig. 5.15, the Dywidags lose their preload when they are strained plastically and experience a residual compression of 145 ksi when the wall rocks in the opposite direction, closing the joint. This residual stress acting in all four bars reduces axial load across the joint by 493 kips. For the north/south load-bearing walls, sufficient compression will remain to transfer the applied horizontal shear; in a non-load-bearing wall, however, the joint may actually remain open after one complete rocking cycle. This issue is explored further when the east/west walls are discussed below.

Bars left unbonded over the full wall height may offer cost and performance advantages over bonded post-tensioning. Eliminating the need for grout injection would speed erection and reduce labor costs. More important, if the elongations due to joint opening were accommodated over the full 148-foot bar length, instead of the 24-inch unbonded length provided in this detail, steel stresses would remain below the yield point. Thus, joint opening would not result in a loss of prestress; even in non-load-bearing walls, prestress would continue to augment frictional shear resistance of the horizontal joints during a damaging earthquake.

The rationale for prescribing seismic design loads, however, presumes that the structure will yield and absorb energy through plastic action. Furthermore, the methodology proposed here for estimating seismic displacement magnitudes assumes elastoplastic behavior. Thus, the behavior assumed

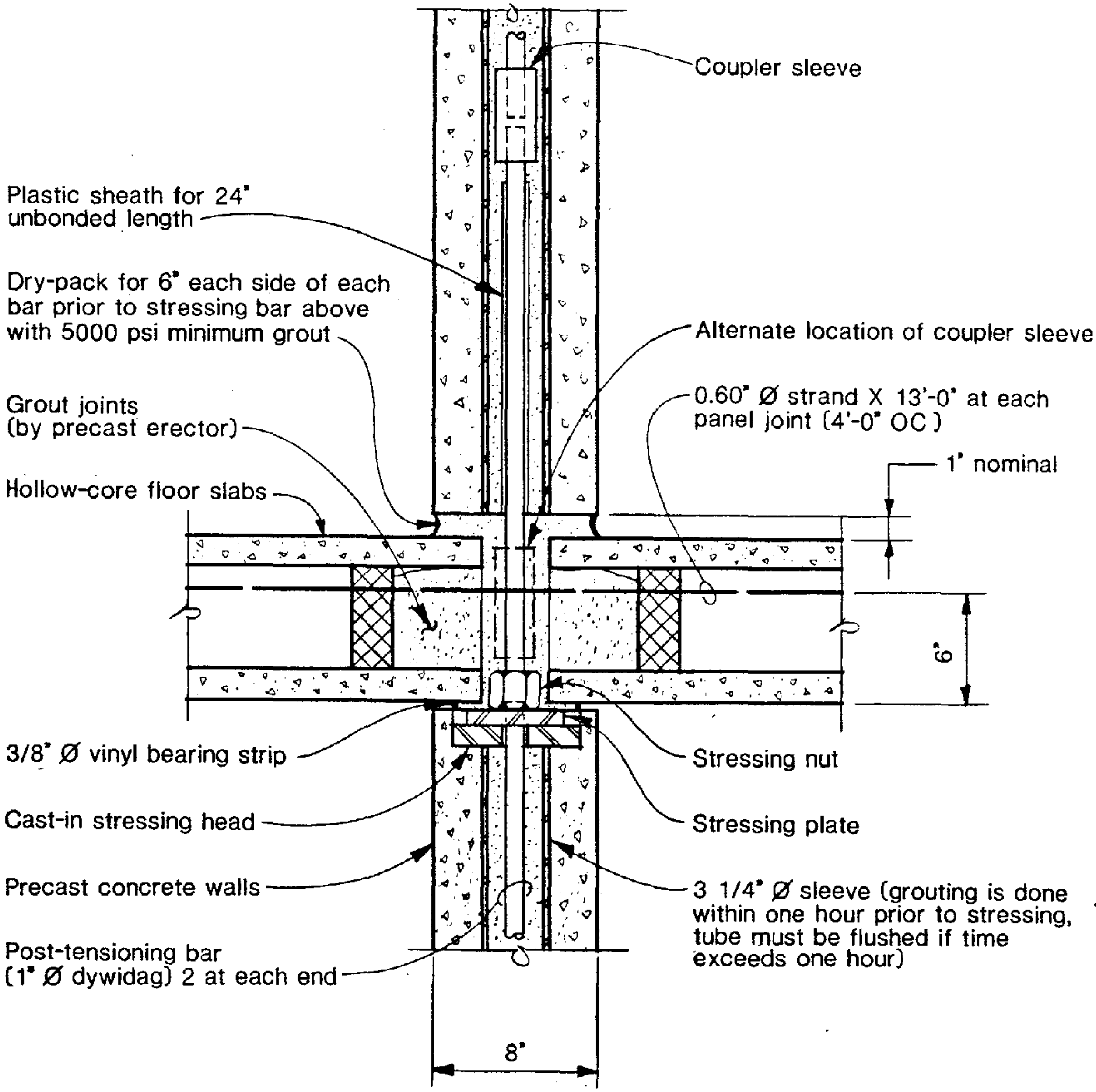


FIGURE 5.14 - LOAD-BEARING WALL TO FLOOR TO WALL

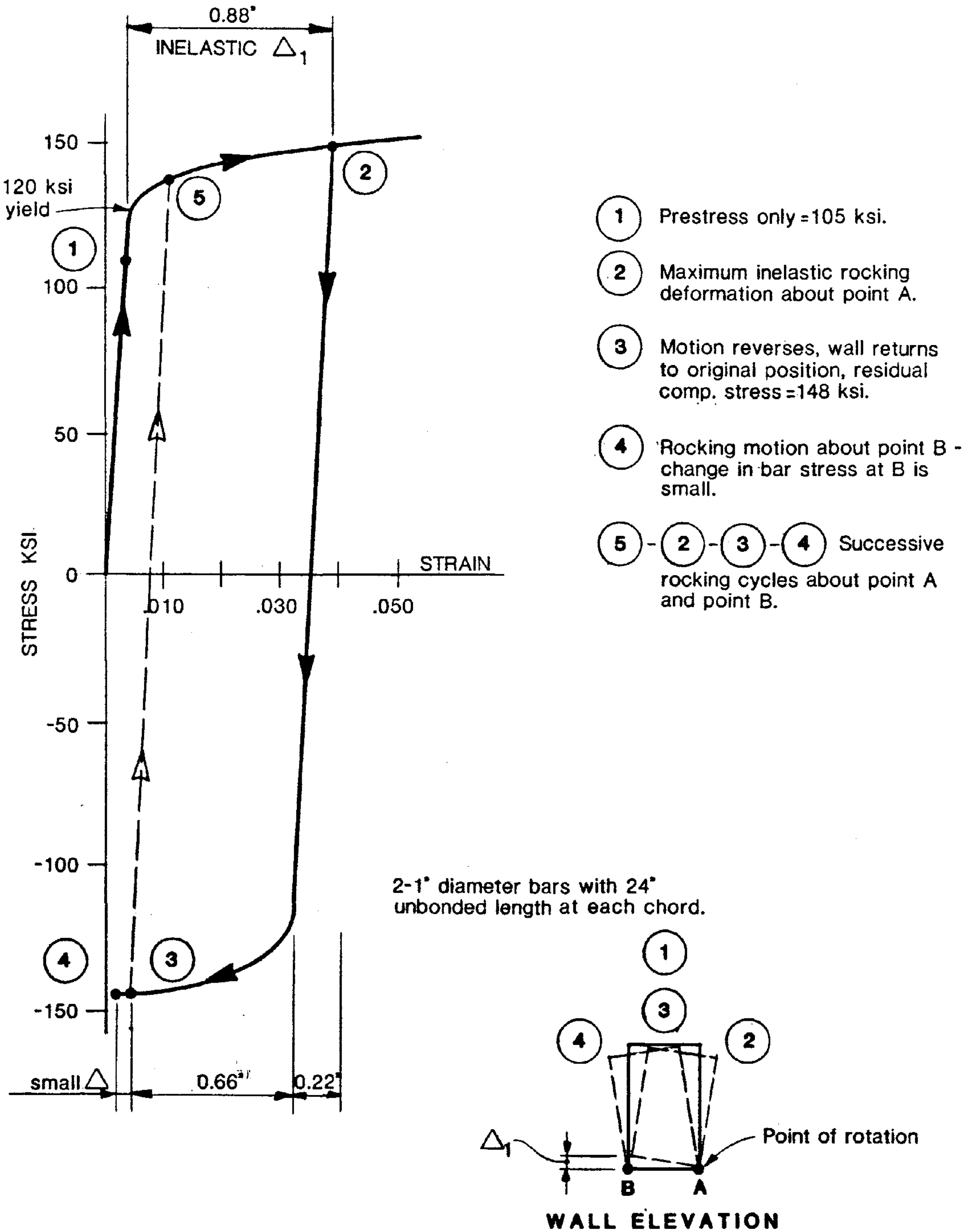


FIGURE 5.15 - DYW DAG BARS HYSTERSIS CURVE

in available design methodologies differs fundamentally from that of nonlinear elastic systems such as precast walls with unbonded post-tensioning bars. Analytical and experimental studies would be needed to justify the application of existing code loading provisions in the design of walls intended to exhibit nonlinear elastic response under strong ground shaking. For this reason, unbonded post-tensioning has not been pursued here; it is suggested that this option be investigated in Phase 3, however.

Mild steel was considered as an alternative to post-tensioning for the vertical reinforcement. Although analyses of inelastic strain (refer to Fig. 5.16) show that the behavior of the two systems differs little after one rocking cycle, it was found that post-tensioning does offer advantages. Under moderate earthquakes and wind load, the post-tensioned wall experiences less cracking. The higher strength of prestressing steel means that fewer bars are required; the tie-down forces can be located nearer to the wall ends, giving a more efficient moment-resisting lever arm; and post-tensioning provides a positive, load-tested vertical tie -- a feature not available with other means of connection.

Floor planks typically are not reinforced for negative moments which occur at the supports; in some cases, this produces cracking in the planks under superimposed dead and live load. As shown in Fig. 5.14, it is suggested that an unstressed 0.60-inch-diameter prestressing strand be located in each joint. Placing strands near the upper edge of the interelement keyway mobilizes negative-moment resistance in the planks. This will also provide continuous reinforcement through the plane of the wall to resist diaphragm forces. It is suggested that strands extend for one fourth the hollow-core span in both directions from the wall. Both the effectiveness of this grouted-in reinforcement and the required development length require experimental investigation in Phase 3.

The integrity of this connection is dependent on compressive and shear strengths of the grout column between the plank ends. Use of a 3/8-inch-diameter vinyl bead when seating the planks on the walls during erection instead of a 2-inch-wide bearing pad (as is common practice) increases the width of the grout column from 4 to 6 inches. This will reduce compressive stresses in the grout and may enhance seismic resistance.

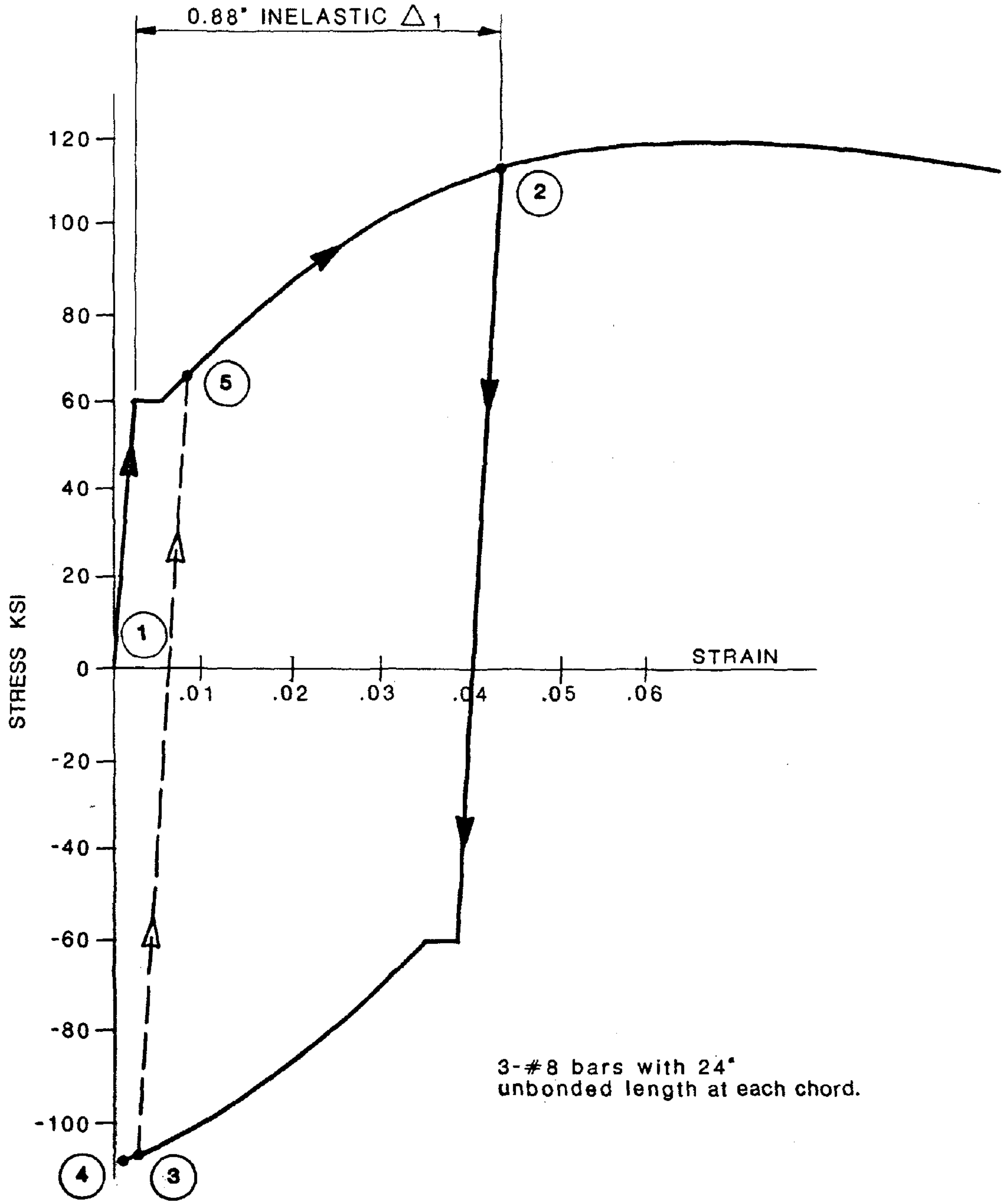


FIGURE 5.16 - GRADE 60 REBAR HYSTERESIS CURVE

Experimental studies are needed to assess the sensitivity of this joint to quality of workmanship, to the thoroughness with which grout flows into the voids of the hollow-core planks and under the planks to the vinyl strip, and to the degree of lateral confinement provided to the grout column by the planks.

5.9.1.5 Alternative Connection Detail

One feature of the typical "platform" connection detail discussed above is that axial forces, shears, and moments in the plane of the wall and in the plane of the diaphragm are all transmitted through the grout column between the plank ends. Thus, diaphragm loads tending to pull the planks out of the joint can have an effect on the capacity of the connection for transmitting axial loads, moments, and shears from the wall panel above to the panel below.

In Fig. 5.17, an alternative connection detail is suggested in which steel angles are used to support the floor planks and the wall panels stack directly, one on top of another. Bolt-on angles allow the wall panels to be erected in two-story heights, thus reducing the number of elements and connections. A shear key can be cast into the wall panels with minimal expense and wall-to-floor plank ties can easily be extended through the keys. In addition, the full 8-inch compression width may be advantageous for structures with higher wall stresses.

A drawback to this approach is that the angles introduce a new element to the structure which will require fire protection and architectural treatment. These functions might be provided by an insulated moulding, however.

5.9.2 Non-Load-Bearing Wall-to-Wall Joint (Fig. 5.18)

5.9.2.1 Description

Walls parallel to the span of the floor planks carry their own weight and participate in resisting lateral loads; they are not subject to dead and live loads of the roof and floors, however, and are therefore referred to as

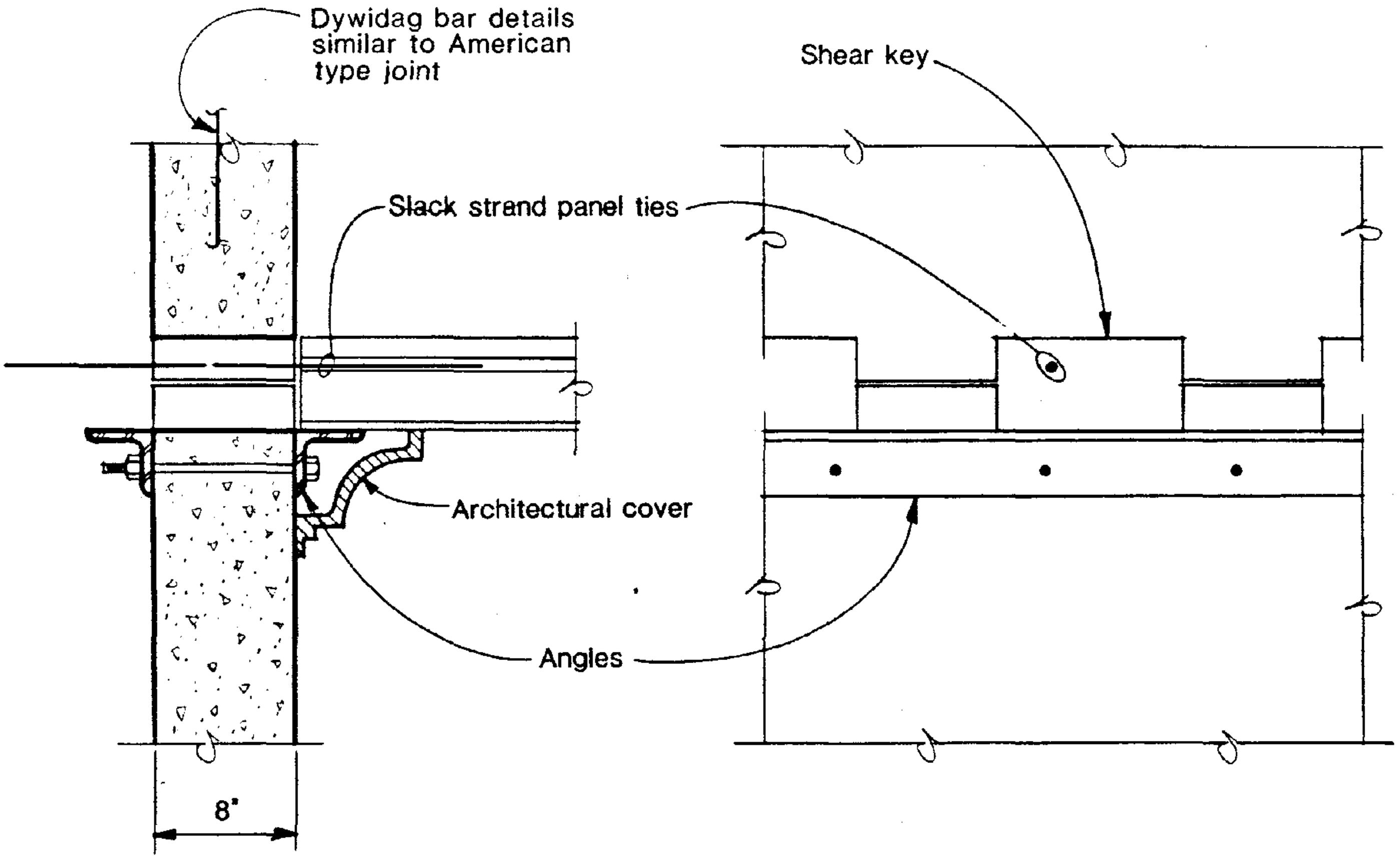


FIGURE 5.17 - LEDGER TYPE BEARING WALL JOINT

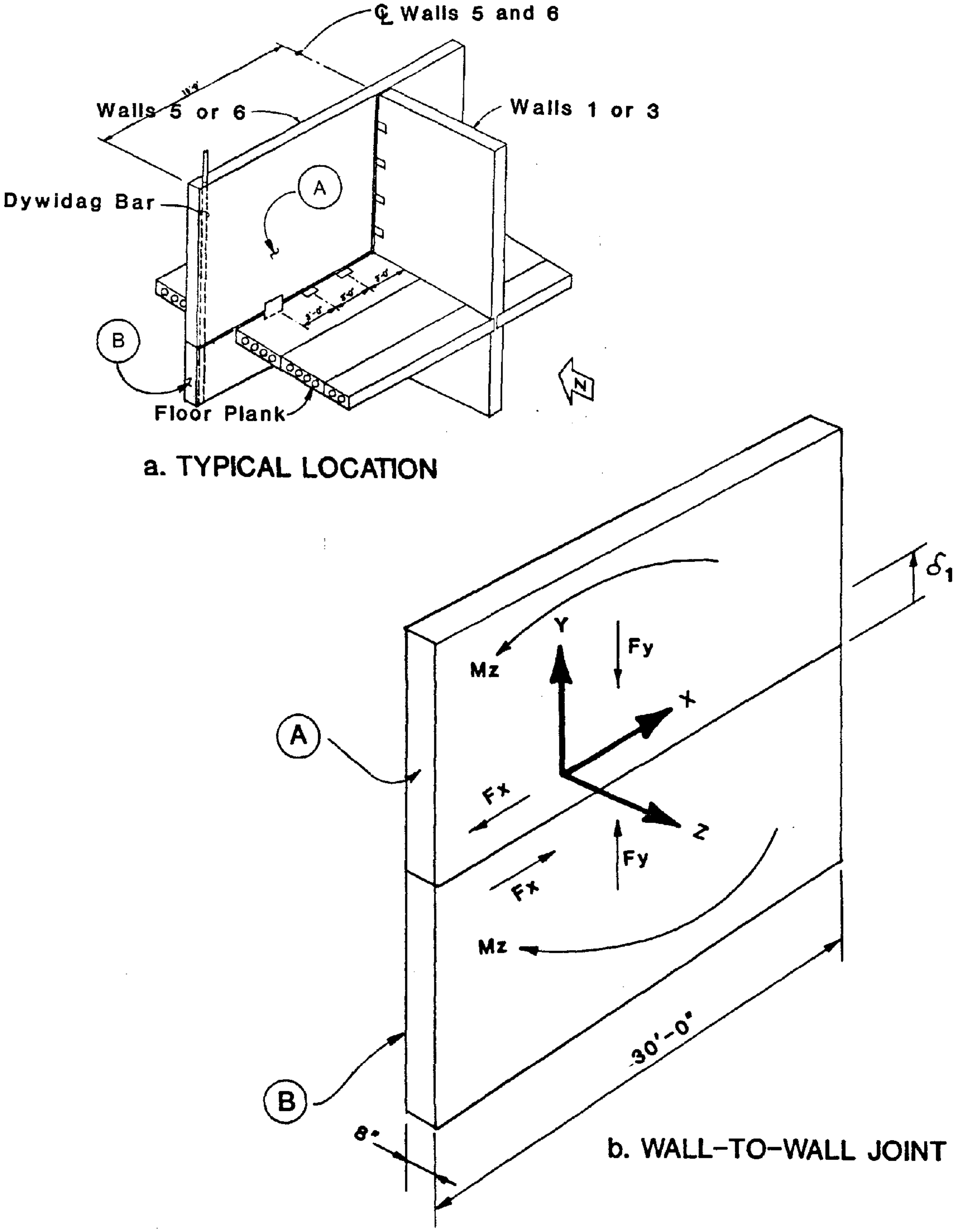


FIGURE 5.18 - NON-LOAD BEARING WALL-TO-WALL JOINT

"non-load-bearing." As shown in the isometric illustrations of Figs. 5.18.a, 5.18.b, and 5.20.b, two types of joint planes associated with non-load-bearing walls: one at the interface of upper and lower wall units, the other at the interface of wall and plank. Note that connections across these two joint types may or may not perform independently, according to the construction details.

5.9.2.2 Joint Actions

Connections across the wall-to-wall joint (Fig. 5.18.b) must transmit wall dead loads in the Y-direction and in-plane wall shears in the X-direction. Additionally, because non-load-bearing walls in the Chameau Condominiums are elements of the east/west lateral force resisting system, they are expected to undergo flexural yielding in their lower stories during a major earthquake. Inelastic action is expected to concentrate at the joints; thus, the connectors must sustain plastic deformations associated with inelastic Z-rotation in addition to developing moments about the Z-axis.

5.9.2.3 Seismic Performance Criteria

Performance criteria for connections across the non-load-bearing wall-to-wall joint are summarized in Table 5.3. The values presented describe conditions near the base of Wall 6 (refer to Fig. 5.2). As shown in Fig. 5.18.b, loads to be transmitted across the joint include a horizontal shear force, F_x, of 200 kips and an in-plane moment, M_z, of 15,400 kip-ft.

Vertical connections through the joint are expected to yield under this moment and the plastic component of joint rotation will produce a 1-inch vertical displacement at the wall end, as shown by δ_1. Required plastic elongations of individual connectors between the wall ends may be determined by linear interpolation, assuming the wall above rotates about its lower corner at the compression end of the horizontal joint.

The axial load, F_y = 1500 kips, results from several effects. Included are 446 kips of wall dead load, 167 kips of post-tensioning due to one 1-inch-diameter deformed stressing bar at each end of the wall, and 927 kips of "hold-down" from Wall 3 (the north/south cross wall that frames

TABLE 5.3. SEISMIC PERFORMANCE REQUIREMENTS FOR NON-LOAD-BEARING WALL-TO-WALL JOINT NEAR TOP OF STRUCTURE

	Actions of Wall Above Relative to Wall Below Degree of Freedom					
	Translation			Rotation		
Action	X	Y	Z	X	Y	Z
Load	F_x 200 kips	F_y 1,540 kips				M_z 15,400 kip-ft
Plastic Disp		δ_1 1.00 in.				

into Wall 6 at its longitudinal centerline). The hold-down force is generated by special connections between these walls. These connections yield under vertical displacements along the Wall 6 centerline which are associated with rocking under east/west excitation. This action and details of the special vertical connections are described in the discussion of Fig. 5.25, which illustrates the vertical wall-to-wall joint.

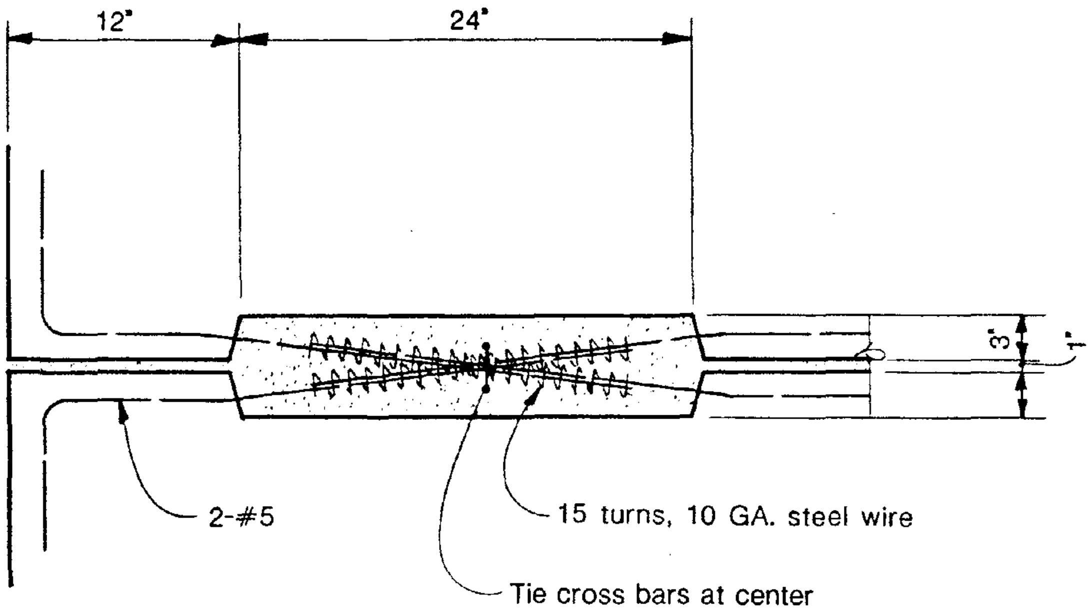
5.9.2.4 Suggested Connection Detail

A connection detail suitable for the non-load-bearing wall-to-wall joint is shown in Fig. 5.19. Flexural loads are resisted by "semicomposite" action of the intersecting load-bearing (i.e., Walls 1 and 3) and non-load-bearing walls (i.e., Walls 5 and 6), as explained below. A 1-inch-diameter Dywidag bar is provided at each end of the wall.

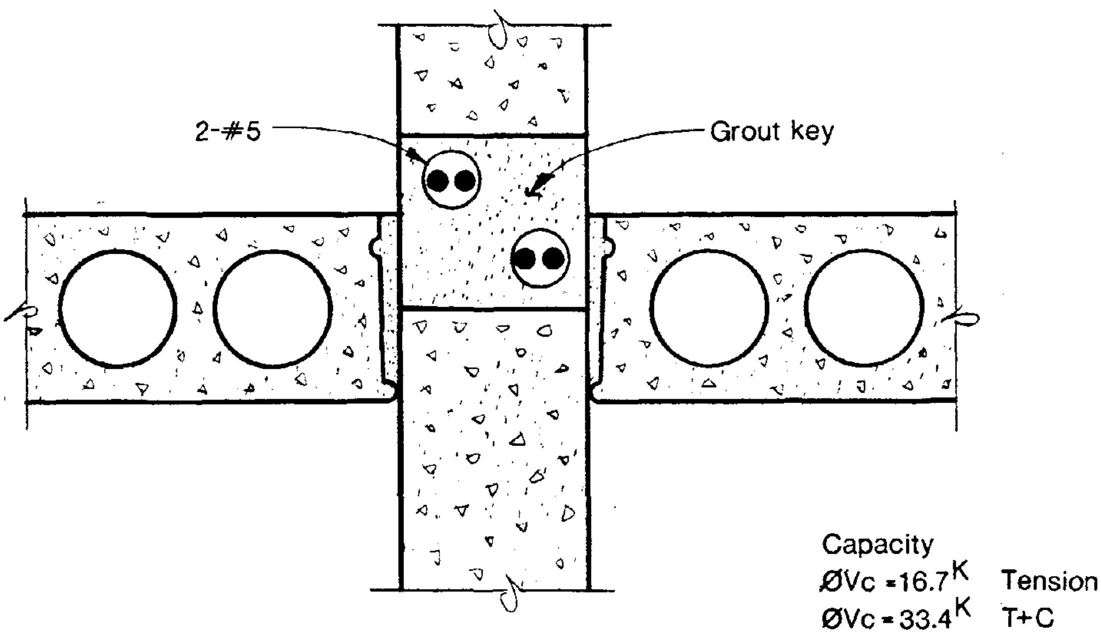
As described above for the load-bearing wall, joints near the base of the structure are expected to open under lateral excitation. For east/west walls, an inelastic elongation of 1 inch is predicted across the joint and the Dywidag bars are expected to be in compression following one complete rocking cycle. This produces a net joint compression of 927 + 446 - 167 = 1206 kips acting at the center of Wall 6. The resisting moment of this axial load is approximately 0.9 x 1206 x 15 feet = 16,281 kip-feet, which exceeds the applied ultimate moment of 15,400 kip-feet. Hence, the Dywidag bars are not required for flexural resistance but are provided to reduce cracking under wind load and minor earthquakes and to assure a positive, load-tested connection between wall panels.

A portion of the horizontal joint shear can be carried by friction. Because the Dywidags are under compression after one rocking cycle, the net dead load available for frictional resistance is 446 - 167 = 279 kips. For a friction coefficient of 0.35, the resisting friction is (0.85) (279) (0.35) = 83 kips, which leaves 117 kips to be carried by shear connectors.

Panel-to-panel shear connectors must be detailed to accommodate the vertical movement predicted to occur across the joints under seismic flexural loads. A suitable detail is shown in Fig. 5.19. The No. 5 bars are installed at the precast plant and are well anchored in the wall panels. To effect the connection, the following field operations are performed. One coil of precut,



ELEVATION



SECTION

FIGURE 5.19 - NON-LOAD BEARING WALL TO WALL CONNECTION DETAIL

preformed, 10-gage spiral reinforcement is passed to the inboard end of each of the projecting bars in the upper panel. Opposing bars from the lower panel are bent to form lap splices. Each 10-gage spiral is stretched out over the full length of the lap. Finally, the voids are grouted.

The crossed bars permit vertical movement across the joint (as the wall panel above rocks) while they transmit horizontal shear through direct tension. If the joint is overloaded, the rebars will yield, avoiding a brittle failure. This detail requires experimental validation during Phase 3.

5.9.3 Non-Load-Bearing Wall to Hollow-Core Floor Joint (Fig. 5.20)

5.9.3.1 Description

Non-load-bearing walls parallel to the span of the floor planks occur along the central corridor of the Chameau Condominiums. Fig. 5.20 shows a typical joint between such a wall and the adjacent floor elements.

5.9.3.2 Joint Actions

Shear in the X-direction is the primary load to be transmitted across the joint. Relative vertical movements will occur, however, due to wall or floor deformations and these will result in secondary forces if they are restrained by vertical stiffness of the shear connectors.

Rocking associated with elastoplastic wall flexure produces vertical displacements at the wall ends. Because most of the wall rotation is due to the opening of joints near the base, these vertical displacements occur with roughly the same magnitude over the full height of the structure.

Depending on the horizontal location of a given connector with respect to the point about which the wall rocks, east/west wall motion will produce either a pure torsional deformation (for a connector vertically above the rotation point) or a combination of torsion and vertical shearing distortion (for connectors located at some horizontal offset from the rotation point). North/south motion produces a pure vertical shearing distortion.

The forces that develop depend on connector location, geometry, configuration, and stiffness relative to the vertical stiffness of the planks.

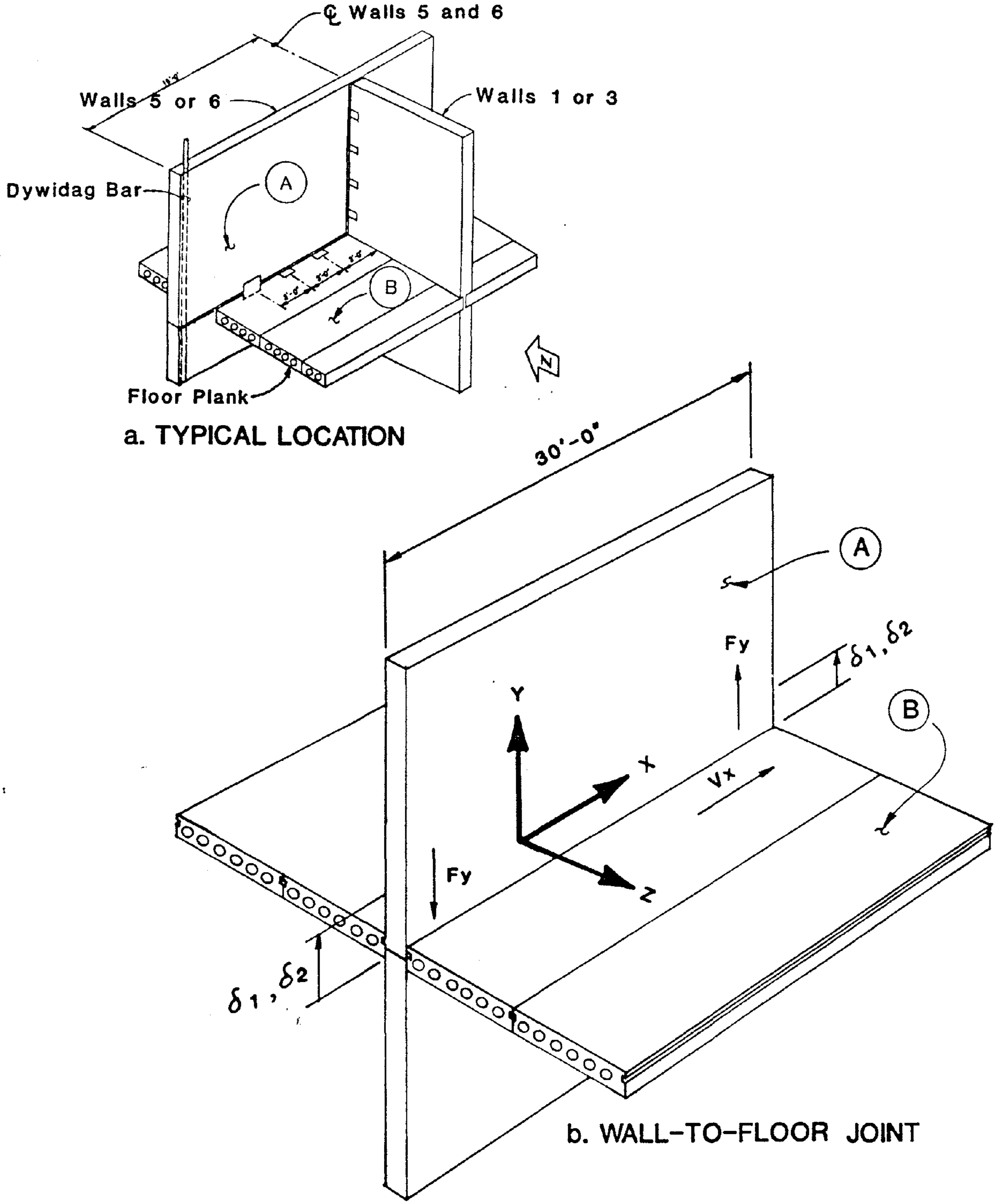


FIGURE 5.20 - NON- LOAD BEARING WALL TO HOLLOW-CORE FLOOR JOINT

5.9.3.3 Seismic Performance Criteria

Performance criteria for connections across the non-load-bearing wall to hollow-core floor joint are summarized in Table 5.4. The connectors are intended to deliver about 10 kips of horizontal shear, V_x, to the wall.

Note that connectors between the floors and Walls 5 and 6 are loaded by north/south as well as east/west response. When the structure is deflected to the south, unrestrained joint displacements (δ_1) of about 3 inches occur between the floors and the east/west walls due to rocking of north/south walls about the compression end of horizontal joints near the foundation.

When the structure is deflected toward the north, the point of rotation shifts toward the corridor end of the walls. In this condition, vertical joint displacements are essentially zero, due to the small lateral offset between the vertical joint plane and the rotation point.

Under east/west excitation, floor-to-wall vertical displacements at the ends of Walls 5 and 6 (δ_2) have a magnitude of 3-1/2 inches, while displacements at the vertical centerlines of these walls are half as large, at 1-3/4 inches. Note, as shown in Fig. 5.21, that vertical displacements vary linearly along the wall-to-wall joint, from a value of zero at the rotation point to a maximum at the opposite end of the wall. Because the rotation point migrates from one end of the wall to the other as the structure sways, connectors at different locations along the wall experience different deformation histories.

The hollow-core floor planks offer significant resistance to vertical displacement. If wall-to-floor shear connectors are stiff with respect to relative vertical movement across the joint, potentially damaging secondary forces can develop.

If the connectors were rigid and infinitely strong, vertical shear forces due to wall motions would be limited by the diaphragm's out-of-plane flexural strength. Asuming that only one plank is deformed by movement of the longitudinal wall, the diaphragm's ultimate out-of-plane moment is reached with vertical shears (labeled F_y in the figure) equal to 6 kips. For comparison, a 3-1/2-inch vertical deflection would produce a shear force of 58 kips if the plank remained elastic.

TABLE 5.4. SEISMIC PERFORMANCE REQUIREMENTS FOR NON-LOAD-BEARING WALL TO HOLLOW-CORE FLOOR JOINT

Condition	Degree of Freedom					
	Translation			Rotation		
	X	Y	Z	X	Y	Z
Fixed	V_x 10 kips	F_y 6 kips				
Free		δ_2 3.5 in.				δ_1 3.0 in.

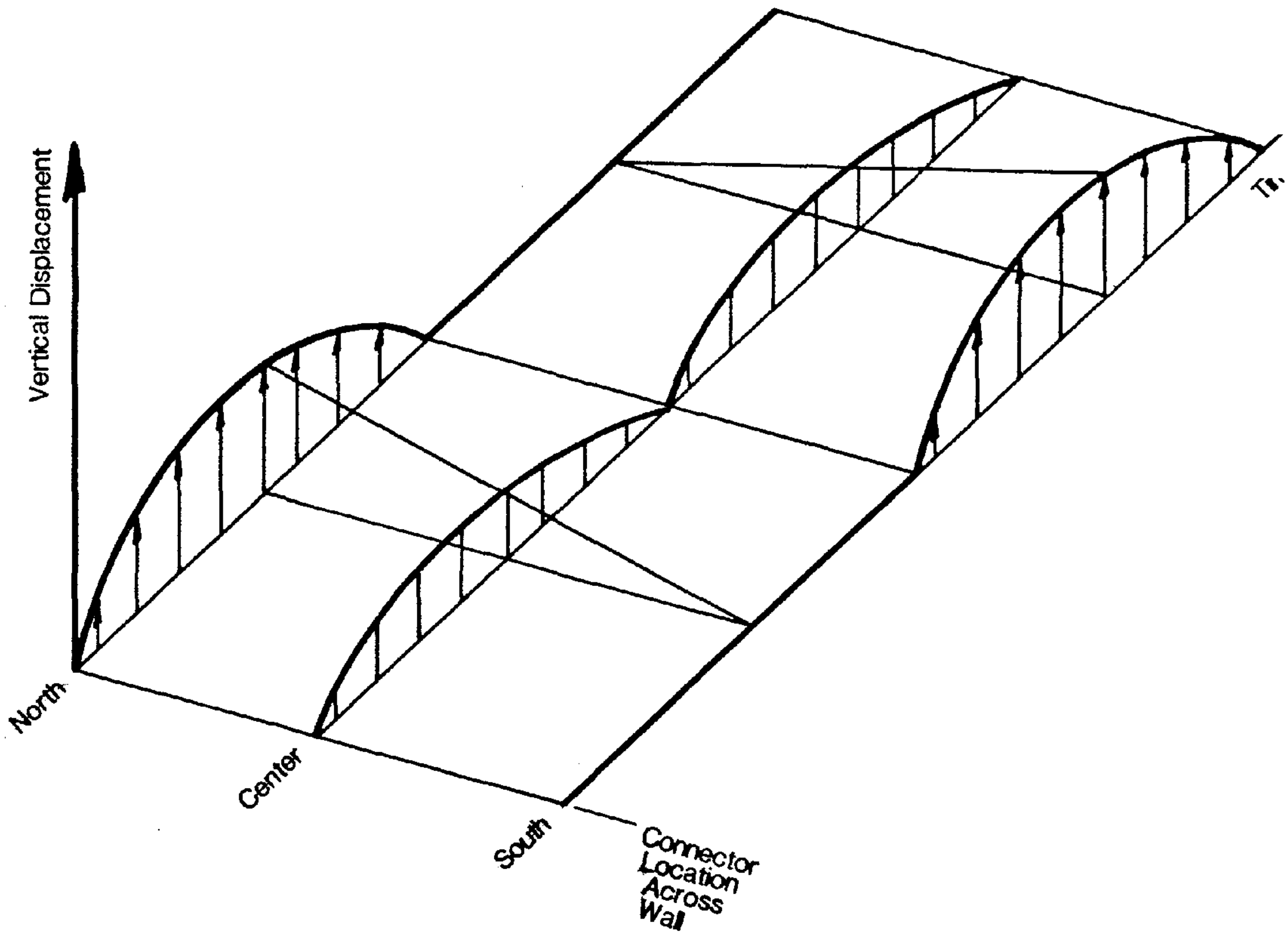


FIGURE 5.21 - EFFECT OF CONNECTOR LOCATION ON PLASTIC DEFORMATION TIME-HISTORY WITH ROCKING WALL

Thus, the connectors need to be flexible with respect to vertical displacements across the floor-to-wall joint. On the other hand, it is intended that horizontal shears be transmitted across the joint without significant relative displacement in the X-direction.

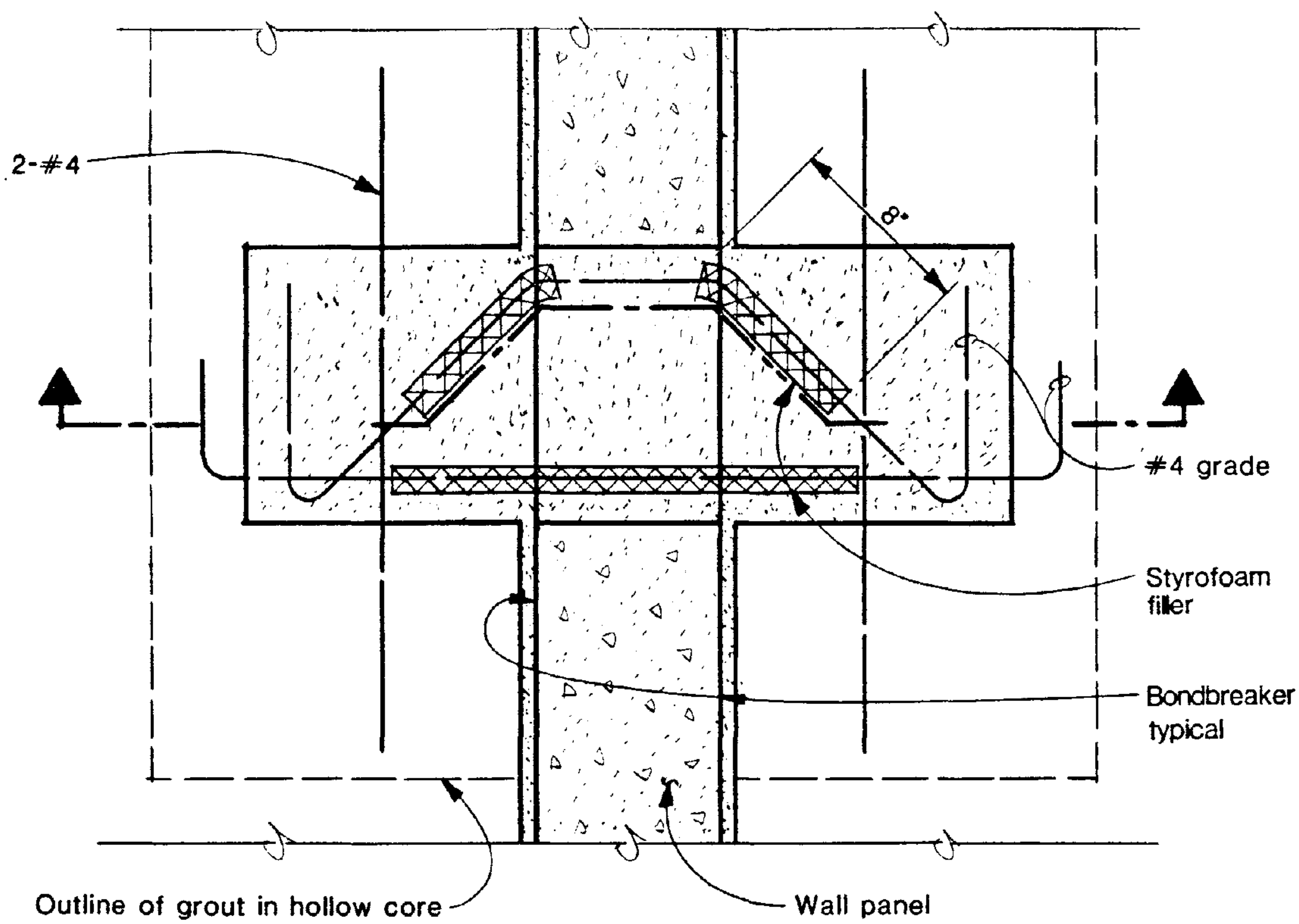
Depending on design details, strength and stiffness of connections across the diaphragm-to-wall joint described here may be affected by loads and displacements across the wall-to-wall joint shown in Fig. 5.18 and specified above. Unless connections across the joints of Figs. 5.18 and 5.20 are detailed to behave independently, they must be evaluated together using a test assemblage as shown in Fig. 5.20 and subjected to the combined loads and deformations of their respective performance criteria.

5.9.3.4 Suggested Connection Detail

A detail suitable for the connection between floors and non-load-bearing walls is presented in Fig. 5.22. The connector shown will allow an inelastic vertical displacement of about ±4 inches, while providing a horizontal shear capacity of 4.5 kips. Grade 40 steel should be used to provide sufficient plastic hinge ductility for the bent bar to displace 4 inches vertically over an 8-inch length.

An additional No. 4 tie completes the force triangle and provides a nominal panel-to-panel connection through the plane of the wall. Two No. 4 bars are provided in the blockout to distribute forces into the unreinforced floor plank. Both the bent bar and the straight bar must be encased in a styrofoam blockout to permit the necessary vertical movements.

Note that these connectors are to be used in opposite hand pairs, even though calculations show that the diagonal bar can carry the floor-to-wall horizontal shear load in tension or compression.



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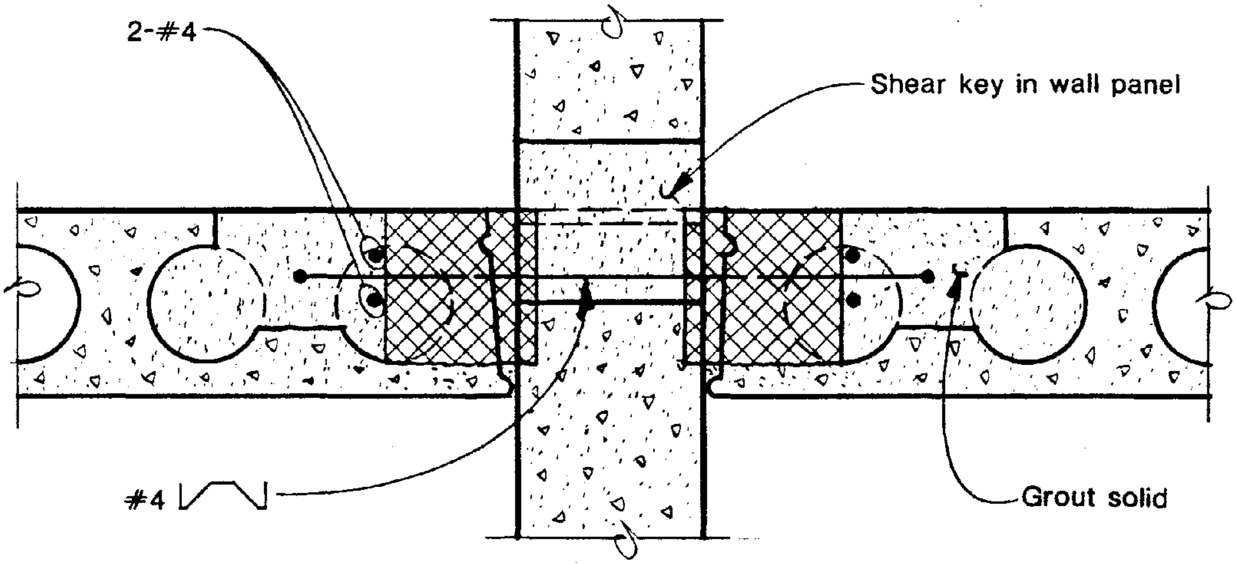


SECTION

FIGURE 5.22 – NON LOAD BEARING WALL TO FLOOR CONNECTION DETAIL

5.9.4 Corridor Support to Wall Joint (Fig. 5.23)

Fig. 5.23 depicts a typical corridor support detail and cross corridor tension tie.

5.9.4.1 Joint Actions

The primary load to be transmitted across this joint is vertical (Y-axis) bearing. Due to rocking action of the walls, however, a secondary Z-axis rotation of about 3 degrees must be accommodated. This rotation occurs as opposite sides of the corridor alternately move upward 3.4 inches and settle back to their original elevation.

Due to nonuniform support conditions or random variations in ultimate wall strengths, behavior of the actual structure will depart from the ideal symmetry implied by its layout. Thus, in addition to Y-axis bearing and Z-axis rotation, a tension tie across the corridor is required to prevent the north and south halves from moving apart. Such a tie could be provided by the corridor support member itself or by a separate tie element; for example, slack strand or mild reinforcing anchored into the vertical reinforcement which passes through the horizontal wall joints at either side of the corridor could be employed.

To estimate the magnitude of required tie forces, several nonlinear analyses were made with a 10-percent difference in ultimate moment capacities between a pair of walls flanking the corridor. Footing stiffnesses differing by a similar amount would produce the same effect.

Cross-corridor connections between the walls are highly redundant and the predicted tie forces depend strongly on assumed tie stiffness. With elastic ties of 2-square-inch cross section, a tie force on the order of 9 kips in the lower floors, decreasing to about 0.5 kip near the top, is obtained. If the ties in the first nine floors are assumed to fail, forces on the order of 1.5 kips are required in Floors 10 through 17. If all the ties are assumed to fail, a relative lateral movement of 3/4 inch is predicted between the two walls at roof level. Differences of more than the assumed 10 percent in properties of the north and south halves of the structure would increase the predicted cross-corridor loads and deformations.

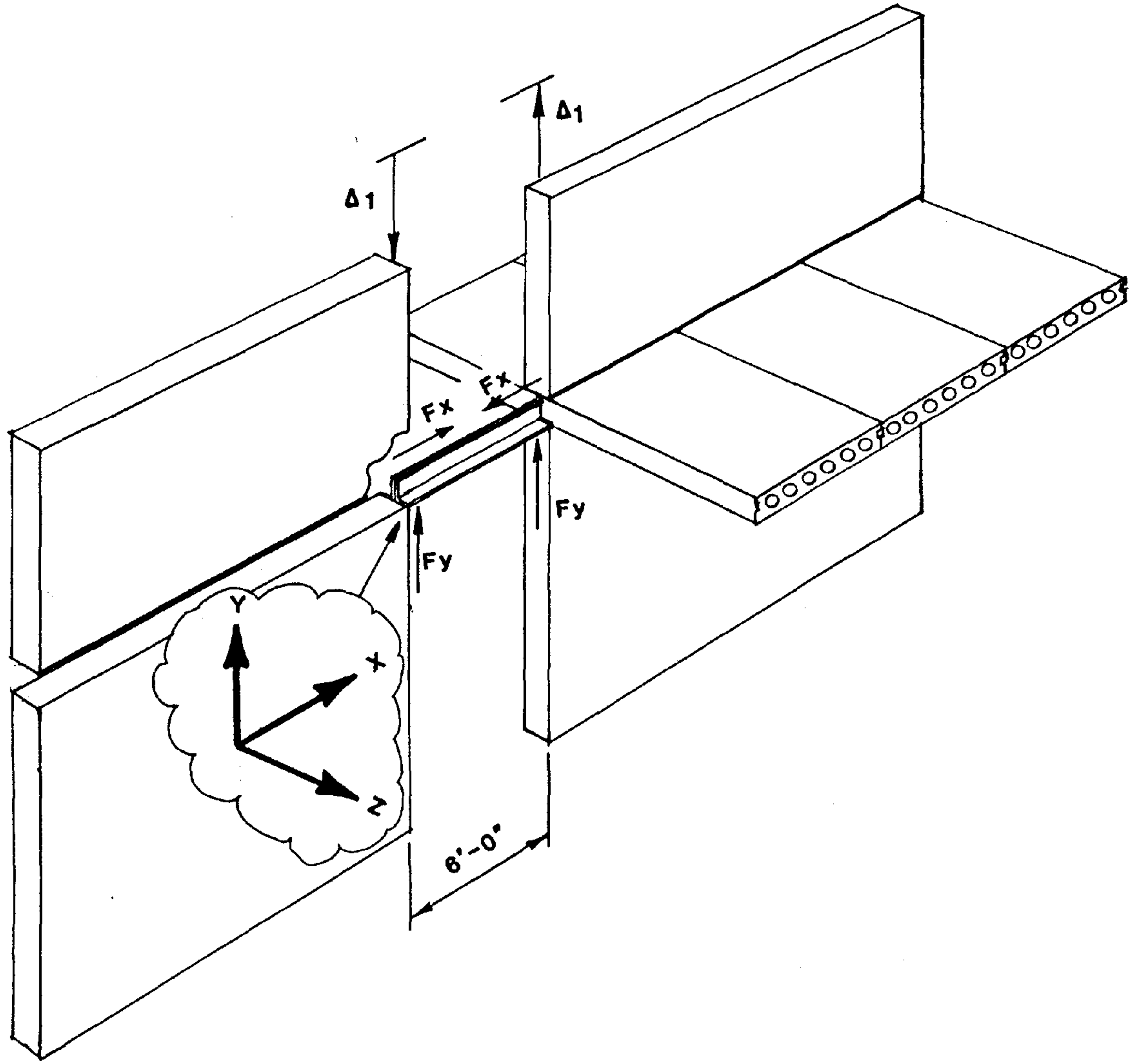


FIGURE 5.23 – CORRIDOR SUPPORT TO WALL JOINT

5.9.4.2 Seismic Performance Criteria

Performance criteria for connections across the corridor support to wall joint are summarized in Table 5.5. It is intended that diaphragms not be pulled apart during an earthquake. Because the walls are stiff and elastic tie forces in the lower stories are due primarily to compatibility effects, cross-corridor wall displacements do not increase appreciably if the ties yield. Thus, satisfactory performance will be obtained as long as tension ties in the upper stories remain elastic.

Accordingly, an elastic capacity of $F_x = 2$ kips is specified for the tension ties. This must be sustained in combination with the relative vertical displacements, δ_1, of 3.4 inches across the corridor, and vertical support reactions, F_y, of about 10 kips at each side of the corridor due to dead and live load.

5.9.4.3 Suggested Connection Detail

The detail shown in Fig. 5.24 serves both as a corridor tie and as a bearing plate for the Dywidag stressing nut at the corridor end of each of the north/south walls. The tie is fabricated from a pair of angles welded back-to-back to form an inverted T. The bolted attachment of the tie to the bearing plate accommodates the required 3-degree rotation, while the bearing plate can be a standardized item which is shipped loose and installed just prior to post-tensioning.

5.9.5 <u>Wall-to-Wall Joint</u> (Fig. 5.25)

5.9.5.1 Description

Fig. 5.25 shows a typical vertical joint between intersecting precast wall units. The Y-Z plane of the coordinate system is parallel to the contact surface between the units with the Y-axis pointing up.

TABLE 5.5. SEISMIC PERFORMANCE REQUIREMENTS FOR CORRIDOR SUPPORT TO WALL JOINT

	Degree of Freedom					
	Translation			Rotation		
Condition	X	Y	Z	X	Y	Z
Fixed	F_x 2 kips	F_y 10 kips				
Free		δ_1 3.4 in.				

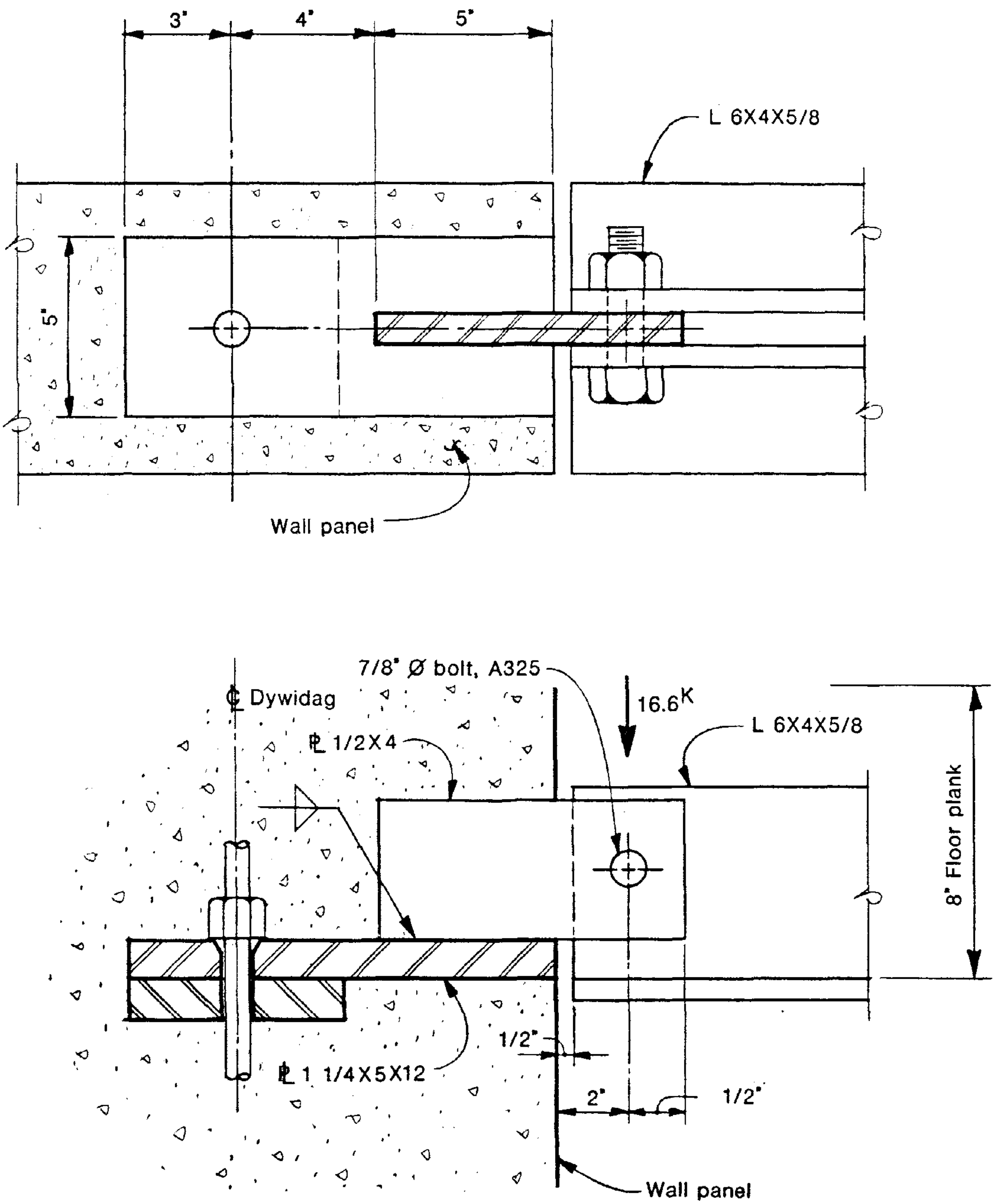


FIGURE 5.24 - CORRIDOR SUPPORT DETAIL

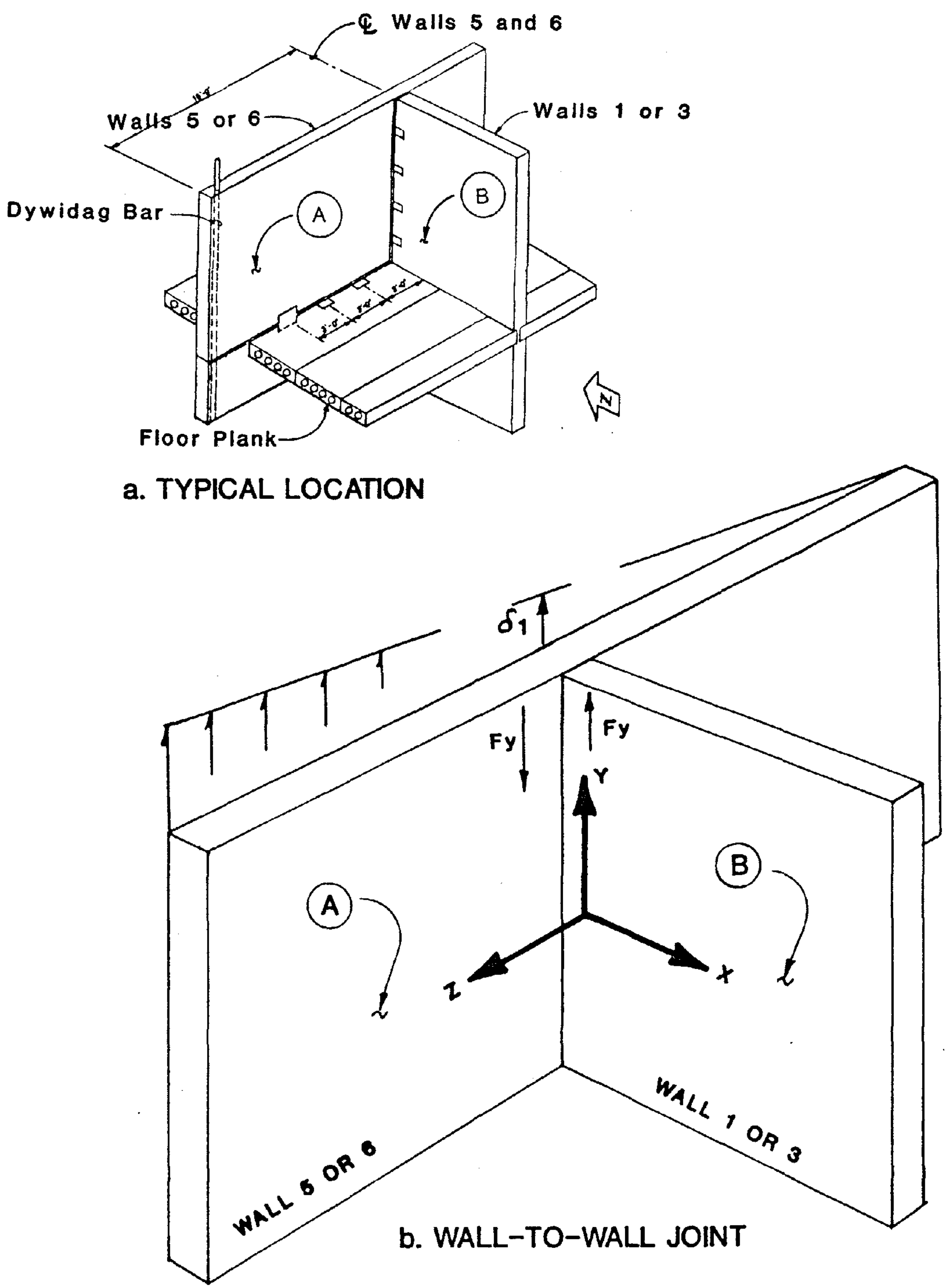


FIGURE 5.25 - VERTICAL WALL-TO-WALL JOINT

5.9.5.2 Joint Actions

In the Chameau Condominiums, vertical joints occur only where walls intersect at right angles. It is not intended that connections across these joints develop composite flexural action in the walls. At the intersections of Walls 4 and 7 and Walls 4 and 8, connections across the vertical joint are desired for stability during erection and for structural integrity in general.

However, at the intersection of Wall 3, a north/south cross wall, with Wall 6, a non-load-bearing longitudinal wall (and similarly for Walls 1 and 5), connections across the vertical joint are intended to develop "hold-down" forces that will contribute to the overturning resistance of the nonbearing wall under east/west seismic loads.

In-plane, inelastic deformation of Wall 6 may be visualized as a rotation about the compression end of the wall-to-footing joint. The rotation results in an upward displacement along the interface with Wall 3, the north/south cross wall. Connectors across this vertical joint resist the relative movement; a portion of the cross-wall dead load is mobilized to resist overturning of the longitudinal wall.

5.9.5.3 Seismic Performance Criteria

Performance criteria for connections across the vertical wall-to-wall joint are summarized in Table 5.6. Wall 6 has been designed assuming that half of the dead load in Wall 3 will be mobilized by connectors across the vertical joint. This means the connectors in each story must develop a total elastoplastic resistance to relative vertical movements between the walls of F_y = 55 kips.

Note that connectors in the vertical joints between Walls 1 and 5 and Walls 3 and 6 are loaded by north/south as well as east/west response. When the structure is deflected to the south, unrestrained joint displacements of about 3-1/4 inches between Walls 1 and 5, and 2-3/4 inches between Walls 3 and 6, occur due to wall rotations about the compression end of horizontal joints near the foundation.

When the structure is deflected toward the north, the point of rotation shifts toward the corridor end of the walls. In this condition, vertical

TABLE 5.6. SEISMIC PERFORMANCE REQUIREMENTS FOR VERTICAL WALL-TO-WALL JOINT

	Degree of Freedom					
	Translation			Rotation		
Condition	X	Y	Z	X	Y	Z
Fixed		F_y 55 kips				
Free		δ_1 3.25 in.				

joint displacements are essentially zero due to the small lateral offset between the vertical joint plane and the rotation point.

In accordance with predicted elastoplastic lateral displacements under east/west excitation, connectors across the vertical joints between Walls 1 and 5, or Walls 3 and 6, must sustain plastic shearing deformations, δ_1, of 1-3/4 inches. Note that this deformation occurs when the structure is deflected either toward the east or toward the west.

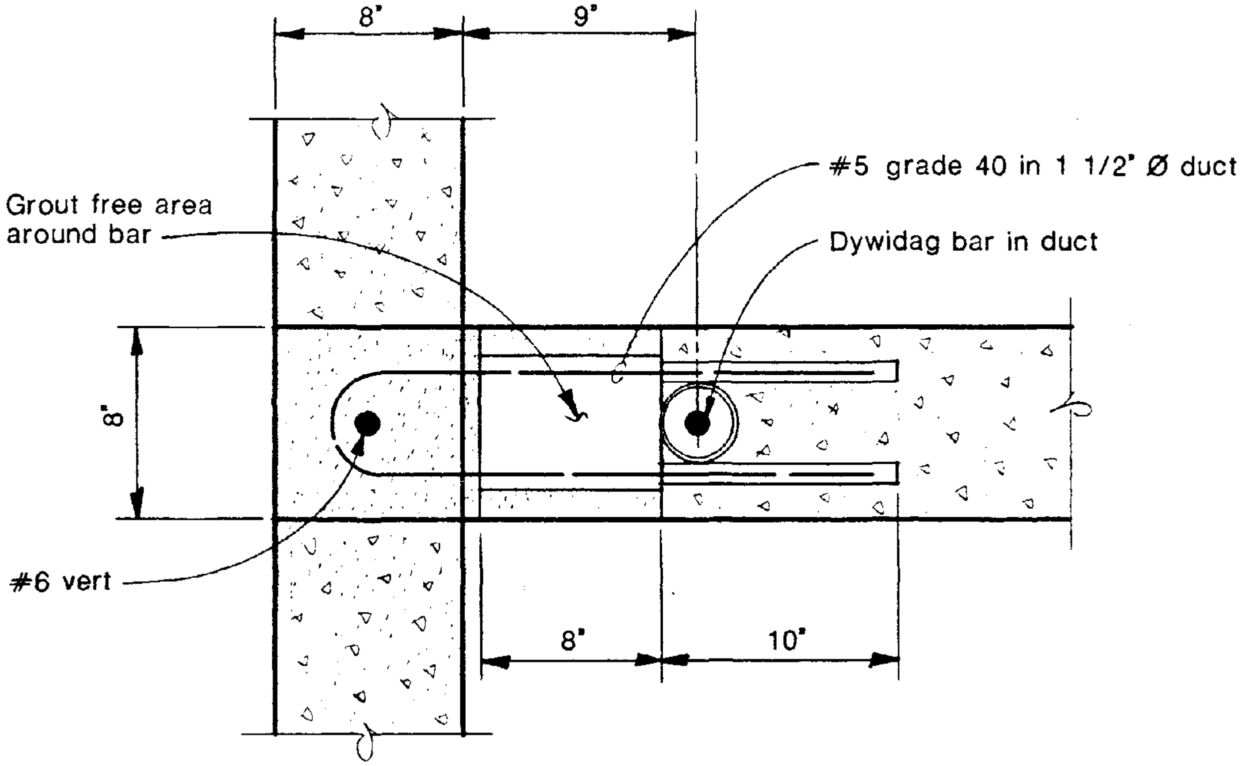
Overturning stability of Walls 7 and 8 is provided by their own dead loads, so the development of vertical connector forces across their joints with Wall 4 is not required. Under east/west excitation, vertical displacements across these joints have an unrestrained amplitude of 3-1/4 inches when the building is deflected toward the east, and an amplitude of essentially zero when the building is deflected toward the west.

5.9.5.4 Suggested Connection Detail

Fig. 5.26 illustrates a connection detail conceived to deliver the required hold-down force while accommodating the movements anticipated across the vertical wall joints. As configured, the connection can sustain a relative movement of 4 inches, which is about equal to the full east/west displacement plus 70 percent of the maximum anticipated north/south displacement.

This connection is simple and easy to install using materials and methods common to the precast industry. The Grade 40 bar provides ample ductility to deform as shown in the figure. The total strain due to length change for a 4-inch vertical displacement is 0.118 inch/inch, well within the ultimate strain of 0.19 for Grade 40 reinforcing steel. The corresponding stress is 72 ksi. Further, the connection will accommodate the full displacement of 3-1/4 inches expected during a north/south earthquake.

Some cautions are appropriate. To allow room for vertical movement along its exposed length, it is critical that grout be kept out of the blockout area around the bar. Some sort of filler material, such as styrofoam, will probably be required. Further, as emphasized earlier, all predicted loads and deformations must be sustained for seven fully reversed cycles. It is



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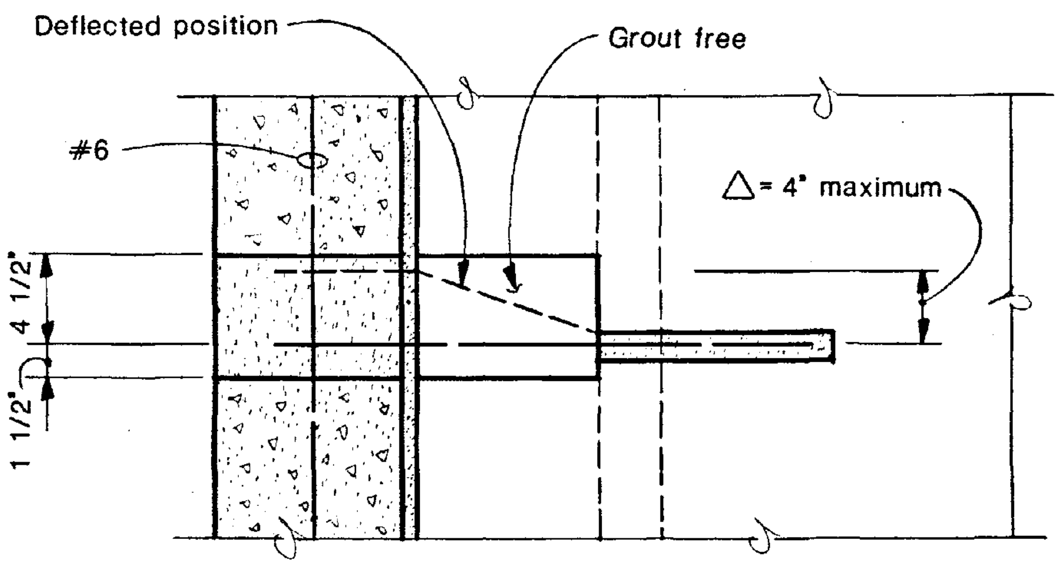


SECTION

FIGURE 5.26 - NON-LOAD BEARING WALL TO LOAD BEARING WALL

suggested that strength, ductility, and reliability of the connection be experimentally verified by tests during Phase 3.

Ultimate capacity of the connection may be computed using a truss action analogy. Under the imposed joint displacement, the vertical component of force in the bent bar resists the applied shear. Plastic moment resistance in the bar is of negligible importance. For a 4-inch relative displacement, the shear resistance offered by one connection is

$$V_n = A_v * F_s * \sin(\theta) = 0.62 * 72 * 4.0/8.94 = 20 \text{ kips}$$

Thus, three or four connectors per wall panel should provide the required 55-kip hold-down force. An appropriate strength reduction factor of this connection should be verified during testing in order to establish a joint capacity for design.

A hysteresis effect, similar to that described for vertical Dywidag connections in the precast walls, occurs in this joint. This will result in opening of the vertical joint between walls after one rocking cycle. However, the ultimate strength of the connection should not degrade significantly during the seven anticipated loading cycles if Grade 40 bar is used.

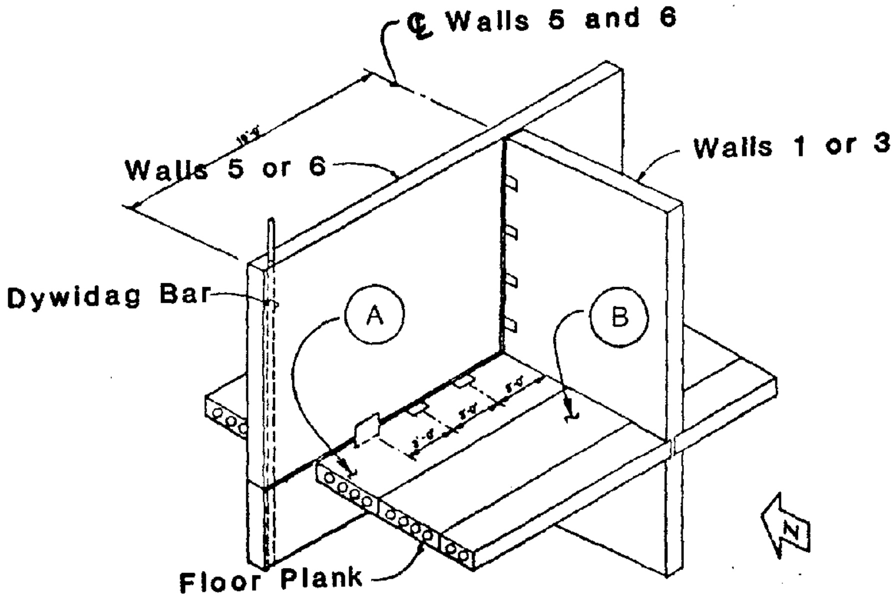
5.9.6 Hollow-Core to Hollow-Core Edge Joint

5.9.6.1 Description

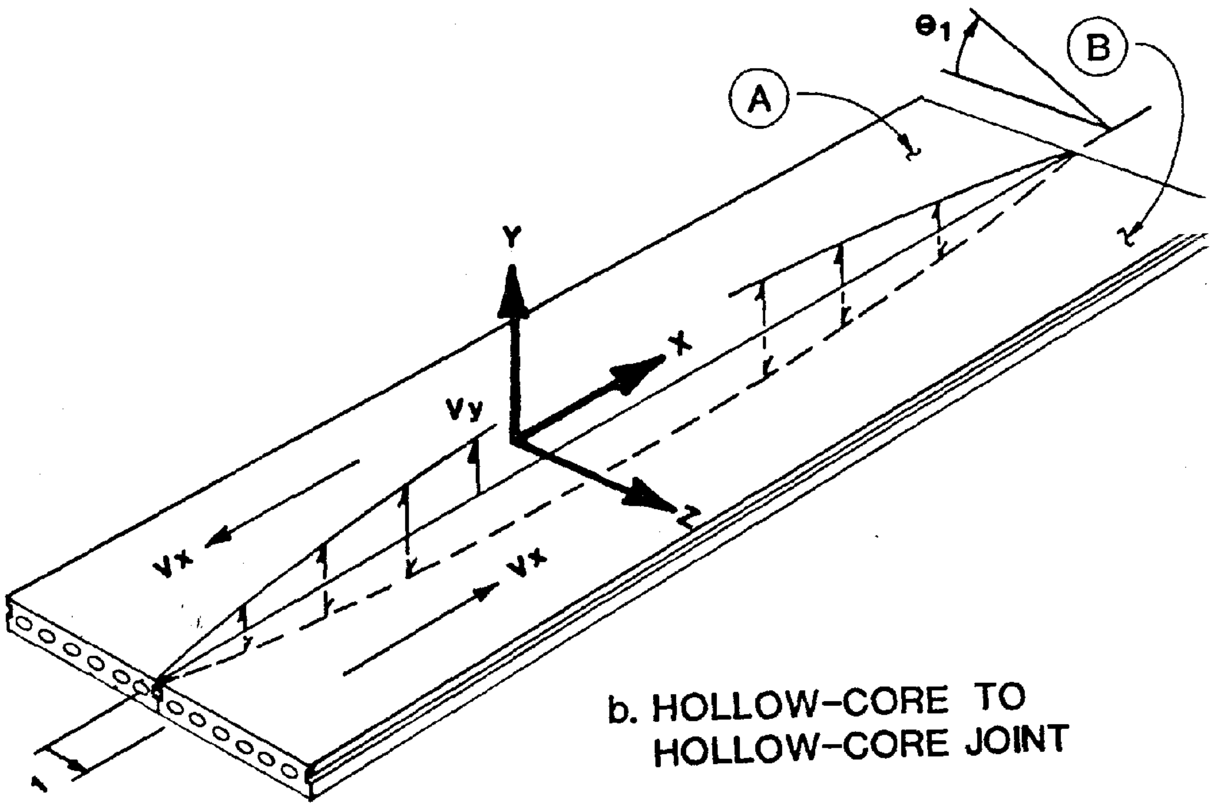
Roof and floors of the Chameau Condominiums are constructed of 8-inch-thick hollow-core planks with typical spans of 26 feet between the precast bearing wall units. Fig. 5.27 depicts a longitudinal joint between adjacent planks.

5.9.6.2 Joint Actions

It was assumed that hollow-core edge joints would be grouted and that there would be no topping slab. Analytically, each bay of the roof and floors has been modeled as a rigid frame assuming full rotational fixity about the Y-axis of each plank end where it frames into a load-bearing wall; in



a. TYPICAL LOCATION



b. HOLLOW-CORE TO HOLLOW-CORE JOINT

FIGURE 5.27 - HOLLOW-CORE TO HOLLOW-CORE JOINT

consideration of service-load cracking, it was further assumed that there is no longitudinal sliding resistance between adjacent planks within the interior of the span.

5.9.6.3 Seismic Performance Criteria

If the plank-end vertical moment is viewed as a couple, with an X-axis "push" at one edge and a "pull" at the other, another possible behavior mechanism presents itself. The transverse diaphragm shear, V_x = 10 kips (shown in Fig. 5.13) resolves into a longitudinal shear, V_x', of 6.8 kips, as indicated in Table 5.7.

Actual behavior of the diaphragm depends on the presence or absence of shear resistance on the longitudinal joints between planks. If the joints are uncracked or if a viable shear-friction mechanism exists, the 6.8-kip shear may be carried by the grout key along the span. If no longitudinal shear resistance exists, Y-moments at the plank ends are necessary to equilibrate the 10-kip transverse diaphragm shear of Fig. 5.13.

In combination with the longitudinal shear, a vertical shear, V_y, may be present due to locked-in forces developed by correction of differential camber, differential live load on adjacent planks, or restrained vertical deflections where the diaphragm is connected to nonbearing walls parallel to the plank span.

Relative vertical displacements along joints between intersecting walls were discussed with Fig. 5.25, above. In-plane shear connections between diaphragms and longitudinal walls were discussed with Fig. 5.20. The potential for large vertical diaphragm forces under north/south seismic excitation, where the diaphragm is tied to longitudinal walls, is clear. Because these connections are intended for east/west resistance, their failure under north/south response seems acceptable and is preferable to damaging the diaphragm by the imposition of large vertical deformations.

On the other hand, vertical shears due to restraint of live load deflections must not produce damage. For plank with a nominal 30-foot span, 4-foot width, 8-inch depth, and 40-psf uniform live load, Spencer [19] predicts maximum vertical shear stresses of 3.4 psi at the longitudinal joint

TABLE 5.7. SEISMIC PERFORMANCE REQUIREMENTS FOR HOLLOW-CORE TO HOLLOW-CORE EDGE JOINT

	Degree of Freedom					
	Translation			Rotation		
Condition	X	Y	Z	X	Y	Z
Fixed	V_x 6.8 kips	V_y 3.4 psi				
Free			δ_1 cracked	θ_1 3 degrees		

one plank width away from a rigid connection between diaphragm and longitudinal wall.

Because relative rotations with respect to the X-axis are not restrained by the anticipated plank-to-plank edge connection, there is a possibility of joint deformation in this coordinate. The largest deformation which has been calculated is an X-rotation of one plank with respect to its neighbor at the edge of the corridor. Due to flexural rotations of the north/south cross walls under north/south seismic loading, one side of the corridor moves up 3.4 inches with respect to the other. This results in a relative X-rotation, θ_1, of about 3 degrees.

Seismic performance criteria for connections across the hollow-core to hollow-core edge joint are summarized in Table 5.7.

5.9.7 Other Locations

In these examples, seismic performance criteria for some important interelement connections have been presented. While the list is not exhaustive, systematic consideration of the structure's seismic forces and deformations, using the approach illustrated above, would enable performance requirements for connections across other joints of interest to be deduced.

5.10 SUMMARY

The rational methodology for the derivation of connector performance requirements described in Chapter 3 has been illustrated by application to a 17-story precast bearing wall building. Beginning with an investigation of the effects of various modeling assumptions, it was concluded that foundation flexibility has a significant influence on the predicted distribution of lateral loads among the walls, and that coupling of north/south cross walls due to out-of-plane stiffness of the floors has a significant influence on predicted vibration period.

Appropriate choices were made for these parameters and, based on computed vibration periods, lateral forces for the design ultimate and elastic demand limit states were computed using provisions of the Uniform Building Code and ATC-3, respectively. Then, using the ratio of elastic strength demand to design ultimate strength (i.e., the R-value), the equal energy principle was applied to estimate the maximum plastic displacement at the top of the structure during the code-specified, once-in-a-lifetime earthquake.

According to the information presented in Table 3.2, the lateral vibration periods of 0.5 and 0.8 second in the north/south and east/west directions, respectively, together with the chosen R-value of 2.6, suggest that the structure must sustain seven fully reversed cycles of loading and deformation.

Kinematic principles were applied to translate the gross lateral movement at the top of the structure into displacements and rotations across individual joints. These values, combined with predicted forces and moments obtained in the lateral load analyses, were used to quantify the seismic performance requirements for some of the important connections between wall panels and between walls and floors.

Conceptual sketches of appropriate connection details were presented for each of the joints. For this structure, the detailing philosophy was to provide for significant ductile action in the connections between precast elements, while the elements themselves are intended to remain elastic. Dywidag post-tensioning bars running continuously over the height of the structure are the principal ductile elements of the lateral force resisting system.

In all other connections, Grade 40 reinforcing bars have been used. The intention is that the connection loads develop relatively high stresses in the bars, that sufficient unbonded lengths be provided to permit significant plastic elongations without exceeding the ultimate tensile strain, and that the bars be anchored well enough that they will not pull out. Thus, these connectors are viewed as ductile fibers between comparatively brittle precast elements; their principal function is to prevent relative displacements that would jeopardize the structure's stability.

Although considerable attention was devoted to the conceptual development of suitable connection details, the primary objective was to present quantitative seismic performance criteria by which any connection detail proposed for a given joint could be evaluated by physical testing in Phase 3.

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 SUMMARY

Seismicity maps of the United States show a likelihood of major earthquake damage along much of the west coast, on isolated stretches of the east coast, and within small regions of the east and west central states. Over the rest of the country, however, the expected intensity of seismic damage is moderate or less, and this environment has provided the greatest market for precast concrete buildings.

Earthquake engineering was in its infancy when the precast buildings industry evolved and seismic issues attracted little attention. Consequently, precast components, connection details, production tooling, and basic framing schemes of some common precast building systems in use today reflect an investment in technology not ideally suited to earthquake resistance.

This report was funded by the National Science Foundation (NSF) and was conducted during Phase 2 of a three-part program conceived by the Prestressed Concrete Institute (PCI) to advance the state-of-the-art of connection design for seismic-resistant precast concrete structures. The Phase 1 report, also funded by NSF, presents an overview of the current state-of-the-art. Physical testing of connections is to be performed by others in Phase 3.

One objective of this study was to explain seismic response of buildings in a way that is useful to design practitioners having no formal training in earthquake engineering or structural dynamics. Earthquakes and their capacity to inflict damage have been described in terms of energy. Due to their flexibility, structures have a capacity for absorbing elastic strain energy. Due to their mass, they have a capacity for absorbing kinetic energy. Due to damping and inelastic action, seismic energy which has been absorbed can be dissipated.

Seismic design was likened to the task of sizing a reservoir system to accommodate an uncertain volume of water. Two approaches could be considered. In the first, a single tank would be provided which would need sufficient capacity to hold the largest volume anticipated. In the second approach, a primary tank would be sized to accommodate the most likely storage demand while a less costly secondary reservoir would be provided to catch any excess flow after the main tank has been filled.

Designing a structure to remain elastic during an earthquake is analogous to the single-tank approach. The strength provided would need to match the largest anticipated seismic forces. Usually the expense to ensure elastic behavior (i.e., single tank) would not be justified because well-detailed structures possess ductility and thus an inherent capacity for energy dissipation (i.e., an overflow reservoir) at no extra cost.

From this perspective, the seismic design approach of modern building codes was explained. Structures are designed to yield under the most severe ground shaking anticipated at the site. Code detailing requirements ensure sufficient ductility to dissipate the seismic energy input to the structure after its elastic limit has been exceeded in a strong earthquake. Code strength requirements, on the other hand, are large enough to ensure elastic behavior under minor ground shaking.

The important design issue is one of selecting the most favorable combination of elastic energy storage capacity (strength) and inelastic energy dissipation capacity (ductility) for the intended construction materials and framing scheme.

Building code requirements for cast-in-place concrete are sufficiently advanced to assure satisfactory performance under usual circumstances. In contrast, for some categories of precast construction, knowledge of seismic behavior has not been adequate to support the formulation of seismic code provisions.

To clarify the applicability of typical building codes in the design of precast structures, two categories of precast construction, "jointed" and "monolithic," were identified. In monolithic construction, precast elements are joined by well-reinforced connections possessing continuity of stiffness, strength, and ductility comparable to well-designed cast-in-place concrete. Jointed construction describes all means of connecting precast components in which the interelement boundaries behave as zones of significantly reduced stiffness, strength, or ductility under the ultimate design limit loads and deformations.

Seismic behavior of jointed and monolithic structures differs in two regards. On one hand, the nonlinear stiffness properties of jointed construction (due to the opening and closing of joints during ground shaking) can result in reduced seismic energy input. On the other hand, the relatively small volumes of material mobilized for inelastic action (due to stress concentrations at the discrete connections) means jointed structures have a smaller capacity for energy dissipation than their conventionally detailed monolithic counterparts. Building code provisions for cast-in-place reinforced concrete are appropriate for the design of monolithic precast structures. Most precast buildings, however, are of jointed construction.

While the precast industry seeks expanded markets in more seismically active regions, seismic risk assessments for regions traditionally viewed as earthquake-free are being revised upward as seismologists continue to compile and evaluate geological and historical data. To meet these challenges, studies are needed to quantify seismic demands for strength and ductility in the connections of jointed precast buildings, and to develop cost-effective adaptations of existing technology.

Thus, a second Phase 2 objective was to develop a rational design methodology which enables building engineers to estimate the magnitude of inelastic deformations which will occur in the connections of a jointed structure during a strong earthquake. A simple approach was described which begins with the selection of a suitable lateral force resisting system. Methods for computing design loads and estimating a structure's global inelastic displacements during a damaging earthquake were presented. Finally, a kinematic analysis procedure was described by which motions of the yielded lateral resisting system are studied and inelastic deformations of connectors in specific joints are estimated.

To illustrate the benefits of a connection design approach which considers inelastic deformations due to earthquakes as well as seismic forces, the proposed methodology was applied to two structures typical of precast construction in the United States. At the suggestion of the PCI Technical Input Group established for this project, a precast parking garage with seven levels above grade and a 17-story precast bearing wall apartment building were selected for study. Conceptual sketches of appropriate connection details were presented for each of the joints considered. In addition, the effort focused on the derivation of quantitative seismic performance criteria by which any connection detail proposed for a given joint could be evaluated by physical testing in Phase 3.

6.2 CONCLUSIONS

The economic and functional success of a jointed precast structure depends to a great degree on the connections. Designing connections that are easily fabricated, speedily erected, stable, strong, and ductile is a demanding task. Due to the lack of guidelines, the requirement of substantial ductility in the connections of jointed precast structures, when design strengths are set by the usual code criteria, often is overlooked.

Research is needed to identify the proper balance between strength and ductility for jointed precast concrete. Is the apparent disadvantage of reduced energy-dissipating capacity offset by the apparent advantage of reduced seismic energy input? This question challenges the validity of traditional design procedures which proportion jointed construction according to lateral force specifications for monolithic concrete, without ensuring comparable capacity for inelastic energy dissipation.

In the past, it appears that much of the experimental data on connections for precast structures has been obtained through privately funded research and has been held as proprietary. If improved design procedures and code provisions for precast structures are desired, improved communication between researchers, producers, designers, and code officials seems likely to accelerate their development.

In this report, the basis of code seismic provisions has been explained and a design methodology which addresses connection ductility requirements has been presented. Rational performance criteria derived for connections in the example structures, intended for service in zones of moderate seismicity, will provide a quantitative basis for the evaluation of physical specimens in Phase 3. Also, while the consideration of jointed structures in regions of high seismic risk is beyond the scope of this study, guidelines were developed which suggest the nature of changes required in adapting traditional jointed construction for service in higher seismic zones; connector strength must increase directly with the base shear ratio while the volume of material mobilized for plastic energy dissipation must increase as the square of the base shear ratio. Further, the number of inelastic deformation cycles which the damaged connectors must endure increases, as described in Table 3.2.

Dissemination of these results may help to promote uniformity of approach and attention to seismic-resistant detailing in the design of precast concrete structures, large and small. This will increase the opportunities for observation and evaluation of state-of-the-art connection performance, enhance the credibility of jointed precast construction in seismic regions, and accelerate the evolution of connection design technology.

Technological evolution is iterative, involving hypothesis, experimentation, evaluation, and deduction. Although it has been necessary in this work to assume answers for some important questions, the essential considerations for rational detailing of connections in seismic-resistant jointed precast structures have been identified. While it may be necessary to modify details of the methdology as improved understanding is gained, the basic framework should remain useful.

Evolution of seismic-resistant design technology is the product of effective communication between industry, government, the research community, and private consulting practice, motivated by a shared commitment to earthquake safety. This report is offered as evidence of technological evolution in progress.

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