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A procedure for estimating the effects of a single, predefined, earthquake on a facility used by a high-technology industry has been developed. The procedure is capable of making four types of seismic damage predictions: building repair costs, equipment repair costs, length of business interruption, and total corporate financial losses. The component approach is used to make all four types of damage prediction. The basic premise of the component approach is that a seismic loss prediction for a facility can be made by first estimating the damage to all of the facility's components and subsequently combining the component damage estimates through a consequence analysis. The key to the component approach is having damageability models (relations between local component demand and component damage) available for all the components found in the facility. As a portion of this project, damageability models have been compiled from other sources for most typical components found in high-technology facilities.

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ABSTRACT

A procedure for estimating the effects of a single, predefined, earthquake on a facility used by a high-technology industry has been developed. The procedure is capable of making four types of seismic damage predictions: building repair costs, equipment repair costs, length of business interruption, and total corporate financial losses.

The component approach is used to make all four types of damage prediction. The basic premise of the component approach is that a seismic loss prediction for a facility can be made by first estimating the damage to all of the facility's components and subsequently combining the component damage estimates through a consequence analysis.

The key to the component approach is having damageability models (relations between local component demand and component damage) available for all the components found in the facility. As a portion of this project, damageability models have been compiled from other sources for most typical components found in high-technology facilities.

EXECUTIVE SUMMARY

This report presents the results of Phase I of a proposed two-phase research program conducted by EQE Incorporated, for the National Science Foundation (NSF). The overall objective of the program is to develop a technology for predicting the potential impact of a catastrophic earthquake on high-technology industries and also for predicting the resulting effects on the local and national economies.

Phase I deals with a single facility. Phase II will use the information developed for single facilities in Phase I to estimate, through a systems analysis, the impact of an earthquake on an industry and on the local and national economies.

A procedure for estimating the effects of a single, predefined, earthquake on a facility used by a high technology industry is developed in this report. The procedure is capable of making four types of seismic damage predictions for a single facility:

- Building repair costs
- Equipment repair costs
- Length of business interruption
- Total corporate financial losses

The four types are interrelated. The results from one of the predictions may be a key input to another. Localized ground rupture or liquefaction effects are not expressly considered; rather, only vibratory ground motion is. These other secondary effects must be handled case by case.

A single seismic damage prediction philosophy has been adopted (the component approach) to make all four types of damage prediction. The basic premise of the component approach is that, by making a seismic damage prediction for each component of a facility, and combining the component damage estimates (through a consequence analysis), damage to the facility can be estimated.

The general seismic risk analysis methodology presented here includes the following steps:

1. Prediction of seismic hazard at the sites under study

2. Inventory and classification of all relevant entities (e.g., building components and equipment)

3. Development of damageability models (motion-damage relationships or fragility functions) for the entities

4. Estimation of damage to the entities inventoried in Step 2 when subjected to the ground motions predicted in Step 1

5. A consequence analysis to determine the effect of the estimated damage on the operation of the facility and on personnel

Step 1, prediction of seismic hazard at the site under study, requires establishing a peak ground acceleration and response spectra for the site during a specific predefined seismic event. The parameters of importance in this step are the magnitude of the seismic event, the source-to-site attenuation distance, and local soil conditions.

Next, the analyst must determine what building components and equipment at the facility are present and will be affected by a seismic event. This information may be obtained by field surveys of the building components and equipment or by analogy with facilities of a similar character for which the inventory is known. This report presents a building component and equipment classification scheme.

Damageability models are presented for the generic building component and equipment classes discussed in the report. The loss-prediction estimates can be improved if the analyst is able to develop damageability models for the specific building components and equipment present at the facility under study. The damageability models can be developed from a combination of theoretical considerations, empirical data, and expert judgment.

Damage to building components and equipment can be estimated from the established ground motions through a structural analysis of the facility. The structural analysis will give the analyst an estimate of the local demand placed on each component. The demand is then related to the component damage through the damageability models already developed.

Finally, a consequence analysis can be applied to the physical damage estimates made in the preceding steps to determine building and equipment repair costs, length of business interruption, and total corporate financial losses.

1. INTRODUCTION

1.1 Statement of the Problem

This report presents the results of Phase I of a proposed two-phase research program conducted by EQE Incorporated, for the National Science Foundation. The overall objective of the program is to develop the technology for predicting the potential impact of a catastrophic earthquake on high-technology industries and also for predicting the resulting effects on the local and national economies.

The industries that are the target of this study -- the electronics, aerospace, defense, biogenetic, and chemical industries -- represent a significant segment of the overall economy of the United States. A major interruption in any of them could conceivably have crippling effects on the nation's economy and security. They also have other unique characteristics that merit them an industry-specific study. Some of these characteristics are as follows:

- High-technology industries are very competitive. Each company in the industry depends on continued research and development to preserve its market position and growth. An interruption of its research and development functions for any significant length of time could be detrimental to the company. This may be the most important characteristic listed here.

- High-technology industries are complex systems of many companies, small and large, each specializing in some aspect of a product's design, manufacture, or assembly. Thus, each company in a high-technology industry must interact with other companies in that industry and this interdependence complicates any attempt to evaluate seismic risk.

- High-technology industries tend to concentrate in certain areas of the country because of the need for interaction between companies, the need to be near major universities and research centers, and the need to be in a geographically desirable location to attract highly qualified personnel. Some of these concentrated industrial developments, such as the electronics industry in the Santa Clara Valley of Northern California and the aerospace industry in the Western Los Angeles Basin of Southern California, are in areas of high seismicity. A major damaging earthquake is almost certain to occur within the next decade or two and may have devastating effects on these industries.

- Certain types of construction seem to be prevalent in most high-technology industrial developments. For example, most of the buildings housing the electronics industry in the Santa Clara Valley are concrete tilt-up structures of one or two stories. In a fast growing, highly competitive industry a particular construction type is typically selected not for its seismic resistance but rather for the speed and cost of construction. Concrete tilt-up structures performed poorly in the 1971 San Fernando, California earthquake and may do the same in an earthquake of similar or larger magnitude near the Santa Clara Valley.

- High-technology industries depend on costly manufacturing and process equipment. Presently available seismic damage-prediction methodologies are oriented towards structural damage. Most post-earthquake damage reports concentrate on structural damage only; there is very little experience data available on the performance of complex manufacturing and process equipment in past earthquakes or on the economic consequences of structural and equipment damage.

The problem of estimating earthquake effects on high-technology industrial facilities and on the economy in general is complicated by the items mentioned above. All of these concerns must be reasonably represented in any methodology used to predict earthquake damage to high-technology facilities.

1.2 Purpose

The objective of Phase I of the two-phase study was to develop a procedure for estimating the seismic damage inflicted on a single facility during a specific predefined seismic event. Phase II will employ the Phase I methodology to assess the effects that damage to one facility will have on other facilities and on the broader economy.

The users of the methodology will be industry decision makers and the public. The methodology can be used for the following purposes:

- To predict dollar damage that industrial facilities may have suffered. This information is valuable to decision makers for estimating their organizations' financial vulnerability and risk-reduction needs and for planning future expansion.

- To evaluate the costs and benefits of various alternative damage-mitigation schemes. If the initial study indicates unacceptably high expected damage for a certain manufacturer, he may wish to reduce his expected losses through selected structural and equipment strengthening, geographical dispersion of facilities, or introducing redundancies into his interactions with other manufacturers who may be vulnerable to earthquake effects.

- To evaluate the social and economic effects on the local community of the earthquake damages to the industry. In communities where the industry is the major source of employment, these effects may be devastating and longlasting. The use of the methodology in estimating earthquake effects would provide local authorities with information necessary for city and regional planning and emergency preparedness.

- To evaluate the effect on the national economy. A significant interruption in one high-technology industry may cause interruptions and setbacks in other industries and affect national and international trade as well as national security. If these effects can be predicted in advance, mitigative actions can be taken, and emergency plans can be prepared at the national level.

1.3 Scope

The methodology developed in this research program is intended for high-technology industries although it could be applied to other industries and other general earthquake risk analyses.

The methodology is demonstrated in reference to the electronics industry in the Santa Clara Valley. Data on prevalent structural and equipment types were developed for this specific industry and location. The procedures used in the development of these data are described in sufficient detail to enable potential users to develop similar data for other industries and locations.

The work described in this report consisted of four major tasks:

- Task 1: Procedures were developed to identify typical structures and equipment used by a specific industry. Procedures were also developed to determine average response and damageability characteristics of the typical structures and equipment.

- Task 2: The state of the art in earthquake ground motion prediction was reviewed, and a step-by-step procedure for use in the methodology was recommended.

- Task 3: A procedure was developed: (1) to combine the ground motion prediction with the structural and equipment damage models and (2) to estimate damage for a single facility.

- Task 4: The methodology developed in the three previous tasks was demonstrated for a hypothetical electronics facility representing average characteristics of the reviewed electronic manufacturing facilities in the Santa Clara Valley.

- Task 5: After the Morgan Hill, California earthquake (April 24, 1984), the scope of the work was extended to include a comparison of predicted losses for structural and non-structural components for concrete tilt-up structures to actual losses suffered in the earthquake. Such comparisons for two buildings are included in Appendix B.

2. OVERVIEW

The procedure presented in this report is capable of making four types of seismic damage predictions for a single high-technology facility:

- Building repair costs
- Equipment repair costs
- Length of business interruption
- Total corporate financial losses

The four damage predictions are interrelated. The results from one may be a key input to another.

Localized ground rupture or liquefaction effects are not expressly considered; rather, only vibratory ground motion is. These other secondary effects must be handled on a case-by-case basis.

A single seismic damage-prediction philosophy has been adopted (the component approach) to make all four types of damage predictions. The basic premise of the component approach is that, by making a seismic damage prediction for each component of a facility, and combining the component damage estimates (through a consequence analysis), damage to the facility can be estimated. Figure 1 is a flowchart of the procedure.

The same procedure will be used to make all four of the desired seismic damage predictions for a single facility. As can be seen in Figure 1, there are two parallel initial tasks: determining the seismic hazard at the site of the facility and developing inventory and damageability models for all relevant components.

The ground motion input to the facility is determined by the site seismic hazard model, which describes the expected ground motion intensity at a site from a given earthquake. The model takes into consideration the geologic characteristics of the region, the historical seismicity data, and the local site characteristics.

The inventory model identifies all relevant physical components in a facility. The physical components may include structural members such as beams, columns, and shear walls and nonstructural members such as interior partitions, glass, curtain walls, HVAC equipment, plumbing, completed products, raw materials, critical production equipment, and other building contents.

The damageability model describes the relationship between the strong-motion input to a component (demand) and the damage expected to be incurred by that component. The component damageability models are developed through a combination of theoretical and empirical means as well as through the judgment of experienced professionals.

Because most high-technology facilities are composed of a large number of components, gathering detailed data for each component is not feasible or required. To simplify the inventory effort, a set of typical component classifications is developed (see Tables 5, 6, and 10 in Chapter 6), and the components of each class are modeled by the average values of the relevant parameters.

The next step in the procedure is calculation of response. At the component level, the seismic input is the local demand the component is subjected to, rather than the site ground motion. The local demand must be computed from the site ground motion. This is a routine structural engineering problem that can be solved using one of many methods, depending on the accuracy and sophistication required.

Next the analyst computes the damage to components. The computation of component damage, given the local demand (derived from the response computations) and the inventory and damageability models, is a straightforward, although repetitive, task. It may be desirable, at some future date, to develop computer programs to carry it out.

The most important step in the component approach to damage prediction is the consequence analysis. For two of the desired damage predictions (i.e., cost of building repair and cost of equipment repair), the consequence analysis is not difficult; it is a simple summation of the building or equipment component repair costs, with one exception: the analyst must look at the total picture to see whether there are any interactions between types of component damage. For example, equipment damage may result from building damage (e.g., building collapse), or one piece of equipment may damage another (e.g., toppling storage racks).

The consequence analyses for the other two types of damage prediction, the length of business interruption, and the total corporate monetary loss are significantly different from those discussed in the previous paragraph. Causal models (i.e., fixed, well-known relationships between component damage and consequences) can only be developed for these damage predictions at considerable expense. At this time, it is not deemed desirable or necessary to do this, at least not until the phenomena involved are better understood. For both damage types, qualitative methods of analysis are preferable, in particular the Delphi method and the panel consensus method. In both, a group of engineers and managers from the facility under study are assembled and presented with a seismic damage scenario. Through their knowledge of the facility and their expertise, the group estimates the expected length of business interruption and the total expected corporate monetary loss.

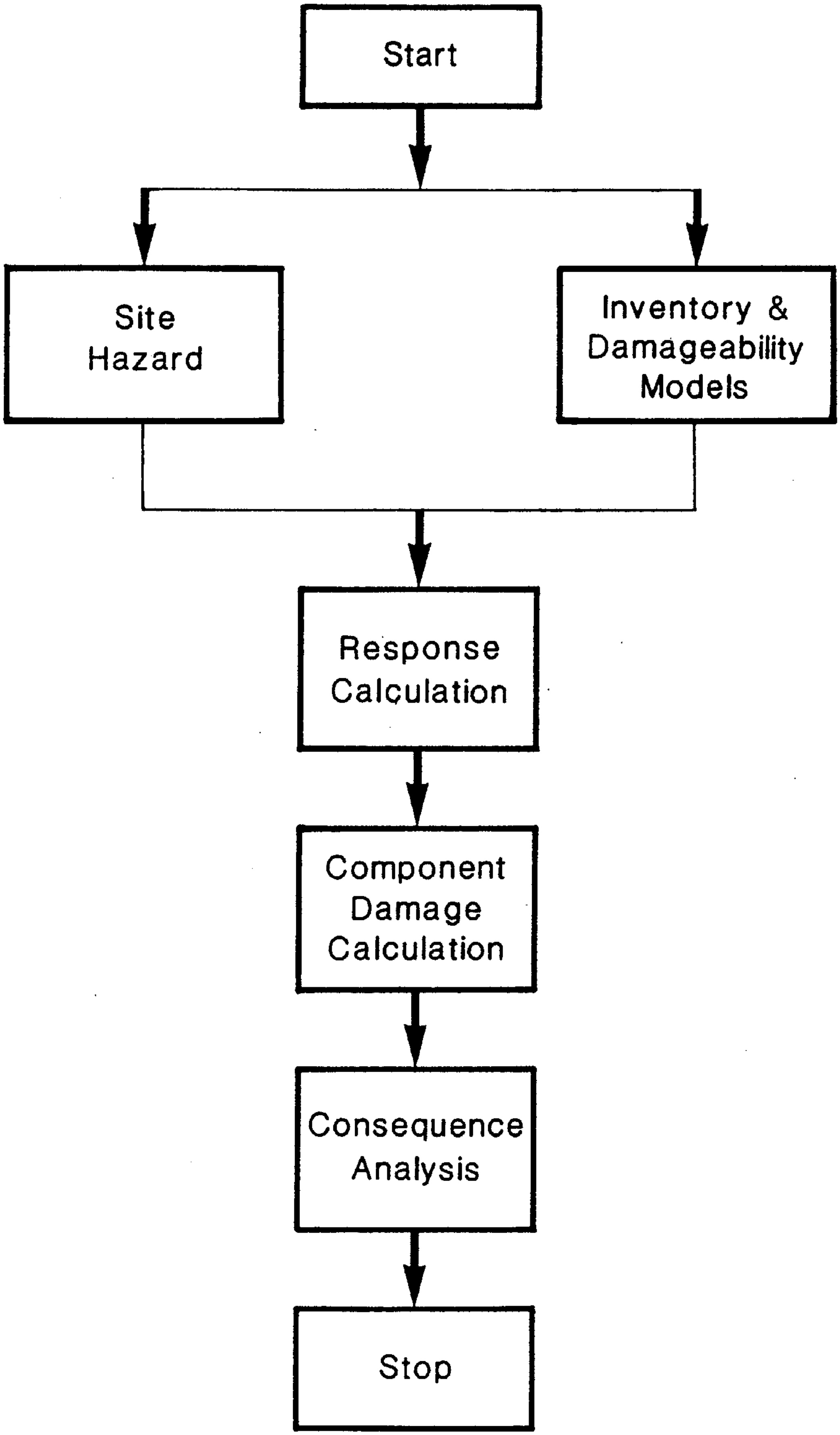

Figure 1: Procedure for Estimating the Effects of a Large Earthquake on a High-Technology Facility

3. REVIEW OF THE STATE-OF-THE-ART GROUND MOTION PREDICTION METHODS

3.1 Introduction

3.1.1 General. Ground motion is produced by the passage of seismic waves emanating from a region of the earth's crust where fault movement has occurred. The estimation of future ground motion requires consideration of a rapidly expanding body of information and the interaction of several disciplines, such as geology, geophysics, seismology, and geotechnical and structural engineering. Although knowledge is progressing at a fast pace, numerous uncertainties exist, so that the complex nature of ground motion can only be simulated through processes that decouple generally recognized but often insufficiently understood effects.

This chapter reviews our knowledge of these processes and presents methods of estimating their effects. Based on the review a methodology is developed for the estimation of ground motion that is appropriate for application to high-technology industries.

3.1.2 Factors Influencing Ground Motion. Ground motion is influenced by the seismic event's source characteristics, the properties of the transmission path, and the local soil conditions.

Source characteristics include fault type, rupture dimensions and mechanism, focal depth, stress drop, and energy released (measured by the magnitude or the seismic moment).

The effects of transmission path properties relate to the spreading or absorption of earthquake energy as the seismic waves travel away from the source. They also include phenomena due to crust inhomogeneities and directivity effects (Singh, 1982).

Local effects result from the geological conditions present at the site (Seed and Idriss, 1983) and from the possible interaction between structures and the surrounding media.

3.1.3 Prediction of Ground Motion. All factors cannot, practically, be included in the estimation of future ground motion. Typically, one source factor only -- magnitude -- and a single transmission path parameter -- distance -- are considered. Local effects are often disregarded or limited to the simple distinction between rock and soil sites.

The ground motion variables of engineering interest include absolute or effective maximum acceleration, velocity, and displacement; frequency content; and duration. Phase characteristics of ground motion are of considerable interest, but they have not been systematically included in the specification of seismic design criteria.

Several subjective factors play a role in the selection of ground motion variables for design purposes (Seed, 1983). These factors include the overall estimate of the seismic hazard, the postulated magnitude and frequency of occurrence of the maximum event, the use and commercial value of structures and contents, the level of risk acceptable to society and its representatives, and the level of risk acceptable to the owners of the facility, including the consequences of underestimating or overestimating it.

The seven basic steps necessary to predict ground motion have been described by Hays (1980). They are:

1. Determination of the seismicity
2. Identification of seismotectonic features
3. Estimation of regional attenuation
4. Estimation of ground-shaking parameters
5. Definition of ground response spectra
6. Evaluation of local effects
7. Estimation of uncertainties

Steps 1 and 2 consist of historic earthquake and geologic data reviews, at a level of effort consistent with the damage prediction to be made.

Steps 3 through 6 are usually performed using a deterministic or probabilistic approach. The estimation of uncertainties, Step 7, implies probabilistic considerations.

3.2 Seismicity and Seismotectonic Features

Knowledge of historical earthquake data and the identification of seismotectonic features, such as capable or active faults, provide initial estimates of potential earthquake hazard and background information for more detailed studies.

3.2.1 Historical Earthquake Data. Compilation of historical earthquake data gives an indication of the severity of future earthquake motion, under the assumption that past events could reoccur at or near the same location. Isoseismals, or curves of equal felt or observed damage and earthquake effects, have been compiled for areas such as California (California Division of Mines and Geology, 1981). Isoseismals of significant historical earthquakes remain the best way to derive intensity attenuation functions in the absence of other data.

Since the determination of isoseismals is a subjective process, emphasis is now given to recorded earthquake data. Extensive earthquake catalogs, such as those maintained by the National Oceanic and Atmospheric Administration (NOAA) and the California Division of Mines and Geology (CDMG), provide information on earthquake sizes and locations and on such other parameters as source mechanisms, dimensions, magnitudes, focal depths, and epicentral intensities.

Of particular significance are any available acceleration records of strong earthquakes in the region of interest. Since the initial deployment of the strong-motion accelerograph network about 45 years ago, nearly 300 free-field or basement motions have been recorded, processed, and analyzed. More limited information exists regarding large earthquakes or motions at short distances from the earthquake source (near-field motions). Furthermore, the correction and digitization procedures implemented until recently have been questioned

(Trifunac and Lee, 1978), which further emphasizes a pressing need for more complete and reliable earthquake information. United States earthquake data are supplemented by ground-motion records recovered during experimental nuclear explosions and by information pertaining to foreign earthquakes (Crouse et al., 1980).

Despite significant progress achieved in the past 10 years regarding the gathering of earthquake data, few records, if any, are usually available that apply to a specific study area.

3.2.2 Geological and Geophysical Data. The importance of determining upper bounds of magnitude and seismicity from geological evidence, rather than from limited historical records, has been unequivocally recognized. Geologic mapping data, low-angle and infrared photography, and local trenching and geophysical measurements are interpreted to identify active faults, which control the occurrence of most earthquakes. This information concerns fault length, degree of activity (seismicity estimated from rate of deformation), geometry, amount and direction of movement, temporal history, correlation with historical seismicity, and estimate of upper bound of potential energy release. Geological, historical, and seismological factors are therefore necessary to measure fault activity and differentiate faults with different rates of slippage.

The need to properly identify fault types has acquired increased recognition because strike-slip, normal, and reverse faults may be associated with different types of ground motion (McGarr, 1982).

Locations of future large earthquakes can also be inferred from the recent concept of seismic gaps, which integrates the history of previous large earthquakes with the rates of tectonic plate motion and geologic slip. Current understanding of seismic gaps, summarized by McNally (1982), demonstrates the significance of seismic quiescence and locked zones along tectonic boundaries with respect to future earthquake occurrence.

3.3 Deterministic Estimates of Ground Motion

The most widely used procedure for defining ground motion parameters is the use of empirical relationships, which are based on statistical regressions of interpreted trends and derived from observed strong-motion data. General estimates of the frequency contents of ground motion are also used.

3.3.1 Empirical Relationships. Available earthquake data have been used to establish relations between earthquake variables, such as distance and observed intensity; magnitude and length of fault rupture (Mark and Bonilla, 1977; Slemmons, 1982); magnitude and duration (Bolt, 1973); etc. Empirical relationships, termed attenuation equations, relate expected peak motion parameters, such as acceleration, velocity, or displacement, to distance and magnitude. These equations are sensitive to the estimates of these latter quantities, and the scatter between observed and predicted values is usually fairly significant because many factors, including but not limited to local site conditions and the conditions of installation of the recording instruments, affect strong-motion measurements. Attenuation relationships provide estimates of potential earthquake ground shaking at some distance away from recognized potential earthquake sources. Frequently used attenuation relationships are those established for peak horizontal ground acceleration; the most widely accepted equations for this variable can be found in Donovan and Bornstein (1977), Blume (1977c), Campbell (1981), Joyner and Boore (1981), and Bolt and Abrahamson (1982).

The applicability of empirical predictions provided by several of the above equations has been critically reviewed by Bernreuter (1977), Boore and Joyner (1982), Bolt (1982), and Donovan (1982a). These predictions have been shown to be sensitive to many factors, confirming that an essential parameter of attenuation equations is the associated error term.

In an attempt to reduce uncertainties associated with attenuation equations, weighted averages of applicable relationships for a particular mode of motion, such as acceleration, velocity, or displacement have been used (Eguchi, 1980), or the data have been treated with partitioning in distance (Blume, 1980) or magnitude (Joyner and Boore, 1981). Partitioning and weighted averages generally provide lower estimates at short distances from the postulated source. So far, few attenuation relationships have been developed for peak velocity and displacement or for the vertical component of ground motion, although vertical motion possibly predominates in the near field (Bolt, 1981).

3.3.2 Duration of Strong Ground Motion. The duration of earthquakes can be measured in different ways. Of significance to engineers are the bracketed duration, measured between the first and the last occurrence of acceleration pulses greater than 0.05g, at frequencies above 2 Hz (Bolt, 1973), and the Husid duration (Husid, 1973). Chang and Kritnitzsky (1977) reviewed several empirical relationships between magnitude and duration and developed curves relating bracketed duration, magnitude, and epicentral distance that differentiate between soil and rock conditions. Housner (1980) provided revised estimates of the near-field duration of strong motion. Detailed studies, such as the work described in Westermo and Trifunac (1978), concentrate on the influence of other variables, such as frequency and depth of sediments at the recording site, on the duration of earthquake shaking.

3.3.3 Spectral Characteristics. In addition to peak values and duration, the specification of earthquake motion should include the frequency content and predominant period characteristics of seismic waves. Strong ground motion can be resolved into an infinite series of simple harmonic functions in the frequency domain (Fourier spectrum), but engineers prefer response spectra, which represent the maximum response of a single-degree-of-freedom system as a function of period or frequency.

Spectral characteristics of earthquake motion may be determined from peak ground velocity and displacement considerations (Hall, Mohraz and Newmark, 1975; Newmark and Hall, 1982), or by using site-dependent generalized spectral shapes. Studies performed by McGuire (1978) indicate that for intermediate frequencies, spectral response is not necessarily well represented by peak velocity, and generalized spectral shapes appear to be preferable.

Seed, Ugas and Lysmer (1974) developed mean and mean-plus-one-standard-deviation general spectral shapes applicable to rock and other sites. These results are particularly applicable when the design event magnitude is close to 6.5. Similar studies were subsequently performed by Mohraz (1976) and Kiremidjian and Shah (1978). The Mohraz spectra are somewhat similar to the Seed spectra, although they tend to be more conservative in the long-period range. This similarity reflects the use of generally comparable data bases.

Mohraz (1978) extended his work to evaluate the influence of magnitude and duration on spectral shapes. Spectral characteristics of near-source motion at moderate to large magnitudes have been empirically established for rock sites (Johnson and Traubenik, 1978) and other site conditions (Johnson, 1980). Vertical spectral shapes have been published by Mohraz (1976) and Rizzo, Shaw and Snyder (1976). Fundamentally, it is now recognized that the use of generalized spectral shapes remains acceptable when large-magnitude events are expected but that it is overly conservative when the controlling magnitude does not exceed 6.0 (Kennedy et al., 1983). Recent emphasis in the profession has been to give added credibility to site-dependent response spectra (Lawrence Livermore National Laboratory, 1980).

Spectral shapes are generally provided in normalized format (i.e., scaled to 1g). In order to define earthquake motion, they are scaled uniformly and independently from the period considered to a specified peak acceleration. The scaling of site-dependent or independent spectral shapes to a specified peak ground acceleration has often been questioned, whether peak ground acceleration is selected deterministically or probabilistically. A more consistent approach is the use of spectral envelopes derived from a selected number of records closely matching the conditions anticipated at the site (Guzman and Jennings, 1976). Another improved procedure involves the development of consistent spectral probabilities (McGuire, 1979; Anderson, 1980; Vanmarcke, 1980), which provide equal risk estimates regardless of the frequency considered.

An alternative to the use of spectral shapes is that of attenuation equations directly applicable to spectral amplitudes (McGuire, 1977; Tera Corporation, 1982; Campbell, 1983), thereby defining spectral response parameters as a function of magnitude and distance. In attempts to improve these estimates similar to those implemented for acceleration attenuation relationships, some spectral amplitude attenuation studies have been limited to narrow frequency ranges (Hanks and McGuire, 1981) or include magnitude, distance, and soil condition partitioning (Katamaya, 1982).

4. LOSS PREDICTION: A REVIEW OF THE STATE OF THE ART

One of the earliest methods devised for estimating earthquake-induced losses, described in Steinbrugge, McClure and Snow (1969), was developed to aid in analyzing the feasibility and effectiveness of earthquake insurance. The method was developed to predict earthquake losses to wood-frame dwellings in California, using the Modified Mercalli Intensity (MMI) Scale to describe the intensity of ground motion. Because MMI is directly related to damage, no structural response calculation is necessary.

The wood-frame dwellings are divided into four components, and damage is estimated separately for each component. The four components are: structure, interior finish, exterior finish, and chimney. These component categories can be further subdivided to account for major variations within a component (discriminating the components by age is one possible subdivision). For each component, the degree of damage is described by such terms as slight, moderate, severe, and total.

The relationship of MMI to degree of component damage is estimated using limited available data. These MMI-damage relationships are converted into relationships between MMI and repair cost, again through estimates.

The method is good for the intended type of building. The sources of information identified can be of value in similar studies. However, the method requires a great deal of knowledge that can only be provided by experts in the fields of engineering, statistics, and risk analysis. Also, the method cannot be applied to large structures without modifications extensive enough to be considered a completely independent method.

Studies have been performed to improve Steinbrugge's method and to apply it to other types of structures. For instance, Rinehart, Algermissen and Gibbons (1976) performed a sensitivity analysis to determine the relative significance of certain parameters with respect to losses. This analysis has led to improvements in the method. Algermissen,

McGrath and Hanson (1978a) extended the previous work to cover buildings other than single-family dwellings.

Culver et al. (1975) described a set of three methods that are useful for surveying and evaluating existing buildings to determine the risk to life and the expected damage. In each method, damage to both structural and nonstructural building components resulting from earthquakes, hurricanes, and tornadoes is considered. The methods are designed to treat a wide variety of structural types, including:

- Braced and unbraced steel frames
- Concrete frames with and without shear walls
- Bearing-wall structures
- Structures with long-span roofs

The first two methods, the Field Evaluation Method and the Approximate Analytical Evaluation Method do not estimate the extent of damage quantitatively and are of no interest here. The third, the Detailed Analytical Evaluation Method (DAEM), is based on a computer analysis and provides quantitative estimates of damage. In the DAEM, the ground motion at a site is expressed in terms of a site velocity response spectrum.

A response-spectrum approach with provisions for amplitude-dependent damping and stiffness characteristics is suggested in the DAEM for calculating the response of the structure to the prescribed ground motion. The response parameters used to predict damage are: maximum floor acceleration, floor velocity, and interstory drift. Three types of damage, namely, structural, nonstructural partition, and nonstructural window damage, are related to the response parameters. Structural damage and window damage are assumed to be functions of interstory drift whereas nonstructural partition damage is assumed to be related to the maximum floor velocity and acceleration (it is assumed that the nonstructural partitions do not span the entire floor-to-floor-height).

The relationship between the percentage of structural damage at a given story level and the maximum drift at that level is assumed to be a normally distributed curve, defined by a mean ductility to failure and an associated coefficient of variation. Ductility to failure is determined empirically, and professional judgment is exercised in selecting the proper coefficient of variation. The relationship of story drift to glass damage is treated similarly to structural damage. A defined drift-to-failure value, and an associated coefficient of variation are used, and a normal distribution is assumed.

Nonstructural damage at a floor level is estimated by treating the floor level in question as a site on the ground subjected to an effective floor MMI. The floor MMI is empirically related to maximum floor acceleration and velocity. The relationship between the floor MMI and the percentage of nonstructural damage to the floor is also given by an empirical formula, which includes a parameter (quality factor) reflecting the damageability of a specific construction type.

The DAEM attempts to relate engineering parameters to the damage suffered by components of a given structure. Damage is expressed in "percentage" only; unfortunately, the way percent damage is related to monetary loss is not well defined.

An extensive research program, directed by R. V. Whitman, J. M. Biggs, C. A. Cornell, and E. H. Vanmarcke, was undertaken at the Massachusetts Institute of Technology (MIT) to develop a method titled Optimum Seismic Protection and Building Damage Statistics. The title was later changed (Whitman, 1973) to Seismic Design Decision Analysis (SDDA). To select the level of seismic resistance to be required for an individual structure or a group of structures, the SDDA method considers the following:

- The cost of providing increased seismic resistance
- The damage that may occur during future earthquakes
- The social consequences of such damage

Many studies have been performed and reports published as part of the SDDA program. A description of the program, as originally conceived, is given in Report (Whitman et al., 1972). Theoretical structural response studies are detailed in Reports 3 and 4 (Anagnostopoulos, 1972; Biggs and Grace, 1973). Damage data and statistics obtained from the 1971 San Fernando, California earthquake are reported in Report 7 (Whitman Hong and Reed 1973). Report 8 (Whitman, 1973) gives damage probability matrices for multi-story buildings. Two reports attempt to correlate earthquake damage to tall buildings with strong ground motion parameters (Wong, 1975; Whitman, Aziz and Wong 1977).

Czarnecki (1973) developed a damage-prediction method, as part of MIT's SDDA program, that is based on engineering principles and is oriented toward high-rise buildings. In this method, the damage is related to structural response parameters. The building can be analyzed for a given earthquake using any acceptable dynamic analysis technique, such as response-spectrum analysis or linear or nonlinear time-history analysis. Total damage to a given building is the sum of damage to components. Components suggested for high-rise buildings are: structural components (steel frames, concrete frames, braced frames, and shear walls); for nonstructural components (drywall partitions, exterior glazing, brick masonry walls, and concrete block walls); and other components. Structural damage is fully attributed to the vertical structural elements, i.e., columns, shear walls, etc., and is assumed to be proportional to the inelastic energy absorbed by that element. Nonstructural damage is associated with maximum interstory drift. Drift-damage curves are developed from actual data and engineering design practices. No attempt is made to consider the variabilities of either the parameters used in the damage prediction or the final results.

Bertero and Bresler (1977) introduced the concepts of local and global damageability indices. The local damageability indices are defined, for both structural and nonstructural elements, as the ratio of building response demand to its corresponding capacity. The global damageability index is obtained by summing the local damageability indices for all structural and nonstructural elements. The local indices are weighted by an importance factor to reflect life hazard, cost, etc. Also, a cumulative damage index is defined as a measure of the cumulative damage to a structure as a result of all previous loadings or hazardous events.

Blejwas and Bresler (1979) developed a method for assessing the earthquake damageability of existing structures using the damageability indices defined by Bertero and Bresler (1977). The method considers both structural and nonstructural building elements.

Hasselman, Eguchi and Wiggins (1980) developed a computer code to be used in assessing the damageability of individual buildings exposed to earthquake, severe wind, and tornado forces. They adopted interstory drift as a basis for measuring building performance in earthquake and wind environments. The determination of damageability characteristics of building components in this method is based on the expert judgment of professionals, supplemented by limited data. Component damage is calculated floor by floor as a function of interstory drift.

Kustu, Miller and Brokken (1982a) developed damage functions for building components. Each damage function defines a relationship between a local response parameter (e.g., floor acceleration, floor velocity, or interstory drift) and the component damage factor (DF). Component DFs are defined as the component repair cost normalized by the initial component construction cost. Published laboratory test data for reinforced concrete, steel frame components, shear walls, masonry walls, drywall partitions, and glass were collected. The test data were statistically analyzed to determine damage threshold values (local response parameters at which a particular component damage state is reached) and their variabilities. The probability distributions of damage thresholds were, in turn, used to derive component damage functions that define the expected value of component DF as a function of a local building response parameter, e.g., tangential interstory drift angle.

Scholl et al. (1982) conducted a study to improve empirical and theoretical procedures for predicting losses to high-rise buildings damaged by earthquakes. A computerized damage data base for high-rise buildings was developed using data from past earthquakes occurring worldwide. The damage data base was then used to generate damage probability matrices (DPMs). DPMs define a probabilistic relationship between site ground motion and structural damage. The ground motion intensity was expressed in both the MMI and the Engineering Intensity (EI) scales.

The theoretical studies included examining the potential for developing component damage functions based on laboratory test data and fundamental concepts of structural dynamics. On the basis of this examination, a probabilistic method of damage prediction that estimates earthquake damage to various structural and nonstructural building components was adopted and recommended for further development. With the use of available building component damage data, various practical applications of this method were demonstrated.

The lack of complete data, the nonuniformity in reporting, and other factors led to the conclusion that the empirical approach has limited potential for future use in reliable damage predictions, but the theoretical approach has great potential for reliable application to earthquake damage-prediction problems.

Three methods for predicting damage to structures due to large underground nuclear explosions were developed by Blume (URS/Blume, 1975). These methods are equally applicable to predicting damage due to earthquakes. The three methods -- the Engineering Intensity Scale, the Spectral Matrix Method, and the Threshold Evaluation Method -- provide a means for making progressively more detailed predictions of structural effects due to seismic ground motion.

The Engineering Intensity Scale (Blume, 1970) is used to estimate the extent of the land area in which structures might be damaged and to make a general evaluation of the incidence and degree of damage to structures within that area.

In the formulation of the Engineering Intensity Scale, ground motion is characterized by 5%-damped spectral velocity (S_v), and structures are characterized by their fundamental-mode vibration properties expressed as natural period (T). Engineering intensity numbers are assigned to various spectral velocity bands. The range of S_v and T applicable to civil engineering structures is divided into a 10-by-9 matrix with ten intensity levels, from 0 through 9, and nine period bands, I through IX, in the period range from 0.01 sec to 10 sec.

A significant number of data on ground motion caused by underground nuclear explosions and corresponding damage data are available for establishing the incidence and degree of damage for various engineering intensity ranges for low-rise buildings (Hafen and Kintzer, 1977; URS/Blume, 1975). In addition, motion and damage data are available from the 1971 San Fernando earthquake for low-rise buildings (Hafen and Kintzer, 1977; Scholl, 1974) and high-rise buildings (Hafen and Kintzer, 1977; Wong, 1975). Motion-damage relationship information for high-rise buildings provided by Whitman, Aziz and Wong (1977) and the additional correlation work by Scholl et al. (1982) should provide sufficient information for this class of building.

The Spectral Matrix Method (SMM) was presented in 1967 (Blume, 1967). The method has subsequently been simplified and further developed (Blume, 1968; Blume and Monroe, 1971; URS/Blume, 1975).

The SMM is a generalized, statistical, computer-based procedure developed for the purpose of quantitatively predicting damage on a large scale. The procedure is applicable to predictions involving a large number of structures, including structures of several different classes and types. A fundamental philosophy of the procedure is that both structural resistance (capacity) and ground motion (demand) are random variables. Damage prediction, therefore, becomes a problem of joint probabilities.

Although the SMM is theoretically based, it is essential that the procedure be calibrated. Substantial low-rise building motion and damage data from underground nuclear explosions (URS/Blume, 1975) and some data from earthquakes (URS/Blume, 1975; Scholl, 1974) have been used for this purpose.

The Threshold Evaluation Method (TEM; Blume, 1969) for predicting the effects of dynamic ground motion on structures involves a systematic and detailed dynamic structural analysis of each individual structure. This method is used to identify both the potential risk from a structure's failure and modifications that might reduce the risk of structural failure. Basically, the TEM is an extension of conventional structural analysis procedures used in design. It requires the identification of the likelihood of exceeding these thresholds for a given seismic event. It is intended to provide detailed insight into the structural behavior of an individual building under lateral loading and to take advantage of several mitigating factors that are normally ignored in structural design practice in the interest of providing additional margins of safety.

Scawthorn, Iemura and Yamada (1981) developed damage models for typical low-rise and mid-rise buildings found in Sendai City, Japan. Both models are largely based on empirical relationships and may not be applicable to structures in other parts of the world. However, the procedures used to develop the damage models are general in nature and can be used to develop damage models for other structures.

The low-rise damage model has a theoretical component to its derivation. Scawthorn, Iemura and Yamada assumed that lateral resistance in the average low-rise Japanese building is derived from three elements: (1) solid walls of shinkabe (older, bamboo lattice-mud) or okabe (newer, similar to stucco on lath), (2) pierced walls of the same material, and (3) walls of the above materials with diagonal wood bracing. The damage data collected after the June 12, 1978, Miyagi-Ken-oki (Sendai) earthquake was used by the authors to develop a probabilistic relationship between estimated low-rise building displacement and damage.

In addition to the low-rise damage model, Scawthorn developed a damage model for mid-rise buildings. In this phase of the study, they developed an empirical relationship between a building's fundamental period and the number of stories. Using that relation, they performed a regression analysis on damage data from Sendai mid-rise buildings and estimated a relation between spectral acceleration, velocity, displacement, and building damage. The relationship between building damage and spectral displacement showed the largest correlation.

Kustu, Miller and Brokken (1982b) and Kuster, Miller and Scholl (1983) summarized two years of work directed toward the development of a methodology for predicting earthquake losses in urban areas. The main concern of the method is the prediction of direct monetary losses. Indirect losses caused by events initiated by earthquakes (such as fires, floods, and landslides) and nonmonetary losses (such as deaths and injuries) can be addressed to the extent permitted by available data. All types of structures are considered.

Kustu et al. developed damage models for a set of typical buildings found in urban areas from publicly available data collected for low-rise residential and high-rise buildings supplemented by engineering principles and expert judgment.

A modular program (SIMPLE -- SIMulation Program for Loss Estimation) automates the loss-prediction computations using a probabilistic sampling or simulation scheme. The program consists of three modules. The ground motion simulation module simulates the ground motion at a specific site due to a predefined seismic event, taking into account source characteristics, transmission path, and local soil conditions. A damage simulation module simulates the damage to individual structures or typical groups of structures for the computed site ground motion. This module requires as input data the damageability characteristics of the structures expressed in terms of DPMs. The component damage statistics group module computes the damage to any group of structures from the damage simulated in the previous module.

To predict losses due to a specific predefined earthquake, these three modules are used to simulate the urban area damage several times, thus accumulating loss samples that are later analyzed to compute the predicted loss. To estimate average annual losses, seismic events are simulated using historical seismicity data and fed into the three modules to generate a damage sample. Damage samples summed over one-year periods are taken as statistical annual loss samples, and many annual loss samples are accumulated and analyzed to make an annualized loss prediction.

It is clear that a wide variety of approaches have been used to make earthquake loss predictions of different types. Each approach outlined in this chapter has elements of three basic loss prediction types: empirical (i.e., collecting data from past events and drawing correlations between past experience and possible future experiences), theoretical (i.e., analyzing buildings for earthquake losses and developing relationships between significant building performance parameters and damage), and expert judgment (i.e., obtaining the consensus opinion of a group of experts). Each of these three possible approaches has advantages and disadvantages, as listed in Table 1; all three of them have been incorporated into the loss-prediction methodology adopted in this study.

TABLE 1

ADVANTAGES AND DISADVANTAGES OF EMPIRICAL, THEORETICAL, AND EXPERT JUDGMENT-BASED EARTHQUAKE LOSS PREDICTIONS

EMPIRICAL APPROACH

Advantages

1. Based on past experience; works quite well, and is simple.
2. Probabilistic risk assessment is possible.

Disadvantages

1. Lack of data to develop these relations for all types of buildings or components.
2. Based on past experience; will be difficult to extrapolate into future if construction practices change or into events with which we have no experience.

THEORETICAL APPROACH

Advantages

1. Lack of data is not too important.
2. We can extrapolate into areas where there are no existing data.

Disadvantages

1. Lack of knowledge of all phenomena involved.

EXPERT JUDGMENT APPROACH

Advantages

1. Readily available.
2. Extrapolations into unknown areas of seismic load or building type can be easily accomplished.

Disadvantages

1. Not completely trustworthy.
2. Can be difficult to get an unbiased consensus from a group of experts.
3. Not entirely reproducible.

5. THE COMPONENT APPROACH

The component approach to damage prediction is characterized by combining empirical, theoretical, and expert judgment elements. The basic premise of the component approach is that the total damage to an entity* can be estimated by: (1) predicting damage for each component of the entity and (2) combining the component damage estimates through a consequence analysis. Making a damage prediction for each component is not, in itself, a simple task. However, this approach is, conceptually simpler, and data for the procedure are more readily available or may be obtained through laboratory tests.

The steps in the component approach are as follows:

1. Inventory the components of the entity under study. This may require the development of a component classification scheme.

2. Perform a seismic response analysis of the entity and determine all relevant local entity responses. This step assumes that site ground motions have been estimated so that the response analysis can proceed.

3. Knowing the local response, and using damage to local response relationships based on empirical data, theoretical reasoning, or expert judgement, estimate the component damages.

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* The term entity is used because the component approach is very general in its range of applications. An entity may be any physical object such as a single building, a group of buildings, a single piece of critical equipment, or all critical and noncriticial equipment in a building.

4. Perform a systems (i.e., consequence) analysis to combine the component damages and make an entity damage prediction.

An advantage of this procedure is that, although every entity is different, there is a limited number of components used to assemble entities. Because the number of components is limited and the number of ways these components can be assembled to form an entity is unlimited, it is a simpler task to develop empirical, theoretical, or expert-judgment damage-response relationships for the components of an entity than it would be for the entity as a whole. Although this statement will have exceptions, it is valid for a large variety of entities.

A technical difficulty with this procedure is that entity and component damage and response are not decoupled phenomena. It may be necessary to iterate Steps 2 and 3, terminating the iteration when the local damage and response converge on the solution.

One of the primary benefits of this damage-prediction procedure is its capacity to make all damage predictions required. The approach can easily be extended and used to predict equipment and lifeline damage (i.e., electric power and gas systems). This flexibility makes it a powerful tool in the earthquake damage-prediction field. The following sections describe, in general terms, each step of the component approach.

5.1 Inventory and Classification

In the component approach to earthquake damage prediction, it is necessary to inventory the components of the entity under study. For buildings, the components are beams, columns, curtain wall panels, windows, elevators, cornices, etc. Because most entities are composed of a large number of components, it is impractical to gather detailed information on each. The analyst must therefore develop a component classification system. Rather than a detailed description, it is only necessary to gather sufficient data that each component can be classified. The components of each class are modeled by the average characteristics of the member components.

It is apparent that the data-gathering requirements of the inventory procedure are closely related to the component classification system. In addition, there is a relationship between the classification system and the analyst's ability to model each component's damageability characteristics. For instance, if the analyst can not quantify the damageability differences between a tilt-up wall and a reinforced block wall, the classification system need not discriminate between these wall types.

Currently, there is not enough known about component fragilities to justify a detailed accounting of all components in an entity; i.e., it is not necessary to account for every partition wall in a building. However, building damage predictions have been refined to the point where the performance of groups of components can be separated from the performance of the overall entity. Thus, for example, it makes sense to count all partitions on a single floor of a building as one component.

It is essential to develop refined damage models for the components that contribute substantially to the predicted damage. If most of the component damages are modeled accurately, the overall damage prediction will not not be adversely affected by neglecting some details. Therefore, it is important to determine which components contribute most significantly to the entity damage.

In some cases, owing to an entity's complexity or the analyst's lack of experience with the entity, it will not be practical to predict component damage. This will most frequently occur in complex pieces of machinery, such as diffusion furnaces. In these cases, the component should be considered an entity, and the analyst should develop a damage model for the entity as a whole, using empirical data, theoretical reasoning, or an expert's judgment.

5.2 Response Computations

The objective of the response computation is to determine the local response, or demand, each component is subjected to by an earthquake (later, the local responses are used to calculate the damage each component sustains). Detailed discussion of the computation is left to Chapter 6, where a response calculation procedure is presented for each of the four damage types of interest. Standard structural analysis methods are used, thus requiring the analyst to develop a structural analysis model of the entity under consideration. The accuracy of, and effort expended on, the structural analysis must be consistent with the detail of the damage prediction sought and knowledge of the damage phenomena involved.

5.3 Component Damages

Component damage is estimated through the use of a damage model, represented by damage curves. Damage curves show the damage performance of entities such as individual building components, entire buildings, individual pieces of equipment, or entire high-technology facilities and define the relationship between a demand and a damage attribute. Table 2 lists possible attributes of damage and demand for many different entities.

A commonly used damage attribute is the damage factor (DF). A DF is defined as the cost of repairing an entity divided by the original construction cost of the entity, adjusted for inflation. The DF is a convenient nondimensional parameter for representing the degree of damage to a component. By using the DF as the damage attribute, the damage curves can be isolated from economic effects, such as inflation. Defined in this way, the DF may exceed a value of 1.0, owing to the cost of removing a severely damaged component.

The damage curves are developed through either empirical data, theoretical reasoning, or expert judgment. In many cases, none of these sources of information will be sufficient to give the analyst confidence in his damage curve, and it is possible to combine the empirically, theoretically, and judgmentally derived damage curves into a single "best-guess" damage curve. An example of the empirical development of damage curves can be found in Kustu, Miller and Brokken (1982a). Similarly, an example of the development of expert judgment damage curves can be found in Kustu, Miller and Scholl (1983). In the latter report, expert judgment was used to develop damage probability matrices (DPMs), which are a digitized probabilistic representation of a damage curve.

5.4 Entity Damage Prediction (Consequence Analysis)

In the final step of the component approach to damage prediction, the analyst combines the individual component damage estimates to arrive at an entity damage prediction. The form of this analysis can vary widely depending on the components and the entity under consideration as well as on the type of damage prediction to be made. The entity damage prediction or consequence analysis can employ any of the following three different techniques (listed in order of increasing reliability):

- Qualitative analysis
- Projection of past experience
- Causal relationships

In a qualitative analysis, the analyst uses qualitative data and expert judgment. He may or may not consider previously collected empirical data. Projection of past experiences relies entirely on historical patterns, and, therefore, previously collected empirical data are a prerequisite for the use of this technique. Causal analysis uses highly refined and specific information about relationships between components. These techniques are well known in the field of product forecasting; many of the ideas for qualitative analysis discussed here were inspired by Chambers, Mullick and Smith (1974).

For two of the types of damage prediction possible with this approach -- building repair cost and equipment repair cost -- the analyst can make use of a causal relationship. For each of these two damage predictions, the total repair cost is simply the sum of the individual component repair costs carefully applied to the situation at hand,(i.e., accounting for intercomponent damage).

For the other two damage prediction types -- length of business interruption and total corporate financial losses -- there are no known causal models. The next most reliable method is projection of past experience. This technique is also of limited usefulness; in most cases there is no past experience with expected length of business interruption or expected corporate financial damages in high-technology facilities. This leaves us with a qualitative analysis to predict these types of damage.

There are many useful methods of qualitative analysis. Among the most useful for this work are the Delphi method and panel consensus. The objective of these methods is to bring together in a logical, unbiased, and systematic way all information and judgments that relate to the damage type under study.

The objective of the Delphi method is to gain the consensus of a group of experts on the expected damage. This is accomplished by questioning them individually and providing them with anonymous feedback information from other members of the group until the damage estimates converge. To do this, the component damage predictions made in the previous steps of the component approach are given to all experts, enabling each expert to have all the component damage information relevant to the prediction. Any scenarios, analogies, or information developed by one expert are passed on to the others, thus ensuring that each expert is formulating his opinions based on all available information. All questioning is handled impersonally by a coordinator. This technique eliminates committee activity almost entirely, thus reducing the influence of certain psychological factors, such as specious persuasion, unwillingness to abandon publicly expressed opinions, and the bandwagon effect of majority opinions.

In the panel consensus method, as in the Delphi method, the objective is to obtain a consensus or at least some agreement between experts. The technique itself is based on the assumption that several experts, by collectively considering all relevant factors, can arrive at a better forecast than one person. However it is not done in an unbiased way. There is no secrecy, and communication is encouraged among the panel members. Thus, the damage prediction may not reflect a true consensus, since it is affected by the group dynamics that the Delphi method is meant to circumvent. Obviously, personalities will enter into the consensus because the persons are brought together for one or a few meetings. This technique is primarily used for expediency.

TABLE 2

POSSIBLE DAMAGE CURVE ATTRIBUTES

Entity	Damage Attribute	Demand Attribute
Building Component (i.e., beam, column, shear wall)	- Repair Cost - Damage Factor - Damage of Energy Absorption Capacity	- Relative Displacement - Absolute Acceleration - Applied Forces
Building	- Life Damage - Number of Injuries - Repair Cost - Damage Factor - Degradation of Structural Performance - Damage of Usefulness	- Engineering Intensity - PGA - Duration - Frequency Content - Spectral Velocity - MMI
Critical Production Equipment	- Damage of Functionality - Repair Cost - Recovery Time	- Peak Floor Acceleration - Duration - Frequency Content - Spectral Velocity
High-Technology Facility	- Building Repair Costs - Equipment Repair Costs - Expected Business Interruption - Total Corporate Financial Dollar Damage	- Specific Seismic Event - Seismic Environment (Annualized Damage)

6. APPLICATION OF THE COMPONENT APPROACH

This chapter is divided into five parts. Section 6.1 is devoted to a detailed discussion of the recommended procedure for making ground motion predictions for predefined seismic events. Sections 6.2 through 6.5 present the procedures recommended for estimating seismic damage to high-technology facilities. In addition, the necessary data will be presented so that each procedure may be implemented. The four procedures will predict the following types of seismic damage:

- Repair costs for buildings
- Repair costs for equipment
- Length of business interruption
- Total corporate financial losses

The procedure is illustrated by applying it to an example problem. The ground motion used for the problem is that estimated for the 1906 San Francisco earthquake.

The example facility is a "good" quality electronics manufacturing plant located in the Santa Clara Valley of northern California. Although most high-technology facilities in this area are composed of multiple buildings at one site, in order to simplify the example, yet still cover the relevant features of a high-technology facility damage prediction, a single building is used. The example building is used for semi-conductor research and development, and manufacturing, and offices for related personnel. It is of the most commonly occurring building construction type in Santa Clara Valley (see Appendix A). It is one story, and has plan dimensions of 150 x 250 ft. as seen in Figure 2. It is constructed of concrete tilt-up walls with a slab on grade and a plywood roof diaphragm supported by steel columns and glu-lam beams. The building height is 20 ft. and the construction quality is "good".

6.1 Ground Motion Prediction

Ground motion prediction is a necessary first step in evaluating any of the four types of seismic damage listed above (see Figure 1 in Chapter 2). Deterministic, site-dependent response spectra represent an appropriate and cost-effective way to specify earthquake ground motion. The general concepts behind the use of such spectra to predict ground motion were presented in Section 3.3. Specifically, the recommended methodology comprises the following steps:

1. Estimate the site peak ground accleration (PGA).

2. Select site-dependent ground motion ratios v/a and ad/v^2 and spectral amplification factors.

3. Construct the corresponding response spectrum on the basis of predicted ground motion and the applicable damping ratios.

These steps are discussed in the following subsections.

6.1.1 Estimation of Site PGA. It is assumed that the analyst has a particular seismic event in mind for which a seismic damage prediction is desired. To fully define the earthquake, the analyst should have the earthquake magnitude, the site's hypocentral distance, and knowledge of the soil conditions at the site.

Use of the relationship between earthquake magnitude, acceleration, and distance developed by Donovan and Bornstein (1977) is recommended. This relationship, which applies to rock and to firm soils, is given by:

$$y = b_1 e^{b_2 M} (R + 25)^{-b_3} \tag{1}$$

where:

y = bedrock horizontal acceleration (cm/sec^2)
M = magnitude
R = hypocentral distance, in kilometers
b_1 = $2{,}154{,}000\ R^{-2.1}$
b_2 = $0.046 + 0.445 \log R$
b_3 = $2.515 - 0.486 \log R$

It should be noted that Equation (1) represents a mean level of acceleration, which can be exceeded by high-frequency acceleration pulses of little engineering significance.

Soil deposits modify earthquake motion through filtering and amplification effects. Accelerations at the surface of such sites generally differ from those recorded on rock at the same distance from the causative fault rupture. Graphical correlations for soil and bedrock surface acceleration, presented in Seed and Idriss (1983) and reproduced in Figure 3, are convenient for estimating surface accelerations for soil sites from those computed for rock sites.

6.1.2 Ground Motion Ratios and Spectral Amplification Factors. The procedure described by Hall, Mohraz and Newmark (1976) provides an approximate method for estimating ground motion ratios and spectral amplification factors. It relies upon statistical studies of earthquake motion and on the fact that spectral amplification primarily depends on PGA, peak ground velocity, or peak ground displacement, depending on the frequency range considered. In this procedure, peak ground velocity and displacement are estimated for the design motions using average values determined for the ratios v/a and ad/v^2, where a, v, and d, respectively, represent peak ground acceleration, velocity, and displacement. Spectral amplification factors are then selected for the portions of the spectrum controlled by acceleration, velocity, and displacement.

The study by Mohraz (1976) represents the most complete analysis of ground motion ratios and spectral amplification factors. Mohraz differentiated between rock sites, sites with less than 30 ft of alluvium underlain by rock, sites with 30 to 200 ft of alluvium underlain by rock, and "alluvium" sites, which presumably include a significant number of very deep soil sites. Mohraz further treated the two horizontal and the vertical components of earthquake motion separately. Mohraz's ground motion ratios include 50th- and 84th-percentile values. 50th- and 84th-percentile spectral amplification factors are presented for five different damping ratios.

The present procedure considers only horizontal motion and does not differentiate between the larger and smaller components. Therefore, the average of the ground motion ratios tabulated by Mohraz for the larger and smaller component is recommended to obtain peak velocity and displacement. To maintain a consistent level of probability between acceleration, velocity, and displacement, the 50th-percentile ratios should be used. The resulting v/a and ad/v^2 ratios are shown in Table 3.

Similarly, amplification factors can be obtained using the average of the 50th-percentile factors tabulated by Mohraz for the two horizontal components and the four soil conditions considered. Amplification factors obtained in this manner are shown in Table 4 for 5% of critical damping.

<u>6.1.3 Response Spectra</u>. Once the PGA and the factors from Step 2 have been computed, the analyst can construct a response spectrum composed of straight line segments on a tripartite logarithmic plot. The following assumptions are necessary to implement this procedure:

- No amplification occurs for periods less than 0.03 sec.

- The acceleration-controlled amplifications begin at periods greater than 0.13 sec.

- A linear segment can be traced between 0.03 and 0.13 sec to define the spectrum between the no-amplification and acceleration-controlled segments.

Vertical response spectra, if they are necessary, can be obtained by uniformly scaling down the horizontal response spectra by a factor of two-thirds.

6.1.4 Example. The following example illustrates the use of the methodology. Deterministic, site-dependent response spectra are to be established for the fictitious site in the Santa Clara Valley. The spectra are to be developed for a recurrence of the 1906 San Francisco earthquake.

(Step 1) The bedrock horizontal acceleration computed is:

$$y = b_1 e^{b_2 M} (R + 25)^{-b_3} = 616 \text{ cm/sec} = 0.63g$$

where:

y = bedrock horizontal acceleration (cm/sec^2)

M = magnitude 8.25

R = hypocentral distance, in kilometers = 15

b_1 = $2{,}154{,}000\ R^{-2.1} = 7302$

b_2 = $0.046 + 0.445 \log R = 0.569$

b_3 = $2.515 - 0.486 \log R = 1.94$

Surface accelerations are obtained by extrapolating the curve Figure 3, (deep cohesionless soil) which shows a surface PGA of 0.40g.

(Step 2) The site considered is assumed to be underlain by 100 ft of alluvium. The corresponding v/a and ad/v^2 are 33 and 4.45 (see Table 3).

The damping ratio of interest is 5%. Table 4 provides the applicable amplification factors: 1.78 for displacement, 1.48 for velocity, and 2.32 for acceleration.

(Step 3) The values obtained from Step 2 are used to construct the response spectrum.

The response spectrum parameters are detailed below for a surface PGA of 0.40g.

Maximum ground velocity = 33 (in/sec)/g x 0.40g   12 in/ft = 1.10 ft/sec

Maximum ground displacement = 4.45 x 1.10 (ft/sec)^2/(0.40 x 32.2 ft/sec^2) = 0.42 ft

Maximum spectral acceleration = 2.32 x 0.40g = 0.93g

Maximum spectral velocity = 1.48 x 1.10 ft/sec = 1.63 ft/sec

Maximum spectral displacement = 1.78 x 0.42 ft = 0.75 ft

The corresponding response spectrum is shown in Figure 4.

6.2 Repair Costs for Buildings

The component approach can be directly applied to buildings. To predict repair costs, the analyst estimates repair costs for each component and finalizes the damage prediction by summing all the component repair costs.

6.2.1 Inventory and Classification. Before building repair costs can be predicted, it is necessary for the analyst to inventory and classify all building components. Buildings are composed of both structural and nonstructural components. The structural components include shear walls, columns, and beams; the nonstructural components include windows, curtain walls, suspended ceilings, ductwork, light fixtures, and plumbing. The contents of the buildings are not included; items such as desks, file cabinets, photocopy machines, and production line equipment are treated in the damage prediction for equipment.

The component inventory is best handled by first establishing a building component classification scheme. The classification scheme eliminates the need for the analyst to develop a damage curve for each building component. Rather, a single damage curve is developed for each component class and uniformly applied to all member components. The component inventory is then reduced to tallying, for each class, the number of member components, their location, and their dollar value. The number of member components and dollar values are used in the consequence analysis. The component location information is needed for the response computations.

Of course, the detail for each component classification scheme is entirely dependent upon the analyst's ability to develop component damage curves and conduct component inventories. For example, if, during the building inventory, the analyst cannot determine whether partition walls are constructed of plaster on metal lath or drywall, there is no need to develop two component classifications for the partition walls. Also, if the analyst is unable to develop significantly different damage curves for two window types, there is no need to define two window classes.

Tables 5 and 6 are based on a suggested component classification scheme. Whether this scheme is adopted by the analyst directly or modified depends on the exact nature of the building for which a damage prediction is desired. The scheme presented is thought to be general enough for direct adoption into many loss predictions.

For many building components, but by no means all, researchers have developed damage curves. The desired form of a damage curve is a plot of a local component response parameter (in this report, either relative displacement or absolute acceleration) versus the component's damage factor. Tables 5 and 6 show building component damage versus component demand estimates. Figure 5 shows how the tabulated thresholds and damage factors are interpreted as damage curves. Where there were no available data, a damage curve has been assigned to the component based on calculation, engineering judgment, or both.

Of course, it is most important to have accurate damage curves for those building components that account for the majority of the total building repair costs. For example, it is more important to accurately model damage to partition walls than damage to ceilings because the partition walls, in general, are more severely damaged in seismic events and therefore contribute more to the total building repair cost. The components that add little to the total repair costs may be approximately modeled without significantly affecting the building damage prediction.

Most high-technology facilities do not have a large number of buildings, and it is therefore feasible for the analyst to inventory the building components by conducting either field surveys or a careful review of the structural, mechanical, electrical, plumbing, and architectural design drawings. A detailed account of the number, value, and position of each building component is not necessary. Even if it were available, the accuracy of the damage prediction does not warrant such detailed information. What is required is an approximate value to be associated with each component type on a floor-by-floor basis.

In some cases, it may be possible to draw analogies between two facilities and use the inventory of one building as a good approximation of the inventory of another facility. This type of analogy will have to be handled case by case. For building types that are typical in design, it is possible to develop inventories that can be applied to a building given its type, use, and age.

The inventory for the example building is presented in Table 7. (Note: All dollar values used are 1984 dollars.) Appendix A presents the statistics that identify the configuration of a typical building in the high-technology facilities of northern California.

6.2.2 Response Computation. The objective of this phase of the building damage prediction is to estimate local (i.e., component) responses given the ground motion input and a structural analysis model of the building. The structural analysis must take into account material and geometric nonlinearities. Total and partial collapse of a building must also be identified and predicted. The response computation is an essential step in making realistic seismic damage predictions.

Blejwas and Bresler (1978) discuss methods of response analysis and their application to building damage prediction. They suggest that the analyst perform a series of analyses (screenings), each successive analysis more refined than the preceding, until he is satisfied that the response prediction is realistic and the response calculation is of the required detail. With this screening process, the analyst can determine the precise analytic requirements of the response computations before embarking on time-consuming and costly methods of analysis.

Beginning with a simple linear elastic response-spectrum analysis, the analyst can determine whether the structure will exhibit nonlinear behavior. If it does not, no further analysis need be performed.

If some of the components do exceed the range of linear elastic response, another stage of analysis may be called for. Clough and Penzien (1975) outline a simplified nonlinear analysis (the Ductility Factor Method) that can be used in building design work. The basic assumption of this method is that the deflections produced by a given earthquake are essentially the same, whether the structure responds elastically or yields significantly. This assumption, if true, can also be applied to damage predictions: the analyst can make use of the local responses predicted by a linear elastic response-spectrum analysis to determine component damage. Caution is needed because the method assumes that the inelastic deformations will be distributed uniformly over the entire structure; hence, any mechanism that leads to local strain concentrations will not be accounted for. It provides a basis for determining local responses in regular structures, but it clearly cannot cope with systems having pronounced strength discontinuities, such as buildings deliberately designed to yield only at the first-story level.

For the next stage of the structural response screening process, Blejwas and Bresler (1978) suggest a "quasi-static" analysis that accounts for material nonlinearties. If this quasi-static analysis does not provide the analyst with a response prediction of the required accuracy, a fully nonlinear time-history analysis is suggested. The details of the recommended analysis procedures will not be presented here; rather, the reader is directed to the appropriate references. The time history analysis is typically necessary only for extremely valuable structures; it is not usually use for tilt-ups. A linear elastic response-spectrum analysis is used in this section.

Total and partial structural collapse predictions are important because of the "extra" damage inflicted on a structure by the collapse. In a situation of partial structural collapse, the failing component is not the only component involved, and thus it is not the only component incurring a repair cost. For example, if a floor beam fails, the total repair cost is not simply the cost of replacing the beam but also the cost of replacing the supported floor, partition walls, and other structural or nonstructural components supported by the floor. This effect is accounted for in the component damage prediction phase of the damage prediction task. The components involved in a collapse, however, must be identified in the present phase of the damage prediction.

Total and partial structural collapse is strongly dependent on the duration of the strong shaking among many other parameters. Unfortunately it is not presently possible to conveniently account for duration in a structural analysis. The only practical means presently at our disposal to predict building collapse and partial collapse is as follows. When evaluating damage to building components, identify which members fail. Stresses should subsequently be redistributed throughout the structure. The failing members will indicate either local or global collapse. The structural analysis and component damage predictions are thus coupled and form an iterative analysis/loss-prediction algorithm. A thorough reliability analysis would be better than this approach, but, in most cases, the analyst will not have the time or budget available to pursue it.

The structural model used to perform damage predictions may differ from the structural model a building designer might use. In the design process, the standard practice is for the designer to make many conservative, simplifying assumptions in the structural model. For example, it is common practice for a designer to neglect nonstructural walls in the stiffness model. These assumptions are inappropriate for damage prediction.

In the example problem, rather than modeling the structure as a single-degree-of-freedom system, a more representative model for a low-rise building can be developed by treating the roof diaphragm as a degree of freedom. Figure 14 represents this arrangement, which is analogous to a flexible beam supported on springs. This system becomes uncoupled when the ratio of the natural frequencies exceeds three (Englekirk, 1979). The two uncoupled models are then the shear wall and a flexible beam (the diaphragm) on rigid supports (the shear wall).

Figures 15 and 16 present the calculations used to estimate the plywood diaphragm's period in the two principal directions of response for the example building. Figures 17 and 18 present the calculations used to estimate the periods of the shear walls. Since the ratio of the natural frequencies of the shear walls and the diaphragms exceeds three, the assumption of uncoupled response holds. The diaphragm and shear wall responses are then estimated through a response-spectrum analysis. The site response spectrum for the example building is shown in Figure 4. The results of the analysis are presented in Figure 19. It is important to keep in mind that the example structure is of "good" quality construction, i.e., no partial or total structural collapse has been predicted even though a lesser quality structure may collapse under the severe ground motions predicted.

6.2.3 Component Damage Estimates

With peak local responses estimated, and a building component inventory with associated construction costs and damage curves, the analyst can estimate the damage sustained by each structural and nonstructural component by simply looking it up in Tables 5 and 6, which give the damageability of many building components. However, some engineering judgment is required when a partial or total structural collapse is predicted. The analyst must determine which components of the building, other than the component initiating the collapse, will be affected and then assign to them the appropriate repair cost, or damage factor.

Tables 5 and 6 show that there are up to three distinct states of damage - i.e., damage states (DS) - for each building component. For each component, the table suggests a local demand parameter that is strongly correlated with component damage. Typically, for structural components, the local demand is a relative displacement parameter. For nonstructural components, the local demand parameter is typically the peak base acceleration. For each damage state, the tables indicate a threshold value (THD) and damage factor (DF). The THD is the value of displacement or accleration must be exceeded before the component enters that DS. Once the component has reached a particular DS, the component will have incurred a repair cost of the DF times the replacement value of that particular component.

The THD and DF values in the tables are estimates for an average component of the indicated class. The actual values that should be used will vary to account for different qualities of construction and materials. The analyst should judge whether the components in the structure under study are of average quality, inferior quality, or superior quality. The variation in the THD and DF values presented in the tables due to component quality may be as large as ± 50%.

Table 8 presents the estimated building component repair costs for the example structure. Note that partial or total collapse of the structure is not predicted.

6.2.4 Building Repair Cost Prediction. The final step, the consequence analysis, involves the summation of repair costs over all building components. The analyst must be cautious with such an analysis since there may be economies of scale in repairing many components at one location over repairing just a few components. If so, the total repair cost must be reduced by an appropriate factor.

As mentioned earlier, if the analyst is dealing with a large number of buildings in a facility analysis, it is possible to develop typical building classifications and typical building component inventories. This concept can be carried one step further, and the analyst can develop typical building damage curves. Each damage curve would then correspond to a particular building classification. For typical Santa Clara Valley electronics manufacturers, the prevalent construction type is the one-story tilt-up building. For this geographical area, it is possible to develop one damage curve to represent the damageability of most buildings. This concept will be used extensively in the proposed Phase II research.

In the example problem, no structural collapse is anticipated, and no economies of scale in repair are assumed; the consequence analysis is a simple summation. Table 9 summarizes the repair costs for the example structure. The total DF for the structure is 11%. Note that a lesser quality structure may have suffered a total or partial collapse. In this case the total DF would be much higher. A good quality structure was chosen for the example so that the component approach could be demonstrated without the added complication of component damage interaction.

6.3 Repair Costs for Equipment

The objective of this procedure to predict the dollar repair cost of all equipment in a high-technology facility. This is an important part of the overall damage prediction because, in many cases, the equipment housed by buildings in the facility will be several times more expensive to repair or replace than the buildings themselves.

The component approach is applied to equipment damage in the same way it is applied to building damage: the damage prediction begins with an inventory and classification task, followed by response computation, equipment damage prediction, and summation of equipment damage. The components involved are the individual items of equipment.

6.3.1 Inventory and Classification. A suggested equipment classification scheme is presented in Table 10. The analyst is, of course, free to modify this system. The detail and accuracy of the scheme finally used must be in harmony with the available inventory data and the analyst's ability to develop damage curves for each class. The suggested classification scheme should work well for most of the equipment found in high-technology facilities; however, there will be exceptions. When an item of equipment does not lend itself to being classified in the proposed scheme, the analyst must add a special class.

Table 10 estimates the damageability of each of the proposed equipment classes. These estimates are based on experience, calculation, or engineering judgment. All of the equipment is sensitive to support accelerations and not relative displacements. It is up to the analyst to estimate the damageability of each special equipment class. As discussed in Chapter 4, these curves may be developed from empirical data, theoretical reasoning, or an expert's judgment.

Since high-technology facilities have large inventories of equipment, it is not feasible to perform a detailed inventory of all equipment. Reasonable estimates of the number, value, and position of each class of equipment are the only required data for the damage-prediction procedure outlined here. A few expensive items and support systems control the cost.

Again, as for the building component inventory, it may be possible to draw analogies between two facilities and use the inventory of equipment found in one building as a good approximation of the equipment inventory of another building. These analogies must be handled case by case.

An equipment inventory, at least for the large expensive equipment, should be available for the facility. If these data are not available in a readily interpreted or centralized area, the analyst will have to obtain the inventory by some other means. One suggested approach is to perform a detailed equipment inventory of randomly selected areas of the facility. Each area of the facility should be representative of some function (i.e., semiconductor production, administration, warehouse, maintenance, etc.). Then, by assuming that other sections of the facility that perform the same function will have similar inventories, the analyst can project the area inventories onto the rest of the facility to estimate the equipment inventory for the entire facility.

The example facility's critical equipment inventory is presented in Table 11. For the example problem, the equipment inventory has been kept to a manageable size.

6.3.2 Response Computation. The objective of this step of the prediction of equipment repair costs is to evaluate the motion that will damage the individual pieces of equipment. Most equipment of importance is sensitive to support accelerations; however, some of the equipment and its supporting systems will be damaged by relative displacements. It is the task of this step to determine either the support acceleration or the relative displacement. This task is most effectively dealt with by a standard structural analysis of the building or structure that houses the equipment.

If both building repair cost and the equipment repair cost predictions are to be made, it would be most economical to make the response computations required here at the same time that the local building responses are estimated. See Section 6.2.2 for more details about this procedure. It is important to assess the potential for partial or total collapse of the building housing the equipment because damage in addition to that caused by vibratory ground motion can be suffered. Collapse of a building could mean total loss of equipment.

The ground and roof accelerations and relative displacements for the example problem were presented in Section 6.2.2. Local demands are listed in Table 8 for each item of equipment. Note that all the equipment rests directly on a slab on grade.

6.3.3 Component Damage Estimates. With equipment responses estimated, and an equipment inventory available with associated replacement values and damage curves, the analyst can estimate the damage sustained by each item of equipment by referring to Table 10, which lists the estimated damageabilities for each item.

Some engineering judgment is required when a partial or total structural collapse is predicted. The analyst must determine which equipment will be damaged by the collapse and then assign the items the appropriate repair cost, or DF.

The damage to critical equipment for the example facility is shown in Table 11.

6.3.4 Equipment Repair Cost Prediction. The final step in the procedure is to make an overall estimate of equipment repair costs given the individual equipment repair costs by summation of the repair costs over all items of equipment. The analyst must be cautious with such an analysis since there may be economies of scale in repairing many items of equipment at one location over repairing just a few items of equipment. If so, the total repair cost must be reduced by an appropriate factor.

In the example problem, the consequence analysis is a simple summation of the critical equipment damage. Table 11 summarizes the results and shows a total DF of 48% for the equipment listed.

6.4 Length of Business Interruption

In predicting the length of business interruption, the consequence analysis step of the component approach takes on a dramatically different appearance. There are no causal models available (at a reasonable cost of implementation), and it also seems impractical to develop causal models because of the difficulty in forecasting the course of actions taken to repair a facility after a seismic event. For instance, in a causal model, it would be difficult to account for the cleverness of production line managers in making use of available resources to recover quickly from the earthquake damage. Given identical situations, different production line managers might pursue different courses of recovery. Also, the occurrence of post-event fires and other hazards are difficult to predict. Yet these other hazards can seriously affect the ability of a high-technology facility to recover from an earthquake. As a result, a quantitative estimate of the length of business interuption is a difficult task at best, and qualitative techniques are preferable.

Five component categories specific to high-technology facilities, and their relevant damage types, are identified in Table 12. Buildings and equipment have already been discussed. In addition to dollar damage, the analyst must be able to determine the time required to restore the equipment and buildings to their pre-event status. The remaining categories (i.e., number of injured employees, damage to production materials, and external influences) are discussed here.

Rather than dividing this section into the steps required by the component approach, as was done in the previous two discussions, the following sections discuss each component category and then specify how the consequence analysis should be carried out.

6.4.1 Recovery Time of Buildings and Equipment. This task requires the analyst to identify the building and equipment damage that will affect the operation of a facility. Damage such as cracks in partition walls will not stop production lines or research activities. However, if the building suffers a partial collapse or if equipment is completely destroyed, this damage will halt a production line or interrupt research activities until the repairs are completed.

Once the significant damage has been identified, the facility managers should be consulted to estimate the time it would take for maintenance crews to perform the necessary repairs. These estimates are best handled by a qualitative analysis method, such as the Delphi or panel consensus methods. If the expected damage to the facility is severe enough, and outside help, spare parts, or equipment are required to make the repairs, the analyst should consider alternative sources of the materials and services in making a repair time estimate. Of course, the analyst must bear in mind that there will be a large demand for repair equipment and services following a seismic event and that this demand will influence the time at which repairs may begin. The suppliers may also have suffered a business interruption. Therefore, the analyst needs to query the supplier as to their own estimated down time. Given the interelatedness of high tech firms, the suppliers may also have difficulties getting back into production because of damage to their own supplies as well.

6.4.2 Deaths and Injuries Among Employees

The number of dead and injured employees will have a direct bearing on a facility's recovery time; a facility cannot begin to recover, or operate production lines, without people. The loss of people is of most concern when a building has collapsed, in which case a large number of employees could be affected.

The number of employee deaths and injuries is difficult to evaluate. It is a simpler task to determine the number of employees who have been severely injured without making a distinction between those who survive an injury and those who do not. For our purposes, the number of severely injured employees can be related to building collapse or partial collapse (see Section 6.2.2) although there are other hazards to be found in high-technology facilities, including spillage of toxic or explosive materials and toppling equipment. This relation is by no means accurate when one looks at the fine details and distribution of injured employees, but, on the whole, the predicted number of injuries will closely match those actually incurred, i.e., the number of people who miraculously escape injury in a collapsed building will compensate for the number of people who are injured by other hazards.

In addition to knowledge of building collapse, the analyst must know how many people are in a facility at any one time and how these people are distributed throughout the facility to make a severe injury prediction. The employee population of a facility will, in turn, depend on the time of day a seismic event occurs and on the day of the week. This information will be available from the facility managers.

The rule then is that no severe injuries will occur unless there is a partial or total building collapse, and all employees in the collapsed part of the building will suffer a severe injury.

6.4.3 Damage to Production Materials. Damage to production materials, whether raw, in process, or completed products, can be estimated by the approach outlined for equipment. The analyst should not model each material separately but rather should group the materials in some fashion and compute damage for the group. For instance, instead of modeling each completed product separately, the analyst will model all the completed products in a particular storage unit. The damage of concern here is the quantity of material destroyed.

Production materials are important to recovery time because a production line cannot operate unless it has raw materials to transform into completed products. The damage to in-process materials will determine how quickly the facility can get back to a normal level of operation, and the destruction to completed products will determine how quickly the facility can start shipping orders again. An extreme example of the importance of damage to in-process material can be found in the aerospace industry, in which only one or a few products are in process simultaneously. If this in-process product is destroyed, production will suffer a major setback.

6.4.4 External Influences. External entities such as utilities (electricity, gas, water), suppliers of raw materials, and consumers, will influence the recovery of a high-technology facility. These damages are by far the most difficult to estimate, and the task might best be accomplished through discussions with the managers of the affected utilities or facilities.

6.4.5 Estimation of Facility Recovery Time. Section 5.4 outlined two tools that can be used in a qualitative analysis: the Delphi Method and the panel consensus method. Each of these methods benefits tremendously from the input of damage predictions made for objects that affect a high-technology facility's recovery time.

With all the data on damage estimated, the analyst must use one of the qualitative methods of analysis to determine the estimated recovery time for the high-technology facility. The preferred method is the Delphi method; however, the panel consensus method produces results more quickly and with reasonable reliability. The analyst must identify a group of people inside the high-technology facility who would be capable of making recovery time estimates given the damage estimates already made. This group would be composed of production line engineers, managers at all levels of operation, and others.

6.5 Total Corporate Financial Losses

The total monetary damage a corporation suffers is not simply the summation of all the repair costs required to restore the facilities to their present condition after a seismic event. There are many other types of damage that only officers and managers of the corporation can assess. For instance, if the corporation were about to introduce a new product into a fast-moving marketplace (such as microcomputers), and loses six months before the introduction because of an earthquake, the potential cost (or loss of market share) of this delay would far outweigh the facility repair costs. The factors that must be considered when making this type of damage prediction include:

- Facility repair costs
- Loss of market share
- Loss of research momentum

The only conceivable approach to this problem is a qualitative approach. In both the Delphi method and panel consensus analysis the analyst begins by identifying employees of the corporation who will have good insight into the potential costs of these other types of damage. It is the responsibility of the analyst to present these employees with a scenario of the facility's postearthquake condition in a form suitable for further consideration. This might include estimates of the total facility repair cost, facility recovery time (possibly divided into production line recovery, research recovery, warehouse recovery, etc.), and a general description of damage to the facilities of competitors.

With the scenario, the analyst must organize and administer either the panel consensus or the Delphi procedure in order for the key employees to arrive at their estimates of total financial cost of the seismic event to the corporation. This is a subjective process, and, as such, can only make an order-of-magnitude estimate of the total financial cost of the seismic event.

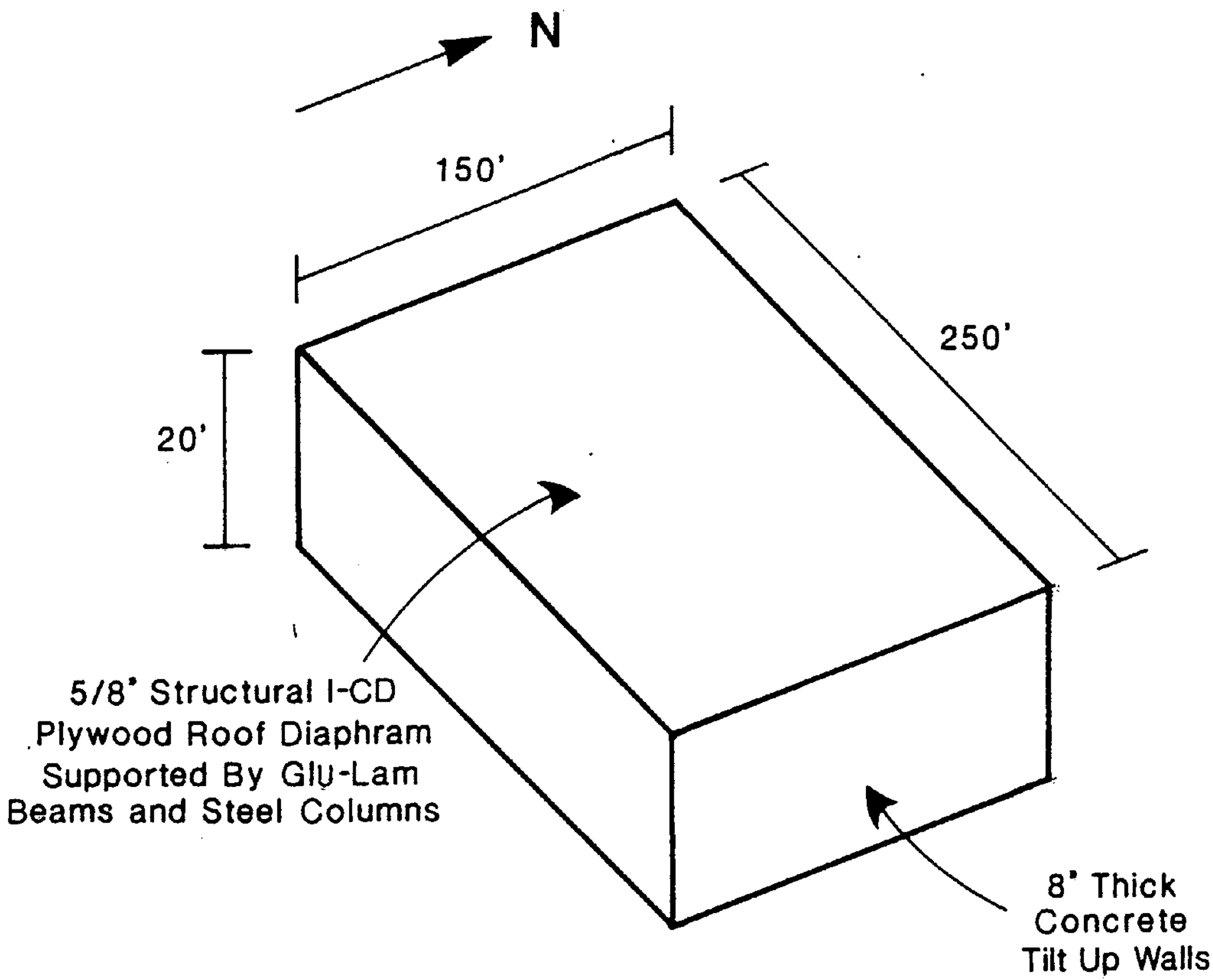


Figure 2: Example Building

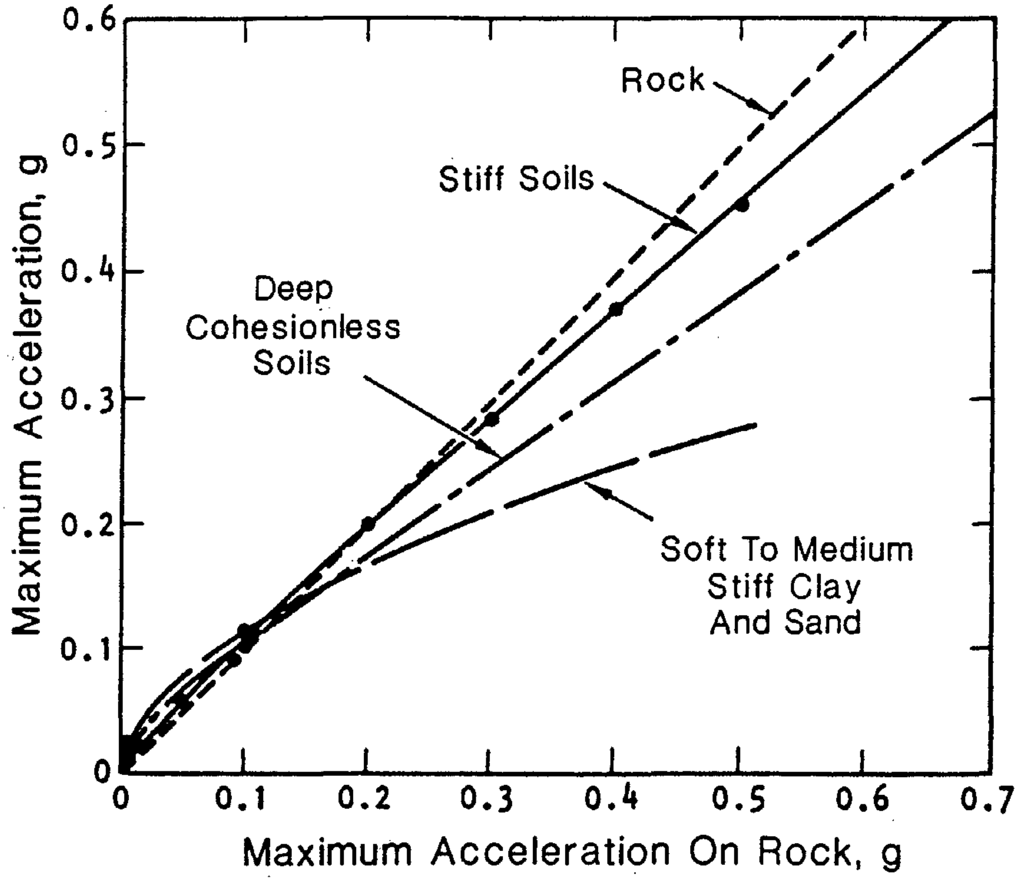


Figure 3: Relationships between soil and bedrock surface accelerations.

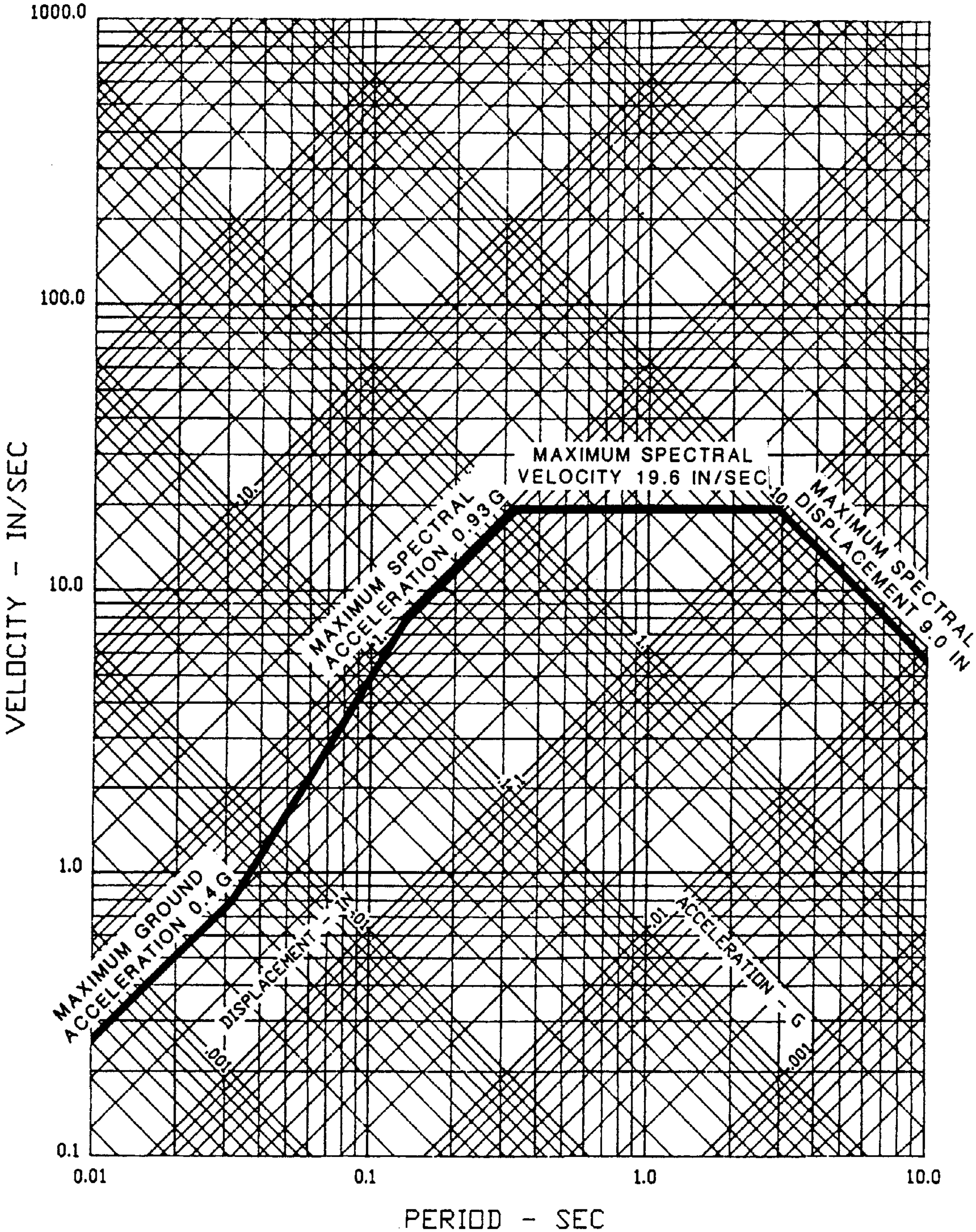


Figure 4: Example site response spectra ... 5% damping

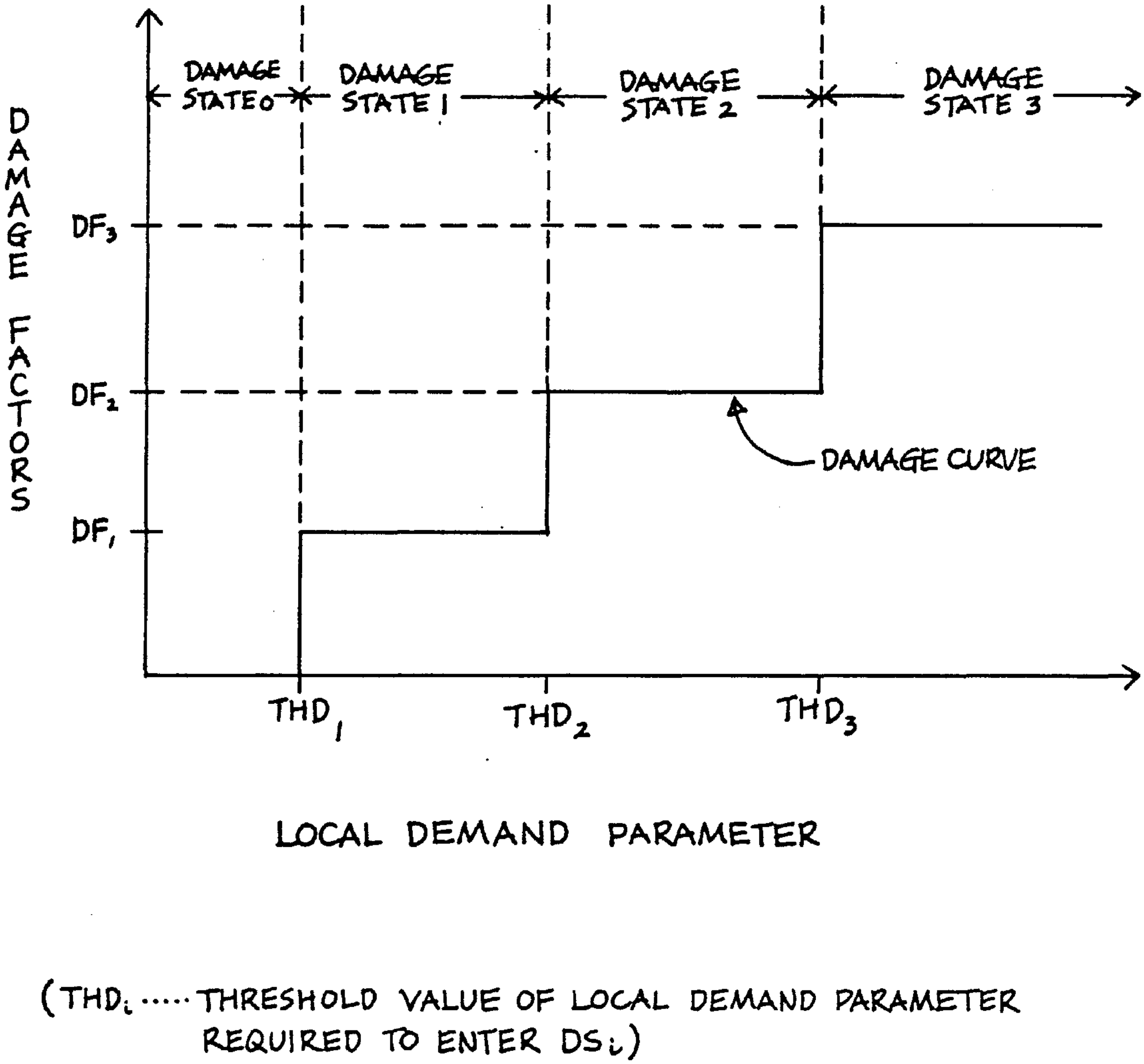
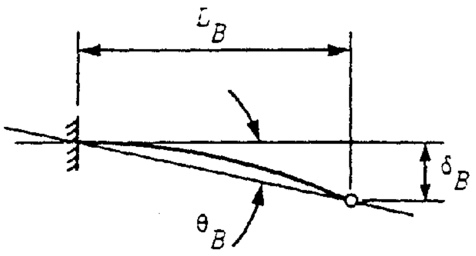


Figure 5: Interpretation of tabulated thresholds and damage factors as damage curves.



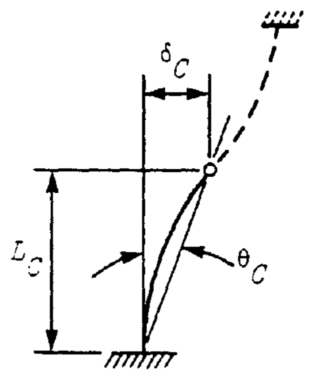
$$\theta_B = \frac{\delta_B}{L_B}$$

where:

δ_B = Cantilever beam deflection

L_B = Cantilever length of beam

a. Beams, Cantilever Deflection Angle (θ_B)



$$\theta_C = \frac{\delta_C}{L_C}$$

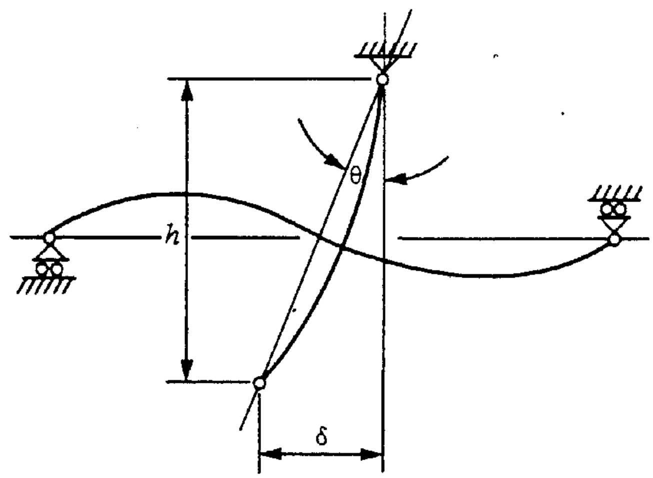
where:

δ_C = Cantilever column deflection

L_C = Cantilever column height

b. Columns, Cantilever Deflection Angle (θ_C)

Figure 6: Local response parameters for reinforced concrete beams and columns (adapted from Kustu, et al., 1982).



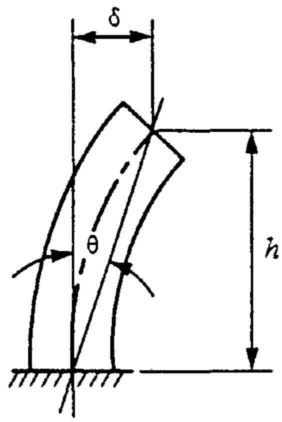
$$\theta = \frac{\delta}{h}$$

where:

δ = Tangential interstory drift

h = Story height

Figure 7: Local response parameter for shell frames: tangential interstory drift angle (Θ) (adapted from Kustu, et. al., 1982).



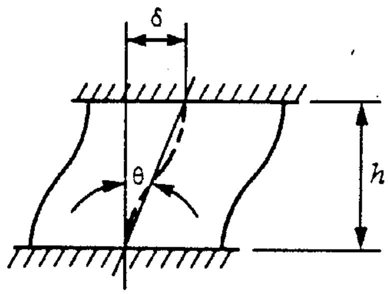
$$\theta = \frac{\delta}{h}$$

where:

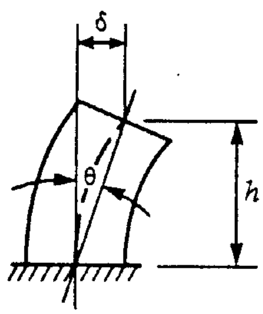
δ = Tangential interstory drift

h = Story height

Figure 8: Local response parameter for reinforced concrete shear walls: tangential interstory drift angle (θ) (adapted from Kustu, et al., 1982).



a. Infill Walls or Racking Tested Walls



b. Pier Walls

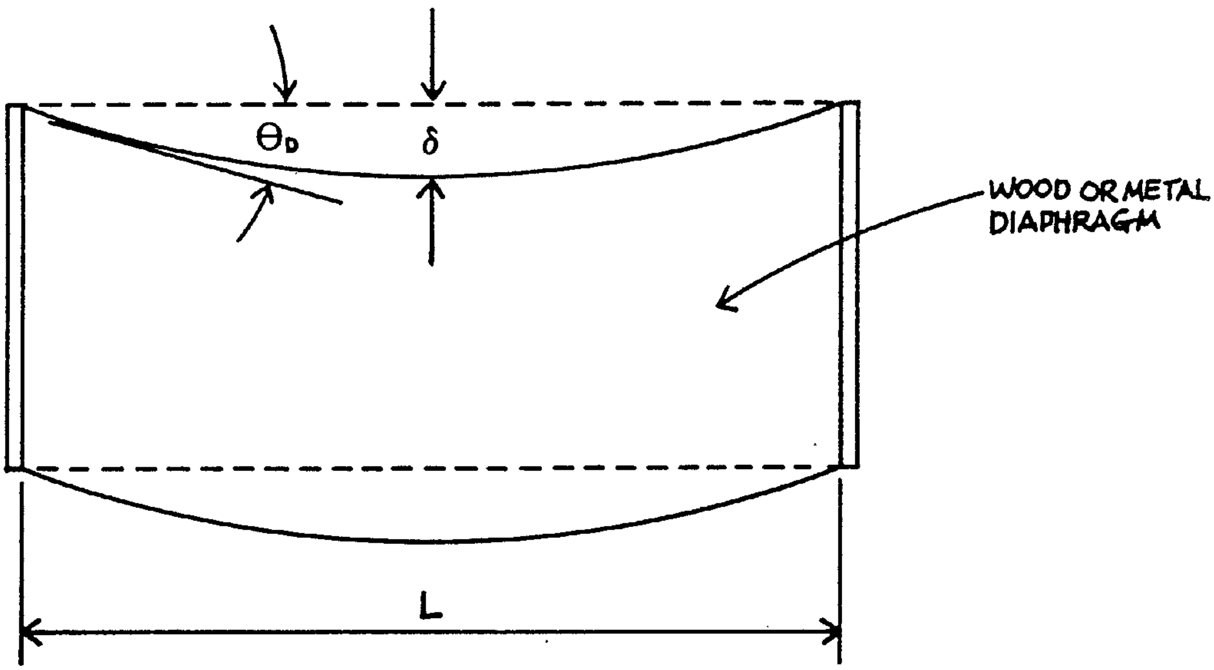
$$\theta = \frac{\delta}{h}$$

where:

δ = Tangential interstory drift

h = Story height

Figure 9: Local response parameter for brick and block walls: tangential interstory drift angle (Θ) (adapted from Kustu, et al., 1982).



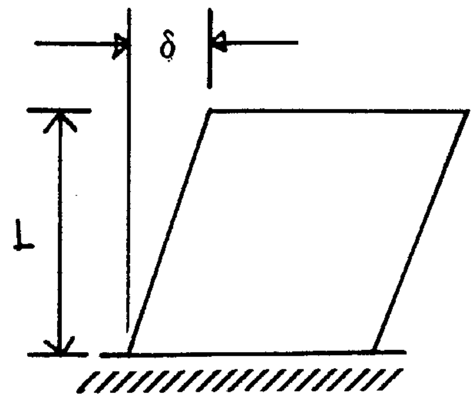
$$\Theta_D = \frac{\delta}{\frac{L}{2}}$$

WHERE:

δ = DIAPHRAGM DEFLECTION

L = DIAPHRAGM SHEAR SPAN.

Figure 10: Local response parameters for flexible diaphragms, wood or metal deck.



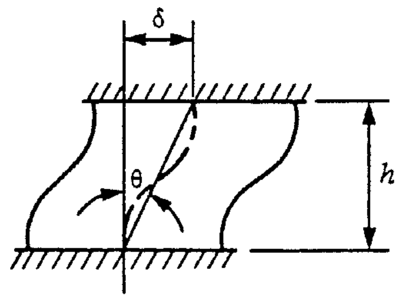
$$\theta = \frac{\delta}{L}$$

WHERE:

δ = SHEAR WALL DEFLECTION

L = SHEAR WALL HEIGHT

Figure 11: Local response parameters for plywood shearwalls.



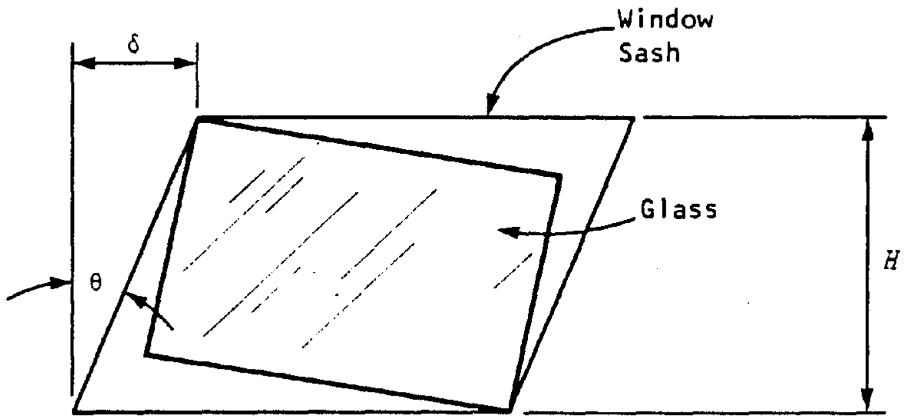
$$\theta = \frac{\delta}{h}$$

where:

δ = Tangential interstory drift

h = Story height

Figure 12: Local response parameter for drywall partitions: tangential interstory drift angle (θ) (adapted from Kustu, et al., 1982).



$$\theta = \frac{\delta}{H}$$

where:

δ = Tangential deflection along H

H = Height of glass

Figure 13: Local response parameter for glazing: deflection angle (Θ) (adapted from Kustu, et al, 1982).

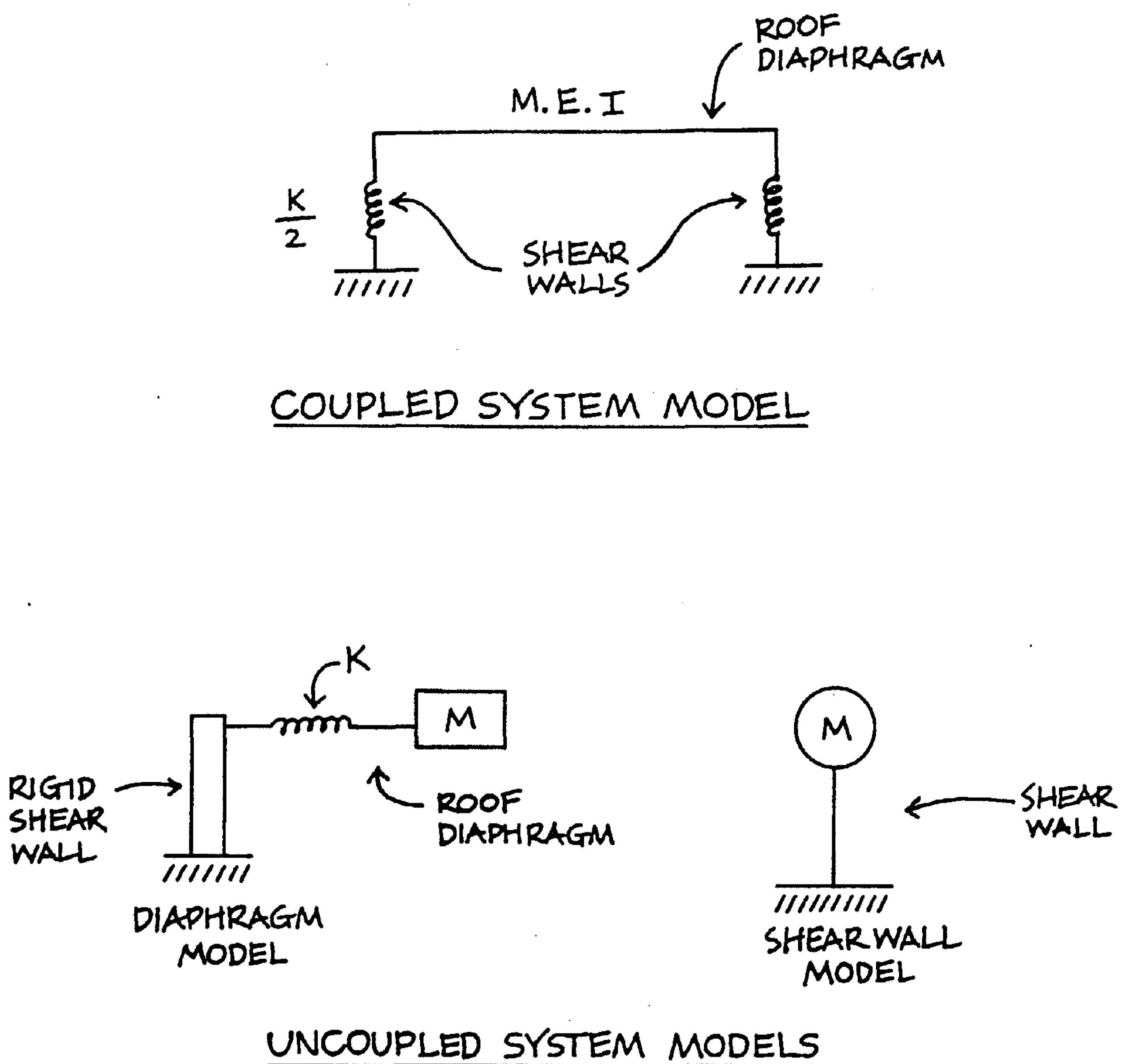
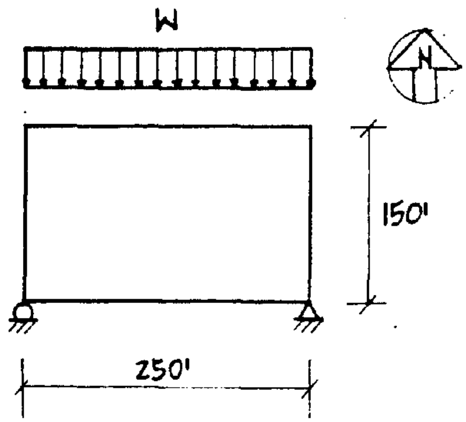


Figure 14: Model including diaphragm as flexible beam.



ROOF DEAD LOAD = 12 PSF

1/2" STRUCTURAL I C-D PLYWOOD DIAPHRAGM
8d NAILS @ 2 1/2", 4", 12" O.C.
$E = 1,800,000$ PSI
$G = 75,000$ PSI

8" CONCRETE TILT-UP WALLS
UNIT WEIGHT = 150 PCF ; $f'_c = 4000$ PSI

LOADS

$$W = \Big[\underbrace{(12\,PSF \times 150')}_{ROOF} + \underbrace{2(10' \times \frac{8''}{12''/1'} \times 150\,PCF)}_{TILT\text{-}UP\ WALLS}\Big]\underbrace{(0.186)}_{CODE\ SEISMIC\ COEFFICIENT} = 707\ PLF$$

$$V = \frac{WL}{2d} = \frac{(707\,PLF)(250')}{2(150')} = 589\ PLF$$

DIAPHRAGM STATIC DEFLECTION

$$\Delta = \underbrace{\frac{5VL^3}{8EAb}}_{\substack{BENDING\\DEFLECTION}} + \underbrace{\frac{VL}{4Gt}}_{\substack{SHEAR\\DEFLECTION}} + \underbrace{0.094\,L e_n}_{NAIL\ SLIP} + \frac{\Sigma \Delta_c X}{2b} \quad (REFERENCE\ 14)$$

$\frac{\Sigma \Delta_c X}{2b} \rightarrow$ ASSUME = 0

$$= \frac{5(589\,PLF)(250')^3}{8(3.6\times10^6\,PSI)(8''\times12'')(150')} + \frac{(589\,PLF)(250')}{4(75{,}000\,PSI)(0.545'')} + 0.094(250')(0.03)$$

$$= 0.11'' + 0.90'' + 0.71'' = 1.94''$$

EQUIVALENT BEAM DEFLECTION

$$\Delta = \frac{5WL^4}{384EI} \Longrightarrow I = \frac{5WL^4}{384E\Delta} = \frac{5(707\,PLF)(250'\times12''/1')^4}{384(1.8\times10^6\,PSI)(1.94'')(12''/1')} = 1.78\times10^7\,IN^4$$

DIAPHRAGM PERIOD T_D

$$\omega = \frac{\pi^2}{L^2}\sqrt{\frac{EI}{m}} \quad (SIMPLY\ SUPPORTED\ BEAM\text{-}UNIFORM\ LOAD)$$

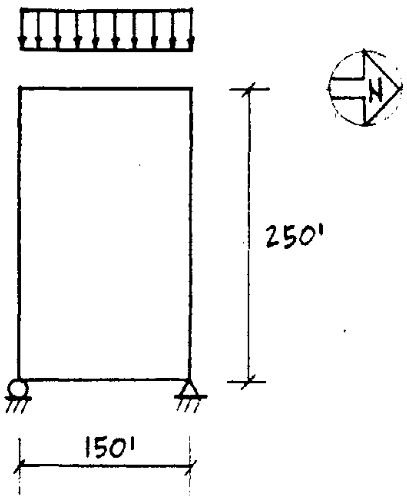
$$m = \frac{(707\,PLF)(1/0.186)(1'/12'')}{(32.2\,FT/SEC^2)(12''/1')} = 0.820\ \frac{\#\cdot SEC^2}{IN^2}$$

$$\omega = \frac{\pi^2}{(250'\times12''/1')}\sqrt{\frac{(1.8\times10^6\,PSI)(1.78\times10^7\,IN^4)}{0.820\ \#\cdot SEC^2/IN^2}} = 6.85\ RAD/SEC$$

$$T_D = \frac{2\pi}{\omega} = 0.92\ SEC$$

Figure 15: Plywood Diaphragm Period, North-South Direction

EQE



ROOF DEAD LOAD = 12 PSF

$\frac{1}{2}$" STRUCTURAL I C-D PLYWOOD DIAPHRAGM
8d NAILS @ $2\frac{1}{2}$", 4", 12" O.C.
E = 1,800,000 PSI
G = 75,000 PSI

8" CONCRETE TILT-UP WALLS
UNIT WEIGHT = 150 PCF
$f_c' = 4000$ PSI

LOADS

$$w = \left[(12\ \text{PSF} \times 250') + 2\left(10' \times \frac{8''}{12''/1'} \times 150\ \text{PCF}\right)\right](0.186) = 930\ \text{PLF}$$

$$V = \frac{wL}{2d} = \frac{(930\ \text{PLF})(150')}{2(250')} = 279\ \text{PLF}$$

DIAPHRAGM STATIC DEFLECTION

$$\Delta = \frac{5VL^3}{8EAb} + \frac{VL}{4Gt} + 0.094\,L\,e_n$$

$$= \frac{5(279\ \text{PLF})(150')^3}{8(3.6\times 10^6\ \text{PSI})(8''\times 12'')(250')} + \frac{(279\ \text{PLF})(150')}{4(75{,}000\ \text{PSI})(0.545'')} + 0.094(150')(0.01)$$

$$= 0.01'' + 0.26'' + 0.14'' = 0.41''$$

EQUIVALENT BEAM DEFLECTION

$$\Delta = \frac{5wL^4}{384EI} \implies I = \frac{5wL^4}{384E\Delta} = \frac{5(279\ \text{PLF})(150'\times 12''/1')^4(1'/12'')}{384(1.8\times 10^6\ \text{PSI})(0.41'')} = 4.31\times 10^6\ \text{IN}^4$$

DIAPHRAGM PERIOD T_D

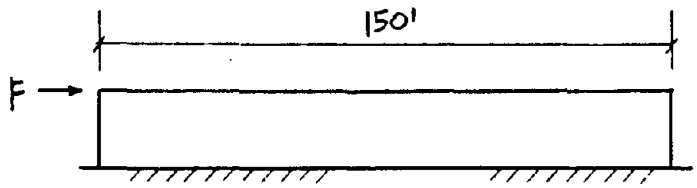
$$\omega = \frac{\pi^2}{L^2}\sqrt{\frac{EI}{m}}$$

$$m = \frac{(930\ \text{PLF})(1/0.186)(1'/12'')}{32.2\ \text{FT/SEC}^2 \cdot 12''/1'} = 1.08\ \frac{\#\cdot\text{SEC}^2}{\text{IN}^2}$$

$$\omega = \frac{\pi^2}{(150'\times 12''/1')^2}\sqrt{\frac{(1.8\times 10^6\ \text{PSI})(4.31\times 10^6\ \text{IN}^4)}{1.08\ \#\cdot\text{SEC}^2/\text{IN}^2}} = 8.16\ \text{RAD/SEC}$$

$$T_D = \frac{2\pi}{\omega} = 0.77\ \text{SEC}$$

Figure 16: Plywood Diaphragm Period, East-West Direction



$t = 8''$

$E = 3{,}600{,}000\ \text{PSI}$

NEGLECT BENDING

$$\Delta = \frac{1.2\,FH}{AG} = \frac{1.2\,F\,(20')(12''/')}{(150' \times 12''/' \times 8'')\left[\frac{3.6 \times 10^{6}\ \text{PSI}}{2(1+0.17)}\right]} = (1.30 \times 10^{-8})\,F$$

$$K = \frac{F}{\Delta} = \frac{1}{1.30 \times 10^{-8}} = 7.69 \times 10^{7}\ \#/\text{IN}$$

$$m = \frac{(12\ \text{PSF} \times 125' \times 150') + (150' + 250') \times 10' \times 8'' \times (1'/12'') \times 150\ \text{PCF}}{32.2\ \text{FT}/\text{SEC}^2 \times 12''/'}$$

$$= 1618\ \frac{\#\cdot\text{SEC}^2}{\text{IN}}$$

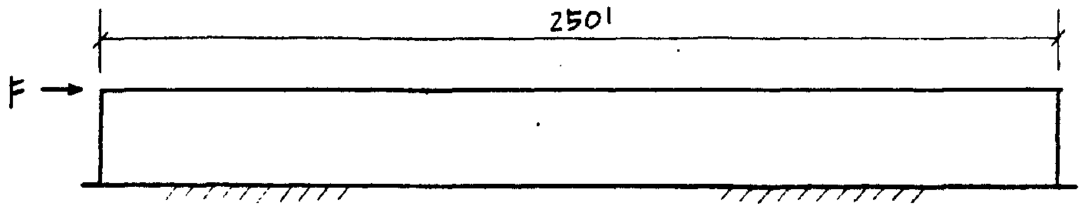
$$\omega = \sqrt{\frac{K}{m}} = \sqrt{\frac{7.69 \times 10^{7}\ \#/\text{IN}}{1618\ \#\cdot\text{SEC}^2/\text{IN}}} = 218\ \text{RAD}/\text{SEC}$$

$$T_W = \frac{2\pi}{\omega} = 0.029\ \text{SEC}$$

NOTE: $\underbrace{3 \times 0.029\ \text{SEC}}_{3T_W} = 0.087\ \text{SEC} \ll \underbrace{0.92\ \text{SEC}}_{T_D}$

THEREFORE DIAPHRAGM AND WALLS CAN BE CONSIDERED AS UNCOUPLED

Figure 17: Shear Wall Period, North-South Direction



t = 8"
E = 3,600,000 PSI

NEGLECT BENDING

$$\Delta = \frac{1.2\,FH}{AG} = \frac{1.2\,(20')(12''/1')\,F}{(250' \times 12''/1' \times 8'')\left[\frac{3.6 \times 10^{6}\ \text{PSI}}{2(1+0.17)}\right]} = (7.80 \times 10^{-9})\,F$$

$$K = \frac{F}{\Delta} = \frac{1}{7.80 \times 10^{-9}} = 1.28 \times 10^{8}\ \#/\text{IN}$$

$$m = \frac{(12\text{PSF} \times 75' \times 250') + (150' + 250') \times 10' \times 8'' \times (1'/12'') \times 150\text{PCF}}{32.2\ \text{FT/SEC}^2 \times 12''/1'}$$

$$= 1618\ \frac{\#\cdot\text{SEC}^2}{\text{IN}}$$

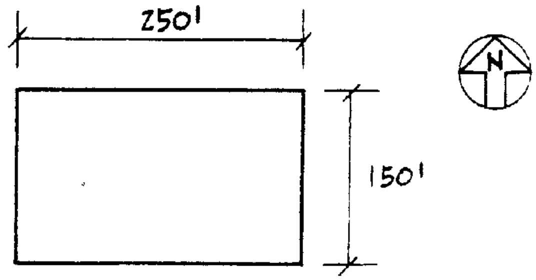
$$\omega = \sqrt{\frac{K}{m}} = \sqrt{\frac{1.28 \times 10^{8}\ \#/\text{IN}}{1.68\ \#\cdot\text{SEC}^2/\text{IN}}} = 281\ \text{RAD/SEC}$$

$$T_W = \frac{2\pi}{\omega} = 0.022\ \text{SEC}$$

NOTE: $3 \times 0.022 = 0.066$ SEC << 0.77 SEC

THEREFORE DIAPHRAGM AND WALLS CAN BE CONSIDERED AS UNCOUPLED

Figure 18: Shear Wall Period, East-West Direction



NORTH-SOUTH DIRECTION

DIAPHRAGM: $T_D = 0.92$ SEC
$S_a = 0.33$ g
$S_v = 19.6$ IN/SEC
$S_d = 2.83$ IN

WALL: $T_W = 0.029$ SEC
$S_a = 0.40$ g
$S_v = 0.43$ IN/SEC
$S_d = 0.0032$ IN

EAST-WEST DIRECTION

DIAPHRAGM: $T_D = 0.77$ SEC
$S_a = 0.45$ g
$S_v = 19.6$ IN/SEC
$S_d = 2.0$ IN

WALL: $T_W = 0.022$ SEC
$S_a = 0.40$ g
$S_v = 0.41$ IN/SEC
$S_d = 0.0020$ IN

Figure 19: Summary of Structure Response

TILT-UP PANELS

NORTH-SOUTH DIRECTION - 37.5 % OF WALLS

$S_d = 0.0032$ IN (FIGURE 19)

$$\theta = S_d / H = \frac{0.0032''}{20' \times 12''/1'} = 1.31 \times 10^{-5}$$

$1.31 \times 10^{-5} < 5 \times 10^{-4}$ NO DAMAGE (FIGURE 5)

EAST-WEST DIRECTION - 62.5% OF WALLS

$S_d = 0.0020$ IN

$\theta = 8.33 \times 10^{-6}$ NO DAMAGE (BY INSPECTION)

ROOF STRUCTURES AND ROOFING

NORTH-SOUTH DIRECTION - 50%

$S_d = 2.83$ IN

$$\theta = \frac{S_d}{L/2} = \frac{2.83''}{(250'/2)(12''/1')} = 0.00189$$

DAMAGE LEVEL 1: THRESHOLD 9.07×10^{-4} D.F. = 0.05

DAMAGE LEVEL 2: THRESHOLD 4.00×10^{-3} D.F. = 0.50

ASSUME D.F. CAN BE LINEARLY INTERPOLATED

$$\frac{0.004 - 0.00189}{0.004 - 0.000907} = \frac{0.50 - D.F.}{0.50 - 0.05} \Longrightarrow D.F. = \underline{\underline{0.193}}$$

EAST-WEST DIRECTION - 50%

$S_d = 2.0$ IN

$$\theta = \frac{S_d}{L/2} = \frac{2.0''}{(150'/2)(12''/1')} = 0.00222$$

$$\frac{0.004 - 0.00222}{0.004 - 0.000907} = \frac{0.50 - D.F.}{0.50 - 0.05} \Longrightarrow D.F. = \underline{\underline{0.241}}$$

Figure 20: Example Calculations for Damage Factors

TABLE 3

RECOMMENDED v/a AND ad/v^2 RATIOS

Site Condition	v/a (in/sec)/g	ad/v^2
Rock	25.5	5.25
Less than 30 feet of alluvium underlain by rock	34.5	4.35
30-200 feet of alluvium underlain by rock	33.0	4.45
Alluvium	52.5	3.70

TABLE 4

RECOMMENDED SPECTRAL AMPLIFICATION (5 PERCENT DAMPING)

Site Condition	Displacement	Velocity	Acceleration
Rock	1.82	1.31	2.09
Less than 30 feet of alluvium underlain by rock	2.53	1.27	2.63
30-200 feet of alluvium underlain by rock	1.78	1.48	2.32
Alluvium	2.05	1.42	2.14

TABLE 5

BUILDING COMPONENTS

Class	Component	Local Demand	Damage State 1 Description	Damage State 1 Threshold	Damage State 1 DF	Damage State 2 Description	Damage State 2 Threshold	Damage State 2 DF	Damage State 3 Description	Damage State 3 Threshold	Damage State 3 DF	Source
B-01	Reinforced concrete beams.	Disp., Fig. 6a	Cracking of concrete.	0.00250	0.06	Yielding of reinforcing bars.	0.00830	0.60	Failure of member.	0.05700	1.25	Ref. 86
B-02	Reinforced concrete columns.	Disp., Fig. 6b	Cracking of concrete.	0.00250	0.06	Yielding of reinforcing bars.	0.00950	0.60	Failure of member.	0.02600	1.50	Ref. 86
B-03	Steel moment resisting sub-assemblies.	Disp., Fig. 7	Buckling of steel flanges and web, yielding of section.	0.01140	0.65	Failure of sub-assembly.	0.05740	1.25	N.A.	-	-	Ref. 86
B-04	Reinforced concrete shear walls.	Disp., Fig. 8	Cracking of concrete.	0.00056	0.06	Yielding of reinforcing steel.	0.00370	0.60	Ultimate.	0.01851	1.25	Ref. 86
B-05	Brick infill wall, reinforced.	Disp., Fig. 9a	Moderate Cracking	0.00422	0.45	Ultimate.	0.03704	1.25	N.A.	-	-	Ref. 86
B-06	Brick infill wall, unreinforced.	Disp., Fig. 9a	Ultimate.	0.00422	1.25	N.A.	-	-	N.A.	-	-	Ref. 86
B-07	Block infill wall, reinforced.	Disp., Fig. 9a	Moderate Cracking	0.00845	0.45	Ultimate.	0.02865	1.25	N.A.	-	-	Ref. 86
B-08	Block infill wall, unreinforced.	Disp., Fig. 9a	Moderate Cracking	0.00264	0.45	Ultimate.	0.01104	1.25	N.A.	-	-	Ref. 86
B-09	Brick and block pier walls, reinforced.	Disp., Fig. 9b	Moderate Cracking	0.00299	0.45	Ultimate.	0.01557	1.25	N.A.	-	-	Ref. 86
B-10	Precast concrete exterior walls.	Disp., Fig. 9a	Cracking of concrete.	0.00050	0.05	Yielding of reinforcing steel.	0.00400	0.60	Failure of connections.	0.00900	1.10	Judgment
B-11	Flexible diaphragms, wood or metal deck.	Disp., Fig. 10	Pulling of nails.	0.00097	0.05	Buckling of panels, splitting of support members, etc.	0.00400	0.50	Ultimate.	0.00924	1.00	Judgment
B-12	Plywood shear walls.	Disp., Fig. 11	Pulling of nails.	0.00097	0.05	Buckling of panels, splitting of support members, crushing of corners.	0.00400	0.20	Ultimate.	0.00920	0.80	Judgment
B-13	Drywall partitions.	Disp., Fig. 12	Minor cracking.	0.00283	0.06	Significant cracking of drywall, separation of drywall from studs, crushing in corners.	0.00617	1.25	N.A.	-	-	Ref. 86

N.A. - Not applicable
Disp. - Displacement
Accel. - Acceleration

TABLE 5 (Continued)

Class	Component	Local Demand	Damage State 1 Description	Threshold	DF	Damage State 2 Description	Threshold	DF	Damage State 3 Description	Threshold	DF	Source
B-14	Glass, window.	Disp., Fig. 13	Realignment.	0.01405	0.10	Broken.	0.02811	0.40	N.A.	-	-	Ref. 86
B-15	Metal exterior cladding.	Disp., Fig. 12	Buckling and plastic deformation of panels.	0.00300	0.10	Failure of connections.	0.01000	1.10	N.A.	-	-	Judgment
B-16	Suspended ceilings	Accel.	Only occasional dislodged tiles.	0.20000	0.10	Falling of some of ceiling, especially at perimeter	0.40000	0.60	Falling of most or all of ceilings tiles, as well as some ceiling-mounted equipment and ceiling frame.	0.50000	1.00	Experience Ref. 86 Ref. 132
B-17	Ornamentation	Accel.	Some falling and breaking.	0.33000	0.20	Most objects fall and break	0.50000	0.80	N.A.	-	-	Ref. 132
B-18	Chillers, boilers, air handlers - skid mounted equipment. On isolation mounts or unanchored.	Accel.	Sliding causes severance of attached conduit or tubing, overturning also possible.	0.20000	0.30	N.A.	-	-	N.A.	-	-	Experience and calculation
B-19	Chillers, boilers, air handlers - skid mounted equipment. Anchored.	Accel.	Sliding causes severance of attached conduit or tubing, overturning is also possible.	0.50000	0.30	N.A.	-	-	N.A.	-	-	Experience and calculation
B-20	Ducting - square or cylindrical sheet metal piping usually supported by wires on rods.	Accel.	Failure of supports.	0.40000	0.10	Inertial failure at points of stress concentration.	0.60000	0.50	N.A.	-	-	Experience and calculation
B-21	Suspended heaters, coolers, fans (steam, hot water, or gas fired) Usually supported by rods.	Accel.	Rupture of attached piping due to swaying of unit.	0.30000	0.20	Falling of unit.	0.50000	1.00	N.A.	-	-	Experience and calculation
B-22	Wall-mounted emergency lighting, P.A. systems, fire alarms bolted to wall.	Accel.	Building will normally collapse prior to unit falling off wall assuming normal bolting.	1.00000	0.05	N.A.	-	-	N.A.	-	-	Experience and judgment

N.A.- Not applicable
Disp. - Displacement
Accel. - Acceleration

TABLE 5 (Continued)

Class	Component	Local Demand	Damage State 1 Description	Threshold	DF	Damage State 2 Description	Threshold	DF	Damage State 3 Description	Threshold	DF	Source
B-23	Cooling towers - forced draft.	Accel.	Leakage of water or rupture of attached piping has been known, or structural damage.	0.30000	0.10	Extensive structural damage to unit.	0.60000	0.30	N.A.	-	-	Experience and judgment
B-24	Elevators.	Accel.	Counterweight derailment, cable misalignment, shifting of motor controller, possible cab derailment, other severe damage, cab may be damaged, elevator room damage.	0.40000	0.60	N.A.	-	-	N.A.	-	-	

N.A. - Not applicable
Disp. - Displacement
Accel. - Acceleration

TABLE 6

GENERAL COMPONENTS

Class	Component	Local Demand	Damage State 1 Description	Damage State 1 Threshold	Damage State 1 DF	Damage State 2 Description	Damage State 2 Threshold	Damage State 2 DF	Damage State 3 Description	Damage State 3 Threshold	Damage State 3 DF	Source
G-01	Equipment with high center of gravity. Height at least equal to three times minimum base dimension. Supported on pads, leveling jacks, or locked rollers (e.g., computer cabinetry control panels.	Accel.	Sliding with some severing of attached conduits or tubing.	0.10	0.20	Sliding and overturning, severing of all attached conduits and tubing.	0.30	0.50	N/A	-	-	Calculation
G-02	Control panels and instrumentation cabinetry. Unanchored. columns.	Accel.	Sliding causes severance of attached conduit or tubing, overturning also possible.	0.30	0.10	Overturning.	0.50	0.50	N/A	-	-	Experience and calculation
G-03	Control panels and instrumentation cabinetry. Anchored.	Accel.	Component malfunction.	0.30	0.05	Sliding causes severance of attached conduits or tubing, overturning also possible.	0.50	0.10	Overturning	-	-	Experience and calculation
G-04	Electrical switchgear - cabinet-mounted equipment bolted to floor.	Accel.	Rocking motion pulls anchor bolts and switchgear slides severing attached conduit.	0.50	0.10	Overturning.	0.60	0.80	N/A	-	-	Experience, calculation, Ref. 132
G-05	Ceiling-suspended installations (e.g., laminar flow hoods, certain types of ducting.	Accel.	Some installations collapsed due to lack of bracing for lateral loads.	0.30	0.10	Most installations collapse due to lack of bracing	0.50	0.80	N/A	-	-	Calculation and experience
G-06	Piping - all sizes and lengths except short attachments to heavy structures such as tanks.	Accel.	Failure of supports.	0.40	0.05	Cracking or rupture at points of stress concentration such as valves or nozzles.	0.60	0.50	N/A	-	-	Experience and Ref. 132
G-07	Conduit - light steel or aluminum piping, threaded connections	Accel.	Failure of supports.	0.40	0.05	Cracking or rupture of stress concentration such as valves or nozzles.	0.60	0.50	N/A	-	-	Judgment

N/A - Not applicable
Disp. - Displacement
Accel. - Acceleration

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TABLE 6 (Continued)

Class	Component	Local Demand	Damage State 1 Description	Damage State 1 Threshold	Damage State 1 DF	Damage State 2 Description	Damage State 2 Threshold	Damage State 2 DF	Damage State 3 Description	Damage State 3 Threshold	Damage State 3 DF	Source
G-08	Cable trays - racks often of Unistrut supported by rods or beams.	Accel.	Failure at support connections with ceiling, especially if only C-clamped.	0.40	0.20	Collapse of large sections.	0.60	0.60	N/A	-	-	Judgment and experience
G-09	Pumps - either horizontal or vertical, all types and sizes.	Accel.	Shaft misalignment.	0.50	0.20	Failure of connecting piping.	0.60	0.50	Failure of pump anchorage	0.70	1.00	Experience
G-10	Transformers - supported on racks or on the ground, assumed to be bolted down.	Accel.	Rocking motion pulls anchor bolts and transformer slides severing attached conduit.	0.30	0.05	Overturning.	0.50	0.50	N/A	-	-	Experience
G-11	Battery racks. Without wraparound.	Accel.	Slight shifting of batteries.	0.10	0.10	Batteries fall from rack.	0.20	1.00	N/A	-	-	Experience
G-12	Battery racks.	Accel.	Slight shifting	0.50	0.10	N/A	-	-	N/A	-	-	Experience

N/A - Not applicable
Disp. - Displacement
Accel. - Acceleration

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TABLE 7

EXAMPLE, BUILDING COMPONENT INVENTORY

Group	Component Class	Replacement Value	Percent of Total
Substructure	Structural Excavation	$ 22,400	1.14
	Footings and Slab	262,000	13.34
		284,400	14.48
Superstructure	Tilt-up Panels	337,240	17.17
	Columns	52,800	2.69
	Roof Structure and Roofing	344,520	17.54
		734,560	37.40
Architectural	Overhead Doors	22,440	1.14
	Windows (Glass and Aluminum)	38,280	1.95
	Suspended Ceilings	64,800	3.30
	Interior Partitions	70,000	3.56
	Carpet	41,940	2.14
	Vinyl Tile	19,800	1.01
		257,260	13.14
Mechanical and Electrical	Roof Drain	7,920	0.40
	Sprinklers and Piping	66,000	3.36
	General and HVAC	210,000	10.69
	Plumbing	20,000	1.02
	Lights and Wiring	165,000	8.40
		468,920	23.87
Other	Grading and Paving	166,320	8.47
	Storm Drainage	14,520	0.74
	Site Work Concrete	38,280	1.95
	TOTAL:	$1,964,260	100%

TABLE 8

ESTIMATED BUILDING COMPONENT DAMAGE FACTORS

Group	Component Class	Replacement Value ($)	Class	Demand	Damage Factor
Substructure	Structural Excavation	22,400	None	--	0.00
	Footings and Slab	262,000	None	--	0.00
Superstructure	Tilt-up Panels	337,520	B-10	38% @ .0000131	0.00
				62% @ .0000083	0.00
	Columns	52,800	B-02	--	0.00
	Roof Structures and	344,520	B-11	50% @ 0.00189	0.193
	Roofing			50% @ 0.00222	0.241
Architectural	Overhead Doors	22,400	None	--	0.1
	Windows (Glass and Aluminum)	38,280	B-14	--	0.00
	Suspended Ceilings	64,800	B-16	50% @ 0.33g	0.43
				50% @ 0.36g	0.50
	Interior Partitions	70,000	B-13	--	0.00
	Carpet	41,940	None	--	1.00
	Vinyl Tile	19,800	None	--	1.00
Mechanical and Electrical	Roof Drain	7,920	G-06	0.40g	0.05
	Sprinkler	66,000	G-06	0.40g	0.05
	General HVAC	210,000	B-20	0.40g	0.10
	Plumbing	20,000	G-06	0.40g	0.05
	Lights and Wiring	165,000	G-06	0.40g	0.10
Other	Grading and Paving	166,320	None	--	0.00
	Storm Drainage	14,520	None	--	0.00
	Site Work Concrete	38,280	None	--	0.00

Note: Some components have two entries, one for each of the two assumed earthquake directions, see Figure 20 for sample calculations of the damage factors for specific components.

TABLE 9

ESTIMATED BUILDING COMPONENT REPAIR COSTS

Group	Component Class	Replacement Value ($)	Repair Costs ($)
Substructure	Structural Excavation	22,400	0
	Footings and Slab	262,000	0
Superstructure	Tilt-up Panels	337,240	0
	Columns	52,800	0
	Roof Structure and Roofing	344,520	74,761
Architectural	Overhead Doors	22,440	2,244
	Windows (Glass and Aluminum)	38,280	0
	Suspended Ceilings	64,800	30,132
	Interior Partitions	70,000	0
	Carpet	41,940	41,940
	Vinyl Tile	19,800	19,800
Mechanical and Electrical	Roof Drain	7,920	396
	Sprinklers and Piping	66,000	3,300
	General and HVAC	210,000	21,000
	Plumbing	20,000	1,000
	Lights and Wiring	165,000	16,500
Other	Grading and Paving	166,320	0
	Storm Drainage	14,520	0
	Site Work Concrete	38,280	0
TOTAL:		$1,964,260	211,073

TOTAL DF = 110%

TOTAL DF = 15% if one excludes "Substructure" and "Other" categories from total replacement cost.

TABLE 10

EQUIPMENT COMPONENTS

Class	Component	Local Demand	Damage State 1 Description	Threshold	DF	Damage State 2 Description	Threshold	DF	Damage State 3 Description	Threshold	DF	Source
E-01	Equipment mounted on air pads or rollers (e.g., computer cabinetry, Perkins Elmers, oscilloscopes on roller racks).	Accel.	Misalignment	0.05	0.02	Sliding with severing of attached conduit or tubing. Rollers fall into floor penetrations, impact with adjacent equipment.	0.10	0.20	Tripping and overturning of most tall equipment.	0.30	0.80	Judgment, Ref. 132
E-02	Equipment mounted on leveling jacks	Accel.	Misalignment	0.10	0.02	Sliding with severing of attached conduit or tubing. Rollers fall into floor penetrations, impact with adjacent equipment.	0.20	0.20	Tripping and overturning of most tall equipment.	0.30	0.80	Judgment
E-03	Equipment mounted on stands, work benches, tables or desks, not bolted down (e.g., CRTs, microscopes, instruments).	Accel.	Shifting of equipment.	0.20	0.05	Sliding and falling off support table.	0.40	0.80	N/A	-	-	Calculation, experience, Ref. 132
E-04	Shelving with high center of gravity (e.g., tape racks, shelving for components).	Accel.	Spilling of shelf contents.	0.10	0.10	Overturning of some shelves.	0.30	0.20	Overturning of most shelves.	0.60	0.80	Calculation, experience, Ref. 132
E-05	Raised floors for computer facilities, etc.	Accel.	Partial collpase of floor from building of support pedestals.	0.30	0.10	Total collapse	0.50	1.00	N/A	-	-	Calculation
E-06	Tanks (all types are included) - ground and leg supported, vertical, and horizontal; contents assumed to be liquid. Tanks are assumed to be anchored to foundation except in case of large oil or water storage tanks.	Accel.	Rocking of tank ruptures short runs of attached piping.	0.20	0.10	Sloshing of tank contents; rupture of connections of roof and wall and base. Sloshing buckles walls.	0.40	0.60	N/A	-	-	Experience Ref. 132

N/A - Not applicable
Disp. - Displacement
Accel. - Acceleration

TABLE 10 (Continued)

EQUIPMENT COMPONENTS

Class	Component	Local Demand	Damage State 1 Description	Damage State 1 Threshold	Damage State 1 DF	Damage State 2 Description	Damage State 2 Threshold	Damage State 2 DF	Damage State 3 Description	Damage State 3 Threshold	Damage State 3 DF	Source
E-07	Office fixtures including desks, tables, files.	Accel.	Sliding of desks or tables; sliding of contents.	0.30	0.05	N/A	-	-	N/A	-	-	Experience, judgment, Ref. 132
E-08	Gas bottles - small vertical tanks with tubing connections. Unchained.	Accel.	Overturning with rupture of attached tubing, toxicity hazard depending on contents.	0.10	0.50	N/A	-	-	N/A	-	-	Experience, calculation
E-09	Gas bottles - small vertical tanks with tubing connections. Chained.	Accel.	Overturning with rupture of attached tubing, toxicity hazard depending on contents.	0.50	0.50	N/A	-	-	N/A	-	-	Experience, calculation
E-10	Freestanding; moveable partitions.	Accel.	Occasional topover.	0.40	0.05	Tipover of most partitions.	0.80	0.20	N/A	-	-	Ref. 132
E-11	Ceiling-mounted light fixtures.	Accel.	Falling of some fixtures.	0.40	0.30	Falling of most fixtures.	0.60	0.80	N/A	-	-	Ref. 132

N/A - Not applicable
Disp. - Displacement
Accel. - Acceleration

TABLE 11

EXAMPLE, CRITICAL EQUIPMENT INVENTORY

Item	Quantity	Replacement Value ($)	Class	Damage Factor (%)	Repair Cost ($)
Ion Implanter	1	500,000	E-2	0.8	400,000
Sputterer	1	125,000	E-1	0.8	100,000
Sputterer	1	100,000	E-1	0.8	80,000
Plasma Therm Etcher	1	90,000	E-2	0.8	72,000
Scanning Electron Microscope	1	75,000	E-1	0.8	60,000
Micralign	4	960,000	E-2	0.8	768,000
Wafertrack	1	85,000	E-2	0.8	68,000
Screen Printer	1	25,000	E-1	0.8	20,000
BTU Engineering Packaging Furnace	3	75,000	E-2	0.8	60,000
Mebes	1	2,500,000	G-2	0.1	250,000
Diffusion Furnaces	4	4,000,000	G-2	0.1	400,000
HYPOX	1	90,000	E-2	0.8	72,000
LPCVD (ASM) Furnace	1	125,000	E-2	0.8	100,000
EPI Reactor	1	275,000	E-2	0.8	220,000
Mask Inspector	1	100,000	E-2	0.8	80,000
Wave Solderer	1	250,000	E-2	0.8	200,000
Auto Bonder	12	1,200,000	E-3	0.8	960,000
TOTAL:		8,077,000		48.0	3,910,000

TABLE 12

SIGNIFICANT COMPONENTS OF A HIGH-TECHNOLOGY FACILITY

Entity	Damage Type
Employees	
- skilled	- injuries, deaths
- unskilled	- injuries, deaths
Buildings	- collapses, condemnation, cost of repair, recovery time
Equipment	
- Critical	- Functionality, cost of repair, recovery time
- Non-critical	- Functionality, cost of repair, recovery time
Materials (on hand)	
- Raw materials	- Quantity destroyed, dollar loss
- In process materials	- Quantity destroyed, dollar loss
- Completed products	- Quantity destroyed, dollar loss
External Influences Affecting Recovery	
- Utilities (electricity, gas, etc.)	- Availability, recovery time
- Suppliers (raw materials)	- Ability to make deliveries, recovery time
- Consumers of completed products	- Rate of product consumption, recovery time

7. CONCLUSIONS AND RECOMENDATIONS FOR FUTURE RESEARCH

The component approach proposed here for use in evaluating seismic damage to high-technology facilities shows promise. The method is capable of assessing all types of physical damage and dollar costs, and a qualitative description of the facility damage is available. The method presents a detailed description of damage, which is necessary to evaluate the nonphysical damage types (i.e., loss of market share, lost research efforts, etc.).

The component approach can be modified so that secondary types of damage may be assessed. For instance, damage caused by postearthquake fires, soil liquefaction, and foundation failures may be assessed if the proposed methodology is appropriately modified. Such modifications should be the subject of future research.

The component classification system needs to be further developed, particularly in the area of critical production and research equipment found in high-technology industries. As yet, there are few or no data available to evaluate this damage on a rational basis. Typical building components also require more substantial evidence of their exact damage levels.

In addition, the component approach lacks a good method of evaluating partial and total structural collapse. This question is important for evaluating cross damage, i.e., the damage inflicted on equipment by the collapse of the structure.

Overall, the component approach is general and flexible. It can be adapted and augmented as more hard data about structural damages are made available. The details of the methodology, however, do require further study and evaluation.

As a part of the research effort reported here, high-technology facilities in the Santa Clara Valley of northern California were surveyed to obtain information relating to building types, use groups,

area (square feet), year of construction, and number of occupants. Table A-1 summarizes the building types used for the data collection. Tables A-2 through A-7 present summaries of the collected data. In many instances the total count (CNT) indicated for each structure type exceeds the sum of the entries for the one- to five-story categories. This is because in some cases detailed information was not available.

The following conclusions can be made from the collected data. First, independent of the use group (i.e., office, laboratory, manufacturing, warehouses, and other), the one-story tilt-up building is the most prevalent type, accounting for roughly 46% of all buildings used. The second most popular structure type is either one-story steel, two-story concrete tilt-up, or two-story concrete, depending on the use group. The two-story tilt-up is clearly the second most prevalent building type overall. This tilt-up buildings in all configurations account for roughly 67% of the buildings used in high-technology facilities. Correlations between building types and industries were not developed.

A similar study for equipment specific to high-technology industries was conducted. The equipment data did not reveal any specific trends except that the equipment used by a high-technology facility is strongly dependent on the nature of the work performed at a facility. For instance, the equipment found commonly in semiconductor fabrication plants is significantly different from the equipment found in aerospace industry facilities. As a result, the data collected are not presented here, for a lack of interesting trends and a lack of sample size (i.e., an individual survey for each industry would have to be conducted to build up a large enough data base to identify equipment trends).

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APPENDIX A

Summary of Typical Building and Equipment Types

TABLE A-1

SUMMARY OF STRUCTURE TYPES

Type	Description
Steel	Steel moment-resisting frames Steel-braced frames
Concrete	Concrete moment-resisting frames Concrete shear wall Concrete and steel (dual)
Tilt-up	Concrete tilt-up Concrete tilt-up and steel frame
Block	Concrete block
Masonry	Unreinforced masonry wall Reinforced masonry wall
Wood	Wood frame

TABLE A-2

STRUCTURE TYPE DATA SUMMARY FOR ALL USE GROUPS

Structure Type	Total		Number of Stories: 1		2		3		4		5+		Square Feet (x 1000)		Year		Number Occupancy	
	CNT	%	CNT	%	CNT	%	CNT	%	CNT	%	CNT	%	CNT	AVG	CNT	AVG	CNT	AVG
Steel	37	18.1	14	60.9	9	39.1	0	-	0	-	0	-	23	56.4	29	1976	20	152
Concrete	21	10.3	0	-	12	63.2	3	15.8	1	5.3	3	15.8	18	121.7	21	1967	18	465
Tilt-Up	137	67.2	48	68.6	22	31.4	0	-	0	-	0	-	73	60.7	88	1973	55	199
Block	4	2.0	4	100.0	0	-	0	-	0	-	0	-	4	32.3	4	1971	4	73
Masonry	3	1.5	3	100.0	0	-	0	-	0	-	0	-	3	28.3	3	1965	3	73
Wood	2	1.0	1	50.0	1	50.0	0	-	0	-	0	-	2	39.1	2	1981	0	-

SUMMARY TABLE

Structure Type	Number of Stories	% of Total
Tilt-Up	1	46.1
Tilt-Up	2	21.1
Steel	1	11.0
Steel	2	7.1
Concrete	2	6.5
Total		91.8%

TABLE A-3

STRUCTURE TYPE SUMMARY FOR OFFICE USE GROUP

Structure Type	Total		Number of Stories: 1		2		3		4		5+		Square Feet (x 1000)		Year		Number Occupancy	
	CNT	%	CNT	%	CNT	%	CNT	%	CNT	%	CNT	%	CNT	AVG	CNT	AVG	CNT	AVG
Steel	12	24.5	8	66.7	4	33.3	0	0	0	0	0	0	12	56.5	12	1976	12	173
Concrete	5	10.2	0	0	4	80.0	0	0	0	0	1	20.0	5	75.7	5	1969	5	298
Tilt-Up	30	61.2	22	75.9	7	24.1	0	0	0	0	0	0	30	40.7	27	1977	24	145
Block	2	4.1	2	100.0	0	0	0	0	0	0	0	0	2	25.1	2	1977	2	76
Masonry	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-	0	-
Wood	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-	0	-

SUMMARY TABLE

Structure Type	Number of Stories	% of Total
Tilt-Up	1	46.5
Steel	1	16.3
Tilt-Up	2	14.7
Concrete	2	8.2
Steel	2	8.2
Block	1	4.1
Total		98%

TABLE A-4

STRUCTURE TYPE SUMMARY FOR LABORATORY USE GROUP

Structure Type	Total		Number of Stories: 1		2		3		4		5+		Square Feet (x 1000)		Year		Number Occupancy	
	CNT	%	CNT	%	CNT	%	CNT	%	CNT	%	CNT	%	CNT	AVG	CNT	AVG	CNT	AVG
Steel	14	50.0	9	64.3	5	35.7	0	-	0	-	0	-	14	56.6	14	1976	14	187
Concrete	2	7.1	0	-	2	100.0	0	-	0	-	0	-	2	98.8	2	1965	2	430
Tilt-Up	11	39.3	9	90.0	1	10.0	0	-	0	-	0	-	11	52.1	10	1977	10	172.5
Block	1	3.6	1	100.0	0	-	0	-	0	-	0	-	1	14.0	1	1974	1	40
Masonry	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-
Wood	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-

SUMMARY TABLE

Structure Type	Number of Stories	% of Total
Tilt-Up	1	35.4
Steel	1	32.2
Steel	2	17.9
Concrete	2	7.1
Tilt-Up	2	3.9
Block	1	3.6
Total		100%

TABLE A-5

STRUCTURE TYPE SUMMARY FOR MANUFACTURING USE GROUP

Structure Type	Total		Number of Stories: 1		2		3		4		5+		Square Feet (x 1000)		Year		Number Occupancy	
	CNT	%	CNT	%	CNT	%	CNT	%	CNT	%	CNT	%	CNT	AVG	CNT	AVG	CNT	AVG
Steel	4	16.0	2	50.0	2	50.0	0	-	0	-	0	-	4	55.0	4	1975	4	150
Concrete	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-
Tilt-Up	20	80.0	13	86.7	2	13.3	0	-	0	-	0	-	18	49.6	20	1975	18	218
Block	1	4.0	1	100.0	0	-	0	-	0	-	0	-	1	65.0	1	1955	1	100
Masonry	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-
Wood	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-

SUMMARY TABLE

Structure Type	Number of Stories	% of Total
Tilt-Up	1	69.4
Tilt-Up	2	10.6
Steel	1	8.0
Steel	2	8.0
Block	1	4.0
Total		100%

TABLE A-6

STRUCTURE TYPE SUMMARY FOR WAREHOUSE USE GROUP

Structure Type	Total		Number of Stories: 1		2		3		4		5+		Square Feet (x 1000)		Year		Number Occupancy	
	CNT	%	CNT	%	CNT	%	CNT	%	CNT	%	CNT	%	CNT	AVG	CNT	AVG	CNT	AVG
Steel	5	27.8	5	100.0	0	-	0	-	0	-	0	-	5	22.8	5	1977	5	79
Concrete	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-
Tilt-Up	13	72.2	10	90.9	1	9.1	0	-	0	-	0	-	13	55.7	12	1976	11	201
Block	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-
Masonry	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-
Wood	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-

SUMMARY TABLE

Structure Type	Number of Stories	% of Total
Tilt-Up	1	65.6
Steel	1	27.8
Tilt-Up	2	6.6
Total		100%

EQE

TABLE A-7

STRUCTURE TYPE SUMMARY FOR OTHER USE GROUPS

Structure Type	Total		Number of Stories: 1		2		3		4		5+		Square Feet (x 1000)		Year		Number Occupancy	
	CNT	%	CNT	%	CNT	%	CNT	%	CNT	%	CNT	%	CNT	AVG	CNT	AVG	CNT	AVG
Steel	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-
Concrete	1	20.0	0	-	1	100.0	0	-	0	-	0	-	1	100.0	1	1963	1	450
Tilt-Up	4	80.0	1	100.0	0	-	0	-	0	-	0	-	1	26.0	4	1972	1	125
Block	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-
Masonry	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-
Wood	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-	0	-

SUMMARY TABLE

Structure Type	Number of Stories	% of Total
Tilt-Up	1	80.0
Concrete	2	20.0
Total		100%

EQE

APPENDIX B

COMPARISONS OF PREDICTED EARTHQUAKE LOSSES SUFFERED IN THE APRIL 24, 1984 MORGAN HILL, CALIFORNIA EARTHQUAKE

CONTENTS

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FIGURES

CONTENTS (Continued)

FIGURES (Continued)

TABLES

120 Mast Street Building

CONTENTS (Continued)

TABLES (Continued)

490 Jarvis Drive Building

PHOTOS

120 Mast Street Building

490 Jarvis Drive Building

B.1 INTRODUCTION

B.1.1 Purpose

The objective of the original program was to develop a methodology for predicting the potential impact of a damaging earthquake on high-technology industries and both local and national economies. Because one-story and two-story concrete tilt-up structures are most prevelant in high-technology industrial parks, the methodology was aimed specifically at predicting damage to these structures. The scope of work was expanded to test this methodology with actual cases from the April 24, 1984 Morgan Hill, California earthquake. Two tilt-up buildings visited by EQE engineers shortly after the earthquake were chosen as test cases. These two structures are located at 120 Mast Street and 490 Jarvis Drive in Morgan Hill. At the time of the earthquake, Wiltron Company (Wiltron) was occupying both buildings. Wiltron is a high-technology company which manufactures microwave communications equipment. Both buildings experienced minor structural and nonstructural damage. The results of the study of the Wiltron buildings are presented in this appendix.

B.1.2 Scope of Work

The scope of work consists of evaluating the developed metholdology using, as test cases, two buildings which experienced damage during the Morgan Hill earthquake. The loss predictions are compared to the actual costs for the repairs performed on the buildings. Only structural and non-structural components of the buildings are included in the comparisons. Repair costs for equipment and costs associated with business interruption are not considered.

The work performed on each selected building can be summarized as follows:

1. Estimate of peak ground acceleration at the site, utilizing empirical relationships.

2. Development of site response spectrum, utilizing empirical ground motion ratios and spectral amplification factors.

3. Development of inventory of structural and nonstructural components and their corresponding replacement costs.

4. Calculation of structural characteristics for structural and nonstructural components by using a simple analysis model.

5. Estimation of damageability factors and repair costs for each structural and nonstructural component.

6. Comparison of predicted repair costs and actual repair costs.

7. Summarization of findings of the work, including conclusions and recommendations.

B.2 DESCRIPTION OF MORGAN HILL EARTHQUAKE

The Morgan Hill, California earthquake occurred at 13:15 PST on April 24, 1984. The earthquake was a moderate event with a mean local magnitude of 6.2 on the Richter scale. The epicenter of the main shock was located at 37.380 N latitude and 121.698 W longitude by the seismographic stations of the University of California at Berkeley. Although surface faulting was not found, the earthquake appears to have been caused by a rupture of the Calaveras fault over a length of 15 miles extending from Halls Valley to Anderson Lake (Figure B.2-1).

The ground motion records indicate that the rupture propagated southeast from the origin of the earthquake. Stations southeast of the epicenter recorded higher accelerations than stations to the northwest recorded. The strong directional dependence exhibited by the Morgan Hill earthquake is one of its most studied characteristics. The effect of this phenomenon was also evident in the geological distribution of earthquake damage.

Most of the damage from the Morgan Hill earthquake occurred in the town of Morgan Hill (population 17,000), located approximately 13 miles south of the epicenter. The total damage to residential buildings and industrial facilities was estimated to be $7.5 million. In contrast, the heavily populated city of San Jose, which is about 10 miles due west of the epicenter, suffered little damage. If the fault rupture had propagated in the opposite direction the effects of the earthquake on the population and the local economy would have been more dramatic. References B.1 through B.4 are sources for more detailed information on the earthquake and its effects.

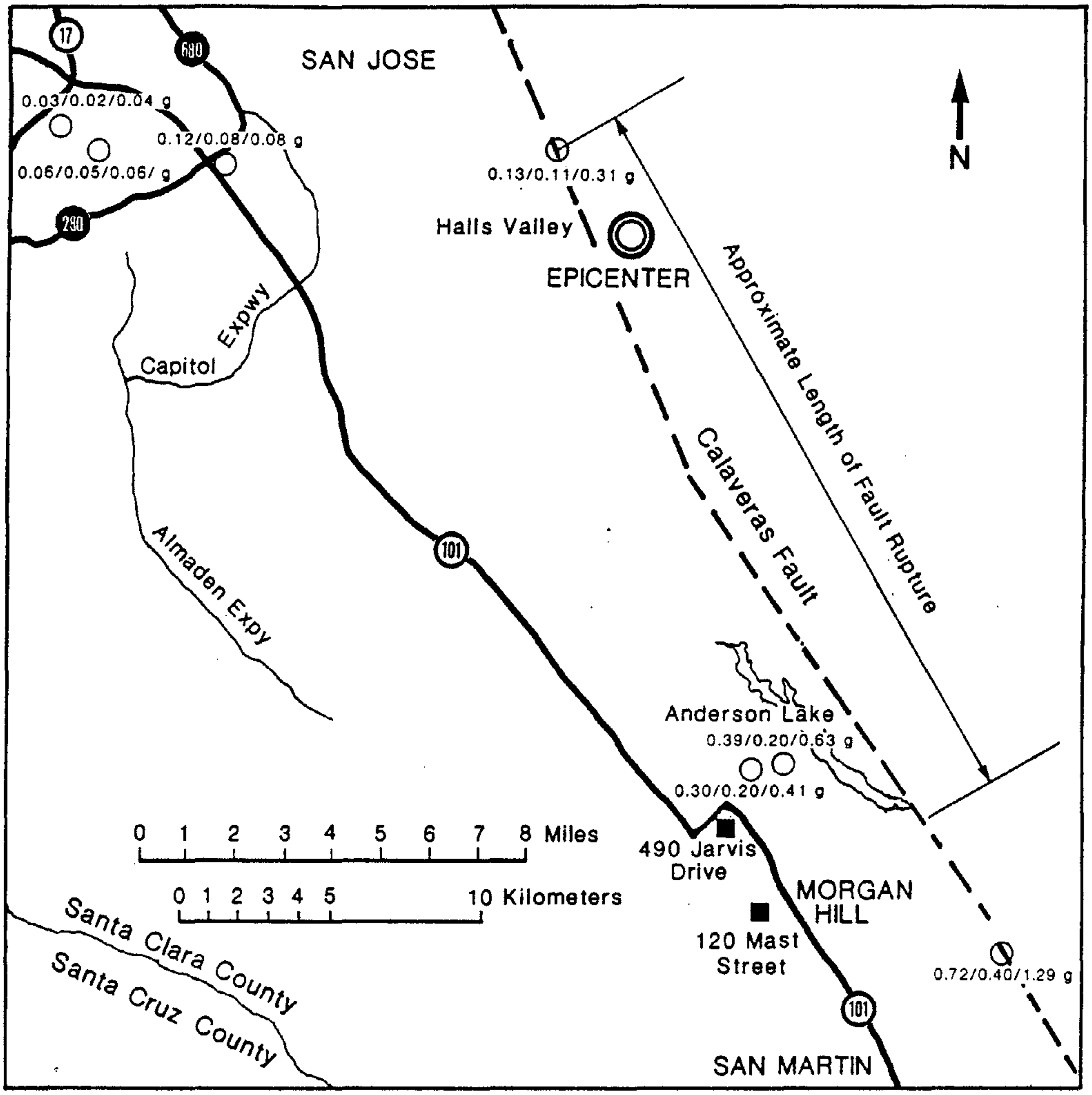


Figure B.2-1: Map shows relative locations of the two Wiltron facilities with respect to the epicenter and the ruptured fault of the Morgan Hill earthquake. Also indicated are the locations and peak acceleration values of the ground motion records from the April 24 event. Recorded peak ground accelerations are listed as: horizontal north-south / vertical / horizontal east-west.

B.3 WILTRON FACILITY AT 120 MAST STREET, MORGAN HILL

B.3.1 Description of Building

The one-story concrete tilt-up structure at 120 Mast Street was designed in 1979 in accordance with the 1976 Uniform Building Code. It is basically rectangular in shape with plan dimensions of approximately 250 feet by 99 feet. The building is 18 feet in height with a 2 foot high wall parapet. A plan view of the building is shown in Figure B.3-1. Photos B.3-1 and B.3-2 are elevations of the building.

The building has a plywood roof diaphragm over a framing system which consists of lumber beams and glulam girders supported by steel columns. The diaphragm and the perimeter tilt-up walls are connected and constitute the lateral load resisting elements (Photo B.3-3). Dowels are provided to transfer shear from the walls into the concrete slab on grade. Reinforcing bars at the tops of the walls act as chords for the roof diaphragm. Other than the welded chord connections, there are no shear connectors between the individual wall panels . Local chords are provided along lines 2 and D to account for the irregularity of the roof diaphragm at the southwest corner of the building. The subdiaphragm concept was apparently utilized to provide for the transfer of out-of-plane forces between the concrete walls and the roof; all wall panels are positively anchored to the beams and girders.

All interior partitions consist of metal studs with gypsum board on both sides. Most are located in the east half of the building. In the analysis these partitions were not considered to be lateral-load-resisting elements. The other half of the building is a large assembly area (Photo B.3-4). Seventy percent of the building area has a suspended ceiling. The only areas within the building without a suspended ceiling are the machine shop and the storage area in the southeast corner of the building. Glass window openings in the perimeter wall panels are relatively small.

B.3.2 Description of Damage

The building at 120 Mast Street was leased from Parkland Properties. Wiltron was scheduled to move to a new facility at 490 Jarvis Drive three days after the Morgan Hill earthquake. The 120 Mast Street structure had been vacated before EQE engineers visited the two buildings approximately one week after the earthquake. Most of the normal furnishings and equipment had been removed from the site. However, no repairs had been made to the building. The following is a summary of the observations made by the EQE engineers during their visit:

- No structural damage was found; there were no cracks in the tilt-up walls or damage to the roof diaphragm. These observations were confirmed by the owner after all necessary repairs were made.

- The plumbing, including the fire sprinkler piping system, and the suspended compressed air lines suffered no damage.

- Most of the damage to the building occurred to the partitions in the north-south direction (Photo B.3-5). At the north side of the building partitions separated from the exterior tilt-up walls. The separations ranged from 0.5 to 1.5 inches. At the south side of the building the partitions were crushed and separated from the wall approximately 0.5 inches. A buckled lightweight metal stud was observed at one location.

- Tiles fell from the suspended ceiling at the perimeter walls (Photo B.3-6).

- The roofing cracked at the southwest corner of the building.

- Distortion of window frames caused the neoprene in some of the steel mullions to pop out. However, no glass windows broke.

- There were two circular ducting failures. An offset occurred in one section of a HVAC duct (Photo B.3-7). A section of a HVAC duct tore and fell during the earthquake (Photo B.3-8).

- A few chain-suspended industrial light fixtures fell from their supports.

- Pieces of unanchored heavy machinery shifted approximately 1 inch due south. Displacements of some tables were approximately 2 inches due south.

- Solder sloshed out of an automatic solderer, splashing onto nearby walls, ceiling and floor.

- No items fell off shelves in the storage area.

The facility manager gave a preliminary repair cost estimate of $20,000. The actual total repair cost was approximately $6,500. This repair cost requires modification, as discussed in Section B.4.5.

B.3.3 Site Response Spectrum

Peak ground acceleration at the site was estimated using the relationship between earthquake magnitude, acceleration, and hypocentral distance developed by Donovan and Bornstein (Reference 53), as discussed in Chapter 6. The value of 0.13g obtained from this formula is not consistent with the ground motions recorded at stations near the site. The three stations that recorded peak horizontal ground acceleration within 5 miles of the facility recorded values above or equal to 0.30g (Figure B.2-1). This large discrepancy may be the result of the directionality of the earthquake and the location of the fault in relation to the site. As is the case for all formulas developed for estimating the average peak ground acceleration, the relationship recommended by Donovan and Bornstein should be applied with considerable caution and judgment, especially for sites very close to the fault where little data is available.

The Campbell formula (Reference B.5) afforded results which are more consistent with the ground motion records. This formula is an empirical relationship between peak horizontal acceleration, Richter magnitude, and distance from the fault rupture zone. The magnitude of the Morgan Hill earthquake is 6.2 and the distance between the fault and the site was estimated at 6 kilometers. The bedrock peak horizontal acceleration was calculated at 0.26g by substituting the above values into the following equation developed by Campbell:

$$PGA = 0.0159\exp(0.868M)[R+0.0606\exp(0.700M)]^{-1.09}$$

where:

PGA = bedrock peak horizontal acceleration

M = magnitude of earthquake, Richter scale

R = fault distance, in kilometers

exp = natural exponent

It is possible that maximum peak ground accelerations significantly different from 0.26g were experienced at the site.

The geological investigation performed before the construction of the building determined that the site is underlain with very stiff soil. Using the graphic correlations presented in Figure 3 (Chapter 6), a surface soil peak horizontal acceleration of 0.25g was obtained. To complete the estimation of the spectral ground motions values, the appropriate ground motion ratios and the spectral amplification factors were selected from Tables 3 and 4 (Chapter 6) respectively. These computations are shown in Figure B.3-2. The site response spectrum was then constructed and is shown in Figure B.3-3.

B.3.4 Predicted Earthquake Losses

Developing an inventory of structural and nonstructural components and their corresponding replacement costs was difficult. Much of the information was not readily available from the owner. Consequently, the total replacement value of the building was approximated by multiplying the total construction cost of the building at 490 Jarvis Drive by the ratio between the floor areas of the two buildings. The percentages of the component replacement value to the total replacement value used for the example building (Table 7, Chapter 6) were used to compute the individual component replacement value. These percentages are based on actual construction data. Sample calculations are illustrated in Figure B.3-4. The complete component inventory of the building is shown in Table B.3-1.

Structural characteristics were calculated by using the simple analysis model in the example problem (Chapter 6). The calculations are shown in Figure B.3-5 through Figure B.3-11. Simplifications were made in the analysis because detailed calculations were not justified. The roof diaphragm and the tilt-up wall panels were determined to be uncoupled because of the large difference in their natural periods. The individual responses of each component to the earthquake were estimated from the site response spectrum shown in Figure B.3-3. A summary of the structural responses is presented in Figure B.3-12.

Figures B.3-13 and B.3-14 show the calculations of estimated damage factors for structural and nonstructural components. Linear interpolation between the three damage states listed in Table 5 was assumed in the calculations. The summaries of the results are presented in Tables B.3-2 and B.3-3.

B.3.5 Comparison of Predicted Earthquake Losses to Actual Losses

A comparison between the approximate actual repair costs of the structural and nonstructural components and the predicted repair costs obtained from application of the methodology is shown in Table B.3-3. Table B.3-4 presents a comparison between the predicted total damage factors and the actual damage factors.

In these examples of the component approach earthquake losses are overestimated by a factor of approximately five. The following factors may have contributed to the discrepancy:

- The building is owned by a construction company which performed all of the necessary repairs.

- The facility was evacuated soon after the earthquake, which eliminated concerns about business interruption and occupant safety.

- The total repair cost value is based on the owner's memory. No verification based on actual construction records was performed.

When these factors are taken into consideration, the reported repair cost of $6,500 should be increased significantly. The facility manager's preliminary cost estimate of $20,000 seems reasonable. This estimate is about two-thirds the total repair cost predicted by the component approach.

Though the dollar values of the predicted and actual repair costs vary significantly, the actual repair cost was only 0.93% of the total structure replacement cost, while the predicted cost was 4.9%. Essentially minor damage was predicted, and minor damage occurred. Therefore, the application of the methodology in this case was a success.

It is significant that all items for which damaged was predicted were damaged (Table B.3-3). Finally, the uniformity of the damage factors listed in Table B.3-4 shows that the thresholds and damage factors developed are consistent, although, perhaps conservative.

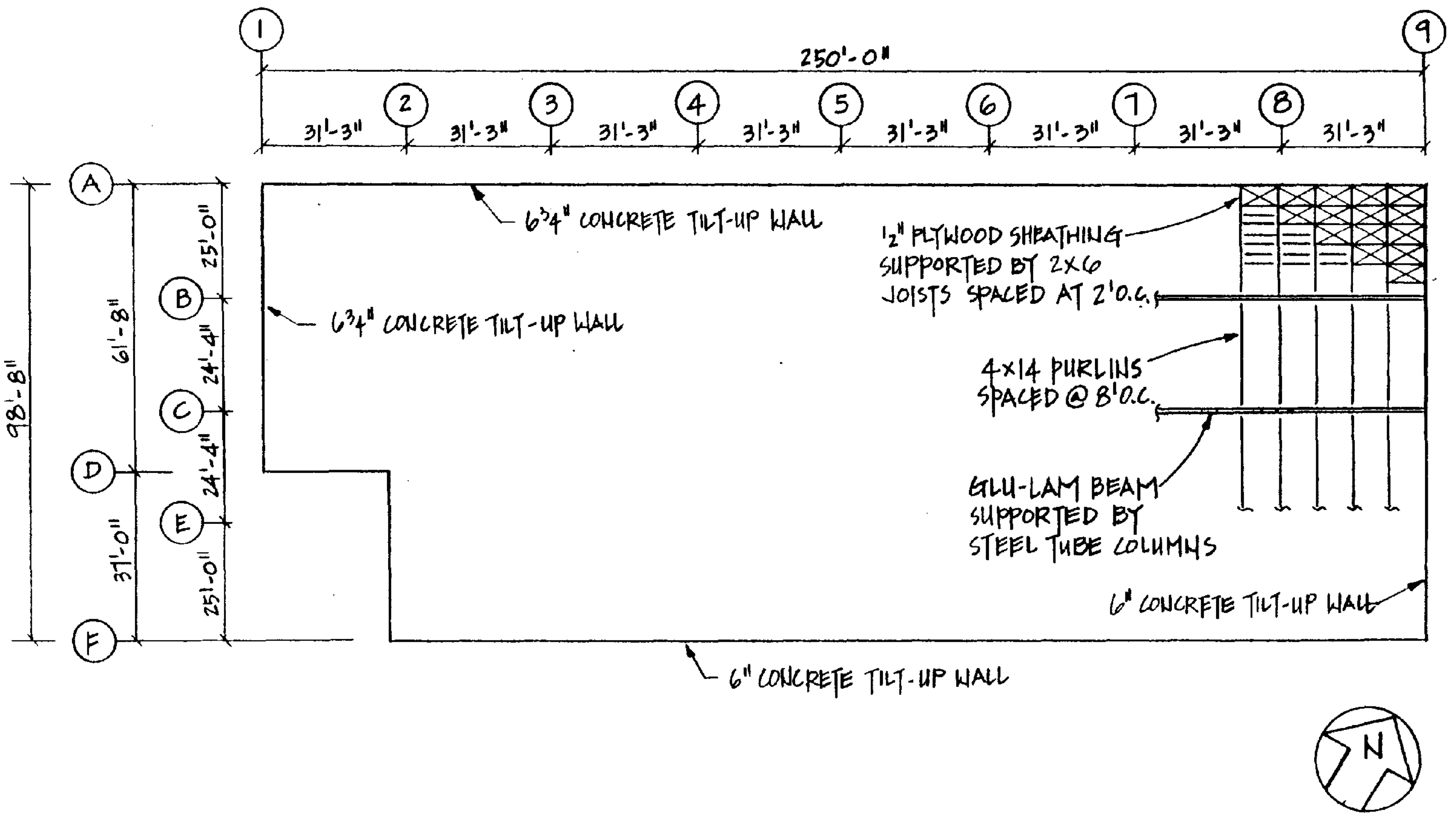


Figure B.3-1: Roof Plan of the 120 Mast Street Building

ESTIMATION OF BEDROCK PEAK HORIZONTAL ACCELERATION

USE CAMPBELL FORMULA (REFERENCE B-5)

$$\text{BEDROCK PGA} = 0.0159\ \text{EXP}(0.868\,M)\,[R + 0.0606\ \text{EXP}(0.700\,M)]^{-1.09}$$
$$= 0.0159\ \text{EXP}(0.868 \times 6.2)\,[6.0 + 0.0606\ \text{EXP}(0.700 \times 6.2)]^{-1.09}$$
$$= 0.262\ g$$

M = MAGNITUDE OF EARTHQUAKE = 6.2

R = FAULT DISTANCE, IN KILOMETERS = 6.0 KM

ESTIMATION OF SOIL PEAK HORIZONTAL ACCELERATION

SOIL CONDITION : STIFF SOILS

SOIL PGA = 0.25 g (FIGURE 3)

GROUND MOTION RATIOS

$$\left.\begin{array}{l} v/a\ (\text{IN/SEC})/g = 34.5 \\ ad/v^2 = 4.35 \end{array}\right\}\ (\text{TABLE 3})$$

MAX. GROUND ACCELERATION = 0.25 g

MAX. GROUND VELOCITY = 34.5 (IN/SEC)/g × 0.25 g = 8.625 IN/SEC

$$\text{MAX. GROUND DISPLACEMENT} = \frac{4.35 \times (8.625\ \text{IN/SEC})^2}{0.25\,g \times 386.4\,(\text{IN/SEC}^2)/g} = 3.350\ \text{IN}$$

SPECTRAL AMPLIFICATION (5 % DAMPING)

$$\left.\begin{array}{ll} \text{ACCELERATION} & 2.63 \\ \text{VELOCITY} & 1.27 \\ \text{DISPLACEMENT} & 2.53 \end{array}\right\}\ (\text{TABLE 4})$$

MAX. SPECTRAL ACCELERATION = 2.63 × 0.25 = 0.66 g

MAX. SPECTRAL VELOCITY = 1.27 × 8.625 = 11 IN/SEC

MAX. SPECTRAL DISPLACEMENT = 2.53 × 3.350 = 8.5 IN

Figure B.3-2: Ground Motion Prediction for the 120 Mast Street Building

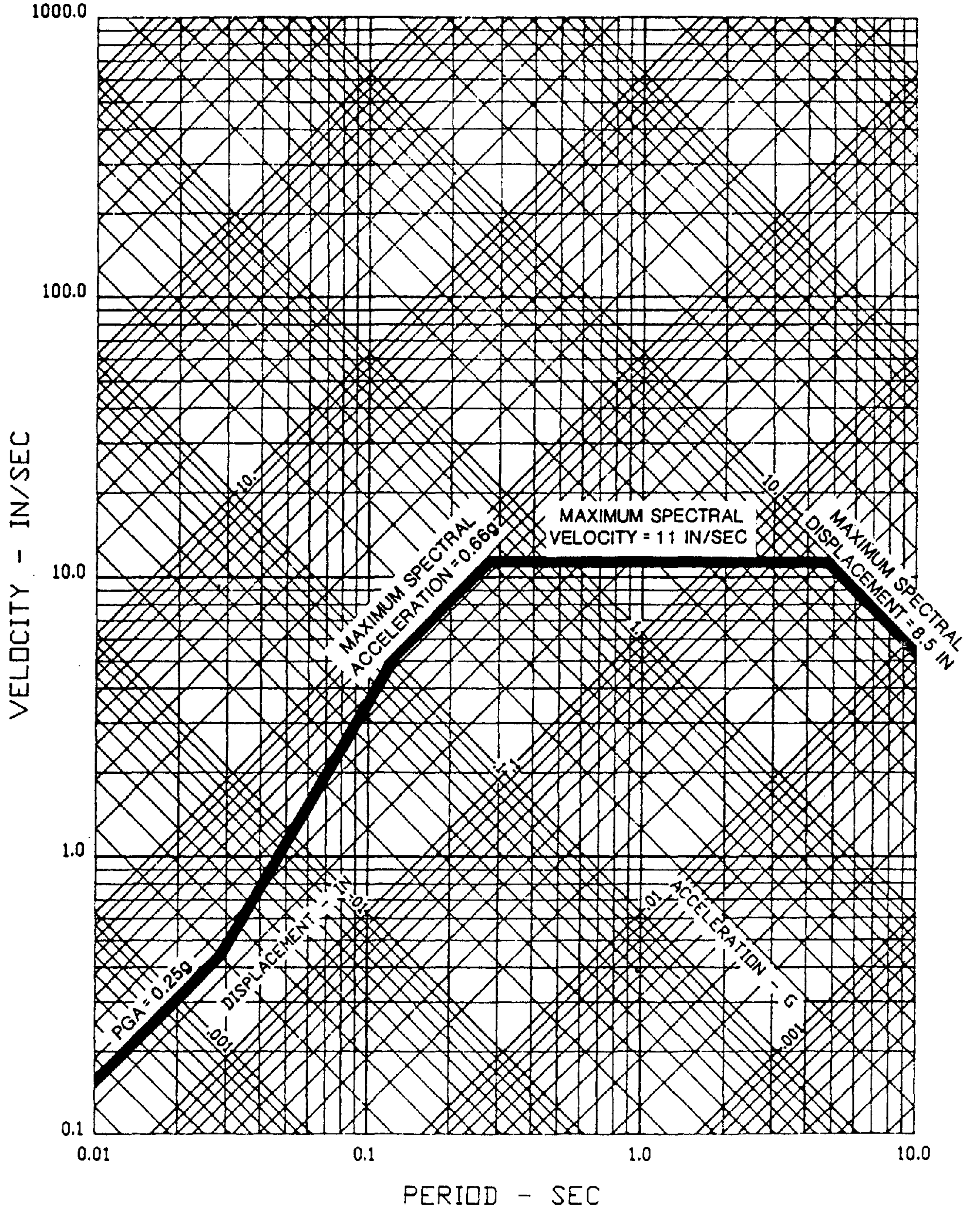


Figure B.3-3: 5% Damping Response Spectrum for the 120 Mast Street Building

ESTIMATION OF BLDG REPLACEMENT VALUE

TOTAL AREA OF BLDG AT 490 JARVIS DRIVE = 75,030 SQ.FT.
(INCLUDES BOTH FLOORS)

REPLACEMENT VALUE = $ 3,408,000 (PROVIDED BY OWNER)

TOTAL AREA OF BLDG AT 120 MAST STREET = 23,629 SQ.FT.

$$\text{REPLACEMENT VALUE} = \$3{,}408{,}000 \times \frac{23{,}629 \text{ SQ.FT.}}{75{,}030 \text{ SQ.FT.}}$$

$$= \$1{,}073{,}270$$

ESTIMATION OF COMPONENT REPLACEMENT VALUE

ROOF

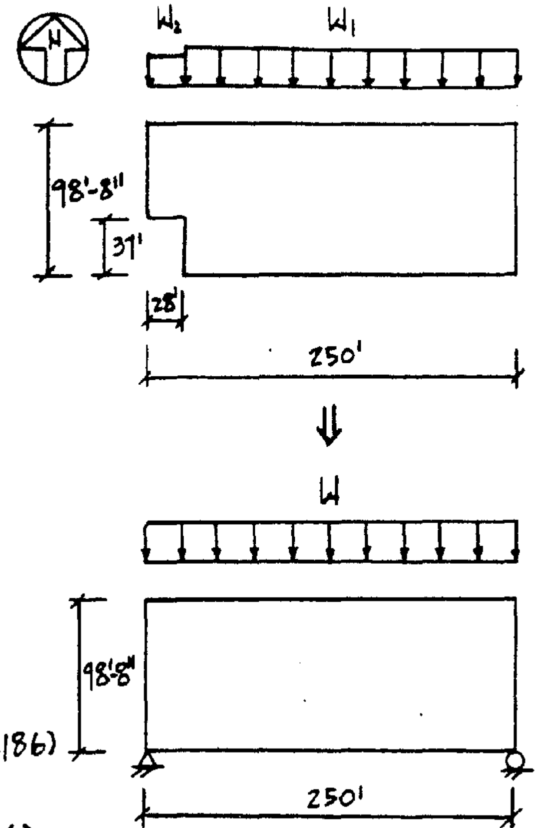
PERCENT OF TOTAL = 17.54 (TABLE 7)

TOTAL REPLACEMENT VALUE = $ 1,073,270

ROOF REPLACEMENT VALUE = $ 1,073,270 × 17.54 %
= $ 188,252

Figure B.3-4: Sample Calculations for Estimating Replacement Values of the 120 Mast Street Building

EQE



ROOF DEAD LOAD = 14 PSF

$\frac{1}{2}$" STRUCTURAL I C-D PLYWOOD DIAPHRAGM
8d NAILS @ 2", 3", 12" O.C.
EFFECTIVE THICKNESS $t = 0.543"$
$G = 90.000$ psi

6" CONCRETE TILT-UP WALLS
UNIT WEIGHT = 150 PCF × (6"/12) = 75 PSF
$f'_c = 3000$ psi
$E = 57000\sqrt{3000} = 3.12 \times 10^6$ psi

$6\frac{3}{4}$" CONCRETE TILT-UP WALLS
UNIT WEIGHT = 150 PCF × (6.75"/12) = 84.4 PSF
$f'_c = 3000$ psi
$E = 3.12 \times 10^6$ psi

LOADS

W_1 = (14 PSF × 98.66' + 75 PSF × 11' + 84.4 PSF × 11')(0.186)
= 584 PLF

W_2 = (14 PSF × 61.66' + 2 × 84.4 PSF × 11')(0.186)
= 506 PLF

USE L = 250' ; b = 98.66' (IGNORE NOTCH IN ROOF)

EQUIVALENT UNIFORM LOAD W = (584 PLF × 222' + 506 PLF × 28')/250' = 575 PLF

$$V = \frac{WL}{2b} = \frac{575 \text{ PLF} \times 250'}{2 \times 98.66'} = 729 \text{ PLF}$$

DIAPHRAGM STATIC DEFLECTION

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.094\,L\,e_n + \underbrace{\frac{\Sigma A_c X}{2b}}_{\text{ASSUME} = 0} \quad \text{(REFERENCE 14)}$$

$$= \frac{5 \times 729 \text{PLF} \times (250')^3}{8 \times (3.12 \times 10^6 \text{psi}) \underbrace{[(8 \times 6")\times 6"]}_{\text{AREA OF CHORD}} (98.66')} + \frac{729 \text{PLF} \times 250'}{4(90000 \text{psi})(0.543")} + 0.094(250')(0.056)$$

AREA OF CHORD = TRIBUTARY WIDTH × WALL THICKNESS
= (8 × WALL THICKNESS) × WALL THICKNESS

$= 0.08" + 0.93" + 1.32"$
$= 2.33"$

Figure B.3-5.1: Roof Diaphragm Period, North-South Direction Earthquake, 120 Mast Street Building

EQUIVALENT BEAM DEFLECTION

$$\Delta = \frac{5WL^4}{384EI}$$

$$\Rightarrow EI = \frac{5WL^4}{384\Delta} = \frac{5(575\,PLF)(250')^4 \times 12^3}{384 \times 2.33''} = 2.169 \times 10^{13}\ \#\text{-}IN^2$$

DIAPHRAGM PERIOD $T_{D(NS)}$

$$\omega = \frac{\pi^2}{L^2}\sqrt{\frac{EI}{m}}$$

$$m = \frac{(575\,PLF)(1/0.186)(1'/12'')}{(32.2\,FT/SEC^2)(12''/1')}$$

$$= 0.666\ \#\text{-}SEC^2/IN^2$$

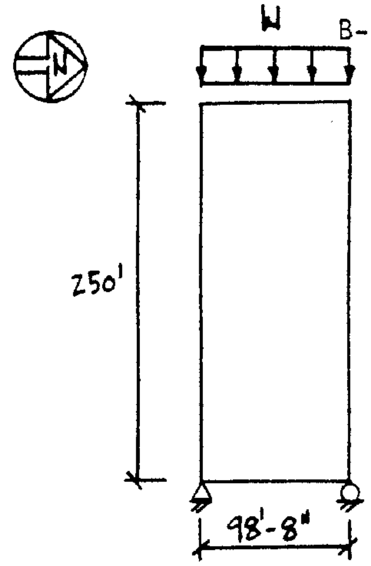
$$\omega = \frac{\pi^2}{(250' \times 12)^2}\sqrt{\frac{2.169 \times 10^{13}\ \#\text{-}IN^2}{0.666\ \#\text{-}SEC^2/IN^2}}$$

$$= 6.25\ RAD/SEC$$

$$T_{D(NS)} = \frac{2\pi}{\omega} = \frac{2\pi\ RAD}{6.25\ RAD/SEC} = 1.00\ SEC$$

THE ROOF DIAPHRAGM PERIOD FOR AN EARTHQUAKE IN THE NORTH-SOUTH DIRECTION IS 1.00 SECONDS

Figure B.3-5.2: Roof Diaphragm Period, North-South Direction Earthquake, 120 Mast Street Building



ROOF DEAD LOAD = 14 PSF

1/2" STRUCTURAL I C-D PLYWOOD DIAPHRAGM
8d@2", 3", 12" O.C.
EFFECTIVE THICKNESS $t = 0.543"$
$G = 90{,}000$ PSI

6" CONCRETE TILT-UP WALLS
UNIT WEIGHT = 75 PSF
$f'_c = 3000$ PSI
$E = 3.12 \times 10^6$ PSI

6 3/4" CONCRETE TILT-UP WALLS
UNIT WEIGHT = 84.4 PSF
$f'_c = 3000$ PSI
$E = 3.12 \times 10^6$ PSI

LOADS

TOTAL ROOF AREA = 23629 □'
EAST WALL WEIGHT = (75 PSF × 73.16' + 84.4 PSF × 25.5') × 11' = 84031 #
WEST WALL WEIGHT = (84.4 PSF × 61.66' + 75 PSF × 37') × 11' = 87770 #
TOTAL ROOF WEIGHT = 14 PSF × 23629 □' + 84031 # + 87770 #
= 502607 #

$$\text{EQUIVALENT UNIFORM LOAD } W = \frac{502607 \times 0.186}{98.66} = 949 \text{ PLF}$$

$$V = \frac{WL}{2b} = \frac{949 \text{ PLF} \times 98.66'}{2 \times 250'} = 187 \text{ PLF}$$

DIAPHRAGM STATIC DEFLECTION

$$\Delta = \frac{5VL^3}{8EAb} + \frac{VL}{4Gt} + 0.094\,L\,e_n + \frac{\Sigma \Delta_c X}{2b}$$

$$= \frac{5(187\text{ PLF})(98.66')^3}{8 \times (3.12 \times 10^6 \text{ PSI})(288\,\square")(250')} + \frac{(187 \text{ PLF})(98.66')}{4 \times 90000 \text{ PSI} \times 0.543"} + 0.094 \times 98.66 \times 0.008$$

$$= .0005" + 0.094" + 0.074" = 0.169"$$

EQUIVALENT BEAM DEFLECTION

$$\Delta = \frac{5WL^4}{384EI} \Rightarrow EI = \frac{5(949\text{ PLF})(98.66')^4 \times 12^3}{384 \times 0.169} = 1.197 \times 10^{13} \text{ \#-IN}^2$$

DIAPHRAGM PERIOD T_D

$$m = \frac{(949 \text{ PLF})(1/0.186)(1'/12")}{(32.2 \text{ FT/SEC}^2)(12"/1')} = 1.100 \text{ \#-SEC}^2/\text{IN}^2$$

$$\omega = \frac{\pi^2}{(98.66 \times 12)^2}\sqrt{\frac{1.197 \times 10^{13} \text{ \#-IN}^2}{1.100 \text{ \#-SEC}^2/\text{IN}^2}} = 23.2 \text{ RAD/SEC}$$

$$T_D = \frac{2\pi}{\omega} = \frac{2\pi \text{ RAD}}{23.2 \text{ RAD/SEC}} = 0.27 \text{ SEC}$$

THE ROOF DIAPHRAGM PERIOD FOR AN EARTHQUAKE IN THE EAST-WEST DIRECTION IS 0.27 SECONDS

Figure B.3-6: Roof Diaphragm Period, East-West Direction Earthquake, 120 Mast Street Building

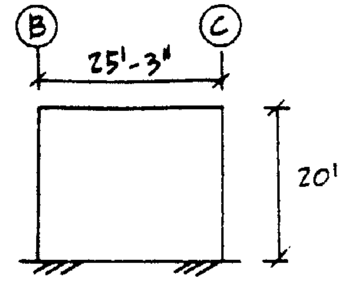
EQE

PANELS HAVE NO SHEAR CONNECTORS
THEREFORE, CALCULATE STIFFNESS INDIVIDUALLY

$$\Delta = \underbrace{\frac{FH^3}{3EI}}_{\text{BENDING}} + \underbrace{\frac{1.2FH}{AG}}_{\text{SHEAR}}$$ (ASSUME CANTILEVER DEFLECTION MODE)

$$= F\left[\frac{H^3}{3\times(3.12\times10^6\,\text{psi})(tB^3/12)} + \frac{1.2\times H}{(tB)(1.33\times10^6\,\text{psi})}\right]$$

$$\frac{1}{K} = \frac{\Delta}{F} = \frac{10^{-6}}{t}\left[1.282\left(\frac{H}{B}\right)^3 + 0.9022\left(\frac{H}{B}\right)\right]$$



$t = 6"$
$E = 3.12 \times 10^6$ psi
$G = 1.33 \times 10^6$ psi

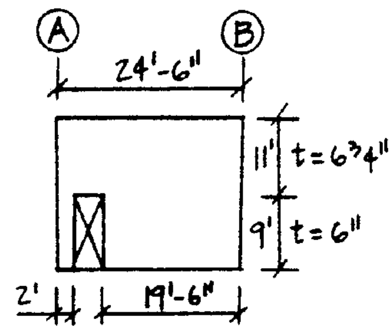
PANEL 9

FOR $t = 6"$, $H = 20'$, $B = 25.25'$, $H/B = 0.7921$

$$\frac{1}{K} = \frac{10^{-6}}{6}\left[1.282(0.7921)^3 + 0.9022(0.7921)\right] = 0.2253\times10^{-6}\ \text{IN}/\#$$

$K = 4.44 \times 10^6$ #/IN

PANEL 8



AVERAGE THICKNESS $t = (6.75"\times11' + 6"\times9')/20'$
$= 6.4"$

IGNORE OPENING
$t = 6.4"$, $H = 20'$, $B = 24.5'$, $H/B = 0.8163$

$$\frac{1}{K} = \frac{10^{-6}}{6.4}\left[1.282(0.8163)^3 + 0.9022(0.8163)\right]$$

$= 0.2240 \times 10^{-6}$ IN/#
$K = 4.46 \times 10^6$ #/IN

ESTIMATION OF EFFECTIVE STIFFNESS

AREA OF OPENING $= 9' \times 3' = 27$ $\square'$
TOTAL AREA $= 20' \times 24.5' = 490$ $\square'$
AREA OF OPENING / TOTAL AREA $= \alpha_1 = 27/490 = 0.05$
$\beta_1 = 1 - \alpha_1 = 0.95$

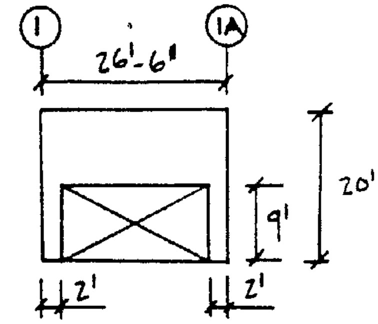
WIDTH OF OPENING $= 3'$
TOTAL WIDTH $= 24.5'$
WIDTH OF OPENING / TOTAL WIDTH $= \alpha_2 = 3/24.5 = 0.12$
$\beta_2 = 1 - \alpha_2 = 0.88$

ASSUME $K_{EFF} = \frac{1}{2}(\beta_1 + \beta_2)K$
$= \frac{1}{2}(0.95 + 0.88)(4.4634 \times 10^6$ #/IN$)$
$= 0.915 \times 4.4634 \times 10^6$ #/IN
$= 4.08 \times 10^6$ #/IN

NOTE: OTHER APPROXIMATE METHODS FOR DETERMINING PANEL STIFFNESS MAY BE USED

Figure B.3-7.1: Sample Calculations for Wall Panel Stiffness, 120 Mast Street Building

PANEL 1



$E = 3.12 \times 10^6$ PSI
$G = 1.33 \times 10^6$ PSI

DUE TO THE LARGE OPENING, CONSIDER COLUMN STIFFNESS ONLY

$$K = 2 \times \underbrace{\frac{12EI}{H^3}}_{\text{STIFFNESS OF FIXED-FIXED COLUMN}}$$

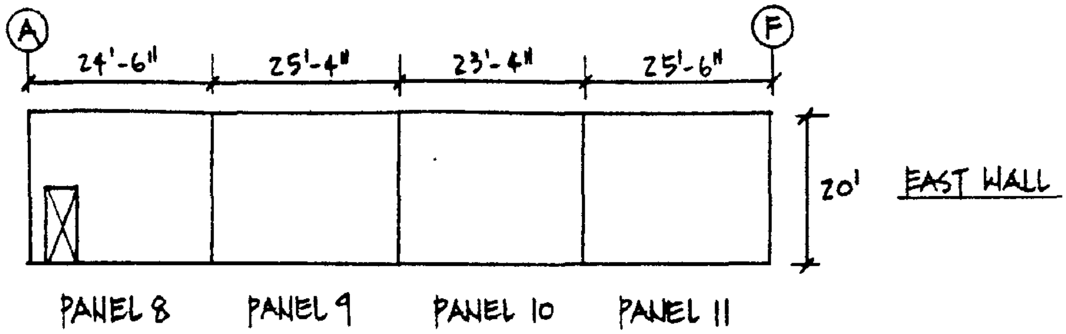
$$= 2 \times \frac{12 \times (3.12 \times 10^6 \text{ PSI})[(6'')(2')^3/12]}{(9')^3}$$

$$= 2 \times (0.2054 \times 10^6 \text{ \#/IN})$$

$$= 0.411 \times 10^6 \text{ \#/IN}$$

NOTE: STIFFNESS CALCULATIONS FOR REMAINING PANELS NOT SHOWN.

Figure B.3-7.2: Sample Calculations for Wall Panel Stiffness, 120 Mast Street Building



TOTAL WALL STIFFNESS

$$K = K_8 + K_9 + K_{10} + K_{11}$$
$$= 10^6 (4.08 + 4.44 + 3.77 + 4.53)$$
$$= 16.82 \times 10^6 \text{ \#/IN}$$

SHEAR WALL PERIOD

$$m = \frac{W_{ROOF} + W_{WALL}}{g} = \frac{1}{g}\left[\frac{wL}{2 \times 0.186} + W_{WALL}\right]$$
$$= \frac{1}{(32.2 \text{ FT/SEC}^2) \times (12"/1')}\left[\frac{575 \text{ PLF} \times 250'}{2 \times 0.186} + 84031\text{ \#}\right]$$
$$= 1216 \text{ \#-SEC}^2/\text{IN}$$

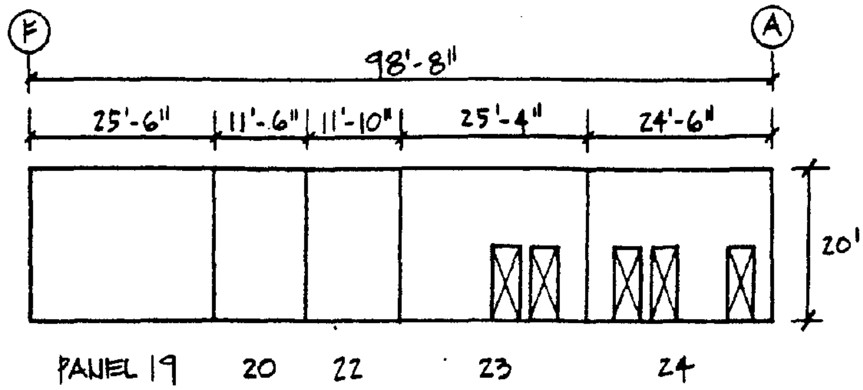
$$\omega = \sqrt{\frac{K}{m}} = \sqrt{\frac{16.82 \times 10^6 \text{ \#/IN}}{1216 \text{ \#-SEC}^2/\text{IN}}} = 118 \text{ RAD/SEC}$$

$$T_W = \frac{2\pi}{\omega} = \frac{2\pi \text{ RAD}}{118 \text{ RAD/SEC}} = 0.053 \text{ SEC}$$

$$3T_W = 3 \times 0.053 = 0.159 \text{ SEC} \ll T_{D(NS)} = 1.00 \text{ SEC}$$

THEREFORE DIAPHRAGM AND EAST WALL CAN BE CONSIDERED AS UNCOUPLED

Figure B.3-8: East Wall Period, North-South Direction Earthquake, 120 Mast Street Building



WEST WALL

TOTAL WALL STIFFNESS

$$K = K_{19} + K_{20} + K_{22} + K_{23} + K_{24}$$
$$= 10^6(4.52 + 0.81 + 0.86 + 3.79 + 3.19)$$
$$= 13.17 \times 10^6 \text{ \#/IN}$$

SHEAR WALL PERIOD

$$m = \frac{W_{ROOF} + W_{WALL}}{g}$$
$$= \frac{1}{g}\left[\frac{WL}{2 \times 0.186} + W_{WALL}\right]$$
$$= \frac{1}{32.2 \text{ FT/SEC}^2) \times (12"/1')}\left[\frac{575 \text{ PLF} \times 250'}{2 \times 0.186} + 87770\text{\#}\right]$$
$$= 1227 \text{ \#-SEC}^2/\text{IN}$$

$$\omega = \sqrt{\frac{K}{m}} = \sqrt{\frac{13.17 \times 10^6 \text{ \#/IN}}{1227 \text{ \#-SEC}^2/\text{IN}}} = 104 \text{ RAD/SEC}$$

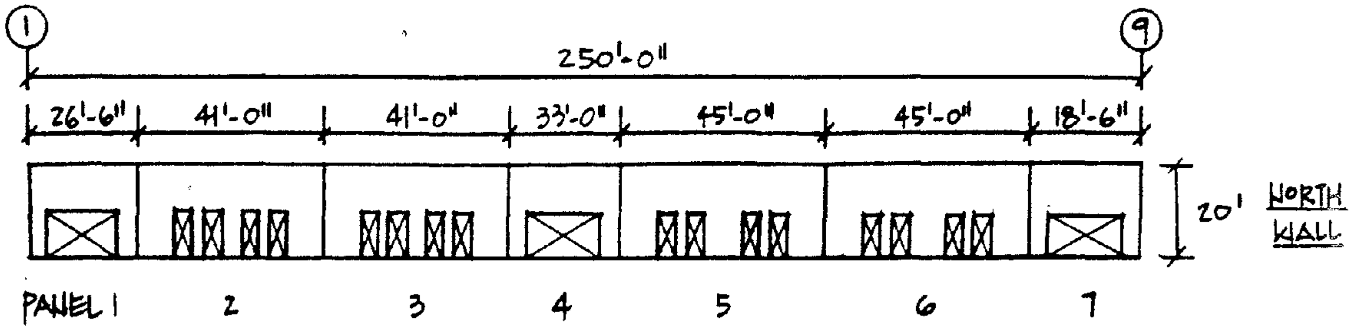
$$T_W = \frac{2\pi}{\omega} = \frac{2\pi \text{ RAD}}{104 \text{ RAD/SEC}} = 0.060 \text{ SEC}$$

$$3T_W = 3 \times 0.060 = 0.180 \text{ SEC} < T_{D(NS)} = 1.00 \text{ SEC}$$

THEREFORE DIAPHRAGM AND WEST WALL CAN BE CONSIDERED AS UNCOUPLED

NOTE: IT IS ALSO POSSIBLE TO COMPARE THREE TIMES THE PERIOD OF THE TWO SHEAR WALLS TOGETHER TO THAT OF THE DIAPHRAGM

Figure B.3-9: West Wall Period, North-South Direction Earthquake, 120 Mast Street Building



TOTAL WALL STIFFNESS

$$K = K_1 + K_2 + K_3 + K_4 + K_5 + K_6 + K_7$$
$$= 10^6(0.41 + 8.20 + 8.20 + 0.41 + 9.66 + 9.66 + 0.41)$$
$$= 36.95 \ \#/\text{IN}$$

SHEAR WALL PERIOD

$$m = \frac{W_{ROOF} + W_{WALL}}{g}$$
$$= \frac{1}{g}\left[\frac{1}{2}(\text{TOTAL ROOF WEIGHT INCLUDING WALLS}) + W_{WALL}\right]$$
$$= \frac{1}{(32.2 \text{ FT/SEC}^2)(12''/1')}\left[\frac{1}{2}(502607\#) + 232100\right]$$

↳ WALL UNIT WT. × LENGTH × TRIBUTARY HT.

$$= 1251 \ \#\text{-SEC}^2/\text{IN}$$

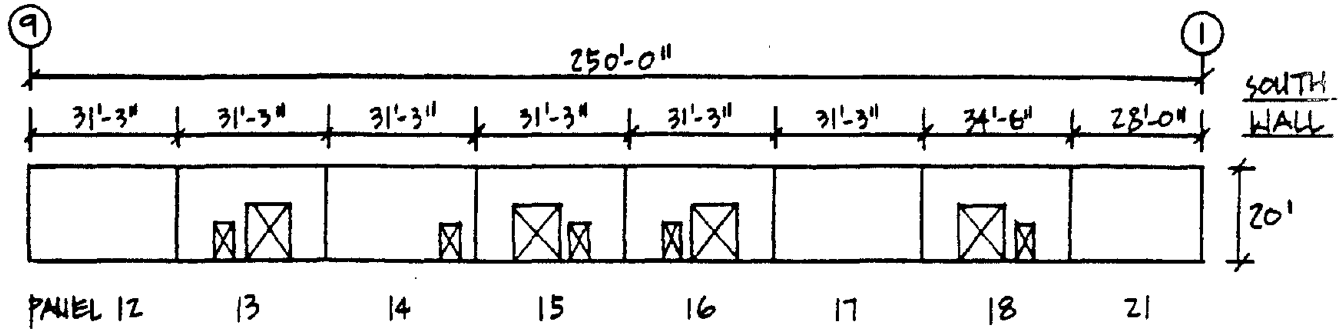
$$\omega = \sqrt{\frac{K}{m}} = \sqrt{\frac{36.96 \times 10^6 \ \#/\text{IN}}{1251 \ \#\text{-SEC}^2/\text{IN}}} = 172 \text{ RAD/SEC}$$

$$T_W = \frac{2\pi}{\omega} = \frac{2\pi \text{ RAD}}{172 \text{ RAD/SEC}} = 0.037 \text{ SEC}$$

$$3T_W = 3 \times 0.037 = 0.111 \text{ SEC} < T_{D(EW)} = 0.27 \text{ SEC}$$

THEREFORE DIAPHRAGM AND NORTH WALL CAN BE CONSIDERED AS UNCOUPLED

Figure B.3-10: North Wall Period, East-West Direction Earthquake, 120 Mast Street Building



TOTAL WALL STIFFNESS

$$K = K_{12} + K_{13} + K_{14} + K_{15} + K_{16} + K_{17} + K_{18} + K_{21}$$
$$= 10^6(6.57 + 4.04 + 6.11 + 4.04 + 4.04 + 6.57 + 5.05 + 5.76)$$
$$= 42.18 \times 10^6 \text{ \#/IN}$$

SHEAR WALL PERIOD

$$m = \frac{W_{ROOF} + W_{WALL}}{g}$$
$$= \frac{1}{g}\left[\frac{1}{2}(\text{TOTAL ROOF WEIGHT}) + W_{WALL}\right]$$
$$= \frac{1}{32.2 \text{ FT/SEC}^2)(12''/1)}\left[\frac{1}{2}(502607) + 209145\right]$$
$$= 1192 \text{ \#-SEC}^2/\text{IN}$$

$$\omega = \sqrt{\frac{K}{m}} = \sqrt{\frac{42.18 \times 10^6 \text{ \#/IN}}{1192 \text{ \#-SEC}^2/\text{IN}}} = 188 \text{ RAD/SEC}$$

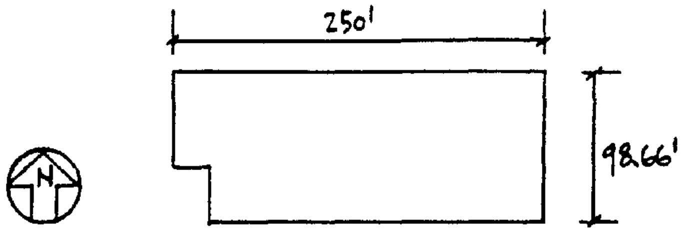
$$T_W = \frac{2\pi}{\omega} = \frac{2\pi \text{ RAD}}{188 \text{ RAD/SEC}} = 0.033 \text{ SEC}$$

$$3T_W = 3 \times 0.033 = 0.099 \text{ SEC} < T_{D(EW)} = 0.27 \text{ SEC}$$

THEREFORE DIAPHRAGM AND SOUTH WALL CAN BE CONSIDERED AS <u>UNCOUPLED</u>

∴ DIAPHRAGMS AND WALLS IN BOTH DIRECTIONS CAN BE CONSIDERED AS UNCOUPLED

Figure B.3-11: South Wall Period, East-West Direction Earthquake, 120 Mast Street Building



NORTH-SOUTH DIRECTION EARTHQUAKE

STRUCTURAL COMPONENT	PERIOD (SEC)	S_a (g)	S_v (IN/SEC)	S_d (IN)
ROOF	1.00	0.18	11.0	1.8
EAST WALL	0.053	0.38	1.24	0.011
WEST WALL	0.060	0.41	1.54	0.015

EAST-WEST DIRECTION EARTHQUAKE

STRUCTURAL COMPONENT	PERIOD (SEC)	S_a (g)	S_v (IN/SEC)	S_d (IN)
ROOF	0.27	0.66	10.9	0.48
NORTH WALL	0.037	0.31	0.77	0.0047
SOUTH WALL	0.033	0.28	0.58	0.0030

SPECTRAL VALUES WERE OBTAINED USING THE CALCULATED PERIODS AND THE DERIVED RESPONSE SPECTRUM ASSUMING THE WALLS AND ROOF ARE UNCOUPLED

Figure B.3-12: Summary of Structure Response for the 120 Mast Street Building

NORTH-SOUTH DIRECTION EARTHQUAKE

ROOF

% OF TOTAL = 50

$$\theta_d = \frac{S_d(\text{ROOF})}{L/2} = \frac{1.8''}{\frac{1}{2} \times 250' \times (12''/1')} = 0.00120 \quad (\text{FIGURE 10})$$

DS1 : THRESHOLD = 0.00097 ; DF = 0.05 (TABLE 5)
DS2 : THRESHOLD = 0.00400 ; DF = 0.50

$$\frac{DF - 0.05}{0.00120 - 0.00097} = \frac{0.50 - 0.05}{0.00400 - 0.00097} \left(\begin{array}{c}\text{ASSUME LINEAR}\\ \text{INTERPOLATION}\end{array}\right) \qquad \underline{DF = 0.084}$$

EAST WALL

LENGTH OF WALL = 98.66'
TOTAL LENGTH OF WALLS = 2(250' + 98.66') = 697.32'

$$\% \text{ OF TOTAL} = \frac{98.66}{697.32} = 14$$

$$\theta_d = \frac{S_d(\text{WALL})}{h} = \frac{0.011}{20' \times (12''/1')} = 0.000046 \quad (\text{FIGURE 8})$$

DS1 : THRESHOLD = 0.00050 ; DF = 0.05
$> \theta_d = 0.000046$ $\qquad$ $\underline{DF = 0}$

WEST WALL

% OF TOTAL = 14

$$\theta_d = \frac{0.015}{20' \times (12''/1')} = 0.000063 < \text{THRESHOLD} = 0.00050 \qquad \underline{DF = 0}$$

Figure B.3-13.1: Calculations of Damage Factors, North-South Direction Earthquake, 120 Mast Street Building

PARTITIONS

% OF TOTAL = 44

$$\theta_d = \frac{\frac{1}{2} \times (S_d)_{ROOF}}{h} = \frac{\frac{1}{2} \times 1.8''}{18' \times (12''/1')} = 0.00417 \text{ (FIGURE 12)}$$

ASSUME HALF OF MAX. DIAPHRAGM DEFLECTION IS AVE. PARTITION DEFLECTION

DS1 : THRESHOLD = 0.00283 ; DF = 0.06

DS2 : THRESHOLD = 0.00617 ; DF = 1.25

(TABLE 5)

$$\frac{DF - 0.06}{0.00417 - 0.00283} = \frac{1.25 - 0.06}{0.00617 - 0.00283}$$

DF = 0.54

SUSPENDED CEILING

% OF TOTAL = 50

$S_a = 0.18g$

DS1 : THRESHOLD = 0.20g > S_a = 0.18g

DF = 0

WINDOWS

EAST WALL

% OF TOTAL = 2

$$\theta_d = \frac{0.011''}{9' \times (12''/1')} = 0.000102$$

DS1 : THRESHOLD = 0.01405 > θ_d = 0.000102

DF = 0

WEST WALL

% OF TOTAL = 12

$$\theta_d = \frac{0.015}{9' \times (12''/1')}$$

= 0.000139 < THRESHOLD = 0.01405

DF = 0

Figure B.3-13.2: Calculations of Damage Factors, North-South Direction Earthquake, 120 Mast Street Building

EQE

EAST-WEST DIRECTION EARTHQUAKE

ROOF

% OF TOTAL = 50

$$\theta_d = \frac{0.48''}{\tfrac{1}{2} \times 98.66' \times (12''/')} = 0.00081$$ DF = 0

NORTH WALL

% OF TOTAL = 36

$$\theta_d = \frac{0.0047''}{20' \times (12''/')} = 0.000020$$ DF = 0

SOUTH WALL

% OF TOTAL = 36

$$\theta_d = \frac{0.0030}{20' \times (12''/')} = 0.000013$$ DF = 0

PARTITIONS

% OF TOTAL = 56

$$\theta_d = \frac{0.5 \times 0.48''}{18' \times (12''/1')} = 0.00111$$ DF = 0

SUSPENDED CEILING

% OF TOTAL = 50

$S_a = 0.66\,g$

DS3 = THRESHOLD = 0.5 g ; DF = 1.0 DF = 1.0

WINDOWS

NORTH WALL

% OF TOTAL = 86

$$\theta_d = \frac{0.0047}{9' \times (12''/')} = 0.000044$$ DF = 0

SOUTH WALL

% OF TOTAL = 0

$$\theta_d = \frac{0.0030}{9' \times (12''/')} = 0.000028$$ DF = 0

Figure B.3-14: Calculations of Damage Factors, East-West Direction Earthquake, 120 Mast Street Building

TABLE B.3-1

BUILDING COMPONENT INVENTORY FOR THE 120 MAST STREET BUILDING

Group	Component Class	Replacement Value ($)	Percent of Total Value
Substructure	Structural Excavation	12,235	1.14
	Footings and Slab	143,174	13.34
		155,409	14.48
Superstructure	Tilt-up Panels	184,280	17.17
	Columns	28,871	2.69
	Roof Structure and Roofing	188,252	17.54
		401,403	37.40
Architectural	Overhead Doors	12,235	1.14
	Windows	20,929	1.95
	Suspended Ceilings	35,418	3.30
	Interior Partitions	38,208	3.56
	Floor Finish	33,808	3.15
		140,598	13.10
Mechanical and Electrical	Roof Drain	4,293	0.40
	Sprinklers and Piping	36,062	3.36
	General and HVAC	114,733	10.69
	Plumbing	10,947	1.02
	Lights and Wiring	90,155	8.40
		256,190	23.87
Other	Grading and Paving	90,906	8.47
	Storm Drainage	7,942	0.74
	Site Work Concrete	20,822	1.94
		119,670	11.15
	TOTAL:	$1,073,270	100%

TABLE B.3-2

ESTIMATED BUILDING COMPONENT DAMAGE FACTORS FOR THE 120 MAST STREET BUILDING

Group	Component Class	Replacement Value ($)	Class	Demand	Damage Factor
Substructure	Structural Excavation	12,235	None	- -	0.00
	Footings and Slab	143,174	None	- -	0.00
Superstructure	Tilt-up Panels	184,280	B-10	14% @ 0.000046 14% @ 0.000063 36% @ 0.000020 36% @ 0.000013	0.00 0.00 0.00 0.00
	Columns	28,871	B-02	- -	0.00
	Roof Structures and Roofing	188,252	B-11	50% @ 0.00120 50% @ 0.00081	0.08 0.00
Architectural	Overhead Doors	12,235	None	- -	0.00
	Windows	20,929	B-14	2% @ 0.000102 12% @ 0.000139 86% @ 0.000044	0.00 0.00 0.00
	Suspended Ceilings	35,418	B-16	50% @ 0.18g 50% @ 0.66g	0.00 1.00
	Interior Partitions	38,208	B-13	44% @ 0.00417 56% @ 0.00111	0.54 0.00
	Floor Finish	33,808	None	- -	0.00

TABLE B.3-3

ESTIMATED BUILDING COMPONENT REPAIR COSTS AND ACTUAL REPAIR COSTS FOR THE 120 MAST STREET BUILDING

Group	Component Class	Replacement Value ($)	Estimated Repair Costs ($)	Actual[1] Repair Costs ($)
Substructure	Structural Excavation	12,235	0	0
	Footings and Slab	143,174	0	0
Superstructure	Tilt-up Panels	184,280	0	0
	Columns	28,871	0	0
	Roof Structure and Roofing	188,252	7,530	1,500
		556,812	7,530	1,500
Architectural	Overhead Doors	12,235	0	0
	Windows	20,929	0	0
	Suspended Ceilings	35,418	17,709	2,688
	Interior Partitions	38,208	9,078	2,312
	Floor Finish	33,808	0	0
		140,598	26,787	5,000
	TOTAL:	697,410	34,317	6,500

[1] This value is approximate.

TABLE B.3-4

COMPARISONS BETWEEN ESTIMATED DAMAGE FACTORS AND ACTUAL DAMAGE FACTORS FOR THE 120 MAST STREET BUILDING

Group	Replacement Value ($)	Estimated[1] Damage Factor	Actual[2] Damage Factor	Estimated D.F.[2] / Actual D.F.
Structural	556,812	0.01352	0.00269	5.0
Architectural	140,598	0.19052	0.03556	5.4
Total[3]	697,410	0.04921	0.00932	5.3

1 Damage Factor = Repair Cost / Replacement Value

2 The Actual Damage Factor is an approximate value.

3 Only the structural and architectural components are included.

Photo B.3-1
120 Mast Street Building
East and north elevations

Photo B.3-2
120 Mast Street Building
Partial north elevation (front entrance)

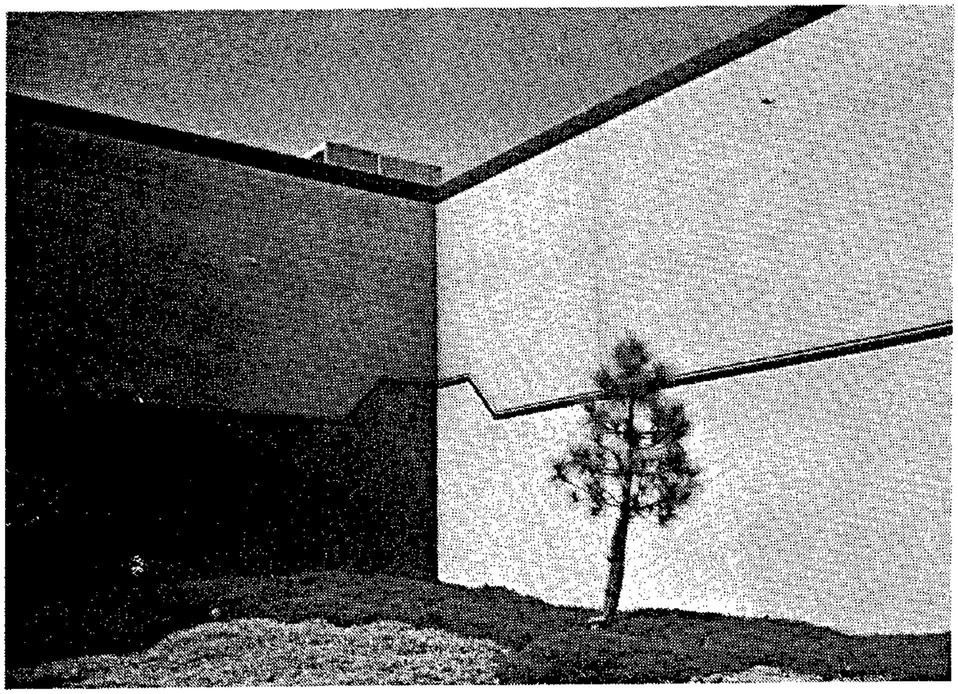


Photo B.3-3
120 Mast Street Building
Typical perimeter concrete tilt-up wall panels

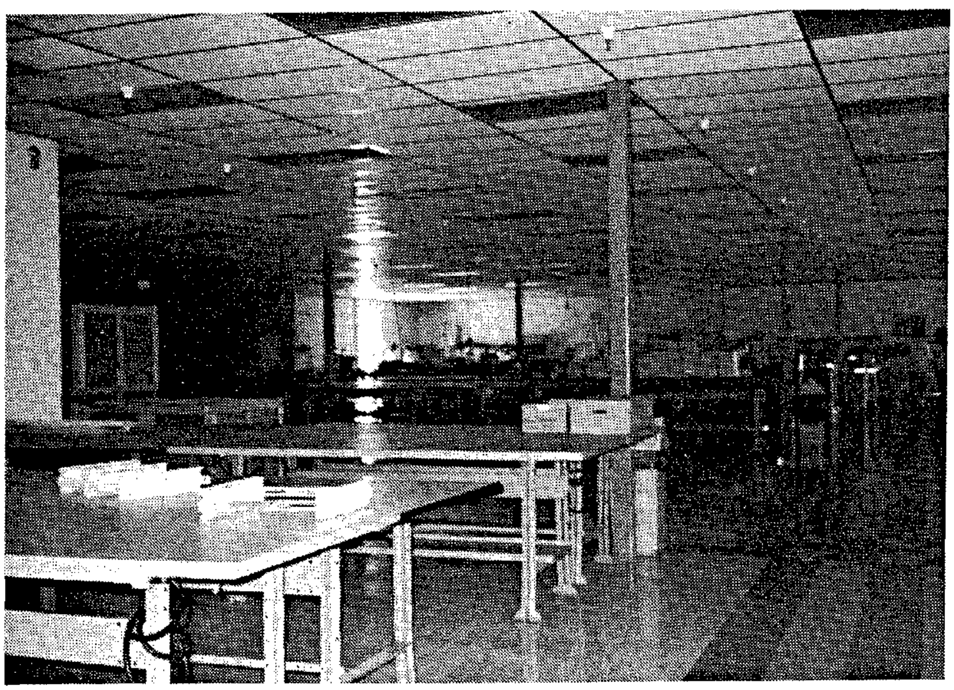


Photo B.3-4
120 Mast Street Building
Interior at the assembly area

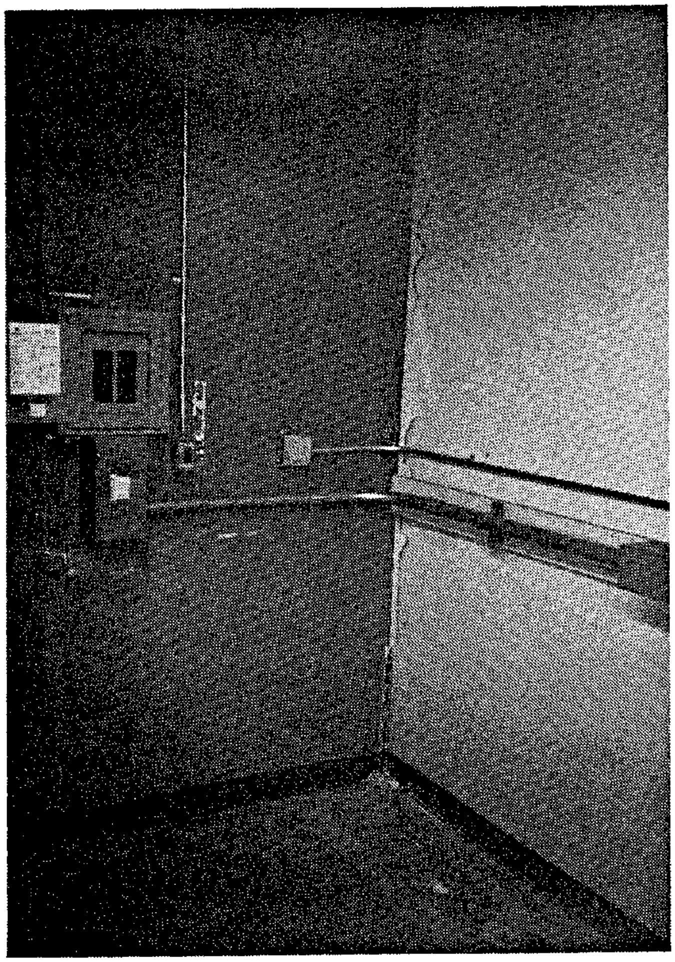


Photo B.3-5
120 Mast Street Building
Most interior partitions suffered significant damage

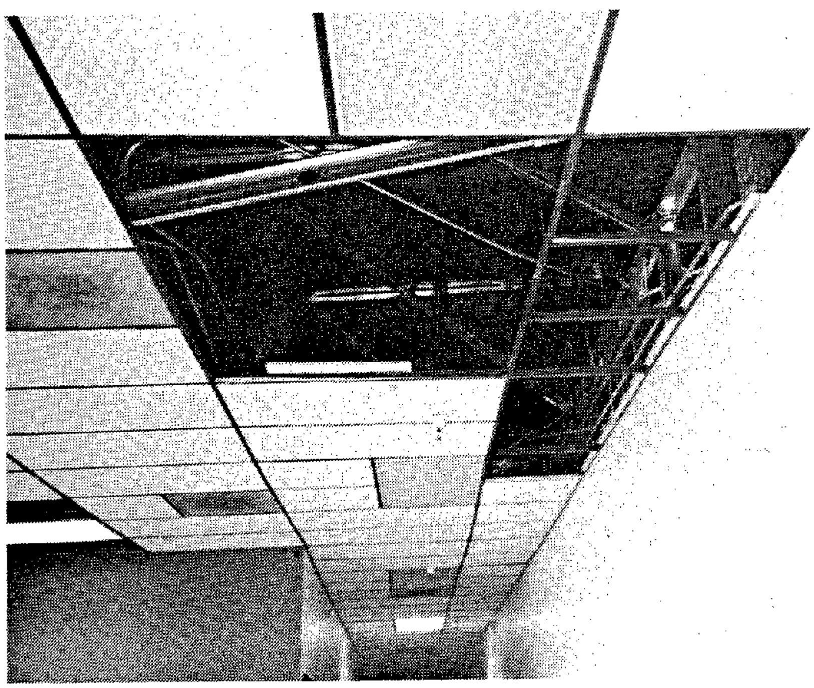


Photo B.3-6
120 Mast Street Building
Many ceiling tiles fell

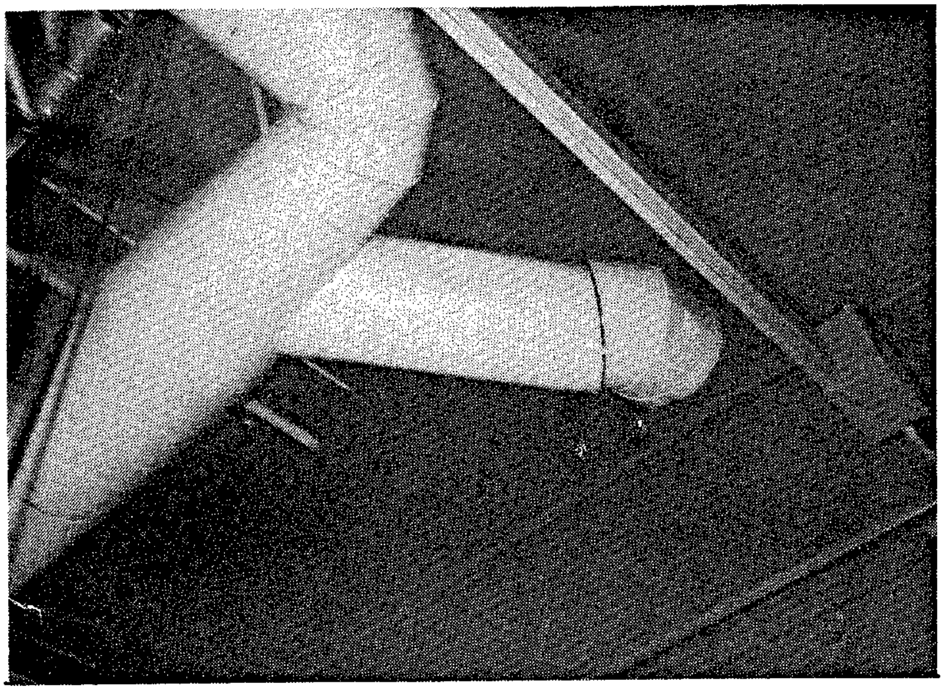


Photo B.3-7
120 Mast Street Building
An offset occurred in one section of a HVAC duct

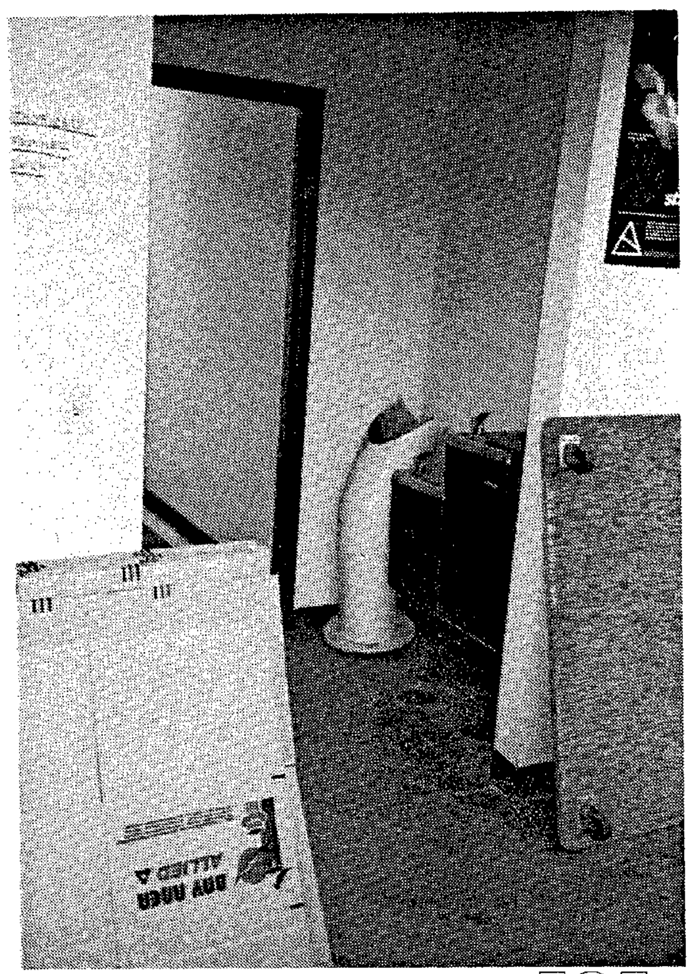


Photo B.3-8
120 Mast Street Building
A section of a HVAC duct broke and fell

B.4 WILTRON FACILITY AT 490 JARVIS DRIVE, MORGAN HILL

B.4.1 Description of Building

The Wiltron facility at 490 Jarvis Drive is a two-story concrete tilt-up structure which was designed in 1983 in accordance with the 1979 Uniform Building Code. The building was completed only days before the Morgan Hill earthquake. Wiltron planned to move operations into the building three days after the earthquake. This structure is irregular in shape with maximum plan dimensions of approximately 192 feet by 240 feet. The perimeter walls are nearly 32 feet in height. A plan view of the building is shown in Figure B.4-1, and a portion of the north elevation is shown in Photo B.4-1.

The building has a plywood roof diaphragm over a framing system which consists of glulam beams and girders supported by steel columns. The floor system consists of a metal deck with lightweight concrete fill supported by open web joists and steel columns (Photo B.4-2). The floor and roof diaphragms and the perimeter tilt-up walls are the lateral load resisting elements (Photo B.4-3). Since there are large window openings in essentially all the wall panels, the building is generally more flexible than a typical shear wall building, such as the one at 120 Mast Street.

Reinforcing bars in the wall panels at roof level act as chords for the roof diaphragm. Only one other shear connector between adjacent concrete tilt-up panels was provided over the entire height of the building. All wall panels are positively anchored to the beams and girders at the roof level to transfer out-of-plane anchorage loads of the wall panels into the roof diaphragm. Continuous ties are also provided at the beam and girder connections along grid lines throughout the building. Local chords are provided along lines 5, 7, PB.7 and PE.7 to accommodate the irregularities of the roof diaphragm. The continuous steel ledgers supporting the metal deck floor are designed to transfer diaphragm shear into the wall panels. They also function as chords for the floor diaphragm.

There is a 20-foot-wide sloped clay-tile roof at the perimeter of the building. Half the roof overhangs the tilt-up walls and is supported by columns. These columns also support concrete planters which are continuous around the building at the upper floor level (Photos B.4-4 and B.4-5).

All interior partitions consist of metal studs with gypsum board on either one or both sides. Seventy percent of the partitions are on the ground floor, and most extend the full height between the floor and the framing above. In the analysis these partitions were not considered to be lateral load resisting components. There is a suspended ceiling above the entire second floor. Only two small areas of the ground floor have a suspended ceiling (Photos B.4-6 and B.4-7).

B.4.2 Description of Damage

This building suffered minimal damage during the Morgan Hill earthquake, includes the following:

- Fallen minor cracking of the drywall partitions
- Fallen ceiling tiles near the perimeter walls
- Minor roofing damage.

All damage was repaired within a few days after the earthquake so that Wiltron could move into the building on schedule. EQE engineers did not have an opportunity to see any of the damage.

B.4.3 Site Response Spectrum

A peak bedrock acceleration of 0.29g was determined using the Campbell formula (Reference B.5). A magnitude of 6.2 and fault-to-site distance of 5.1 kilometers were used in the calculation. A geological investigation performed before construction of the building determined that the site had very stiff soil. Using the graphic correlation presented in Figure 3 (Chapter 6), a surface soil peak horizontal acceleration of 0.27g was obtained. It is possible that significantly different ground accelerations were experienced.

To complete the estimate of ground motions, appropriate ground motion ratios and the spectral amplification factors were selected from Tables 3 and 4 (Chapter 6) respectively. All computations described above are shown in Figure B.4-2. The site response spectrum was then constructed and is shown in Figure B.4-3.

B.4.4 Predicted Earthquake Losses

The total replacement value of the building was assumed to be the total construction cost of $3,408,000. Since the building was being completed when the earthquake occurred and the necessary repairs were done within days after the earthquake, the above assumption is reasonably justified.

Much of the information required for compiling the inventory of structural and nonstructural components and their corresponding replacement costs was not readily available from the owner.

The replacement costs of the structural, non-structural, mechanical, and electrical components are approximately proportional to the number of stories for one-story and two-story buildings. The cost for site work does not increase in proportion to building height. Therefore, the ratios of component replacement values to total replacement value used in the example building (Chapter 6) were modified before being used to estimate component replacement values. The complete component inventory of the building is shown in Table B.4-1.

Structural responses of multi-story buildings are much more complicated than responses of one-story buildings. Elevated floor levels are typically concrete and are similar in stiffness to shear walls. Consequently, it is not always possible to decouple the walls and the floor for analysis purposes. The simple analysis model developed for the example building could not be utilized directly for the Wiltron two-story building. To estimate the natural period of the wall panels for comparison with the roof period, a two-degree-of-freedom model was used. The period of the fundamental mode was then obtained as one of the solutions to the eigenvalue problem. A single-story wall was considered to compare the periods of the shear walls to the floor diaphragm periods.

The irregular building configuration presented another analysis problem which required a solution. For the north-south direction earthquake, the flexible plywood roof diaphragm was modeled as two simply-supported beams (Figure B.4-5). The two beams are assumed to have a common middle support in the center of the building. The metal-deck floor with concrete topping was modeled as a single beam simply supported by the end walls. A concentrated load, representing the lateral resistance provided by the center walls, was placed at midspan. The floor is more rigid than the roof and will likely distribute loads to the walls in proportion to their rigidities.

For the east-west direction earthquake, both the roof diaphragm and the floor diaphragm were assumed to be rectangular in shape. The supports of the diaphragms were located between the two walls of each end so that the distances between the assumed supports and the actual wall locations were inversely proportional to the wall stiffnesses.

Using these simplified models, the periods of the roof diaphragm and the floor diaphragm were computed following the same procedure used in the example. All calculations of the component structural characteristics are shown in Figure B.4-4 through Figure B.4-11.

The roof diaphragm was found to be much more flexible than the concrete walls and was therefore treated as uncoupled. The magnitudes of the natural periods of the metal deck-floor diaphragm with lightweight concrete topping and the two-story walls are similar. The coupling between the two components can only be modeled by a rather complex and elaborate system. The development of such a coupled system is not within the scope of this project. The floor diaphragm period is approximately three times the period of each of the one-story shear walls. As an approximation, it was assumed that the floor diaphragm walls were also decoupled. The structural responses of the individual components were then estimated from the site response spectrum shown in Figure B.4-3. A summary of structural responses is presented in Figure B.4-12.

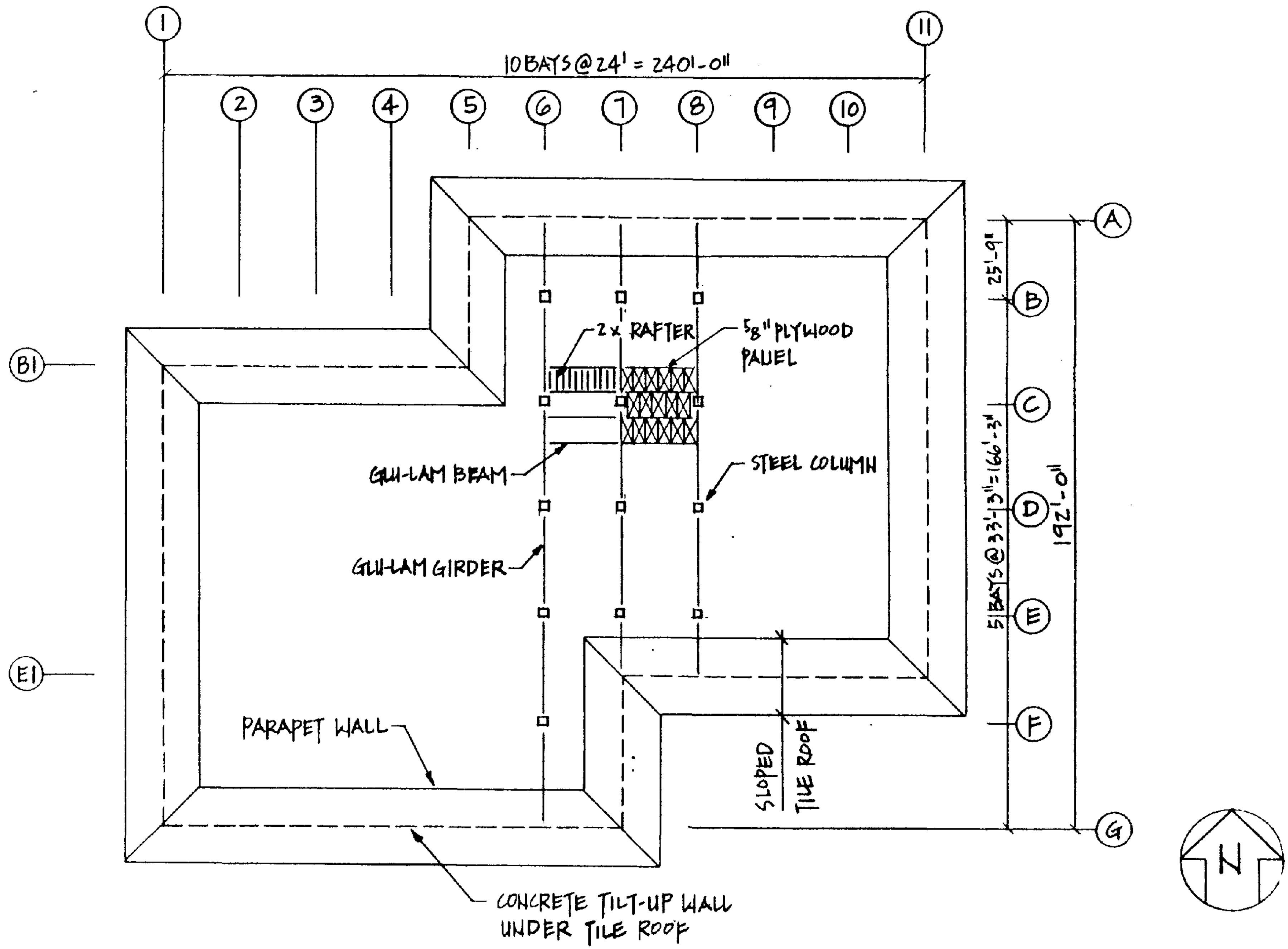
Figures B.4-13 and B.4-14 show the calculations of estimated damage factors of the structural and nonstructural components. Linear interpolation between the three damage thresholds listed in Table 5 was assumed in the calculations. The summaries of the results are presented in Tables B.4-2 and B.4-3.

B.4.5 Comparison of Predicted Earthquake Losses to Actual Losses

Comparisons between the actual repair costs of the structural and nonstructural components and the predicted repair costs calculated by the component approach are shown in Table B.4-3. Table B.4-4 presents a comparison between the predicted total damage factors and the actual damage factors.

The total cost of repair was over-estimated by a factor of 6.0, comparable to the 5.3 over-estimation factor for the 120 Mast Street building. A large portion of the over-estimate was related to predicted damage to the suspended ceiling.

Some discrepancies may have been caused by the oversimplified model used in the analyses for the estimation of the component structural responses. Inaccurate ground acceleration prediction may also have contributed to the large difference between actual and predicted damage values. However, as was the case for the 120 Mast Street building, a minor amount of damage (5.0%) was predicted using the methodology, and a very minor amount (0.84%) occurred .



EQE

Figure B.4-1: Roof Plan of the 490 Jarvis Drive Building

ESTIMATION OF BEDROCK PEAK HORIZONTAL ACCELERATION

USE CAMPBELL FORMULA (REFERENCE B-5)

$$\text{BEDROCK PGA} = 0.0159\ \text{EXP}(0.868\ M)[R + 0.0606\ \text{EXP}(0.700\ M)]^{-1.09}$$
$$= 0.0159\ \text{EXP}(0.868 \times 6.2)[5.1 + 0.0606\ \text{EXP}(0.700 \times 6.2)]^{-1.09}$$
$$= 0.289\ g$$

M = MAGNITUDE OF EARTHQUAKE = 6.2
R = FAULT DISTANCE, IN KILOMETERS = 5.1 KM

ESTIMATION OF SOIL PEAK HORIZONTAL ACCELERATION

SOIL CONDITION : STIFF SOILS

SOIL PGA = 0.275 g (FIGURE 3)

GROUND MOTION RATIOS

$$\left.\begin{array}{l} V/a\ (\text{IN/SEC})/g = 34.5 \\ ad/V^2 = 4.35 \end{array}\right\}\ (\text{TABLE 3})$$

MAX. GROUND ACCELERATION = 0.275 g

MAX. GROUND VELOCITY = 34.5 (IN/SEC)/g × 0.275 g = 9.488 IN/SEC

$$\text{MAX. GROUND DISPLACEMENT} = \frac{4.35 \times (9.488\ \text{IN/SEC})^2}{0.275\ g \times 386.4\ (\text{IN/SEC}^2)/g} = 3.685\ \text{IN}$$

SPECTRAL AMPLIFICATION (5% DAMPING)

$$\left.\begin{array}{ll} \text{ACCELERATION} & 2.63 \\ \text{VELOCITY} & 1.27 \\ \text{DISPLACEMENT} & 2.53 \end{array}\right\}\ (\text{TABLE 4})$$

MAX. SPECTRAL ACCELERATION = 2.63 × 0.275 = 0.72 g

MAX. SPECTRAL VELOCITY = 1.27 × 9.488 = 12 IN/SEC

MAX. SPECTRAL DISPLACEMENT = 2.53 × 3.685 = 9.3 IN

Figure B.4-2: Ground Motion Prediction for the 490 Jarvis Drive Building

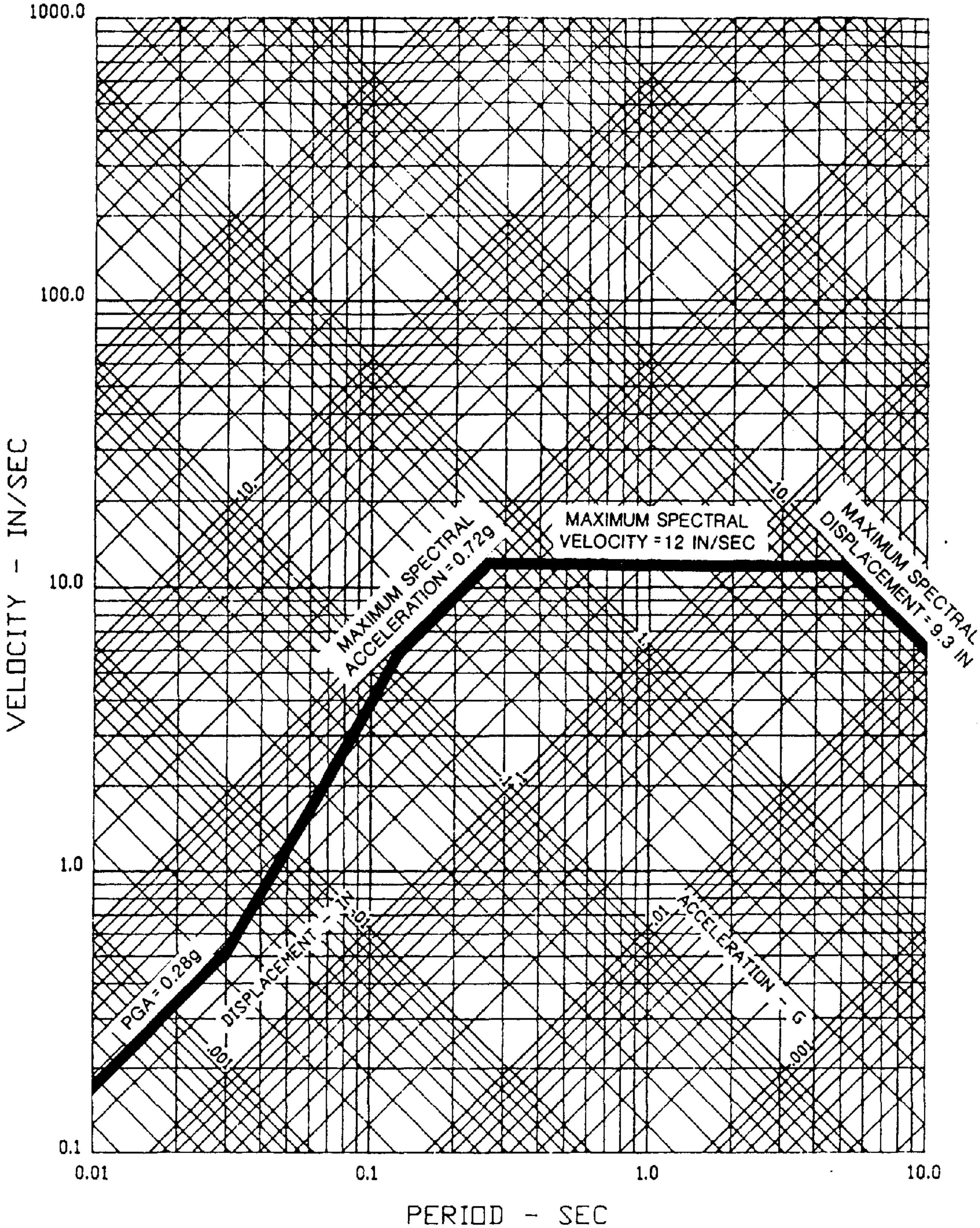


Figure B.4-3: 5% Damping Response Spectrum for the 490 Jarvis Drive Building

EQE

WEIGHTS

(I) ROOF

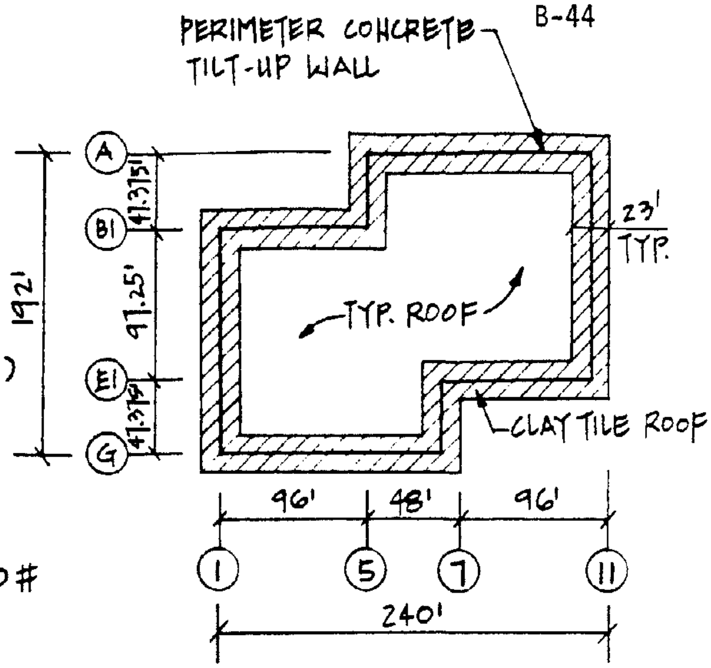
(1) TYP. ROOF

UNIT WEIGHT = 14 PSF
AREA = 36984 □'
TOTAL WT. = 14 PSF x 36984 □'
= 517776 #

(2) CLAY TILE ROOF (INCLUDES FACIA BEAM)

UNIT WEIGHT = 1080 PLF
LENGTH = 864'
TOTAL WT. = 1080 PLF x 864'
= 933120 #

TOTAL ROOF WT. = 517776 # + 933120 #
= 1450896 #

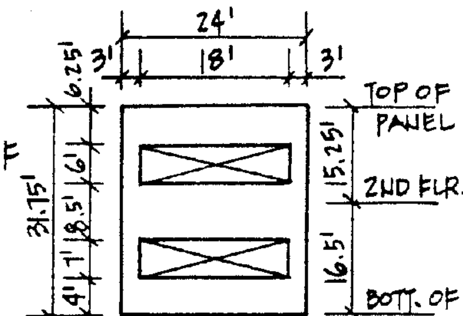


(II) FLOOR

UNIT WEIGHT = 41 PSF
AREA = 36984 □'
TOTAL FLOOR WEIGHT = 41 PSF x 36984 □' = 1516344 #

(III) CONCRETE TILT-UP WALLS

7½" THICK CONCRETE PANEL, TYPICAL
UNIT WT. = 150 PCF x 7.5" x (1'/12") = 93.75 PSF
TYP. PANEL WT. = (24' x 31.75' - 6' x 18' - 7' x 18') 93.75 PSF
= 49500 #
PANEL WT. TRIBUTARY TO ROOF :
W_{ROOF} = (15.25' x 24' - 6' x 18') x 93.75 PSF x ½
= 12094 #
PANEL WT. TRIBUTARY TO FLOOR :
W_{FLOOR} = ½ x 49500 # = 24750 #



TYPICAL PANEL

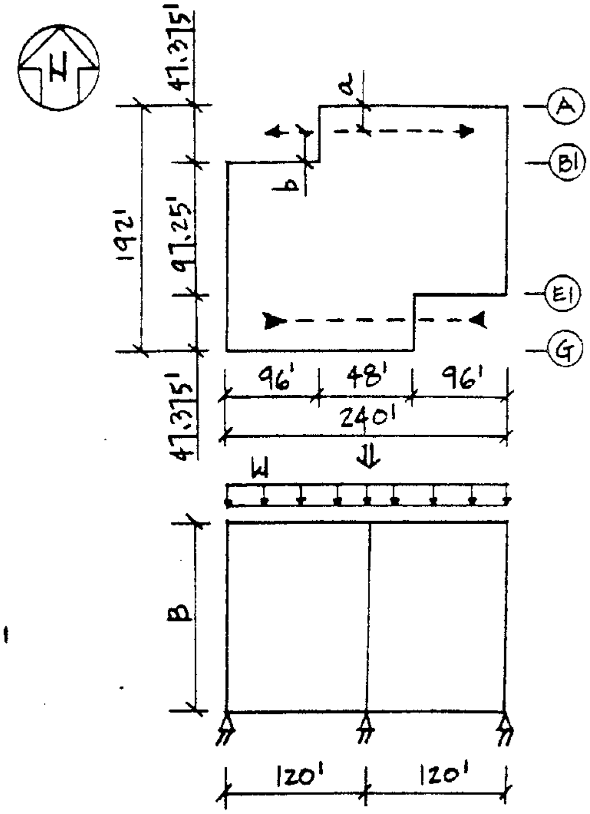
CALCULATIONS FOR OTHER PANELS NOT SHOWN.

WALL WEIGHTS:

LINE	NO. OF PANELS	W_{ROOF} (#)	W_{FLOOR} (#)
1	6	72564	148500
5	2	29026 *	59400 *
7	2	35100 *	66488 *
11	6	72564	148500
A	6	72564	148500
B1	4	48376	99000
E1	4	48376	99000
G	6	72564	148500

* THERE ARE IRREGULAR PANELS.

Figure B.4-4: Weights of Structural Components, 490 Jarvis Drive Building



ROOF DIAPHRAGM (NORTH-SOUTH EQ)

ASSUME 2 BEAMS WITH EQUAL SPANS OF 120' AND EFFECTIVE CHORD RESISTANCE LIES BETWEEN LINES A AND B1

CALCULATE EFFECTIVE BEAM WIDTH

LET a = DISTANCE TO WALL @ LINE A
b = DISTANCE TO WALL @ LINE B1

ASSUME DISTANCE TO WALL $\propto$ 1/ WALL STIFFNESS AND WALL STIFFNESS $\propto$ LENGTH OF WALL

THEREFORE, $\frac{a}{b} = \frac{96'}{144'}$

ALSO, $a + b = 47.375'$

SOLVE SIMULTANEOUS EQUATIONS

$a = 19'$, $b = 28.375'$

EFFECTIVE BEAM WIDTH $B = 97.25' + 2b$
$= 97.25' + 2 \times 28.375'$
$= 154'$

5/8" STRUCTURAL I C-D PLYWOOD DIAPHRAGM
10d NAILS @ 2½", 3", 12" O.C.
EFFECTIVE THICKNESS $t = 0.715"$
$G = 90{,}000$ PSI

7½" CONCRETE TILT-UP WALLS
$f'_c = 3000$ PSI
$E = 57000\sqrt{3000} = 3.12 \times 10^6$ PSI

LOADS

TOTAL ROOF WEIGHT = 1450896 #
TOTAL WALL WEIGHT = (72564 # + 48376 #) × 2 = 241880 #
$W = (1450896\# + 241880\#) \times \frac{1}{2} \times 0.186 / 120'$
$= 1692776 \times \frac{1}{2} \times 0.186 / 120' = 1312$ PLF

$V = \frac{WL}{2B} = \frac{1312 \text{ PLF} \times 120'}{2 \times 154'} = 511$ PLF

Figure B.4-5.1: Roof Diaphragm Period, North-South Direction Earthquake, 490 Jarvis Drive Building

DIAPHRAGM STATIC DEFLECTION

$$\Delta = \frac{5VL^3}{8EAB} + \frac{VL}{4Gt} + 0.094\,L\,e_n + \underbrace{\frac{\Sigma A_c X}{2B}}_{\text{ASSUME} = 0}$$

$$= \frac{5 \times 511\text{ PLF} \times (120')^3}{8 \times (3.12 \times 10^6\text{ PSI})\underbrace{[(8 \times 7.5'') \times 7.5''](154')}_{\text{AREA OF CHORD}}} + \frac{(511\text{ PLF})(120')}{4 \times (90{,}000\text{ PSI})(0.715'')} + 0.094(120')(0.021)$$

AREA OF CHORD = TRIBUTARY WIDTH × WALL THICKNESS
= (8 × WALL THICKNESS) × WALL THICKNESS = 450□"

$$= 0.0026'' + 0.2382'' + 0.2369''$$

$$= 0.478''$$

EQUIVALENT BEAM DEFLECTION

$$\Delta = \frac{5WL^4}{384EI}$$

$$\Rightarrow EI = \frac{5WL^4}{384\,\Delta} = \frac{5(1312\text{ PLF})(120')^4 \times 12^3}{384\,(0.478'')} = 1.28 \times 10^{13}\ \#\text{-IN}^2$$

DIAPHRAGM PERIOD

$$\omega = \frac{\pi^2}{L^2}\sqrt{\frac{EI}{m}}$$

$$m = \frac{(1312\text{ PLF})(1/0.186)(1'/12'')}{(32.2\text{ FT/SEC}^2)(12''/1')} = 1.52\ \#\text{-SEC}^2/\text{IN}^2$$

$$\omega = \frac{\pi^2}{(120' \times 12)^2}\sqrt{\frac{1.28 \times 10^{13}\ \#\text{-IN}^2}{1.52\ \#\text{-SEC}^2/\text{IN}^2}} = 13.81\text{ RAD/SEC}$$

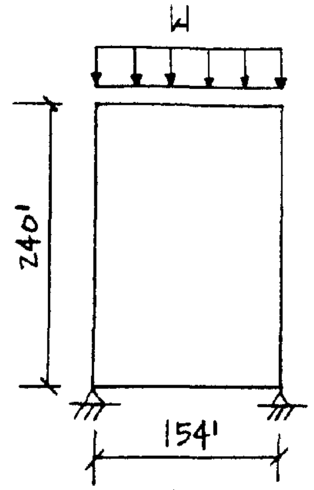
$$T^R_{D(NS)} = \frac{2\pi}{\omega} = \frac{2\pi\text{ RAD}}{13.81\text{ RAD/SEC}} = 0.45\text{ SEC}$$

THE ROOF DIAPHRAGM PERIOD FOR AN EARTHQUAKE IN THE NORTH-SOUTH DIRECTION IS 0.45 SECONDS

Figure B.4-5.2: Roof Diaphragm Period, North-South Direction Earthquake, 490 Jarvis Drive Building

EQE

ROOF DIAPHRAGM (EAST-WEST EQ)



5/8" STRUCTURAL I C-D PLYWOOD DIAPHRAGM
10d NAILS @ 2½", 3", 12" O.C.
EFFECTIVE THICKNESS $t = 0.715''$
$G = 90{,}000$ PSI

7½" CONCRETE TILT-UP WALLS
$f'_c = 3000$ PSI
$E = 57000\sqrt{3000} = 3.12 \times 10^6$ PSI

LOADS

TOTAL ROOF WT. = 1450896 #
TOTAL WALL WT. = (72564# + 29026# + 35100# + 72564#)
= 209254 #

$W = (1450896\# + 209254\#) \times 0.186 / 154' = 2005$ PLF (USE 154' LENGTH DERIVED FOR DEPTH OF DIAPHRAGM IN NORTH-SOUTH DIRECTION)

$$V = \frac{WL}{2B} = \frac{2005\text{ PLF} \times 154'}{2 \times 240'} = 643 \text{ PLF}$$

DIAPHRAGM STATIC DEFLECTION

$$\Delta = \frac{5VL^3}{8EAB} + \frac{VL}{4Gt} + 0.094\,L\,e_n + \cancel{\frac{\Sigma \Delta_c X}{2B}}^{\,0}$$

$$= \frac{5(643\text{PLF})(154')^3}{8(3.12\times10^6\text{PSI})(450\square'')(240')} + \frac{(643\text{ PLF})(154')}{4(90000\text{PSI})(0.715'')} + (0.094)(154')(0.029)$$

$$= 0.0044'' + 0.3847'' + 0.4198''$$

$$= 0.81''$$

EQUIVALENT BEAM DEFLECTION

$$\Delta = \frac{5WL^4}{384EI} \Rightarrow EI = \frac{5WL^4}{384\Delta} = \frac{5(2005\text{PLF})(154')^4 \times 12^3}{384(0.8089'')} = 3.14 \times 10^{13}\ \#\text{-IN}^2$$

DIAPHRAGM PERIOD

$$m = \frac{(2005\text{PLF})(1/0.186)(1'/12'')}{(32.2\text{ FT/SEC}^2)(12''/1')} = 2.32\ \#\text{-SEC}^2/\text{IN}^2$$

$$\omega = \frac{\pi^2}{L^2}\sqrt{\frac{EI}{m}} = \frac{\pi^2}{(154'\times12)^2}\sqrt{\frac{3.14\times10^{13}\ \#\text{-IN}^2}{2.32\ \#\text{-SEC}^2/\text{IN}^2}} = 10.62 \text{ RAD/SEC}$$

$$T^R_{D(EW)} = \frac{2\pi}{\omega} = 0.59 \text{ SEC}$$

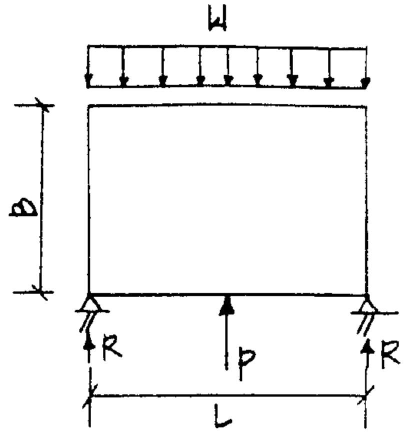
THE ROOF DIAPHRAGM PERIOD FOR AN EARTHQUAKE IN THE EAST-WEST DIRECTION IS 0.59 SECONDS

Figure B.4-6: Roof Diaphragm Period, East-West Direction Earthquake, 490 Jarvis Drive Building

FLOOR DIAPHRAGM (NORTH-SOUTH EQ)

$2\frac{1}{2}$" CONCRETE TOPPING OVER $1\frac{1}{2}$" METAL DECK

ASSUME SIMPLY-SUPPORTED BEAM WITH
SPAN L = 240' ; WIDTH B = 154'
DIAPHRAGM IS SOMEWHERE BETWEEN FLEXIBLE AND RIGID



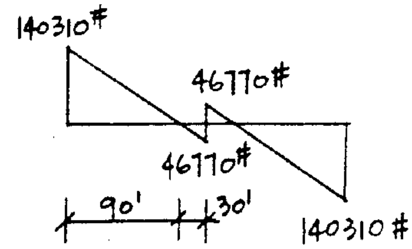
LOADS

TOTAL FLOOR WEIGHT = 1516344 #
TOTAL WALL WEIGHT = (148500 + 99000) x 2 = 495000#
W = (1516344 + 495000) 0.186 / 240' = 1559 PLF

CONCENTRATED LOAD P AT MID-SPAN REPRESENTS
LATERAL RESISTANCE FROM CENTER WALLS AT LINES 5 & 7

ASSUME WALL STIFFNESS $\propto$ NO. OF PANELS
$\because$ THERE ARE NO SHEAR CONNECTORS BETWEEN PANELS

NO. OF PANELS AT EA. END WALL = 6
NO. OF PANELS AT CENTER WALL = 4
TOTAL NO. OF PANEL = 2x6 + 4 = 16



SHEAR DIAGRAM

ASSUME P = (1559 PLF x 240') 4/16 = 93540 #
R = (1559 PLF x 240') 6/16 = 140310 #

$$V = \frac{R}{B} = \frac{140310\#}{154'} = 911 \text{ PLF}$$

DIAPHRAGM STATIC DEFLECTION

BENDING : $\Delta_B = \frac{5WL^4}{384EI} - \frac{PL^3}{48EI}$ WHERE $I = 2A(\frac{B}{2})^2 = 2(450^{\square''})(\frac{154'}{2})^2 = 5336100 \text{ IN}^2\text{-FT}^2$

$$= \frac{5(1559 \text{ PLF})(240')^4 \times 12}{384(3.12\times10^6 \text{ PSI})(5336100 \text{ IN}^2\text{-FT}^2)} - \frac{(93540\#)(240')^3 \times 12}{48(3.12\times10^6 \text{ PSI})(5336100 \text{ IN}^2\text{-FT}^2)}$$

$$= 0.0485'' - 0.0194'' = 0.029''$$

SHEAR : $\Delta_W = \frac{q_{AVE} L_1 F}{10^6}$ (SEE TRI-SERVICE MANUAL, REFERENCE B-6)

$q_{AVE} = \frac{1}{2} V = \frac{1}{2} \times 911 \text{ PLF} = 456 \text{ PLF}$
$L_1 = 90'$
ESTIMATED FROM TABLE BY LINEAR INTERPOLATION

$$\Delta_W = \frac{(456 \text{ PLF})(90')(1.36)}{10^6} = 0.056''$$

$$\Delta = \Delta_B + \Delta_W = 0.029'' + 0.056'' = 0.085''$$

Figure B.4-7.1: Floor Diaphragm Period, North-South Direction Earthquake, 490 Jarvis Drive Building

EQE

EQUIVALENT BEAM DEFLECTION (ASSUME DEFLECTED SHAPE RESULTING FROM UNIFORM AND CONCENTRATED LOAD IS SIMILAR TO DEFLECTION RESULTING FROM UNIFORM LOAD)

$$\Delta = \frac{5WL^4}{384EI}$$

$$\Rightarrow EI = \frac{5WL^4}{384\Delta} = \frac{5(1559)(240')^4 \times 12^3}{384 \times 0.08493''} = 1.37 \times 10^{15} \text{ \#-IN}^2$$

DIAPHRAGM PERIOD T_D

$$\omega = \frac{\pi^2}{L^2}\sqrt{\frac{EI}{m}}$$

$$m = \frac{(1559 \text{ PLF})(1/0.186)(1'/12'')}{(32.2 \text{ FT/SEC}^2)(12''/1')}$$

$$= 1.81 \text{ \#-SEC}^2/\text{IN}^2$$

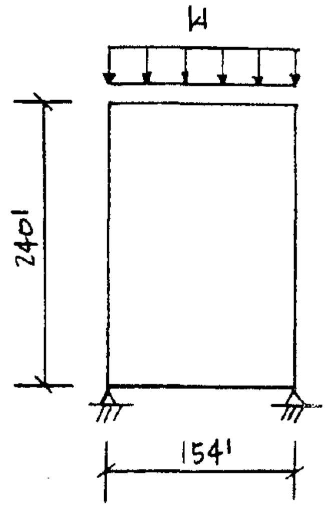
$$\omega = \frac{\pi^2}{(240' \times 12)^2}\sqrt{\frac{1.37 \times 10^{15} \text{ \#-IN}^2}{1.81 \text{ \#-SEC}^2/\text{IN}^2}} = 32.76 \text{ RAD/SEC}$$

$$T^F_{D(NS)} = \frac{2\pi}{\omega} = 0.19 \text{ SEC}$$

THE FLOOR DIAPHRAGM PERIOD FOR AN EARTHQUAKE IN THE NORTH-SOUTH DIRECTION IS 0.19 SECONDS

Figure B.4-7.2: Floor Diaphragm Period, North-South Direction Earthquake, 490 Jarvis Drive Building

EQE



FLOOR DIAPHRAGM (EAST-WEST EQ)

2½" CONCRETE TOPPING OVER 1½" METAL DECK

LOADS

TOTAL FLOOR WT. = 1516344 #

TOTAL WALL WT. = (148500#)x2 + 59400# + 66488#
= 422888#

W = (1516344 # + 422888 #) x 0.186 / 154'
= 2342 PLF

$$V = \frac{WL}{2B} = \frac{(2342\ PLF)(154')}{2(240\ FT)} = 751\ PLF$$

DIAPHRAGM STATIC DEFLECTION

$$\text{BENDING}: \Delta_B = \frac{5VL^3}{8EAB} = \frac{5(751\ PLF)(154')^3}{8(3.12 \times 10^6\ PSI)(450^{b''})(240')} = 0.0051''$$

$$\text{SHEAR}: \Delta_W = \frac{q_{AVE} L_1 F}{10^6}$$

$$q_{AVE} = \tfrac{1}{2}V = \tfrac{1}{2} \times 751\ PLF = 376\ PLF$$

$$L_1 = \tfrac{1}{2}L = \tfrac{1}{2} \times 154' = 77'$$

$$\Delta_W = \frac{(376\ PLF)(77')(1.36)}{10^6} = 0.039''$$

$$\Delta = \Delta_B + \Delta_W = 0.0051'' + 0.039'' = 0.044''$$

EQUIVALENT BEAM DEFLECTION

$$\Delta = \frac{5WL^4}{384EI} \Rightarrow EI = \frac{5(2342\ PLF)(154')^4 \times 12^3}{384(0.044'')} = 6.74 \times 10^{14}\ \#\text{-}IN^2$$

DIAPHRAGM PERIOD

$$m = \frac{(2342\ PLF)(1/0.186)(1'/12'')}{(32.2\ FT/SEC^2)(12''/1')} = 2.72\ \#\text{-}SEC^2/IN^2$$

$$\omega = \frac{\pi^2}{(154 \times 12)^2}\sqrt{\frac{6.74 \times 10^{14}\ \#\text{-}IN^2}{2.72\ \#\text{-}SEC^2/IN^2}} = 45.52\ RAD/SEC$$

$$T^F_{D(EW)} = \frac{2\pi}{\omega} = 0.14\ SEC$$

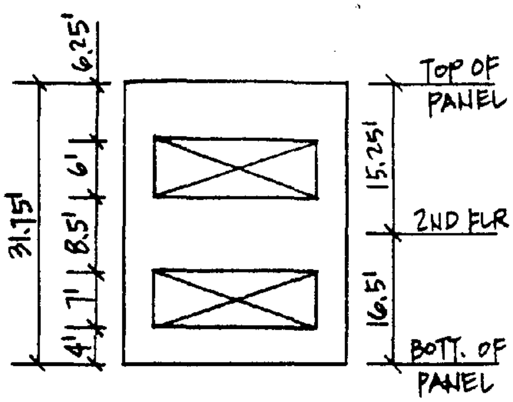
THE FLOOR DIAPHRAGM PERIOD FOR AN EARTHQUAKE IN THE EAST-WEST DIRECTION IS 0.14 SECONDS

Figure B.4-8: Floor Diaphragm Period, East-West Direction Earthquake, 490 Jarvis Drive Building

TYPICAL PANEL STIFFNESS

THICKNESS $t = 7\tfrac{1}{2}''$

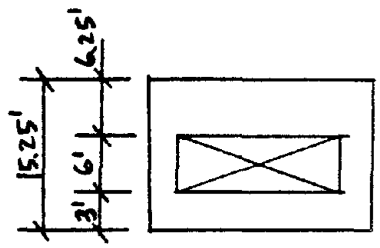
USE 2 DEGREE OF FREEDOM MODEL
FOR COMPARISON WITH ROOF PERIOD



$$\Delta_{FIXED} = \frac{FH^3}{12EI} + \frac{1.2FH}{AG}$$

$$= F\left[\frac{H^3}{12 \times 3.12 \times 10^6 \, tB^3} + \frac{1.2H}{1.33 \times 10^6 \, tB}\right]$$

$$= \frac{F}{t}\left[0.026709\left(\frac{H}{B}\right)^3 + 0.902256\left(\frac{H}{B}\right)\right] \times 10^{-6}$$



$t = 7.5''$

$$\Delta_{FIXED} = F \times 10^{-9}\left[3.56125\left(\frac{H}{B}\right)^3 + 120.30075\left(\frac{H}{B}\right)\right]$$

$$\delta_{FIXED} = \frac{\Delta_{FIXED}}{F} \times 10^9$$

$$\Delta_{CANTILEVER} = \frac{FH^3}{3EI} + \frac{1.2FH}{AG} = 4 \times \frac{FH^3}{12EI} + \frac{1.2FH}{AG}$$

$t = 7.5''$

$$\Delta_{CANTILEVER} = F \times 10^{-9}\left[4 \times 3.56125\left(\frac{H}{B}\right)^3 + 120.30075\left(\frac{H}{B}\right)\right]$$

$$= F \times 10^{-9}\left[14.245\left(\frac{H}{B}\right)^3 + 120.30075\left(\frac{H}{B}\right)\right]$$

$$\delta_{CANTILEVER} = \frac{\Delta_{CANTILEVER}}{F} \times 10^9$$

SECOND FLOOR (CANTILEVER)

CALCULATE PANEL STIFFNESS (METHOD C, CHAPTER 13, TRISERVICE MANUAL, REF. B-6)

STEP 1 : DEFLECTION OF SOLID WALL WITH $H/B = 15.25/24 = 0.6354$
$\delta_1 = 14.245(0.6354)^3 + 120.30075(0.6354) = 80.09$

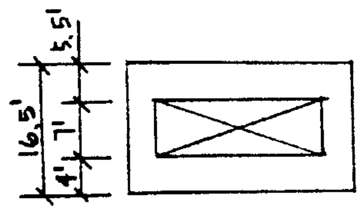
STEP 2 : DEFLECTION OF SOLID WALL WITH $H/B = 6/24 = 0.25$
$\delta_2 = 14.245(0.25)^3 + 120.30075(0.25) = 30.29$

STEP 3 : DEFLECTION OF PIERS WITH $H/B = 6/3 = 2$
$\delta_3 = \tfrac{1}{2}[3.56125(2)^3 + 120.30075(2)] = 134.55$

STEP 4 : $\delta = \delta_1 - \delta_2 + \delta_3 = 80.09 - 30.29 + 134.55 = 184.35$
$K = F/\Delta = 10^9 \times 1/\delta = 5.42 \times 10^6$ #/IN

Figure B.4-9.1: Calculations for Typical Wall Panel Stiffness, 490 Jarvis Drive Building

(2) FIRST FLOOR



STEP 1: $H/B = 16.5/24 = 0.6875$

$\delta_1 = 3.56125(0.6875)^3 + 120.30075(0.6875)$
$= 83.86$

STEP 2: $H/B = 7/24 = 0.2917$

$\delta_2 = 3.56125(0.2917)^3 + 120.30075(0.2917)$
$= 35.18$

STEP 3: $H/B = 7/3 = 2.3333$

$\delta_3 = [3.56125(2.3333)^3 + 120.30075(2.3333)] \times 1/2$
$= 162.97$

STEP 4: $\delta = \delta_1 - \delta_2 + \delta_3$
$= 83.86 - 35.18 + 162.97$
$= 211.65$

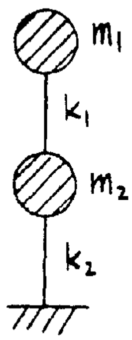
$\therefore K = F/\Delta = 10^9 \times 1/\delta = 4.72 \times 10^6$ #/IN

Figure B.4-9.2: Calculations for Typical Wall Panel Stiffness, 490 Jarvis Drive Building

SHEAR WALLS PERIOD (NORTH-SOUTH EQ)

USE 2 DEGREES OF FREEDOM MODEL

CALCULATE FUNDAMENTAL PERIOD



$$\begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix} \begin{Bmatrix} \ddot{x}_1 \\ \ddot{x}_2 \end{Bmatrix} + \begin{bmatrix} k_1 & -k_1 \\ -k_1 & k_1+k_2 \end{bmatrix} \begin{Bmatrix} x_1 \\ x_2 \end{Bmatrix} = \begin{Bmatrix} F_1 \\ F_2 \end{Bmatrix}$$ - EQUATION OF MOTION

$$\begin{vmatrix} k_1 - m_1\omega^2 & -k_1 \\ -k_1 & (k_1+k_2) - m_2\omega^2 \end{vmatrix} = 0$$ - FREQUENCY EQUATION

LET $x = \omega^2$

$$m_1 m_2 x^2 - [m_1(k_1+k_2) + m_2 k_1]\, x + k_1 k_2 = 0$$

LET $A = m_1 m_2$
$B = m_1(k_1+k_2) + m_2 k_1$
$C = k_1 k_2$

$$Ax^2 - Bx + C = 0$$

$$x = \frac{B - \sqrt{B^2 - 4AC}}{2A}$$

$$\omega = \sqrt{x}$$

WALL PERIOD $T_W = 2\pi/\omega$

END WALLS (EAST AND WEST) PERIOD

k_1 = NO. OF PANELS × TYP. PANEL STIFFNESS ∵ THERE ARE NO SHEAR CONNECTORS
$= 6 \times (5.42 \times 10^6\ \#/\text{IN}) = 32.52 \times 10^6\ \#/\text{IN}$

$k_2 = 6 \times (4.72 \times 10^6\ \#/\text{IN}) = 28.32 \times 10^6\ \#/\text{IN}$

$$m_1 = \frac{W_{ROOF} + W_{WALL}}{g} = \frac{1}{g}\left[\frac{WL}{2 \times 0.186} + W_{WALL}\right]$$

$$= \frac{1}{386.4\ \text{IN/SEC}^2}\left[\frac{1312\ \text{PLF} \times 120'}{2 \times 0.186} + 6 \times 12094\ \#\right] = 1283.1\ \#\text{-SEC}^2/\text{IN}$$

$$m_2 = \frac{1}{386.4\ \text{IN/SEC}^2}\left[\frac{140310\ \#}{0.186} + 6 \times 24750\ \#\right] = 2336.6\ \#\text{-SEC}^2/\text{IN}$$

$A = m_1 m_2 = 2.998 \times 10^6$
$B = m_1(k_1+k_2) + m_2 k_1 = 1.541 \times 10^{11}$
$C = k_1 k_2 = 9.210 \times 10^{14}$

$$x = \frac{B - \sqrt{B^2 - 4AC}}{2A} = 6904$$

$\omega = \sqrt{x} = 83.09$ RAD/SEC

$T_W = 2\pi/\omega = 0.076$ SEC

$3T_W = 3 \times 0.076$ SEC $= 0.228$ SEC < ROOF $T^R_{D(NS)} = 0.45$ SEC <u>UNCOUPLED</u>

Figure B.4-10.1: Shear Wall Period, North-South Direction Earthquake, 490 Jarvis Drive Building

CENTER WALLS PERIOD

$k_1 = 4 \times (5.42 \times 10^6 \text{ \#/IN}) = 21.68 \times 10^6 \text{ \#/IN}$

$k_2 = 4 \times (4.72 \times 10^6 \text{ \#/IN}) = 18.88 \times 10^6 \text{ \#/IN}$

$$m_1 = \left[\frac{2 \times 18720\,\#}{0.186} + 4 \times 12094\,\#\right]\frac{1}{386.4 \text{ IN/SEC}^2} = 2315.8 \text{ \#-SEC}^2\text{/IN}$$

$$m_2 = \left[\frac{93540\,\#}{0.186} + 4 \times 24750\,\#\right]\frac{1}{386.4 \text{ IN/SEC}^2} = 1557.7 \text{ \#-SEC}^2\text{/IN}$$

$A = m_1 m_2 = 3.607 \times 10^6$

$B = m_1(k_1+k_2) + m_2 k_1 = 1.277 \times 10^{11}$

$C = k_1 k_2 = 4.093 \times 10^{14}$

$$x = \frac{B - \sqrt{B^2 - 4AC}}{2A} = 3564$$

$\omega = \sqrt{x} = 59.70$ RAD/SEC

$T_W = 0.11$ SEC

$3T_W = 0.33$ SEC $<$ ROOF $T^R_{D(NS)} = 0.45$ SEC <u>UNCOUPLED</u>

USE SINGLE DEGREE OF FREEDOM MODEL FOR COMPARISON WITH FLOOR PERIOD

END WALLS (EAST AND WEST) PERIOD

$K = 28.32 \times 10^6 \text{ \#/IN}$

$m = 2336 \text{ \#-SEC}^2\text{/IN}$

$$T_W = 2\pi/\omega = 2\pi\sqrt{\frac{m}{K}} = 2\pi\sqrt{\frac{2336}{28.32 \times 10^6}} = 0.057 \text{ SEC}$$

$3T_W = 0.17$ SEC $<$ $T^F_{D(NS)} = 0.19$ SEC <u>UNCOUPLED</u>

CENTER WALLS

$k = 18.88 \times 10^6 \text{ \#/IN}$

$m = 1558 \text{ \#-SEC}^2\text{/IN}$

$$T_W = 2\pi\sqrt{\frac{1558}{18.88 \times 10^6}} = 0.057 \text{ SEC}$$

$3T_W = 0.17$ SEC $<$ $T^F_{D(NS)} = 0.19$ SEC <u>UNCOUPLED</u>

Figure B.4-10.2: Shear Wall Period, North-South Direction Earthquake, 490 Jarvis Drive Building

SHEAR WALLS PERIOD (EAST-WEST EQ)

USE 2 DEGREE OF FREEDOM MODEL FOR COMPARISON WITH ROOF PERIOD

$K_1 = 10 \times (5.42 \times 10^6 \text{ \#/IN}) = 54.2 \times 10^6 \text{ \#/IN}$

$K_2 = 10 \times (4.72 \times 10^6 \text{ \#/IN}) = 47.2 \times 10^6 \text{ \#/IN}$

$$m_1 = \frac{1}{386.4 \text{ IN/SEC}^2}\left[\frac{2005 \text{ PLF} \times 154'}{2 \times 0.186} + 10 \times 12094 \text{ \#}\right] = 2461.1 \text{ \#-SEC}^2/\text{IN}$$

$$m_2 = \frac{1}{386.4 \text{ IN/SEC}^2}\left[\frac{2342 \text{ PLF} \times 154'}{2 \times 0.186} + 10 \times 24750 \text{ \#}\right] = 3149.7 \text{ \#-SEC}^2/\text{IN}$$

$A = m_1 m_2 = 7.752 \times 10^6$

$B = m_1(K_1 + K_2) + m_2 K_1 = 4.203 \times 10^{11}$

$C = K_1 K_2 = 2.558 \times 10^{15}$

$$X = \frac{B - \sqrt{B^2 - 4AC}}{2A} = 6986$$

$\omega = 83.58$ RAD/SEC

$T_W = 0.075$ SEC

$3T_W = 0.225$ SEC $<$ ROOF $T^R_{D(EW)} = 0.59$ SEC <u>UNCOUPLED</u>

USE SINGLE DEGREE OF FREEDOM MODEL FOR COMPARISON WITH FLOOR PERIOD

$K = 47.2 \times 10^6 \text{ \#/IN}$

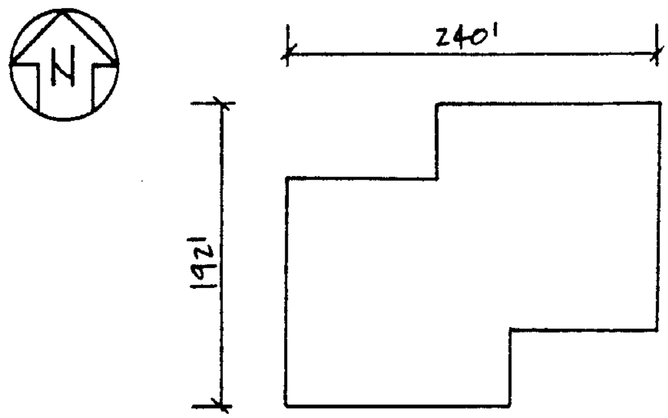
$m = 3149.7 \text{ \#-SEC}^2/\text{IN}$

$$T_W = 2\pi\sqrt{\frac{m}{K}} = \sqrt{\frac{3149.7}{47.2 \times 10^6}} = 0.051 \text{ SEC}$$

$3T_W = 0.15$ SEC $\approx T^F_{D(EW)} = 0.14$ SEC <u>ASSUME UNCOUPLED</u>

$\boxed{\text{DIAPHRAGMS AND WALLS IN BOTH DIRECTIONS ARE ASSUMED TO BE UNCOUPLED.}}$

Figure B.4-11: Shear Wall Period, East-West Direction Earthquake, 490 Jarvis Drive Building



NORTH-SOUTH DIRECTION EARTHQUAKE

STRUCTURAL COMPONENT	PERIOD (SEC)	Sa (g)	Sv (IN/SEC)	Sd (IN)
ROOF	0.45	0.42	12	0.88
FLOOR	0.19	0.72	8.8	0.27
END WALLS	0.076	0.50	2.4	0.029
CENTER WALLS	0.11	0.65	4.4	0.072

EAST-WEST DIRECTION EARTHQUAKE

STRUCTURAL COMPONENT	PERIOD (SEC)	Sa (g)	Sv (IN/SEC)	Sd (IN)
ROOF	0.59	0.32	12	1.2
FLOOR	0.14	0.72	6.4	0.15
END WALLS	0.075	0.50	2.4	0.029

Figure B.4-12: Summary of Structure Response for the 490 Jarvis Drive Building

NORTH-SOUTH DIRECTION EARTHQUAKE

ROOF

% OF TOTAL = 50

$$\theta_d = \frac{0.88''}{1/2 \times (120') \times (12''/1')} = 0.001222$$

DS1 : THRESHOLD = 0.00097 ; DF = 0.05
DS2 : THRESHOLD = 0.00400 ; DF = 0.50

$$\frac{DF - 0.05}{0.001222 - 0.00097} = \frac{0.50 - 0.05}{0.00400 - 0.00097} \qquad \underline{\underline{DF = 0.09}}$$

FLOOR

% OF TOTAL = 50

$$\theta_d = \frac{0.27''}{1/2 \times (240') \times (12''/1')} = 0.000188 < \text{THRESHOLD} = 0.00097 \qquad \underline{\underline{DF = 0}}$$

END WALLS

% OF TOTAL = 33

$$\theta_d = \frac{0.029''}{31.75' \times (12''/1')} = 0.000076 < \text{THRESHOLD} = 0.00050 \qquad \underline{\underline{DF = 0}}$$

CENTER WALLS

% OF TOTAL = 11

$$\theta_d = \frac{0.072}{31.75' \times (12''/1')} = 0.000189 < \text{THRESHOLD} = 0.00050 \qquad \underline{\underline{DF = 0}}$$

Figure B.4-13.1: Calculations of Damage Factors, North-South Direction Earthquake, 490 Jarvis Drive Building

GROUND FLOOR PARTITIONS

% OF TOTAL PARTITIONS = 35

$$\theta_d = \frac{0.5 \times 0.27''}{15.5' \times (12''/1')} = 0.000726 < \text{THRESHOLD} = 0.00283 \qquad \underline{\underline{DF = 0}}$$

SECOND FLOOR PARTITIONS

% OF TOTAL PARTITIONS = 15 (USE ROOF DEFLECTION, ASSUME ROOF AND FLOOR UNCOUPLED)

$$\theta_d = \frac{0.5 \times 0.88''}{14.25' \times (12''/1')} = 0.002573 < \text{THRESHOLD} = 0.00283 \qquad \underline{\underline{DF = 0}}$$

GROUND FLOOR CEILING

% OF TOTAL CEILING = 10

S_a = 0.72 g

DS3 : THRESHOLD = 0.5 g ; DF = 1.0 $\qquad \underline{\underline{DF = 1.0}}$

SECOND FLOOR CEILING

% OF TOTAL CEILING = 40

S_a = 0.42 g

DS2 : THRESHOLD = 0.4 g ; DF = 0.6

DS3 : THRESHOLD = 0.5 g ; DF = 1.0

$$\frac{DF - 0.6}{0.42 - 0.4} = \frac{1 - 0.6}{0.5 - 0.4} \qquad \underline{\underline{DF = 0.68}}$$

WINDOWS AT END WALLS

% OF TOTAL = 33

θ_d = 0.000076 < THRESHOLD = 0.01405 $\qquad \underline{\underline{DF = 0}}$

WINDOWS AT CENTER WALLS

% OF TOTAL = 11

θ_d = 0.000189 < THRESHOLD = 0.01405 $\qquad \underline{\underline{DF = 0}}$

Figure B.4-13.2: Calculations of Damage Factors, North-South Direction Earthquake, 490 Jarvis Drive Building

EQE

EAST-WEST DIRECTION EARTHQUAKE

ROOF

% OF TOTAL = 50

$$\theta_d = \frac{1.2''}{12(154')\times(12''/1')} = 0.001299$$

DS1: THRESHOLD = 0.00097 ; DF = 0.05
DS2: THRESHOLD = 0.00400 ; DF = 0.50

$$\frac{DF - 0.05}{0.001299 - 0.00097} = \frac{0.5 - 0.05}{0.004 - 0.00097}$$ DF = 0.10

FLOOR

% OF TOTAL = 50

$$\theta_d = \frac{0.15''}{12\times 154'\times(12''/1')} = 0.000162 < \text{THRESHOLD} = 0.00097$$ DF = 0

WALLS

% OF TOTAL = 56

$$\theta_d = \frac{0.029}{31.75'\times(12''/1')} = 0.000076 < \text{THRESHOLD} = 0.00050$$ DF = 0

Figure B.4-14.1: Calculations of Damage Factors, East-West Direction Earthquake, 490 Jarvis Drive Building

GROUND FLOOR PARTITIONS

% OF TOTAL = 35

$\theta_d = \frac{0.5 \times 0.15''}{15.5' \times (12''/1')} = 0.000403 < \text{THRESHOLD} = 0.00283$ $\underline{DF = 0}$

SECOND FLOOR PARTITIONS

% OF TOTAL = 15

$\theta_d = \frac{0.5 \times 1.2''}{14.25' \times (12''/1')} = 0.003509$

DS1 : THRESHOLD = 0.00283 ; DF = 0.06
DS2 : THRESHOLD = 0.00617 ; DF = 1.25

$\frac{DF - 0.06}{0.003509 - 0.00283} = \frac{1.25 - 0.06}{0.00617 - 0.00283}$ $\underline{DF = 0.30}$

GROUND FLOOR CEILING

% OF TOTAL = 10

$Sa = 0.72g$

SD3 : THRESHOLD = 0.50g ; DF = 1.0 $\underline{DF = 1.0}$

SECOND FLOOR CEILING

% OF TOTAL = 40

$Sa = 0.32g$

DS1 : THRESHOLD = 0.2 g ; DF = 0.1
DS2 : THRESHOLD = 0.4 g ; DF = 0.6

$\frac{DF - 0.1}{0.32 - 0.2} = \frac{0.6 - 0.1}{0.4 - 0.2}$ $\underline{DF = 0.40}$

WINDOWS

% OF TOTAL = 56

$\theta_d = \frac{0.029''}{31.75' \times 12''/1'} = 0.000076 < \text{THRESHOLD} = 0.01405$ $\underline{DF = 0}$

Figure B.4-14.2: Calculations of Damage Factors, East-West Direction Earthquake, 490 Jarvis Drive Building

TABLE B.4-1

BUILDING COMPONENT INVENTORY FOR THE 490 JARVIS DRIVE BUILDING

Group	Component Class	Replacement Value ($)	Percent of Total Value
Substructure	Structural Excavation	32,035	0.94
	Footings and Slab	375,562	11.02
		407,597	11.96
Superstructure	Tilt-up Panels	644,453	18.91
	Columns	100,877	2.96
	Roof Structure and Roofing	329,213	9.66
	Second Floor	329,213	9.66
		1,403,756	41.19
Architectural	Overhead Doors	42,940	1.26
	Windows	73,272	2.15
	Suspended Ceilings	123,710	3.63
	Interior Partitions	133,594	3.92
	Floor Finish	118,258	3.47
		491,774	14.43
Mechanical and Electrical	Roof Drain	14,994	0.44
	Sprinklers and Piping	126,096	3.70
	General and HVAC	401,122	11.77
	Plumbing	38,170	1.12
	Lights and Wiring	315,240	9.25
		895,622	26.28
Other	Grading and Paving	158,812	4.66
	Storm Drainage	13,973	0.41
	Site Work Concrete	36,466	1.07
		209,251	6.14
	TOTAL:	$3,408,000	100%

TABLE B.4-2

ESTIMATED BUILDING COMPONENT DAMAGE FACTORS FOR THE 490 JARVIS DRIVE BUILDING

Group	Component Class	Replacement Value ($)	Class	Demand	Damage Factor
Substructure	Structural Excavation	32,035	None	- -	0.00
	Footings and Slab	375,562	None	- -	0.00
Superstructure	Tilt-up Panels	644,453	B-10	33% @ 0.000076 11% @ 0.000189 56% @ 0.000076	0.00 0.00 0.00
	Columns	100,877	B-02	- -	0.00
	Roof Structures and Roofing	329,213	B-11	50% @ 0.001222 50% @ 0.001299	0.09 0.10
	Second Floor	329,213	B-11	50% @ 0.000188 50% @ 0.000162	0.00 0.00
Architectural	Overhead Doors	42,940	None	- -	0.00
	Windows	73,272	B-14	33% @ 0.000076 11% @ 0.000189 56% @ 0.000076	0.00 0.00 0.00
	Suspended Ceilings	123,710	B-16	10% @ 0.72g 40% @ 0.42g 10% @ 0.72g 40% @ 0.32g	1.00 0.68 1.00 0.40
	Interior Partitions	133,594	B-13	35% @ 0.000726 15% @ 0.002573 35% @ 0.000403 15% @ 0.003509	0.00 0.00 0.00 0.30
	Floor Finish	118,258	None	- -	0.00

TABLE B.4-3

ESTIMATED BUILDING COMPONENT REPAIR COSTS AND ACTUAL REPAIR COSTS FOR THE 490 JARVIS DRIVE BUILDING

Group	Component Class	Replacement Value ($)	Estimated Repair Costs ($)	Actual[1] Repair Costs ($)
Substructure	Structural Excavation	32,035	0	0
	Footings and Slab	375,562	0	0
Superstructure	Tilt-up Panels	644,453	0	0
	Columns	100,877	0	0
	Roof Structure and Roofing	329,213	31,275	10,889
	Second Floor	329,213	0	0
		1,811,353	31,275	10,889
Architectural	Overhead Doors	42,940	0	0
	Windows	73,272	0	0
	Suspended Ceilings	123,710	78,185	4,092
	Interior Partitions	133,594	6,012	4,419
	Floor Finish	118,258	0	0
		491,774	84,197	8,511
	TOTAL:	2,303,127	115,472	19,400

[1] This value is approximate.

TABLE B.4-4

COMPARISONS BETWEEN ESTIMATED DAMAGE FACTORS AND ACTUAL DAMAGE FACTORS FOR THE 490 JARVIS DRIVE BUILDING

Group	Replacement Value ($)	Estimated[1] Damage Factor	Actual[2] Damage Factor	Estimated D.F.[2] / Actual D.F.
Structural	1,811,353	0.01727	0.00601	2.9
Architectural	491,774	0.17121	0.01731	9.9
Total[3]	2,303,127	0.05014	0.00842	6.0

1 Damage Factor = Repair Cost / Replacement Value

2 The Actual Damage Factor is an approximate value.

3 Only the structural and architectural components are included.

Photo B.4-1
490 Jarvis Drive Building
Partial north elevation

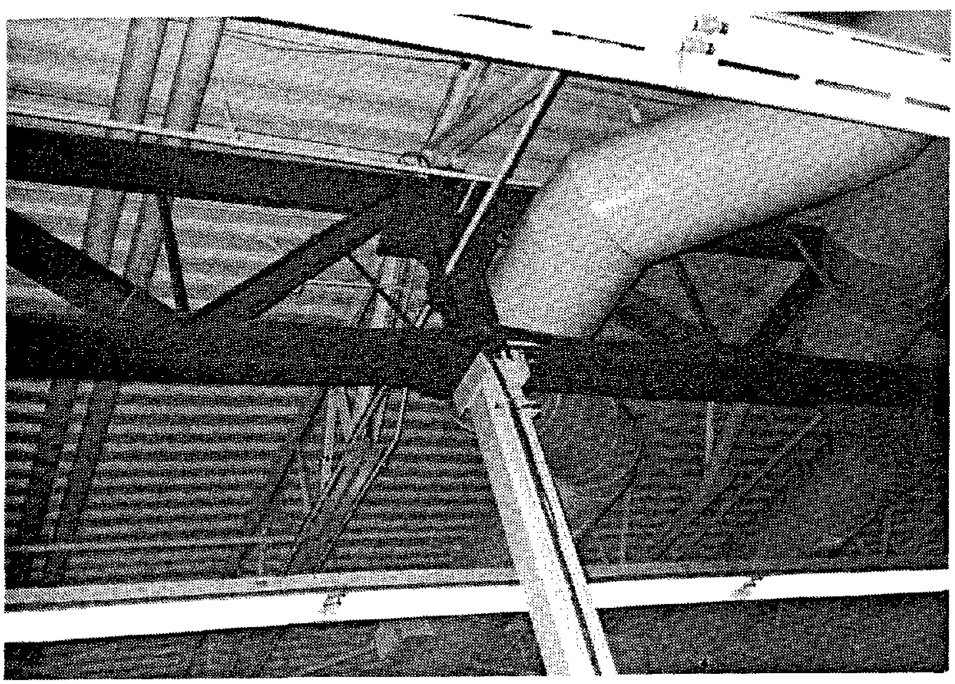


Photo B.4-2
490 Jarvis Drive Building
Second floor metal deck supported on open web joists and steel tube columns

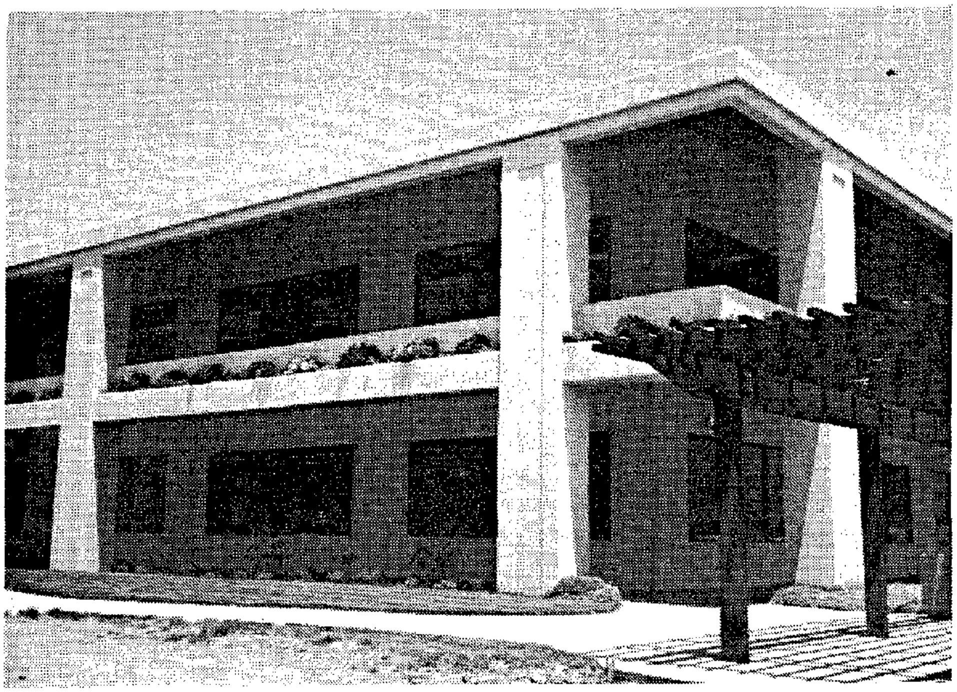


Photo B.4-3
490 Jarvis Drive Building
Typical perimeter concrete tilt-up wall panels with large window openings

Photo B.4-4
490 Jarvis Drive Building
Sloped clay tile roof and the concrete planter at the second floor level

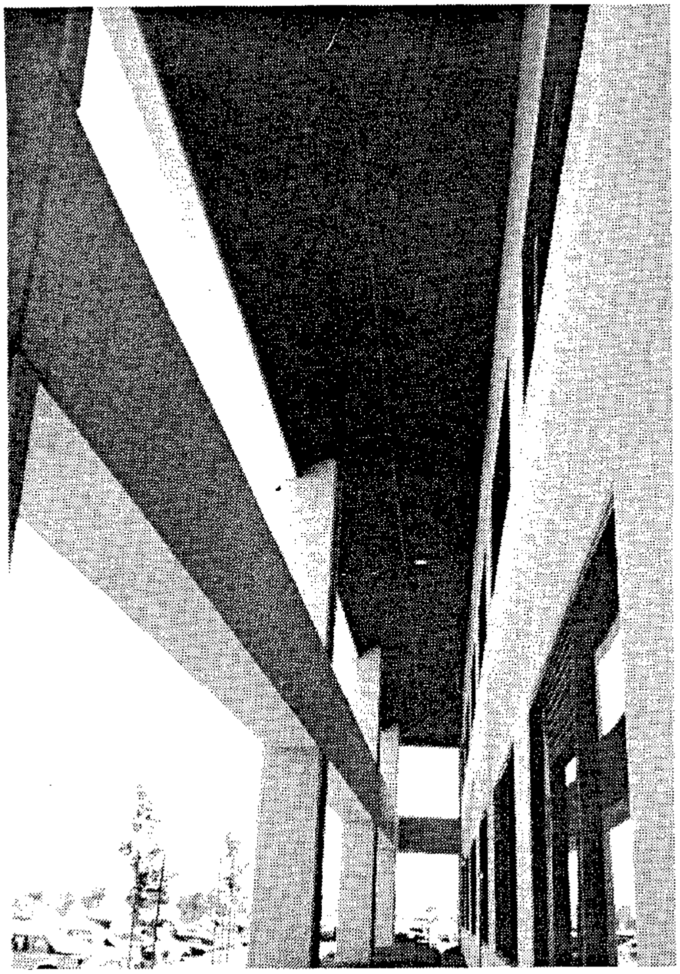


Photo B.4-5
490 Jarvis Drive Building
Concrete columns supporting the sloped clay tile roof and the concrete planter

Photo B.4-6
490 Jarvis Drive Building
General view of interior on second floor

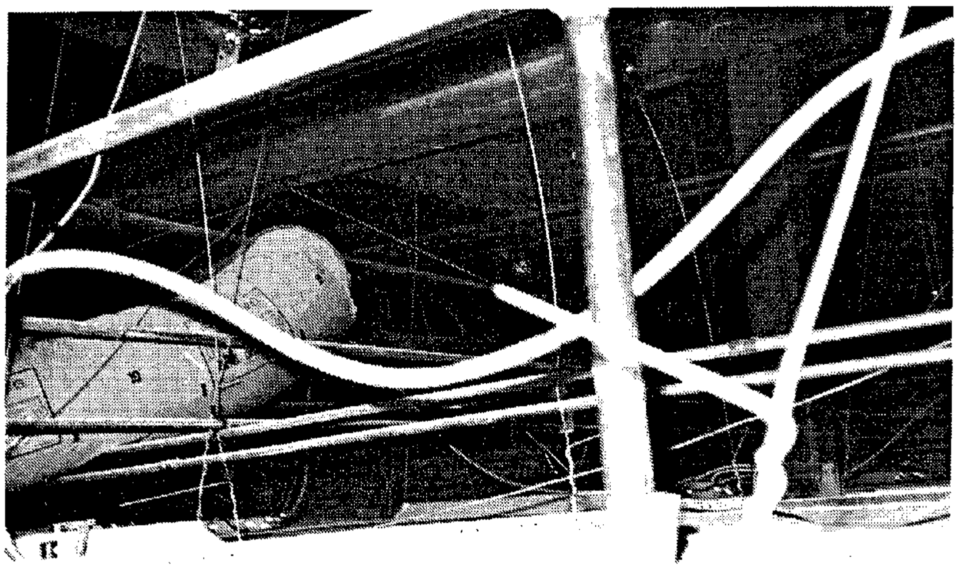


Photo B.4-7
490 Jarvis Drive Building
Wire hangers for suspended ceiling tiles on second floor

B.5 SUMMARY OF FINDINGS

B.5.1 Summary

The methodology for predicting damage to high-technology facilities was tested using two buildings which experienced minor damage during the April 24, 1984 Morgan Hill earthquake. The two buildings were occupied by Wiltron Company which manufactures microwave equipment. Only structural and nonstructural elements, identified in Tables 5 and 6 were treated. No attempt was made to evaluate equipment damage and or loss of revenue. The 120 Mast Street building is a one-story concrete tilt-up structure which is nearly rectangular in plan. The 490 Jarvis Drive building is a two-story concrete tilt-up structure with large window openings in its walls. It is irregular in plan, with two large re-entrant corners. Thus the two structures are a good cross-section representation of tilt-up structures. Both structures are within 15 miles of the epicenter and less than 5 miles from the fault. Empirical relationships were used to predict that the two facilities experienced peak ground accelerations between 0.25g and 0.30g.

During the earthquake, the roofing, interior partitions, and suspended ceiling of the 120 Mast Street building were damaged. The approximate repair cost for this damage was reported to be about $6500, or about 0.93% of the estimated total replacement cost for the building. This percentage should be modified for comparison purposes because the owner is a contractor who made repairs at minimal cost; the facility was vacated soon after the earthquake, expediting repairs. It is believed that the facility manager's preliminary cost estimate of $20,000 is reasonable. In either event, the structural and nonstructural damage was minor.

Application of the methodology resulted in a predicted damage of approximately 5.0% of the total replacement cost. This represents over 5 times the reported damage (and 1.7 times the facility manager's estimate). However, the prediction of only minor damage is in agreement with the actual performance, and in this sense, the methodology was successful. In addition, the three items which experienced damage are the same three items which were predicted to experience damage.

The 490 Mast Drive building experienced $19,400 in damage to structural and nonstructural components. As was the case with the 120 Mast Street building, the repair costs are associated with the roofing, suspended ceilings, and interior partitions. Again, the damage that occurred was very minor and represents only 0.84% of the structure's total replacement cost.

The methodology predicted that repairs amounting to approximately $115,000 (4.9% of the total replacement cost) would be required. This amount of damage is considered minor. Again the amount of damage predicted by the methodology is in agreement with the severity of the actual damage. Damage predictions were made for the roofing, interior partitions, and suspended ceiling: the same three components that actually experienced damage.

B.5.2 Conclusions

As a result of testing the developed methodology on two buildings that experienced damage during the Morgan Hill earthquake, the following was concluded:

- Estimation of peak ground acceleration at a near-fault site can introduce large uncertainties into the damage-prediction methodology. Empirical relationships are based on data which are inadequate for such sites. In addition, uncertainties were complicated further in this study because of the large amount of directionality or energy focusing which occurred during the Morgan Hill earthquake.

- The methodology is straightforward in application to single-story tilt-up structures with reasonably regular geometry. For two-story structures, its application requires judgment. The second floor is usually a concrete slab and, thus, its period is similar in magnitude to the periods of the shear walls. In such cases, simple decoupling is not always possible. For structures with irregular features, such as re-entrant corners, modeling decisions are necessary. The formulas provided in the report assume that the diaphragms can be modeled as simple beams. More complicated geometries would require the use of more complex equations.

- Both of the selected buildings experienced very minor structural and nonstructural damage (less that 1% of the replacement cost). The methodology predicted that minor damage, approximately 5% of the replacement cost, would occur. In this sense, the methodology was successful.

- The structural and nonstructural components which the methodology predicted would experience damage did, in fact, sustain damage to some degree. This indicates that the initial damage state thresholds (Damage State 1) for the damaged items (roof structure and roofing, partitions, and suspended ceilings) are approximately correct.

- The methodolgy is general enough to allow for incorporation of new data and information. New damage-state thresholds or formulas for predicting ground accelerations can be included without changes in procedure.

B.5.3 Recommendations

Recommendations to further test and improve the methodology are made as follows:

- Performance data that become available from past and future earthquakes should be incorporated into the methodology.

- The methodology should be further tested. The ideal structures for further study would be buildings which experienced moderate or major damage on sites for which fairly accurate maximum peak ground accelerations are available. The effectiveness of the methodology can be better evaluated if significant damage has occurred to a structure.

- The component approach should be used by facility managers and government officials to help determine expected direct and indirect losses in the event of a catastrophic earthquake.

B.6 REFERENCES

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