TECHNICAL MANUAL

SEISMIC DESIGN GUIDELINES FOR UPGRADING EXISTING BUILDINGS

DEPARTMENTS OF THE ARMY, THE NAVY, AND THE AIR FORCE
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SEISMIC DESIGN GUIDELINES FOR UPGRADING EXISTING BUILDINGS

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FOREWARD

This manual on seismic upgrade design guidance for existing buildings was developed as a sequel to the manual for new construction, TM 5-809-1/NAVFAC P-355.1/AFM 88-3, Chap 13, Sec. A Seismic Design Guidelines for Essential Buildings. This manual meets one of the objectives of the EARTHQUAKE HAZARDS REDUCTION ACT of 1977 (Publication 95-124), i.e., to provide improved seismic design and construction requirements to upgrade existing buildings in seismic areas.

This manual provides the seismic design upgrade concepts for existing buildings based on the state-of-the-art methodology and past practices by the triservices (Army, Navy and Air Force) and the private sector. It includes the dynamic-analysis approach similar to new construction concepts with some modifications/tolerances on the acceptance criteria. A methodology for screening, evaluating and prioritizing seismically hazardous buildings from a large number of buildings in a military installation and the Navy's Rapid Seismic Analysis Procedure are provided. For existing buildings in seismic zones 1 and 2, a static code procedure is provided to evaluate and upgrade resistance to collapse in the event of a major earthquake, as required by the code provision.

The general direction and development of this manual were under the supervision and guidance of the Office of the Chief of Engineers, Headquarters, Department of the Army, Washington, D.C. Assistance was provided by the Headquarters, Naval Facilities Engineering Command, Department of the Navy, Washington, D.C. and the Directorate of Engineering and Services, Headquarters, Department of the Air Force, Washington, D.C.

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CHAPTER 1

GENERAL

1-1. Purpose

This manual prescribes criteria and furnishes design guidelines, procedures, and strategy to screen, prioritize, evaluate, upgrade, and strengthen existing facilities for seismic resistance. These criteria apply to all elements responsible for the design of military construction in the high seismic regions and will apply to all existing facilities in Seismic Zones 3 and 4, to only existing essential facilities in Seismic Zone 2, and to other facilities designated by the approving agency. These guidelines also provide procedures and guidance for engineers to identify seismically hazardous buildings and to determine the strengthening method to resist the required seismic forces. This manual is a supplement to TM 5-809-10/NAVFAC P-355/AFM 88-3, Chapter 13, referred to herein as the Basic Design Manual (BDM) and TM 5-809-10-1/NAVFAC P-355.1/AFM 88-3, Chapter 13, Section A, referred to herein as the Seismic Design Guidelines (SDG).

1-2. Scope

These guidelines encompass a strategy and method to identify potential seismically hazardous buildings on a priority basis. The guidelines include a step-by-step procedure involving building inventory reduction; preliminary screening; preliminary evaluation; detailed structural analysis; development of design concepts for seismic upgrading/strengthening; cost benefit analysis; final design and preparation of contract documents; and seismic upgrading/strengthening of nonstructural elements. The problems relating to earthquake-induced ground failures and tsunami are stated in the BDM, paragraph 2-7, and will not be covered in this manual. Authorization from HQDA(DAEN-ECE-D) WASH, DC 20314-1000, NAVFAC Code 4BA 200 Stovall Street Alexandria, VA 22332, or HQ USAF/LEEE WASH, DC 20332 is required for the application of the procedures in this manual.

1-3. Definitions, symbols, and notations

Unless otherwise noted in this manual, all definitions, symbols, and notations will be as indicated in chapter 3 of the BDM. Symbols and notations are listed in appendix A.

1-4. Seismic hazard risk levels

The evaluation and upgrading of existing buildings is based on seismic ground motions of two risk levels as specified in chapter 3 of the SDG.

a. The selected risk levels of the two design earthquakes, EQ-I and EQ-II, are based on Department of Defense standards; however, the risk levels may be revised as warranted by approval authorities.

b. As an alternate, the code provisions provided in appendix C may be used for high risk or nonessential buildings in high seismic regions as warranted or deemed appropriate by approval authorities.

1-5. Identification of seismically hazardous buildings

The military has a large inventory of buildings, and an effective strategy method is required to identify potentially hazardous buildings on a priority basis. The objective of this strategy/method is to minimize unnecessary investigations by eliminating buildings of minor importance and low hazard exposure from the large inventory, identifying groups of similar buildings, and prioritizing seismic safety evaluation and hazard mitigation (strengthening) efforts. Since the basic goals of seismic hazard mitigation for existing buildings are to enhance life safety (i.e., protection against collapse) and post-earthquake operational capability, it is essential to identify buildings with post-earthquake operational requirements or high risk (high-loss potential) functions.

a. The essential buildings with post-earthquake operational requirements are:

(1) Hospitals

(2) Fire stations, rescue stations, and structures housing vehicles essential for post-earthquake rescue and relief operations.

(3) Power stations and other utilities required as emergency facilities.

(4) Mission essential facilities. The decision to designate a building as "mission essential" is the responsibility of the operating Command. Since it may be possible to pick up the function of an entire Base at other locations, the decision to designate a structure as mission essential should be confirmed at the major command level or higher.

(5) Primary communications or data-handling facilities. (Some of these may be mission essential, but this category is not limited to mission essential.)

(5) Primary communications or data-handling facilities. (Some of these may be mission essential, but this category is not limited to mission essential.)

(6) Facilities involved in operational missile control, launch, tracking, or other critical defense capabilities.

(7) Facilities involved in handling, processing, or storing sensitive munitions, nuclear weaponry, gas and petroleum fuels, and chemical or biological contaminants.

b. High-risk (high-loss-potential) buildings are those whose primary occupancy is for assembling a large number of people, or where services are provided to a large area having many other buildings. Buildings in this category may suffer damage in an earthquake, but are recognized as warranting a higher level of safety than an ordinary building. Typical examples are:

(1) Buildings whose primary occupancy is that of an auditorium, recreation facility, dining hall, or commissary, any of which may have an occupancy of more than 300 persons.

(2) Confinement facilities.

(3) Central utility facilities (power, heat, water, sewage) that are not required as emergency facilities and that serve large areas.

(4) Buildings housing valuable equipment whose justification is provided by the using agency.

c. All other buildings are considered nonessential, ordinary buildings of lesser importance which will require the life safety provision, i.e., against collapse, unless a higher upgrade is warranted by approving authorities.

d. Hazardous critical facilities (e.g., nuclear power plants, dams, and LNG facilities) are not included within the scope of this manual, but are covered by other publications or regulatory agencies. For any facilities housing hazardous items not covered by criteria, advice should be sought from DAEN-ECE-D (Army), NAVFAC Code 04BA (Navy), or HQ USAF/LEEE (Air Force).

1-6. Background

a. Seismic design criteria. In recent years, developments in earthquake engineering have resulted in substantial changes in seismic design criteria. In the 1960's, major changes began to occur in the seismic design codes. In 1966, the first edition of the "Seismic Design for Buildings" was introduced (TM 5-809-10/NAVDOCKS P-355/AFM 88-3, Chapter 13, March 1966). In 1973, a new revised and expanded edition of the manual was published (TM 5-809-10/NAVFAC P-355/AFM 88-3, Chapter 13, April, 1973) which included ductility provisions for moment resisting space frames. In the February, 1982 edition (i.e., the BDM) substantial changes were made in force levels and seismic detailing requirements. Many of these changes were in response to experiences from the 1971 San Fernando, California earthquake. In the late 1970's, areas in the United States outside of California and the Pacific Coast area began to be aware of the need for earthquake-resistant design requirements for their facilities. In 1978, "Tentative Provisions for the Development of Seismic Regulations for Buildings" was published by the National Bureau of Standards (NBS SP-510; Applied Technology Council, ATC 3-06; and National Science Foundation, 78-8). These provisions were developed through a nationwide effort to improve seismic design and construction building practices and are currently being evaluated by a national committee. In addition to the static force approach used in codes and manuals, there was a need for a dynamic analysis approach to seismic design for essential buildings. In 1986, the Tri-Services published the SDG to provide guidelines for the design of essential buildings, as well as other structures, by means of a two-level dynamic analysis procedure.

b. Existing buildings. Major changes in structural criteria based upon building failures in past earthquakes naturally raise the question of the adequacy of existing buildings. A building designed and constructed prior to the recent changes in seismic design criteria, especially those in high seismic areas, will probably not conform to the requirements of today's criteria. In some cases, the general structural system does not conform, and there are some cases where the new lateral force levels can be 3 or more times greater than forces used in the original design. This does not necessarily mean that all these buildings are unsafe or will not be able to perform adequately when subjected to a major or moderate earthquake. Some of the older buildings may actually perform better than new ones that conform to the latest provisions. Many of the performance capabilities of buildings depend on configuration, details, and ability to act in a tough, ductile, energy absorbing manner rather than on conformance to the minimum standards of the code provisions.

c. Evaluation and upgrading. Current codes are developed for new construction and are not necessarily applicable to existing buildings. An existing building should be evaluated on the basis of its actual performance characteristics, as best as they can be determined, when subjected to a realistic postulated earthquake. Modifications of existing buildings shall take into account the performance characteristics of the existing materials interact-ing with the new materials used to upgrade the structure.

1-7 Methodology for seismic evaluation and upgrading existing facilities

The various steps in the methodology which are outlined below and graphically in figure 1-1, are presented in detail in the following chapters of this manual. It should be noted that the methodology as shown is applicable to a military installation with a large inventory of buildings. The approval authority may direct the omission of one or more steps in the methodology. For example, for an installation with a limited number of buildings (e.g., 25 or less) that are in use, the inventory reduction may not be required. If only essential buildings at a given facility are to be considered for seismic upgrading, the inventory reduction, preliminary screening, and preliminary evaluation may be omitted and the upgrading evaluation would directly begin with the detailed structural analysis, with or without the cost/benefit analyses.

a. Inventory reduction (chapter 2). Prior to beginning the phased seismic evaluation procedure, the overall inventory of the installation is reviewed to select buildings that will be included in the evaluation program. The purpose of reducing the total inventory to a select group is to eliminate unnecessary investigations and to keep the scope of work within reasonable limits.

b. Preliminary screening (chapter 3). A site survey is made to visually inspect all the buildings on the select inventory list. A screening process is used to reduce the number of buildings that require the preliminary evaluation.

c. Preliminary evaluation (chapter 4). A structural analysis of each selected building from the preliminary screening is made using simplified techniques. The purpose of the evaluation is to estimate the vulnerability of the buildings (i.e., damage when subjected to site specific seismic ground motion) and to establish a priority listing for more detailed structural analysis.

d. Detailed structural analysis (chapter 5). Buildings are selected for the detailed analysis on the basis of the priority listing resulting from the preliminary evaluation or by direct request by the authorized agency. The purpose of the detailed structural analysis is to determine if the existing building will satisfy the acceptance criteria, to identify deficiencies, and, if required, to recommend alternatives for seismic upgrading.

e. Development of design concepts for seismic upgrading (chapter 6). On the basis of the detailed structural analysis, methods of seismic strengthening are studied. A general concept is developed as recommended in the detailed structural analysis for seismic upgrading. In some cases, an alternate concept may be included.

f. Cost-benefit analysis (chapter 7). The costs of seismic upgrading are compared to the risk of doing nothing and to the costs of a new building. An evaluation may also be made for various levels of rehabilitation in comparison to the risk of future damage. The results of the cost-benefit analysis will be used for setting priorities in relation to other buildings.

g. Final design and preparation of contract documents (chapter 8). The proposed upgrading concepts will be used as a basis for the development of the final design for seismic upgrading. The final design will include a complete analysis of the modified building to confirm the adequacy of the strengthening measures in accordance with the detailed structural analysis procedure. Contract documents will include drawings and specifications.

h. Nonstructural elements (chapter 9). A qualitative evaluation is made on the basis of available documents and an on-site inspection. Elements identified as being susceptible to damage are subjected to a detailed analytical evaluation by a static or dynamic approach. Recommendations for seismic upgrading are made if required.

i. Evaluation of existing structural materials (appendix E). Where necessary data or information of the existing materials are not available, the materials and structural elements will be tested. Testing procedures and methods for materials and structural elements are provided.

1-8. References and bibliography

Publications that are referenced in the text and are required reading for use of this manual are listed in appendix B. Publications for suggested reading are listed in the bibliography.

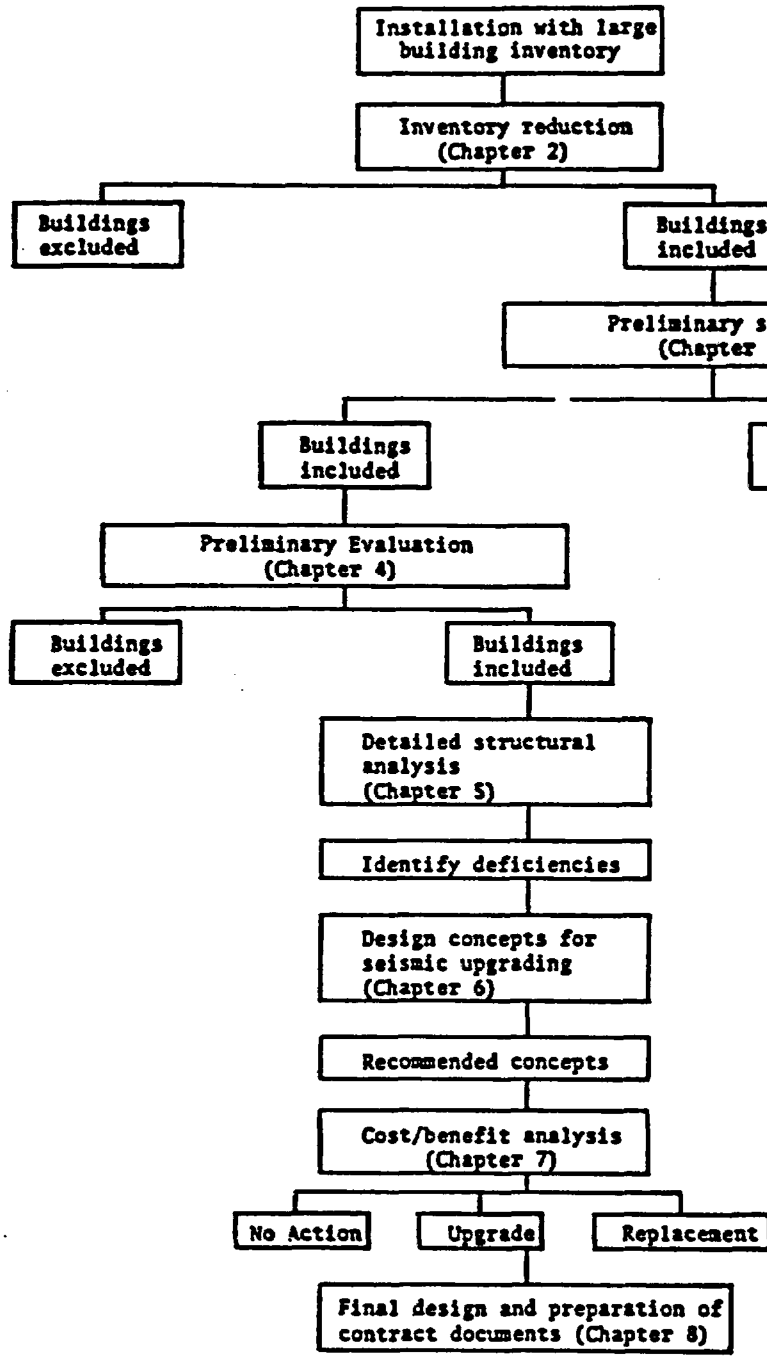


Figure 1-1. Methodology for seismic evaluation and upgrading

CHAPTER 2

INVENTORY REDUCTION

2-1. Introduction.

Generally military installations have a large inventory of existing buildings and the potential high cost of an engineering investigation of all of the buildings located in high seismicity areas makes it necessary to identify seismically hazardous buildings in a carefully planned manner. The first step in dealing with a large inventory of existing buildings is to apply some type of screening process to reduce the inventory and to eliminate unnecessary investigation. Certain categories of existing buildings represent an acceptable level of risk. Criteria for reducing the number of buildings in the inventory to be investigated are given below. However, this does not preclude the investigation of any of these buildings if the responsible agency directs that they remain in the inventory to be investigated.

2-2. Availability of building inventory.

The military maintains a central inventory of real property which is the basic source of information on the status, cost, capacity, condition, use, maintenance, and management of its installations. In addition to specific information regarding the installation, the real property inventory contains a detailed record for buildings and facilities, including information such as type of construction, building/facility number, total area, total capacity, total acquisition cost, year built or acquired, and number of floors.

2-3. Building inventory reduction.

To expedite identification of the more important buildings in the military real property inventory, a procedure was developed to screen the inventory for a given installation or group of installations in a particular high seismicity region. The screening procedure, which is graphically outlined in figure 2-1, utilizes the following criteria for the exclusion of buildings to reduce the inventory. An example is given in appendix F, figure F-1, sheet 1 of 12.

a. Buildings, except essential buildings, that were designed in accordance with the provisions of the 1982 Basic Design Manual (BDM) or equal (e.g., 1976 Uniform Building Code (UBC), or to more stringent requirements).

b. Buildings located in seismic zone 0.

c. One-story wood-frame and one-story preengineered metal buildings, except essential or high risk buildings.

d. Buildings occupied by no more than 5 occupants, except essential or high-risk buildings.

e. One- and two-family housing, two stories or less.

f. Buildings, except essential or high-risk buildings, of no more than 500 square ft or with a replacement cost of less than $50,000.

g. Structures scheduled for replacement within 5 years.

h. Modifications and additions to the above factors authorized by the approval agency.

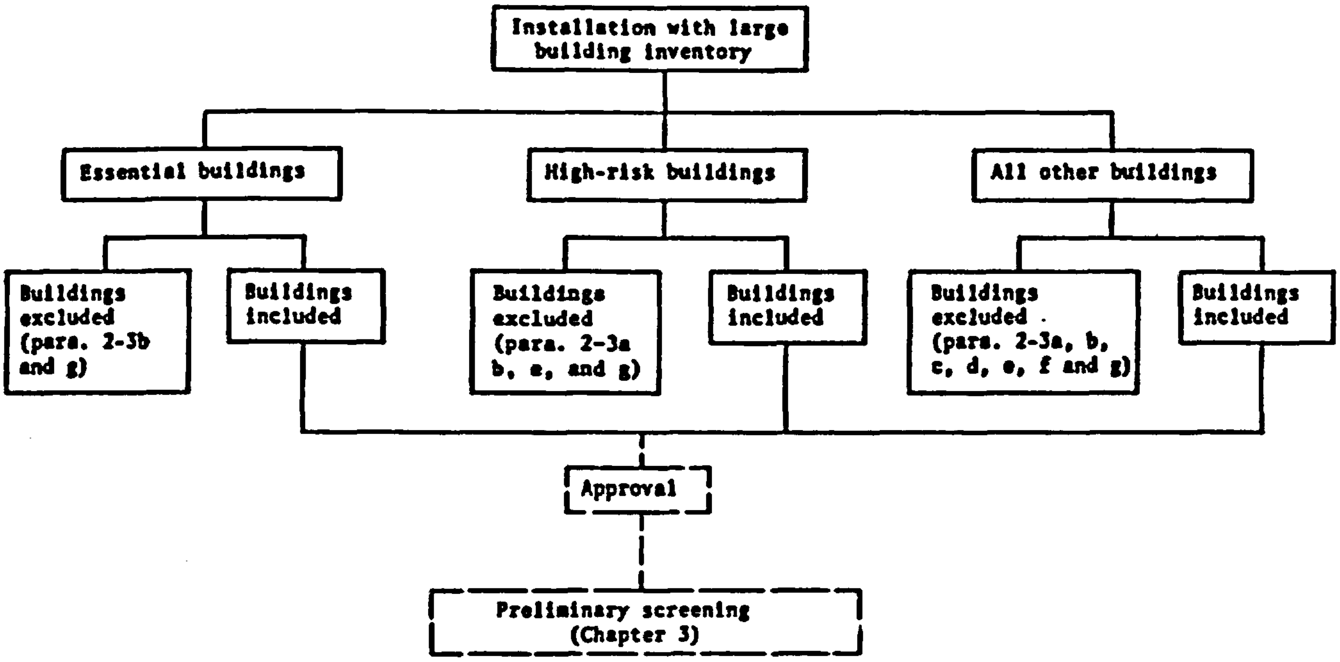


Figure 2-1. Methodology for inventory reduction

CHAPTER 3

PRELIMINARY SCREENING

3-1. Introduction

This chapter describes the general procedures for the preliminary screening of existing buildings. Guidelines are presented for the preliminary screening process to classify and categorize the buildings and criteria are provided for screening of buildings from further consideration.

3-2 Preliminary screening process

Preliminary screening will be used after inventory reduction only if there is a need to further reduce the number of structures to be evaluated. A flow chart is shown in figure 3-1.

a. Classification. The buildings remaining after the inventory reduction will be classified as essen-

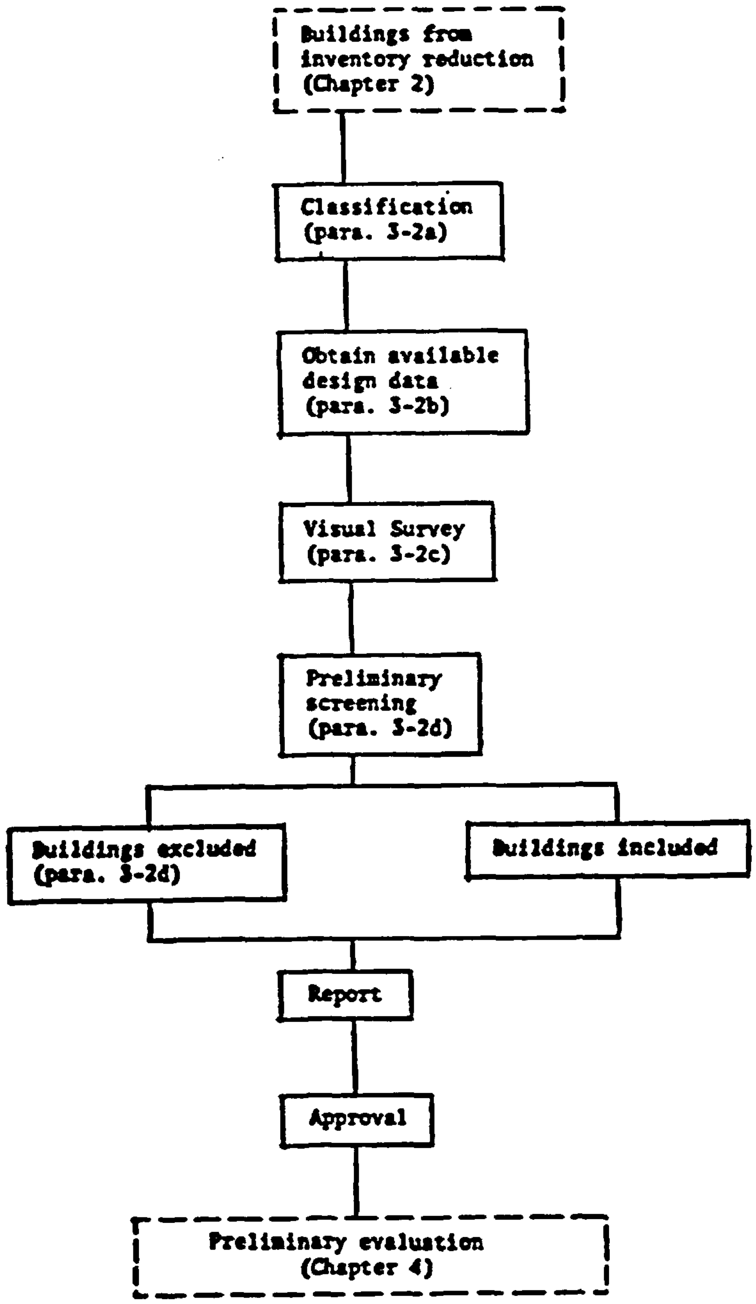


Figure 3-1. Methodology for preliminary screening

tial, high-risk, or all others in accordance with paragraph 1-5. Classification of the buildings will be provided by the using agency or will be performed by the engineer in collaboration with the using agency. The function of the building will be noted by the engineer during the site visit for the preliminary screening and any apparent discrepancy in the classification will be resolved with the using agency.

b. Available design data. The engineer will obtain available design data (e.g., drawings, design criteria, calculations, and specifications). Data pertaining to the "as-built" condition of a building are essential when available. The engineer will notify the using agency so that the assembly of selected available building data can be transferred to the engineer prior to the site visit. These data and information will be reviewed by the engineer and the pertinent information will be transferred to the screening form used in the review process. It is expedient to transfer as much data as possible to the forms. An example is shown in appendix F, figure F-1, sheet 4. When the design data are minimal or if none is available, such as may be for the older buildings, it will be noted on the screening form so that sketches with pertinent dimensions, sizes, and other notes regarding the structural systems can be made during the preliminary screening inspection. Older buildings are more likely to have undergone structural revisions and additions. Indications of such revisions and additions will be noted and confirmed. Data may require revision during the field inspection.

c. Field survey. The purpose of the inspection survey will be to obtain general data regarding each building to facilitate the preliminary screening process. These data will include building identification number, title, general function, size, general structural type (i.e., wood, concrete, steel frame, etc.), general condition, and other pertinent data. The screening forms, such as shown in appendix F, figure F-1, sheets 4 and 5, are used to establish a check list for the visual observations to aid field note taking. The inspection survey need not be detailed. The time allotted for each building will vary, depending on the size and complexity of the structure, but should be between 10 and 30 minutes. A more detailed examination will be made during the preliminary evaluation as described in chapter 4.

d. Screening. The field notes will be systematically reviewed to determine the number of buildings that will remain on the list for the preliminary evaluation process. Justifications for removing buildings from the list include:

(1) Buildings that upon further evaluation are determined to fall within the intent of the inventory reduction criteria of chapter 2.

(2) Buildings of obviously inferior construction or whose structural condition has deteriorated to the point where upgrading is not feasible or cost effective. For this condition the engineer may recommend a course of action. As an example, a building with severe foundation problems, such as extreme ground settlement that resulted in footing or pile damage, may require a nonseismic evaluation to determine if the building should be demolished or repaired.

(3) Buildings that are essentially identical to structures remaining on the list for further evaluation. The site inspection may indicate that groups of buildings are similar or essentially identical. In this case, one building may be selected to represent all the buildings in a group. The other buildings are then placed on hold with the decision for further evaluation dependent on the results of the analysis of the representative building. However, each building must be inspected for any serious deficiency, damage and changes to warrant a separate category outside the group.

3-3. Report

A report will be prepared to summarize the results of the preliminary screening. The report will include the following items.

a. Description of the screening process.

b. Description of screening criteria.

c. Description of each building surveyed, including classification, contents, general structural type, condition, and available design data.

d. Results of screening such as which buildings require analysis and those that were eliminated from further evaluation with justification for the elimination. Identify those eliminated buildings similar to ones that are to have further evaluation. Recommendations on course of action for those buildings eliminated from further seismic evaluation.

e. Provide a plot plan, if information is available, to locate buildings included in inventory, identifying buildings eliminated from further evaluation and those that remain on the list for the preliminary evaluation.

f. Summary table that includes building classifications, structural categories, comments, and recommendations.

CHAPTER 4

PRELIMINARY EVALUATION

4-1. Introduction

This chapter describes the general methodology for the preliminary evaluation of existing buildings using a rapid seismic analysis. The methodology is used when there is a need to establish a priority listing for detailed evaluation and seismic upgrading of a group of buildings vulnerable to seismic ground motion. The simplified techniques provide estimates of the seismic vulnerability/damage for a group of buildings at a fraction of the costs for detailed evaluations. When it has been determined by the preliminary screening or other actions that a building requires a detailed structural analysis, the preliminary evaluation will not be required and the methodology for upgrading will proceed directly to the procedures described in chapter 5. The methodology for preliminary evaluation is summarized in figure 4-1.

a. General procedure. The procedure described in this chapter provides general guidelines for preliminary evaluation, establishing priorities for upgrading, and preparing a report. An example of

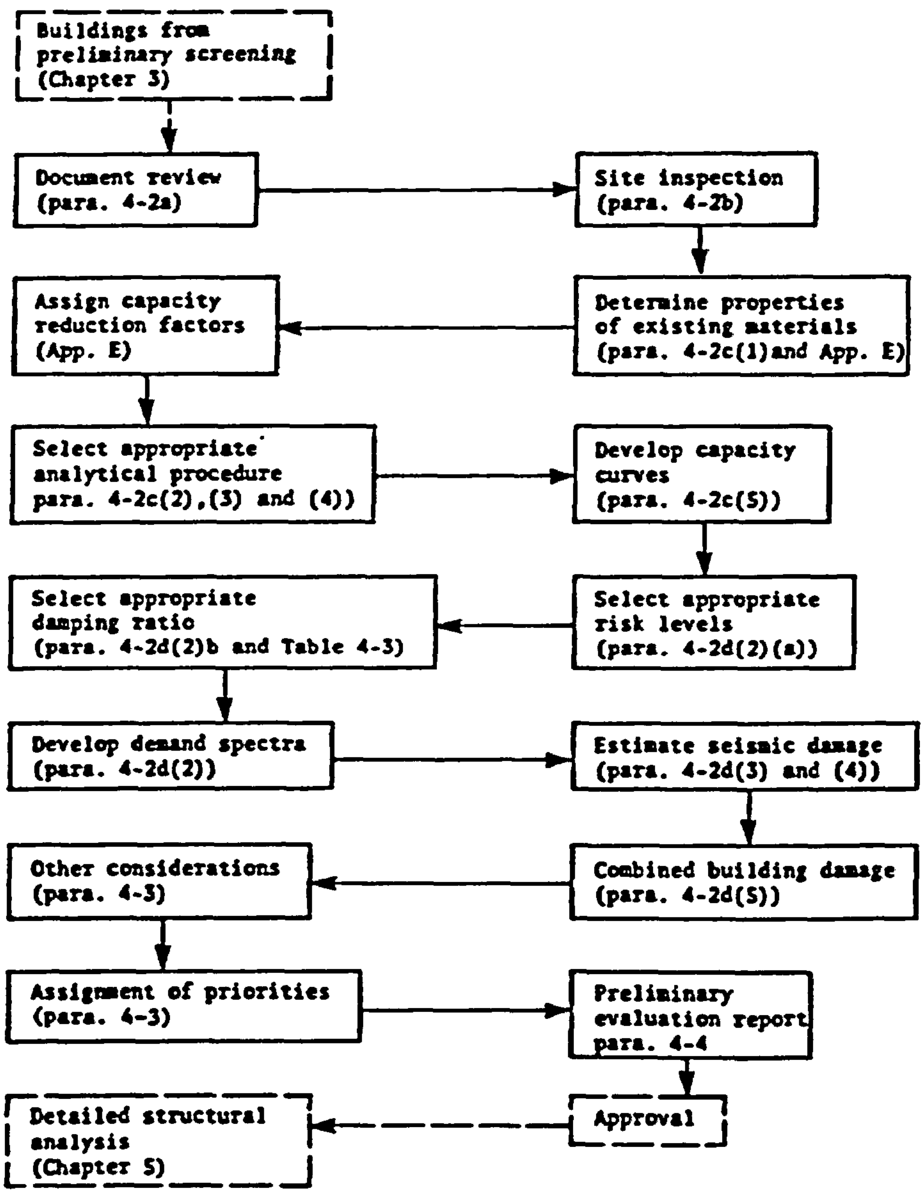


Figure 4-1. Methodology for preliminary evaluation

a preliminary evaluation is given in appendix F, figure F-1, sheet 6.

b. Naval Facilities Engineering Command Rapid Seismic Analysis Procedure (RSAP). The RSAP, which is summarized in appendix D, is a variation of the procedures described in this chapter. For example, it provides empirical formulas for determining natural periods and capacity characteristics, reduction factors to adjust the ultimate site demands, and computer programs for computing damage estimates. These variations may not be applicable to all buildings. Engineering judgment and experience will be used in conjunction with the RSAP to determine rational building structural capacities and demands.

4-2. Preliminary evaluation

The preliminary evaluation provides the initial analytical data for estimating the vulnerability of the selected buildings to seismic damage. This is an important consideration in determining priorities for upgrading within each building classification (i.e., essential, high-risk, all others). When a preliminary evaluation is prescribed, the following basic steps are performed: document review, site inspection, approximation of the capacity of the structure to resist seismic forces, approximation of damage by reconciliation of the structural capacity with the earthquake ground motion demands, and recommendations. The document review and site inspection may not be required if all pertinent data and conditions have been obtained in the preliminary screening process. Generally, it will be done concurrently if the number of buildings is not large.

a. Document review. The available drawings, calculations, specification, and other design documents obtained from the using agency will be reviewed by the engineer to identify the lateral force resisting system and other pertinent information. The information will be summarized in a format that can be used during the site inspection to serve as a checklist.

b. Site inspection. A field examination of each building will be performed to determine the condition of the structural elements and to evaluate its lateral force resisting system. Observations will also be made on nonstructural elements, occupancy level, and value of contents. This inspection will be done in a more detailed manner than the preliminary screening inspection described in chapter 3. The inspection team will include at least one structural engineer experienced in structural evaluation. Also, personnel familiar with the building, such as the building supervisor or building engineer, should accompany the inspection team to provide access to the various areas within the building, describe functional requirements and point out any known areas of damage, deterioration, and modification. The site inspection will normally take one to two hours; however, the time can increase greatly with the complexity and condition of the building. Advance notice will be required to arrange for inspection of buildings with restricted access.

(1) Information obtained from the document review will serve as checklists during the inspection.

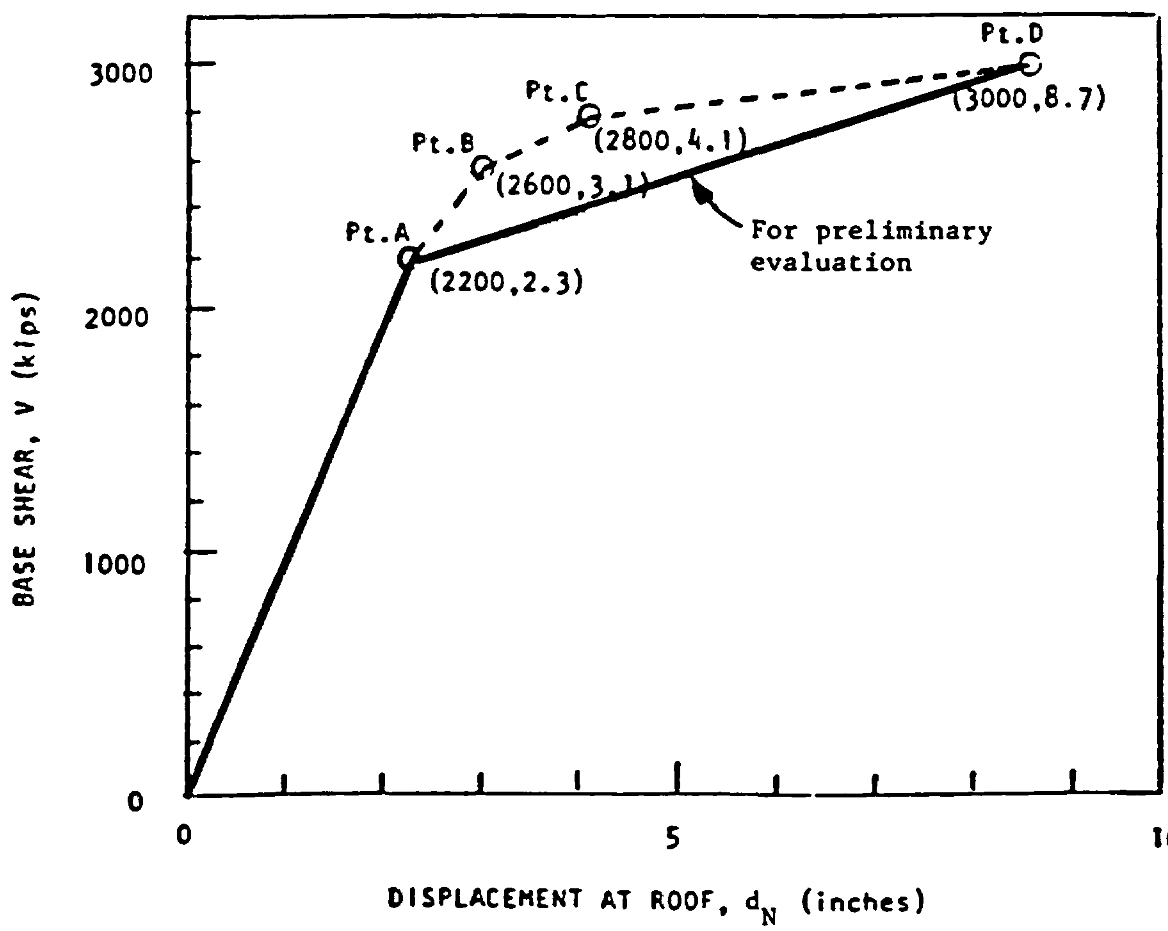
(2) The structural and nonstructural elements of the entire building will be examined from the outside and inside, to the maximum extent practicable. In open buildings such as warehouses and machine shops, the structural elements are fairly well exposed. In closed-in buildings, such as offices and hospitals, structural elements are generally hidden from view by partitions, furred walls, and hung ceilings; therefore, it may be necessary to lift ceiling tiles and go into concealed spaces, closets, mechanical rooms, and other locations where structural elements are likely to be exposed. Except for critical cases, representative samples will be used to establish building characteristics during the preliminary evaluation. Estimates will be made on the normal number of occupants in the building and the costs of the contents in the building.

(3) The existing lateral force resisting system will be confirmed by the on-site inspection. The various load paths by which lateral forces are transferred from the roof and floor systems to the frames or shear wall systems and to the foundations will be determined. Appropriate documentation of any discontinuity in the load paths or weaknesses in the structural connections, and any evidence of redundancy or back-up systems and modifications or additions to the building will be made.

c. Capacity of the structure. The value for capacity is a simplified representation of the capacity of the overall building for a specified level of stress or distortion such as when yielding of major structural members occur or when lateral displacements reach a prescribed limit. On the basis of the available documents and the visual observations, the capacity of the structure to resist lateral forces will be estimated by means of a rapid evaluation technique. For a large group of buildings, the evaluations should average less than one day per building. More time may be spent on a representative building or structural system in order to obtain data that will expedite the analysis of other similar buildings or systems (e.g., see para (3) below). For smaller groups of buildings where there is a large diversity of building types, the average time of the evaluation may be longer. For some buildings that are large or complex a detailed evaluation is required (e.g., see last sentence of para (2) below). For the rapid evaluation tech nique, the capacity is represented by a curve similar to the capacity curve required for method 2, capacity spectrum method, in paragraph 5-5*b* of SDG, except that only the points representing initial major yielding and ultimate strength (near collapse) are required. An example of a capacity curve, a modification of SDG figure 5-5, is shown in figure 4-2. General guidelines for determining the capacity curve are given below. An example is given in appendix F, figure F-1, sheets 6 through 12.

(1) Determination of the lateral force capacity of a structure will include consideration of all elements, structural and nonstructural (e.g., see para (2) below), that contribute to the resistance of lateral forces. Physical properties are generally obtained from existing available data, otherwise assumptions and/or tests must be made. Guidelines for determination of the physical properties of representative structural and nonstructural materials are provided in appendix E of this manual. The analysis must include the evaluation of the most rigid elements resisting the initial lateral distributions, as well as the more flexible elements that resist the lateral distortions after the rigid elements yield or fail. Consideration must also be given to the interaction of various combinations of the structural framing systems and elements which will contribute to the resistance of the lateral loads.

(2) The capacities are generally determined by manual calculation methods. The methods used will vary for different types of buildings and lateral force resisting systems. For shear wall systems, a shear capacity is assumed and an adjustment is made for the flexural capacity that is dependent on the height-to-depth ratio of the piers. For reinforced concrete frame structures, a similar procedure is used to relate the capacity due to flexure of the columns to the capacity due to shear strength in the columns. For steel mill type buildings, the capacity of the steel frame is



Modified from SDG fig 5-5

Figure 4-2. Force-displacement capacity curve dependent on the fixity of the column base. For braced frames, L/r and connections are generally the critical items. The horizontal diaphragm system will be evaluated for its capacity to transfer lateral forces to the vertical resisting elements. If the structure consists of a structural steel frame and nonstructural infill brick walls, the brick must yield and then fail before the steel frame acts. If a system consists of nonstructural brick wall elements and structural steel X-bracing elements, both systems of elements will work until the brick fails, and then the X-bracing will take the load until it fails. For some buildings, because of size or complexity, approximate manual calculations may not be adequate to establish reliable capacities for lateral loads. In these cases, either the procedure described in paragraph (3) below shall be used or the building shall have the detailed structural analysis described in chapter 5.

(3) In some cases, a more detailed analysis may be made for one building (or part of a building) that is representative of several other buildings. The results can be extrapolated to analyze other buildings constructed similarly. For example, there may be many multi-storied reinforced concrete warehouse type buildings with flat slabs, drop panels, and column capitals for interior framing and reinforced concrete walls with large window openings for exterior framing. Computer runs using simplified idealized models can be made to establish guidelines for lateral stiffness characteristics and stress distributions of typical frames of a representative building. The results can be extrapolated to analyze the other buildings with this warehouse type of construction. A similar procedure can be used to establish guidelines for multi-leveled, high-low roof, mill type steel buildings.

(4) The results of the capacity evaluation are expressed in at least two of the following three terms that represent the overall building:

(a) *The base shear coefficient (C_B).* C_B is equal to the base shear capacity divided by the effective weight of the building (V/W). C_B is analogous to the ZIKCS of the BDM, except that C_B represents a capacity value and ZIKCS represents a design value.

(b) *Lateral displacement at the top of the structure (d_N).* d_N is the displacement at the top of the building resulting from the application of the lateral forces associated with C_B.

(c) *Fundamental period of vibration of the structure (T) that is consistent with the values of C_B and d_N.* T is generally calculated from a Rayleigh method of analysis such as formula 3-3 of the BDM; however, in some cases, a value may be assumed based on empirical data.

(5) Two capacities are required for each of the principal directions of the building. One capacity represents initial major yielding (e.g., point A in fig 4-2) and the other represents the ultimate strength or near collapse state (e.g., point D in fig 4-2) of the lateral force resisting system. The C_B, d_N, and T values are used to determine spectral acceleration (S_a) and spectral displacement (S_d) as described in SDG capacity spectrum method, paragraph 5-5*b*, and illustrated in table 4-1.

(6) A table is used to summarize the capacities of a large group of buildings.

d. Damage estimates. A graphical reconciliation between the earthquake demand (site response spectrum) and the building capacity is used to estimate the amount of damage that will occur during a postulated earthquake. The procedure is essentially the same as the Capacity Spectrum Method prescribed in SDG paragraph 4-4*d* and described in SDG paragraph 5-5*b*. In the SDG, the objective is to determine if the building will remain functional during EQ-II and to approximate the lateral deformations (para 4-4*d* (7)). In the evaluation procedure, the objective is to estimate the damage ratio for EQ-II.

(1) *Capacity spectrum.* When the capacity is plotted in terms of V and d_N, as shown in figure 4-2, it is similar to the force-displacement curves used to represent strength of materials; however, instead of plotting the results of a single test element, the curve represents the global capacity of the overall structure. When the V and d_N are converted to S_a and S_d, the shape of the curve remains similar, but the units change. S_a is essentially proportional to V, with some variations due to story mass distribution and modal participation factors and S_d is essentially proportional to d_N. The capacity curve can also be expressed in terms of S_a and T and S_d and T, as shown in figure 4-3. The conversion process is shown in table 4-1. A plot of S_a (acceleration) vs S_d (displacement) for the overall building as illustrated in figure 4-3 is analogous to a force (mass times acceleration) vs displacement curve for a structural material. The secant stiffness of the S_a vs S_d curve is analogous to the secant modulus of the stresses and strains representing a force-displacement curve. T, which represents the mass and stiffness characteristics of the structure, is approximated by using the secant stiffness of the S_a vs S_d curve.

(2) *Demand spectra.* The demands of earthquakes are represented by response spectra. Response spectra are obtained by using procedures described in SDG, chapter 3. They are usually plotted in terms of S_a and T (e.g., see SDG fig 2-8) or on a log-log-log tripartite curve which gives values for S_a, S_d, S_v (spectral velocity) and T, as

Table 4-1. Calculation of S_a and S_d capacity values

Point	V (kips)	d_N (in)	V/W	PF_N	α	S_a (g)	S_d (in)	T (sec)
A	2200	2.3	0.22	1.30	0.78	0.280	1.77	0.80
D	3000	8.7	0.30	1.26	0.83	0.361	6.90	1.40

V/W: V = Base shear, W = Weight = 10,000 Kips

d_N = Lateral roof displacement due to V

PF_N = $(\Sigma m\phi)(\phi_N)/(\Sigma m\phi^2)$, modal roof participation factor

α = $(\Sigma m\phi)^2/(\Sigma m)(\Sigma m\phi^2)$, effective modal weight

S_a = Spectral acceleration = $V/W \div \alpha$

S_d = Spectral displacement = $d_N \div PF_N$

T = $2\pi\sqrt{S_d/(S_a)(g)}$, fundamental period of vibration

$\Sigma m\phi$ = Summation of story mass times mode shape factor from the roof to the base of the building

From SDG Table 5-4

shown in figure 4-4. From the tripartite plot, data can be obtained to plot the curve in terms of S_d and T and S_a and S_d, as shown in figure 4-5. These relationships are consistent with the dynamic analysis formula:

$$S_d = S_a (T/2\pi)^2 g \qquad \text{(eq 4-1)}$$

where g is the acceleration of gravity and S_a is expressed in units of g.

(*a*) *Risk level.* The site response spectra for this evaluation will be representative of ground motion for EQ-II as defined in the SDG.:

(*b*) *Damping.* A set of damping values for each building, as a percentage of critical damping, will be determined from table 4-2. These damping values will be used to define the response spectra representing the ground motion described in paragraph (*a*), above.

(3) *Capacity versus demand.* The capacity curve and the demand curve are plotted on the same graph. Their intersection is considered to be the reconciliation between demand and capacity. The demand is represented by two curves: one represents elastic damping for periods less than the elastic period of the building and the other represents damping at the ultimate capacity. A transition line is drawn connecting the two curves between the elastic period and the ultimate capacity period. The procedure is identical to Method 2, Capacity Spectrum Method, of the SDG, except that the preliminary evaluation uses a more approximate capacity curve. An example, which is a modification of SDG figure 5-6, is shown in figure 4-6.

(4) *Percent damage.* To estimate the amount of damage a building experiences from an earthquake, damage must first be defined. Until the yield capacity of the structure is reached, damage is assumed to be zero. When the ultimate capacity is reached, damage is assumed to be 100 percent. For intermediate values of capacity, the assessment of damage is necessarily somewhat subjective and depends on many factors not amenable to

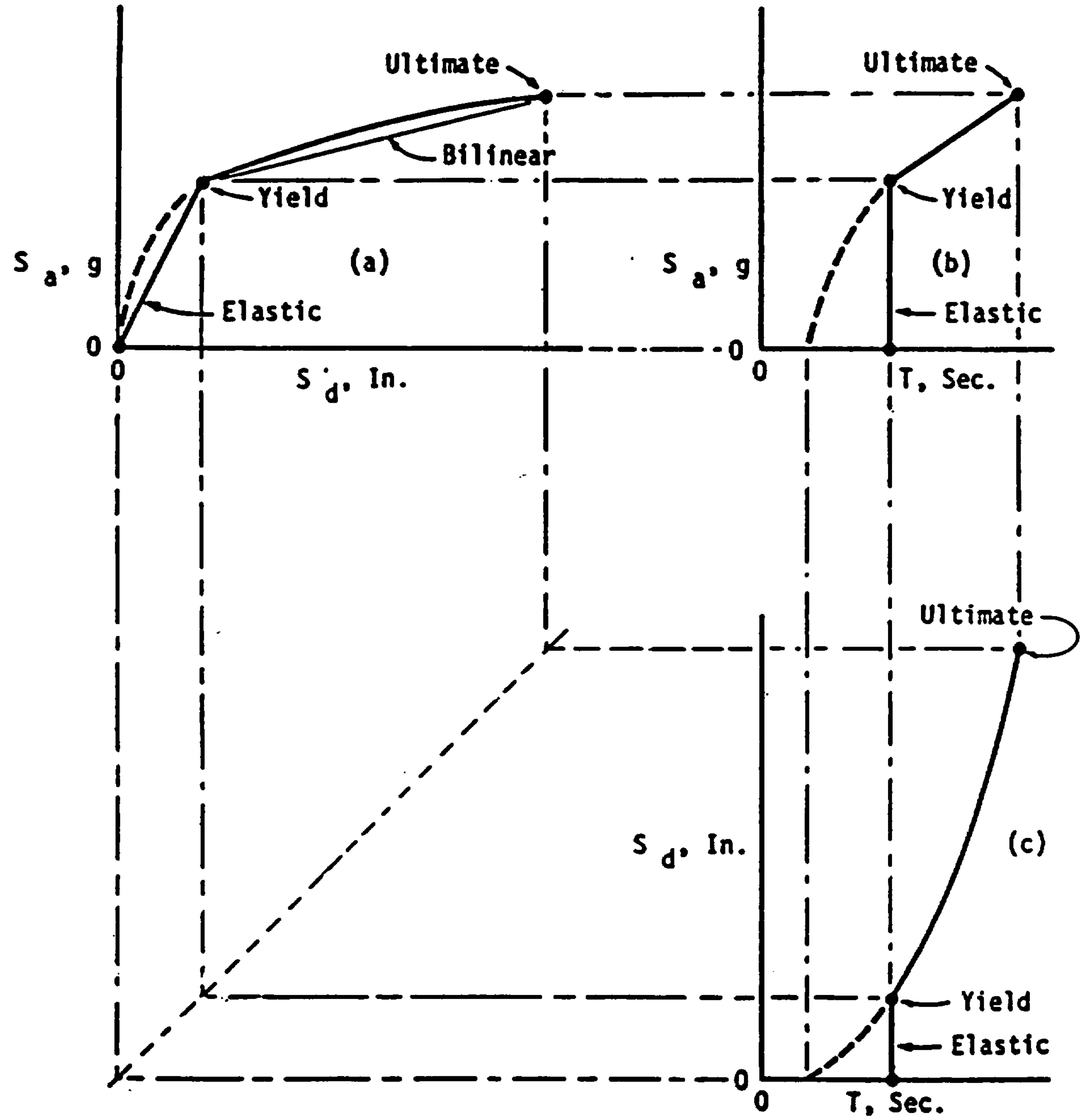


Figure 4-3. Capacity spectrum curves

analytical treatment. In lieu of a better alternative, it is assumed that damage varies linearly between the yield limit and the ultimate limit. The increase in damping beyond the elastic limit can be related to the effects of increased internal energy absorption (e.g., hysteresis loops) and the nonlinear effects (e.g., reduction in harmonic amplification) of the building response. In lieu of a better alternative, damping is also assumed to increase linearly between the yield limit and the ultimate limit values. The percent of damage is estimated from the graphical solution by taking the ratio of the length between the damage reconciliation point and the yield point (length d) to the length between the yield point and the ultimate capacity (length c), as shown in figure 4-6. This procedure is done for each of the principal directions of the building.

(5) *Combined building damage estimate.* For each building, the damage is computed for each of the principal directions of motion, longitudinal and transverse. To determine the combined damage for the two directions, it is assumed that one-third of the building depends on each direction of lateral resistance and the remaining one-third depends on both directions for lateral resistance. That is, if a structural element required for both directions for lateral resistance is damaged by one direction of

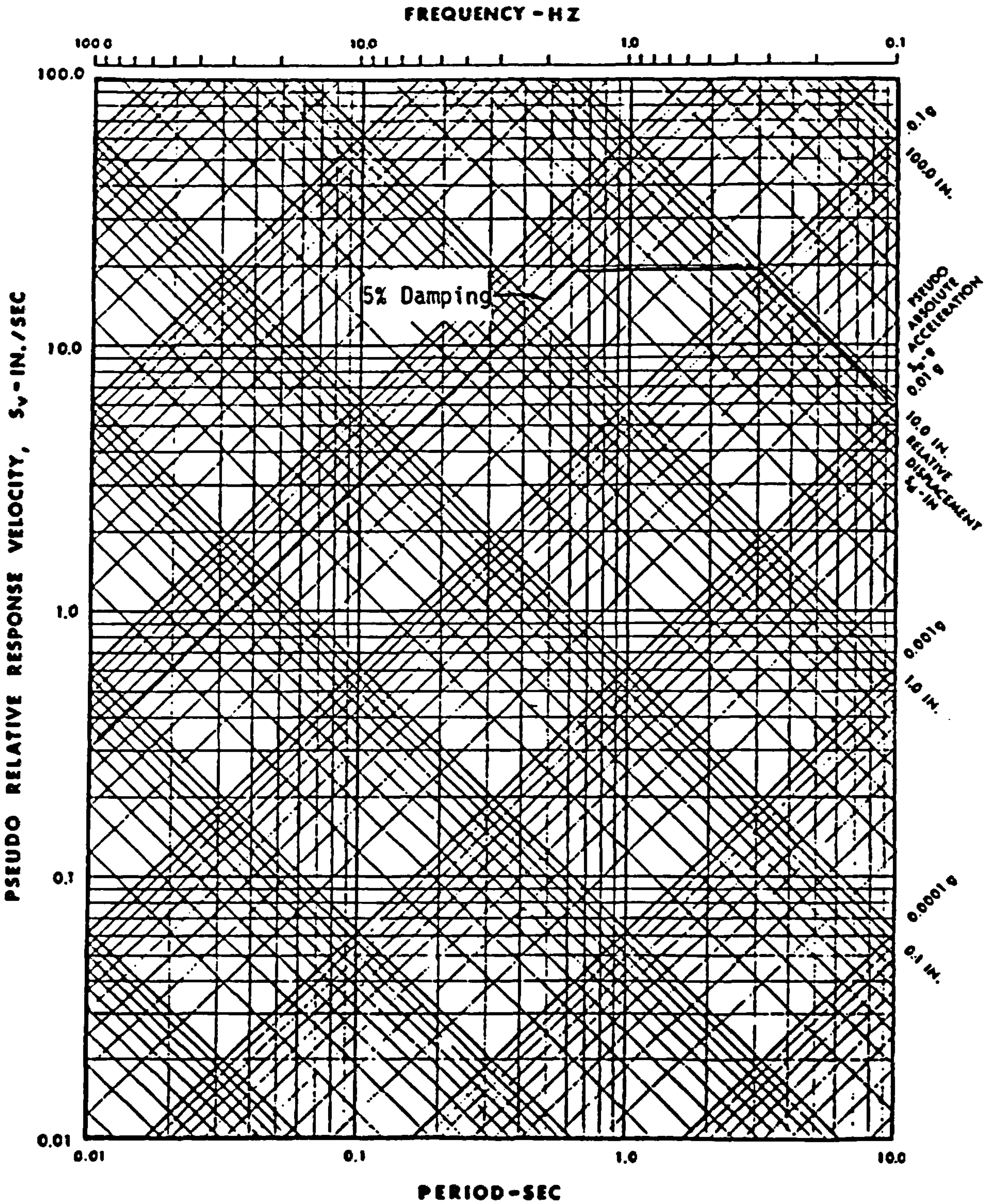


Figure 4-4. Tripartite plot of response spectra shown in SDG figure 2-8

motion, it is also damaged in the other direction. The procedure takes two-thirds of the damage of the more critical direction and one-third of the damage of the other direction to determine the combined damage. For instance, if the damages are 60 percent and 30 percent in the two principal directions, the combined damage is 50 percent.

(6) *Damage vs earthquakes.* The procedure requires damage evaluation for ground motion represented by EQ-II as prescribed in paragraph 4-2d(2)(a). Damage estimates can also be determined, with little additional effort, for smaller

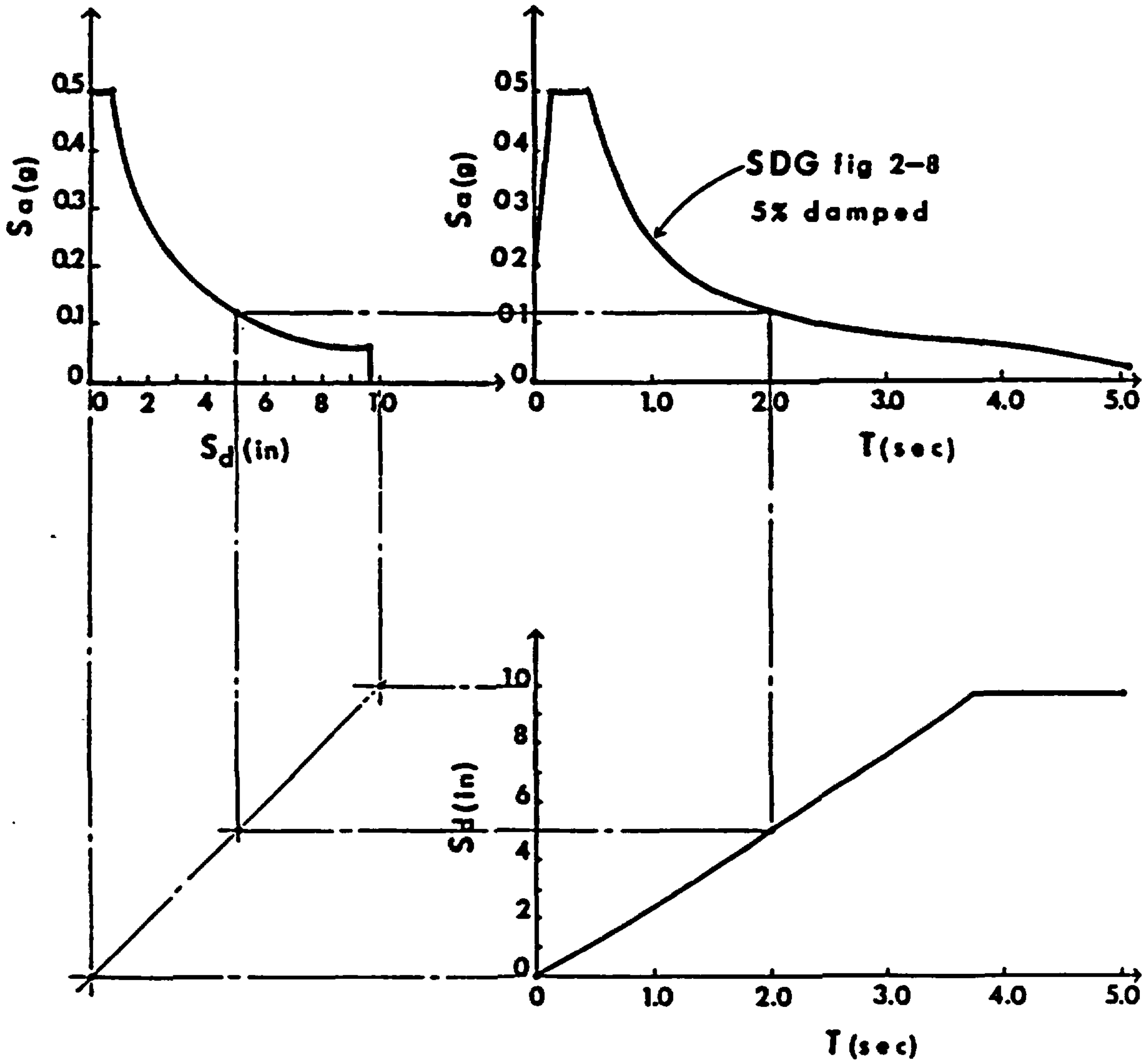


Figure 4-5. Response spectrum plotted in terms of S_d vs T and S_a vs S_d

earthquakes that have a higher probability of occurring at the site. For example, these smaller earthquakes may include response spectra representing ½ and/or ¾ the amplitudes of EQ-II. Sample results are shown in appendix F, figure F-1, sheet 12. Damage estimates for these smaller earthquakes may be necessary in some cases to aid in establishing upgrading priorities between the various buildings. For example, Building A may be very sensitive to the size of the earthquake such that for EQ-II it has 100 percent damage, but for ¾ of EQ-II it has no damage. Building B is estimated to have 80 percent damage at the EQ-II demand, 60 percent damage at ¾ of EQ-II, and 20 percent damage at ½ of EQ-II. In this example, for any earthquake up to ¾ of EQ-II, Building A performs better than Building B (no damage compared to up to 60 percent damage). Only in the event of EQ-II does Building B perform better than Building A (80 percent damage vs. 100 percent damage). Thus, a conclusion may be made that Building B has a higher priority for upgrading than Building A.

e. Results of the preliminary evaluation. The results for all the buildings will be summarized in tabular formats for ease of comparison. An example is shown in appendix F, figure F-1, sheet 12. These tabulations show estimated damage in terms of percentages. If replacement cost data are available, such as from the inventory data base, damage can be shown in dollar costs.

4-3. Priorities for upgrading

The cost associated with a building's vulnerability to seismic damage within each building classification is an important economic consideration in the assignment of priorities for seismic upgrading. Other considerations must also be evaluated in the

Table 4-2. Damping values of structural systems

Structural System	Elastic-Linear	Post Yield
Structural Steel	3%	7%
Reinforced Concrete	5%	10%
Masonry Shear Walls	7%	12%
Wood	10%	15%
Dual Systems	(1)	(2)

1. Use the value of the primary, or more rigid, system. If both systems are participating significantly, a weighted value, proportionate to the relative participation of each system, may be used.

2. The value for the system with the higher damping value may be used.

From SDG Table 4-1

assignment of priorities when limited funds are available. These considerations will include: potential damage to building contents (e.g., a relatively inexpensive warehouse structure may contain very expensive electronic equipment that could be seriously damaged by failure of the building); importance of the function performed by the building to the mission of the installation; number of occupants normally within the building; redundancy (i.e., are there viable alternatives for performance of the function if the building is lost? For example, an urban area may have three or more buildings that perform an essential function, but the temporary loss of function of one building could be offset by the services of the others). The relative weighting of each of the above considerations is somewhat subjective and may vary from one installation to another and therefore should be established in collaboration with a designated representative of the using agency.

4-4. Report

A report will be prepared to summarize the results of the preliminary evaluation. The report will include the following items.

a. Description of preliminary evaluation process.
b. Observation on nonstructural elements, occupancy, and contents.
c. Prioritization criteria and weighting factors used.
d. Criteria for determining which buildings require further analysis and for justifying no further analysis.
e. Method of analysis for each structural type.
f. Description of each building analyzed including lateral force resisting system, assumed structural properties, etc.
g. Copies of the capacity spectra with the graphical estimation of structural damage.
h. Prioritization of buildings within each classification.
i. Results of analyses that include building damage assessments, list of buildings requiring further analysis, and list of buildings not requiring further analysis.

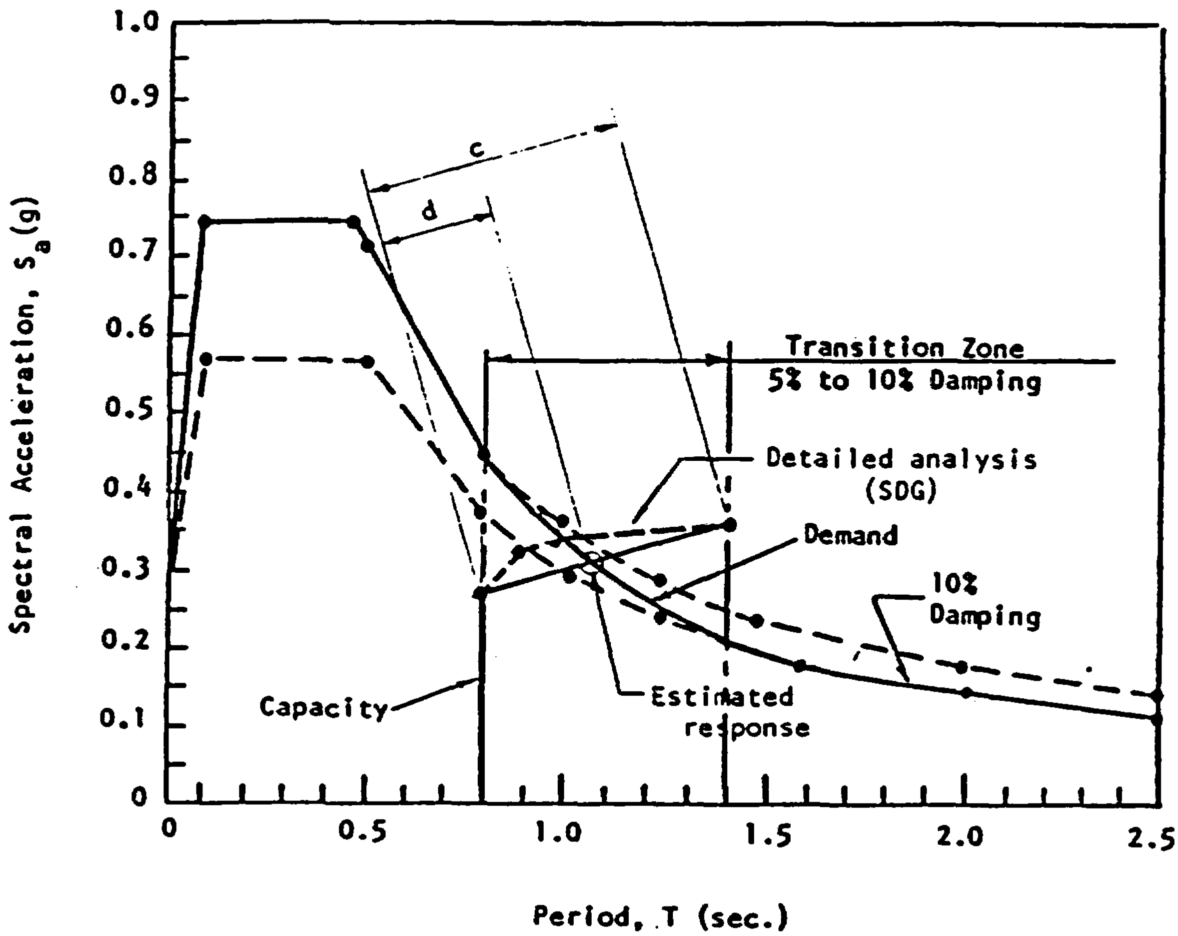


Figure 4-6. Capacity spectrum method for preliminary evaluation

CHAPTER 5

DETAILED STRUCTURAL ANALYSIS

5-1. Introduction

This chapter prescribes acceptance criteria and describes general procedures for detailed structural analysis of existing buildings. Guidelines are provided for determining the capacity of the existing structure to resist seismic forces. The detailed analysis is performed on buildings that have been selected as a result of the evaluation and/or priorities (chapter 4) established by the approval authority or on buildings as directed by higher authority. The purposes of the detailed structural analysis are to determine if the building satisfies the acceptance criteria or if it requires seismic upgrading, and if it requires seismic upgrading to identify the deficiencies and to recommend alternatives for the upgrading (chapter 6). The methodology for the detailed structural analysis is summarized in figure 5-1.

5-2. Acceptance criteria

The acceptance criteria for the seismic resistance of existing buildings will be essentially as prescribed for the post-yield analysis for EQ-II in paragraph 4-4 of the SDG. If an existing building does not conform to the above criteria some latitude is provided in the following paragraphs in recognition that seismic upgrading is an expensive and disruptive process and it may be more cost-effective to accept an existing building that is marginally deficient rather than to enforce strict adherence to the criteria.

a. Conforming systems and materials. When the lateral force resisting structural systems and materials are in compliance with the requirement of the BDM (Refer to BDM paragraph 3-6 for approved structural systems and to BDM chapters 3, 5, 6, 7, and 8 for material requirements), the earthquake demand represented by the EQ-II response spectra may be reduced by a maximum of 15 percent (i.e., to 0.85 EQ-II) and the drift limitations for EQ-II will remain the same as prescribed in SDG paragraph 4-4*e*(2)(*a*) (i.e., story drift ratio 0.010 for essential and 0.015 for others).

b. Nonconforming systems and materials. When the lateral force resisting system or the structural materials do not conform to the approved systems and material specifications of the BDM, justification for acceptability of the existing systems and/or materials is required. Requirements for substantiated data are prescribed below. Acceptance of the approval agency is also required.

(1) Structural systems not specified in the BDM and/or SDG (e.g., "nonductile" moment resistant reinforced concrete frames and unreinforced masonry shear walls) require an analytical evaluation report. The report will include data for establishing the capacity of the system to resist seismic loads and justification for the performance of the system satisfying the intent of the BDM and SDG provisions.

(2) Structural materials not satisfying the minimum requirements of the BDM and SDG require an evaluation report. Guidelines are provided in appendix E.

(3) The acceptance criteria for the substantiated noncomplying structural systems and materials are the same as prescribed in paragraph *a*, above, except that the drift will not be allowed to exceed 60 percent of the drift limits prescribed for conforming systems and materials.

c. Alternative acceptance criteria. In lieu of the above acceptance criteria, at the option of the approval authority, the acceptance criteria for the seismic resistance of specific existing buildings, namely other than essential buildings in seismic zones 3 and 4, may be satisfied by conformance with the provisions of the BDM or the Static Code Procedure of appendix C.

5-3. Methodology for the analysis

The detailed structural analysis follows a procedure similar to that used for the preliminary evaluation for determining the capacity of the structure to resist seismic loads, except that the analysis is done in greater detail and with more accuracy in order to increase the reliability of recommendations for acceptability or upgrading. The procedure extends beyond the scope of the preliminary evaluation by identifying deficiencies and evaluating the effects of correcting deficiencies to improve the overall performance capabilities of the building.

a. Document review. Available drawings, calculations, specifications, and other design and/or construction documents will be reviewed in detail for pertinent information that will aid in the detailed structural analysis. Items not covered by the available documents and required to complete a detailed analysis will be investigated during the site inspection.

b. Site inspection. A detailed site examination will be performed to confirm data contained in the available design and construction documents and

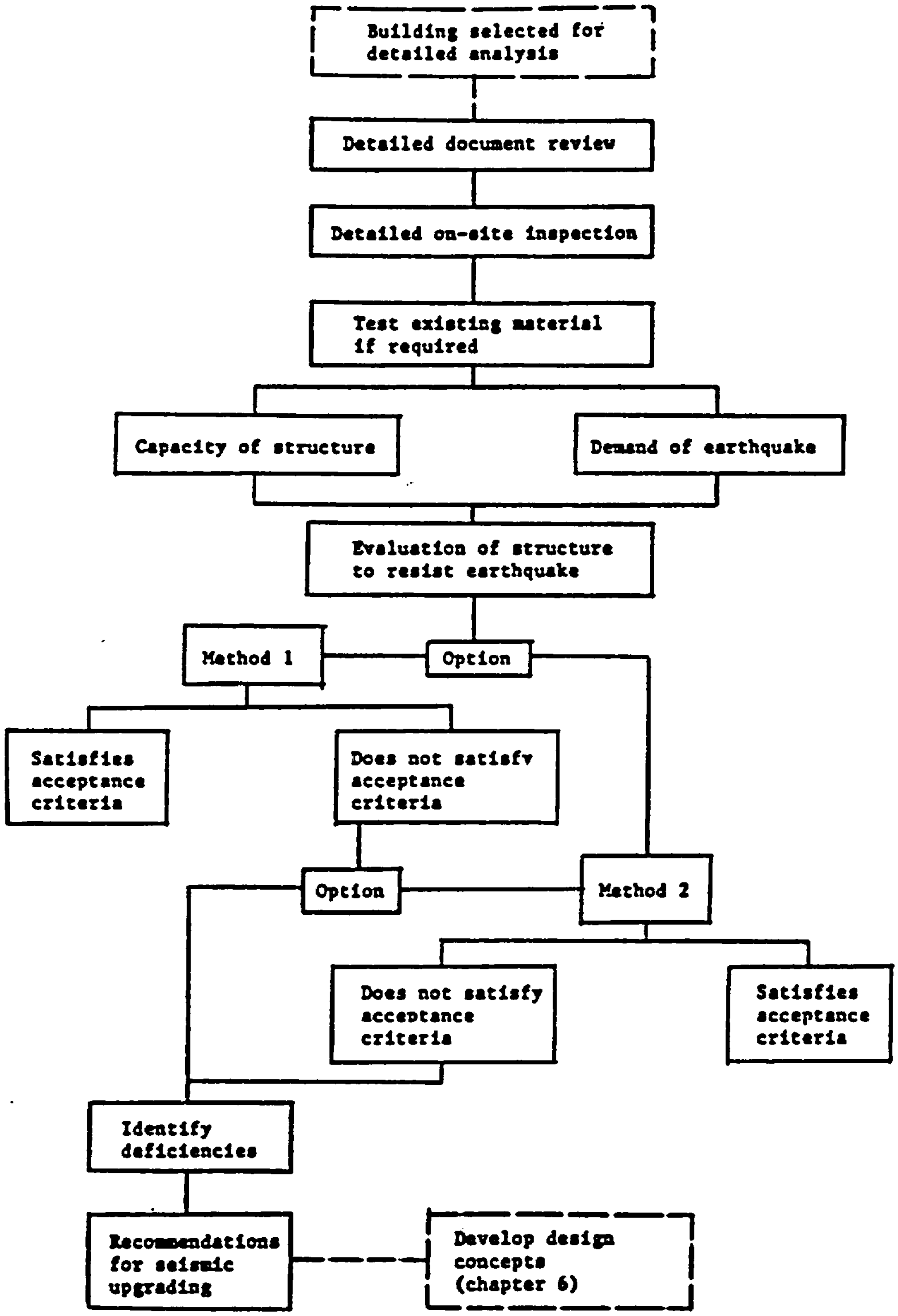


Figure 5-1. Methodology for detailed structural analysis of buildings

the results of any previous inspection and evaluation reports. Special attention will be given to verify the existing lateral force resisting elements and systems (e.g., note any missing bracing members, openings not shown on the drawings, and additions). Testing or special inspection will be made when there is no available data or when as-built conditions are suspect.

c. Testing of existing materials. When economically justified, a testing program may be established to determine the capacity characteristics of nonconforming materials and details, especially when the acceptability of test results can make the difference between accepting an existing building in an as-is condition as opposed to requiring a costly modification. Structural capacities of existing materials will be determined in accordance with criteria and testing requirements of appendix E.

d. Capacity of the structure. The capacity of the structure to resist lateral forces will be determined in accordance with the guidelines provided in the SDG for new construction with the modifications provided in this manual to cover existing materials and structural systems (Refer to para 5-2 for acceptance criteria).

e. Demands of the earthquake. The structure will be subjected to the demands of EQ-II, as defined in the SDG.

f. Evaluation of structure. The structure will be evaluated by a capacity/demand comparison in accordance with the SDG procedures for designing for EQ-II (refer to SDG paras 4-4 and 5-5), using methods 1 or 2 as described below. Examples of procedures are given in SDG appendix E.

(1) *Method 1:* Elastic analysis procedure (refer to SDG para 4-4c). This procedure is used to determine if the existing structure has the required capacity to resist the prescribed earthquake criteria. Table 5-1 is an extended version of SDG table 4-2, inelastic demand ratios.

(*a*) If the structure meets the acceptance criteria of paragraph 5-2 above and does not have any of the deficiencies listed in paragraph 4-4*c*(5) of the SDG, upgrading is not required.

(*b*) If the structure does not conform to the acceptance criteria by means of the Method 1 analysis, seismic upgrading will be required unless it can be demonstrated that the building can satisfy the acceptance criteria by means of Method 2, capacity spectrum method.

(2) *Method 2*: Capacity spectrum method (refer to SDG paras 4-4*d* and 5-5*b*). This procedure is used to compare to earthquake demand as represented by an appropriate response spectrum with the structural capacity as represented by a capacity spectrum with accelerations, S_a, the building can resist when it has fundamental periods, T.

(*a*) If the structure conforms to the acceptance criteria of paragraph 5-2 with the Method 2 analysis, upgrading is not required.

(*b*) If the structure does not conform to the acceptance criteria with the Method 2 analysis, upgrading will be required.

g. Identify deficiencies for structures that are selected for seismic upgrading. The results of the detailed structural analysis from Method 1 or Method 2 will be used to identify the structural deficiencies.

(1) For Method 1, in most cases, structural deficiencies will be identified as those that exceed the allowable inelastic demand ratios given in table 5-1, which is an extended version of table 4-2 of the SDG. The results of the Method 1 analysis will also be evaluated to identify other deficiencies indicated in paragraph 4-4*c*(5) of the SDG.

(2) For Method 2, in most cases, structural deficiencies will be identified as those members that limit the capacity of the structure below the level required by the earthquake demand because of inelastic yielding or rotation. However, care should be exercised in the determination of the structural capacities to confirm that the possibility of the other deficiencies indicated in paragraph 4-4*c*(5) of the SDG have been properly considered in the determination of the structural capacity.

(3) For both Method 1 and Method 2, supplementary structural analyses, as described in the BDM for new construction, must be performed to determine the structural adequacy of an existing building or to identify possible deficiencies. These analyses include:

(*a*) Evaluation of foundations for vertical bearing and the transfer of horizontal forces to the soil.

(*b*) Evaluation of floor and roof diaphragms for shear capacity and shear transfer to vertical resisting members. Also adequacy of diaphragm chords and collector members.

(*c*) Out-of-plane bending of vertical walls and piers, including anchorage and support at floor and roof levels.

(*d*) Adequacy of support and anchorage of equipment, piping, and nonstructural elements as described in chapter 9.

(*e*) Adequacy of bracing or lateral supports to preclude local buckling of steel members.

(*f*) Check of P-delta effects (see SDG para 5-5*d* for additional guidance), local torsion and other secondary stresses.

5-4. Recommendations

On the basis of the detailed structural analysis results, recommend alternatives for seismic upgrading.

Table 5-1. Inelastic demand ratios for existing buildings. (Sheet 1 of 2)

Building System	Element	Essential	High Risk	Others
a. Systems conforming to BDM requirements.				
Steel				
DMRSF	Beams	2.0	2.5	3.0
	Columns*	1.25	1.5	1.75
Braced Frames	Beams	1.5	1.75	2.0
	Columns*	1.25	1.5	1.75
	Diag. Braces**	1.25	1.5	1.5
	K-Braces***	1.0	1.25	1.25
	Connections	1.0	1.25	1.25
Tie Rods	Tension only	1.0	1.1	1.25
Concrete				
DMRSF	Beams	2.0	2.5	3.0
	Columns*	1.25	1.5	1.75
Walls:				
(1) Single curtain of reinforcing	Shear	1.1	1.25	1.5
	Flexure	1.5	1.75	2.0
(2) Double curtain of reinforcing	Shear	1.25	1.5	1.75
	Flexure	2.0	2.5	3.0
Diaphragms	Shear	1.25	1.5	1.75
	Flexure	1.5	1.75	2.0
Masonry Walls	Shear	1.1	1.25	1.5
	Flexure	1.5	1.75	2.0
Wood	Trusses	1.5	1.75	2.0
	Columns*	1.25	1.5	1.75
	Shear Walls and Diaphragms	2.0	2.50	3.0
	Connections (other than nails)	1.25	1.50	2.0

*In no case will axial loads exceed the elastic buckling capacity.

**Full panel diagonal braces with equal number acting in tension and compression for applied lateral loads.

***K-bracing and other concentric bracing systems that depend on compression diagonal to provide vertical reaction for tension diagonal.

Sheet 1 of 2

Table 5-1. Inelastic demand ratios for existing buildings. (Sheet 2 of 2)

Building System	Element	Essential	High Risk	Others
b. Systems not conforming to BDM requirements.+				
Concrete Frames	Beams	1.25	1.5	1.75
	Columns*	1.0	1.25	1.25
Unreinforced Concrete Walls	Shear	1.0	1.1	1.25
	Flexure	1.0	1.0	1.0
Unreinforced Masonry Walls	Shear	1.0	1.1	1.25
	Flexure	1.0	1.0	1.0

*In no case will axial loads exceed the elastic buckling capacity.
+See also paragraph 5-2b for additional acceptance criteria for nonconforming structural systems.

Sheet 2 of 2

CHAPTER 6

DEVELOPMENT OF DESIGN CONCEPTS

6-1. Introduction

This chapter describes general procedures for the development of design concepts for the structural upgrading of existing buildings to comply with the acceptance criteria prescribed in chapter 5. Guide lines are provided for the upgrading of the structural systems, the determination of the capacities

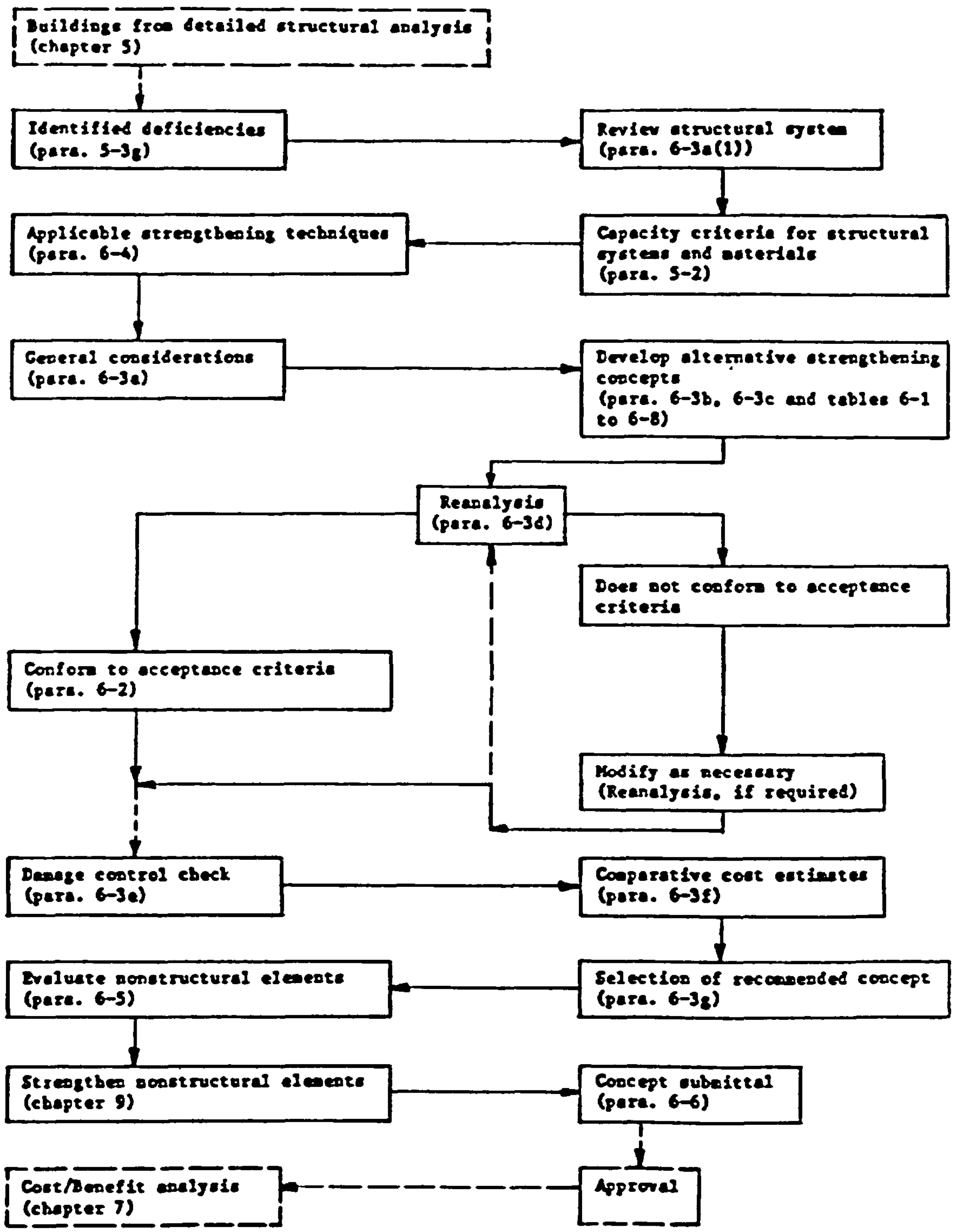


Figure 6-1. Methodology for development of design concepts for seismic upgrading

of new structural elements, and development of strengthening techniques. The methodology for the procedures contained in this chapter is indicated in the flow chart in figure 6-1.

6-2. Acceptance criteria

The design criteria for the development of concepts for seismic upgrading of existing buildings will be in accordance with the applicable provisions of the BDM and/or the SDG as required for new construction. Unless otherwise directed by the approval authority, the minimum acceptance criteria for the conceptual designs will be as indicated in paragraph 5-2 for the detailed structural analysis. The objective of seismic upgrading will be to establish full compliance with SDG provisions for the EQ-II post-yield evaluation (refer to SDG paragraphs 4-4 and 5-5) or, when so directed by the approval authority, the applicable provisions of the BDM or appendix C. In most cases the costs associated with full compliance, as opposed to the reduced force levels permitted by the minimum criteria, will be negligible. However, the allowable reduction will provide a margin for acceptance in those cases where strict adherence to the unreduced criteria would result in much more expensive or disruptive procedures (e.g., a 15 percent reduction in the EQ-II response spectra may make it possible to accept an existing building without strengthening the existing foundations or the construction of an additional shear wall; however, if even with the reduced criteria foundation strengthening or a new wall is required, the upgrading design will be in compliance with the unreduced criteria).

6-3. Development of concepts for seismic upgrading

The results of the detailed structural analysis prescribed in chapter 5 will identify, for a given building, the deficiencies with respect to the acceptance criteria of the various structural elements or systems. These results will be carefully reviewed in the development of alternative design concepts to upgrade the structure to meet the acceptance criteria. Three alternative concepts will be developed for each building unless justification can be shown for fewer alternatives (e.g., it may be shown that the obvious cost effective solutions for a deficient steel braced building are either to replace the existing bracing with stronger bracing members, or to add new bracing so that only two alternatives need to be developed). Each concept will be developed to the extent that will permit a reasonable cost estimate to be made. The extent of removal of existing construction will be indicated; the sizes and locations of new, replaced, or strengthened structural members will be indicated; typical structural connections will be shown; and the extent and schematic details for upgrading nonstructural elements will be indicated.

a. General considerations. In addition to the acceptance criteria of paragraph 6-2, the following general considerations will be addressed in the development of the design concepts:

- Structural systems
- Plan configuration
- Horizontal diaphragms and foundation ties
- Eccentricity
- Deformation compatibility of new and existing materials
- Inelastic demand ratios
- Drift limits
- Base isolation and energy dissipation

(1) *Structural systems.* The development of the structural upgrading concept requires a complete understanding of the existing vertical and lateral load resisting systems of the existing building. The designer must be able to determine the consequences that the removal, addition, or modification of any structural or nonstructural element will have on the performance of the strengthened building.

(*a*) *Gravity load resisting system.* An evaluation of the existing vertical load carrying structural system will be made to determine the effects that the seismic upgrading may have on future performance of the building to resist dead and live gravity loads. The evaluation will include a description of the components of the vertical load carrying system and the load path from the source of the dead and live loads to the foundations.

1. Floor and roof framing. In most buildings, the horizontal framing systems (i.e., floors and roofs) will participate in the lateral force resisting system as a diaphragm in addition to supporting the gravity loads. As part of the seismic upgrading, the floor and roof systems may require modifications (e.g., superimposed diaphragms or horizontal bracing) that will add to the dead load; thus, the capacity of the modified system must be evaluated for the new loading conditions.

2. Vertical structural elements. Vertical load resisting elements such as columns, bearing walls, and framing systems, may also be affected by the seismic upgrading. In addition to the added weight that may be imposed due to the seismic strengthening, these elements may participate in the lateral force resisting system. For example, bearing walls may also be used as shear walls and frames may be strengthened or braced to resist seismic forces. If these framing elements are not used for the lateral force resisting system, they will have to be analyzed for deformation compatibility. This analysis will include the effects of the lateral displacements due to extreme seismic motion on the vertical load carrying capacity of the vertical structural elements.

3. Foundations. If the seismic upgrading adds weight or redistributes the existing gravity loads, the foundations must be analyzed for the additional gravity loads combined with the horizontal and overturning forces associated with the seismic lateral force design.

(*b.*) *Lateral load resisting systems.* The structural system that is designed to resist the seismic forces basically relies on a complete three-dimensional space frame; a coordinated system of shear walls or braced frames with horizontal diaphragms; or a combination of both. Descriptions of these basic systems and their components for new construction can be found in the BDM, paragraph 2-9. In the evaluation and upgrading of an existing structure, it is sometimes difficult to identify an existing lateral force resisting system. Innovative analytical procedures and reliance on existing materials and systems that are not generally considered for new construction are required to determine the load paths and capacities of the existing structures. When an existing structure is not adequate to resist the prescribed lateral forces, as determined by the detailed structural analysis described in chapter 5, strengthening of the existing lateral force resisting system will be required.

(2) *Configuration.* If the existing building is highly irregular in plan configuration or is comprised of units with incompatible seismic response characteristics (e.g., a flexible 6-story steel moment frame connected to a 3-story rigid concrete shear wall unit), severe problems that cannot be resolved by strengthening or upgrading could develop at the connection between two units. In such cases, consideration should be given to separating the two units with a structural expansion joint. Each unit should have a complete system for resisting vertical as well as lateral loads. Structural members bridging the joint with sliding supports on the adjacent unit should be avoided. The criteria for building separations in the SDG (para 4-4*e*(2)(*b*)) apply also to existing buildings. Expansion joints should provide for three-dimensional uncoupled response of each of the separate units of a building, but need not extend through the foundations.

(3) *Horizontal diaphragms and foundation ties.* Every upgraded existing building will have either a rigid or semi-rigid horizontal floor diaphragm as defined in Chapter 5 of the BDM. Roof diaphragms may be flexible or semi-flexible provided they comply with the applicable requirements specified for those diaphragms in the BDM. Foundation ties between pile caps and caissons will be provided in accordance with paragraphs 3-3(*J*)3c and 4-8 of the BDM. Existing diaphragms and foundation ties that do not comply with these requirements will be strengthened or replaced in accordance with the guidelines of paragraph 6-4, unless proper justification can be provided for waiving the deficiency.

(4) *Eccentricity.* Provisions shall be made for the increase in shear resulting from the horizontal torsional moment due to an eccentricity between the center of mass and the center of rigidity, as prescribed in BDM paragraph 3-3(*e*)4 and SDG paragraph 4-3*e*(5). In the development of upgrading concepts for existing buildings, when the vertical shear resisting elements must be strengthened, supplemented, or replaced with new elements, consideration will be given to location of new or strengthened elements so as to reduce any eccentricity between the center of rigidity and the center of mass.

(5) *Deformation compatibility of new and existing materials.* The compatibility of the deformation characteristics of the existing elements and the new strengthening elements will be considered in the strengthening design of the building.

(*a*) *Relative rigidities.* When lateral forces are applied to a building, they will be resisted by the various elements in proportion to their relative rigidities. The lateral stiffness of a structure is calculated on the basis of the deformation characteristics of the lateral force resisting elements. The structure may be flexible (e.g., a light steel frame) or rigid (e.g., a structure with thick masonry walls). If the structure is to be strengthened to resist seismic forces, the new structural elements must be more rigid than the existing elements if they are to take a major portion of the lateral forces and reduce the amount of force that is taken by the existing elements. Both the relative rigidities and strengths of all lateral force resisting elements must be considered. To illustrate, the following two examples are given:

1. Existing steel moment frame strengthened by diagonal steel bracing. Assume an existing steel moment frame building that has a one-inch story displacement due to seismic forces. Diagonal bracing is added to the moment frames to reduce the lateral displacement to 0.1 inch for the same force level. Thus, it can be estimated that the bracing resists about 90 percent of the lateral force and the frame resists about 10 percent. If the original moment frame can safely resist 10 percent of the seismic forces, the bracing system is effective. (Note this example neglects the possible increase in the magnitude of the seismic forces due to a decrease in the period of vibration.)

2. Existing brick building strengthened by a steel braced frame system. Assume an existing brick building that has a 0.01-inch story displacement due to seismic forces. A steel bracing system is added that has a 0.02-inch story displacement for the same force level. In this case, it appears that after strengthening the building, the brick walls will resist approximately two-thirds of the lateral forces until they fail and transfer load to the steel braced frames. Therefore, the steel bracing system is fully utilized prior to cracking of the brick walls; however, if it subsequently can resist the total seismic forces, it will limit the lateral displacements and prevent excess damage to the brick walls (see subpara (7) below for drift limitations).

(*b*) *Deformation characteristics of structural elements.* The accuracy of the relative rigidity calculations is dependent on the accuracy of the assumptions used for determining the stiffness characteristics of each element or system. When all of the lateral force resisting elements are of the same material and have similar deformation characteristics (e.g., flexural and/or shearing deformations), the relative rigidities can be calculated with reasonably good accuracy. However, when there is a variety of materials and cross-sectional shapes, the confidence level on the accuracy of the relative rigidities is greatly reduced. When the confidence level is low, the range of stiffness values should be enveloped and the distribution should be overlapped to account for the inaccuracies. Mathematical modeling guidelines are given in SDG paragraphs 5-4*b* and 5-5*a*(2). Structural elements that require special consideration in determination of relative rigidities include:

Concrete: cracked vs. uncracked

Shear walls: participation of intersecting walls (e.g., effective flange widths) and effects of openings.

Steel frames: participation of concrete fireproofing, floor slab and framing, and infill walls (structural and nonstructural).

(*c*) *Evaluation of structural elements not part of the lateral force resisting system.* Structural elements that are not part of the lateral force resisting system will be evaluated for the effects of the deformation that occur in the lateral force resisting system. These provisions for the EQ-II deformations parallel the deformation compatibility provisions in BDM paragraph 3-3(J)1d.

(*d*) *Protection of existing brittle elements.* Brittle elements that are not part of the lateral force resisting system are susceptible to damage if they are forced to conform to the deformations of the lateral force resisting system. In order to protect these elements from the possibility of being subjected to large distortions, provisions can be made to allow the structural system to distort without forcing distortion on the brittle elements. An example of isolating a masonry wall from the slab soffit is shown in the BDM, figure 9-1. When rigid walls are locked in between columns, a similar method of isolation may be required at each end of the wall.

(6) *Inelastic demand ratios.* Seismic capacity, demand, and inelastic demand ratios shall be calculated in accordance with the provisions of chapter 4 of the SDG and shall not exceed the values given in table 5-1 unless they are supported by other systems that can resist the required lateral forces. For example, in an existing unreinforced masonry bearing wall building with new reinforced concrete shear walls or steel bracing, the masonry walls are assumed to share the lateral forces in proportion to their relative rigidities until the allowable inelastic demand ratio for reinforced masonry is exceeded. At that point, the entire story shear must be resisted by the new shear walls or steel bracing. The masonry wall may be assumed to be capable of supporting the imposed vertical loads, providing the drift limits specified in the following subparagraph are not exceeded.

(7) *Drift limits.* Lateral deflections, or drift, of a story relative to its adjacent stories for EQ-II will be in accordance with the provisions of paragraph 5-2*a*, except that for unreinforced concrete or masonry walls and nonductile reinforced concrete frames where the allowable inelastic demand ratios are exceeded (see subpara (6) above), the interstory drift limits for the EQ-II forces will be reduced to those given in paragraph 5-2*b* (i.e., 60 percent of para 5-2*a*) unless the above nonductile elements are properly anchored to a new structural system (i.e., reinforced concrete or masonry wall, braced steel frame, etc.) that is capable of resisting the entire story shear.

(8) *Base isolation and energy dissipation.*

(*a*) *Base isolation.* Design strategies that significantly modify the dynamic response of a structure at or near the ground level, are generically termed base isolation. This is usually achieved by introduction of additional flexibility at the base of the structure. The objective is to force the entire superstructure to respond to vibratory ground motion as a rigid body with a new fundamental mode based on the stiffness of the isolation devices. This strategy is particularly effective for short stiff buildings (i.e., with a fundamental mode less than 1 sec). For these buildings, it is feasible with the isolation devices to develop a new fundamental mode with a period of about 2 sec. For most sites (e.g., those with a predominant site period less than 1 sec), the new fundamental mode period will be beyond the portion of the response spectrum that is subject to dynamic amplification and the response of the structures will be greatly reduced. The concept of base isolation is not new; for many years it has been used to reduce the vibration of equipment and machinery with springs, resilient mountings, and shock absorbers. Similarly, bridges and other simple structures have been isolated to reduce vibration and noise, and in some instances, to reduce the seismic excitation. The application to complex structures, such as buildings, has been made possible in recent years due to greatly improved computational capability (e.g., high speed, large capacity, digital computers) and development and marketing of the isolation assemblies. A typical installation consists in large pads of natural or synthetic rubber layers bounded to steel plates in a sandwich assembly. The isolator assembly, as well as all connecting elements and building services, must be capable of resisting the design spectral displacement corresponding to the new fundamental mode (a recent California installation has base isolation assemblies that can deflect elastically up to 18 inches). Some base isolation assemblies may have a lead core or other devices to increase damping and thus decrease the response at the isolator. Because of the uncertainties associated with ground motion predictions, most seismic base isolators are designed with fail-safe provisions to arrest the motion of the building prior to development of instability due to excessive displacement of the isolator. Base isolation can be an effective strategy to reduce the seismic response of buildings provided careful consideration is given to the amplitude and frequency content of the expected ground motion; the design of the connecting services to accommodate the expected displacements; and provision of fail-safe mechanisms as described above. The ability of base isolation to reduce seismic response is even more attractive in application to existing buildings with inadequate seismic resistance. However, in addition to the considerations described above, installation of base isolation in an existing building entails accurate determination of the magnitude and location of the vertical loads; a rigid diaphragm above the isolators to collect and distribute the lateral loads; and careful underpinning and jacking of the existing structure in order to effect a systemic transfer of the existing structure in order to effect a systematic transfer of the existing foundation loads to the base isolation device. Base isolation has been investigated for a number of existing structures (base isolation for an historic structure in Salt Lake City is currently under construction), and there are provisions to establish construction feasibility or cost-effectiveness of base isolation for seismic upgrading.

(*b*) *Energy dissipation.* An effective means of providing a substantial level of damping is through hysteretic energy dissipation. Some structures (e.g., properly designed ductile steel and concrete frames) exhibit additional damping and reduced dynamic response as a result of the limited yielding of structural steel or concrete reinforcement. Mechanical devices, designed to increase structural damping, have been developed using mild steel in flexure or torsion and the deformation of lead by extrusion or shear. Viscoelastic materials in shear have been used successfully to control wind vibration in tall buildings and similar installation have been proposed for reducing the seismic response of buildings. Friction is another source of energy dissipation that can be used to reduce dynamic response and limit deflections. However, frictional resistance is difficult to quantify and its reliability may be questionable. Hydraulic damping has been successfully used on machinery and bridge structures, but there are no known applications used to modify building response. The use of structural dampers to reduce the seismic response of existing buildings may be feasible and cost-effective. The installation of viscoelastic structural dampers as an alternative upgrading concept for design example F-3 has been developed in a recent technical article (see biblio Scholl, R.E.).

b. Selection of strengthening technique.

(1) *General.* The selection of an appropriate strengthening technique for the upgrading of an existing building that does not comply with the acceptance criteria of chapter 5 will depend upon the type of structural systems in the existing building, the nature of the deficiency, and the considerations given in subparagraph *a* above. In some cases, the selection may be influenced by other than structural considerations. For example, a requirement that the building be kept operational, with minimal interference to the functions that it provides during the structural modifications, may dictate that the modifications be restricted to the periphery of the building with as much work as possible accomplished on the exterior of the buildings. On the other hand, it may be possible temporarily to relocate the function and occupants of an existing building that is to be upgraded. This, of course, provides more latitude in the selection of appropriate and cost effective strengthening techniques. In many cases, seismic upgrading is accomplished concurrently with func-

tional alterations, renovation, and/or energy retrofits. In these cases, the selected structural modification scheme should be the one that best suits the requirements of all the proposed alterations.

(2) *Examples.*

(*a*) An existing unreinforced masonry building with inadequate shear capacity in walls has reinforced concrete floor and roof diaphragms that are adequate in shear capacity, but do not have adequate chord strength for the flexural action of the diaphragms. A rigid system of new reinforced concrete shear walls or steel braced frames will be required to provide the additional strength and rigidity to protect the masonry walls. Because a chord has to be developed for the existing diaphragms, it may be advantageous to consider strengthening the masonry wall with a new reinforced concrete wall on the inside of the masonry walls as described in paragraph 6–4*b*(4). This will facilitate anchorage of the masonry walls for out-of-plane forces, development of a new diaphragm chord, and shear transfer from the existing diaphragms. A portion of the masonry wall may be removed to reduce the loads on the existing foundations. If the full thickness masonry walls can span vertically for the seismic out-of-plane forces, consideration can be given to providing the new concrete walls in selected locations while minimizing the eccentricity between the center of mass and the new center of rigidity. The diaphragm chords must be continuous to resist the horizontal flexural stresses in the diaphragms and to provide the necessary support to the masonry walls.

(*b*) A two-story steel frame building has 7 frames in the transverse direction and 3 frames in the longitudinal direction. Three of the 7 frames in the transverse direction and 2 of 3 frames in the longitudinal direction are moment frames. The floor and roof diaphragms consist of steel decking without concrete fill. The existing frames and diaphragms are adequate for the acceptance criteria in the longitudinal direction. The 3 existing transverse frames do not meet the acceptance criteria for drift and the diaphragms do not have adequate shear capacity in the transverse direction for the three bays between the moment frames, but they would be adequate for only two bays. There are a wide range of possible solutions to this example: the diaphragms could be strengthened by adding a concrete fill (para 6–4*b*(7)(*c*); the existing transverse moment frames could be strengthened (paras 6–4*b*(1)(*a*) and (*b*); some of the intermediate transverse frames could be made moment-resisting (para 6–4*b*(1)(*d*)); or new reinforced concrete shear walls or steel bracing could be added to reduce the drift. It should be noted that modifying the intermediate frames for moment resistance may not be feasible because of interferences with the steel decking. Adding concrete fill to strengthen the steel decking will require removal and replacement of the second floor and the roof. New concrete shear walls will require new foundations. Considerable cost saving can be achieved by eliminating or minimizing the work on the roof, floor, and foundations. Vertical steel bracing becomes a logical solution. If the bracing is installed at the end transverse frames and at every other frame in between, the existing diaphragm will only have to span two bays and will not need to be strengthened. The bracing will be effective in reducing drift, but the resulting shorter period will probably increase the seismic demand forces that now will be resisted primarily by bracing. However, with 4 lines of bracing, the forces are well distributed and the additional foundation loads (shear and overturning forces) may not be difficult to accommodate with the existing foundations.

(*c*) An existing two-story office building that performs an essential function must be seismically upgraded. The building has an identified deficiency in the transverse direction in which the lateral forces are resisted by nonductile concrete frames. The detailed structural analysis indicates that additional lateral load resistance is required and also that building deformation must be limited so that allowable drift and inelastic demand ratios for the nonductile frames are not exceeded. The structural modification must be accomplished with minimal interference to the functions and occupants in the building. The above restrictions dictate that the optimum solution would be one that provides significant rigidity and can be implemented from the exterior of the building. If the floor and roof diaphragms comply with the requirements of chapter 5 of the BDM, an appropriate scheme for the upgrading would be to provide bracing or shear walls at each end of the building. A potential problem with this scheme might be inadequate resistance to the overturning forces at the foundation level. Possible solutions to the overturning problem would be larger footings; drilled piers to provide tension tie-down; buttresses in the plane of the end walls to increase the resisting lever arm for the overturning moments; or internal shear walls to reduce the lateral forces on the end walls. Prefabricated steel shear walls (para 6–4*d*(2)(*b*)) can be used to minimize the time and area of disruption in an existing building.

(*d*) An existing three-story unreinforced masonry building is to be seismically upgraded because of the historical significance of its external architecture. The building will be used as an administration building after the seismic upgrading. The roof and floor systems are timber post and beam construction and the timber flooring is inadequate for diaphragm action or to anchor the walls. Retention of the timber framing will require installation of a sprinkler system. In this building consideration should be given to reconstruction of the second floor and roof systems in reinforced concrete. The existing exterior walls may need temporary shoring while a new reinforced concrete wall is constructed at the interior face of the existing walls (para 6–4*b*(4)). The new walls and floor and roof framing will provide the lateral force systems and will provide the necessary support to the existing exterior walls.

c. Strengthening options for structural systems. Paragraph 6–4 describes various generic strengthening techniques for structural elements. The preceding subparagraph provides two representative examples for the selection of appropriate strengthening techniques that are compatible with the existing structural system and will correct the deficiencies identified in the detailed structural analysis. Tables 6–1 to 6–8 provide a listing of various strengthening options that may be considered for seismic upgrading. This listing should not be considered to be complete or exclusive and the engineer is encouraged to use his initiative in the development of the three required alternative upgrading concepts from the options described in the tables or by innovative variation of those options.

d. Reanalysis. Each alternative upgrading concept will be evaluated for compliance with the acceptance criteria in chapter 5. Unless the effects of the structural modifications on the mass, stiffness, and load distribution in the building are obviously negligible, a reanalysis of each concept will be required. The reanalysis will be similar to the detailed structural analysis but with the revised structural model resulting from the upgrading modifications. In most cases the effect of strengthening and/or stiffening of an existing building will reduce the modal periods of vibration and increase the spectral demand on the building. One or more analysis iterations may be required to reconcile the modified capacity of the building with the seismic demand.

e. Damage control check. After each alternative upgrading concept has been checked for compliance with the acceptance criteria for EQ-II forces, it will also be checked as follows for essentially elastic response to EQ-I forces:

(1) *EQ-I analysis.* Perform an EQ-I analysis in accordance with paragraph 4-3 of the SDG. The acceptance criteria prescribed in paragraph SDG 4–3*e*(1) will be modified as follows:

(*a*) *Ductile framing systems.* The 25 percent tolerance allowed in excess of the flexural elastic capacity for a limited number of elements may be increased to 30 percent.

(*b*) *Other framing systems.* The 10 percent tolerance allowed in excess of the flexural elastic capacity may be increased to 15 percent.

(*c*) *Box systems.* These systems may not exceed the elastic capacity requirements of the SDG.

(2) *Alternatives to EQ-I analysis.* When the EQ-II reanalysis prescribed in paragraph d above has been performed by Method 2, compliance with EQ-I requirements may be made by comparing the elastic capacities, calculated for the EQ-II reanalysis, with the EQ-I spectral requirements. When the EQ-II reanalysis has performed by Method 1 (conventional elastic analysis), the following procedures may be used:

(*a*) Compare the response spectrum for the EQ-I elastic response to that for EQ-II post-yield response. Determine the spectral acceleration ordinate, S_{aI}, at the building's fundamental period on the EQ-I spectrum and the corresponding ordinate, S_{aII}, on the EQ-II spectrum.

(*b*) Calculate the ratio, $R = \frac{S_{aI}}{S_{aII}}$

(*c*) Examine a representative sample of inelastic demand ratios (IDR) at each level of the building.

(*d*) Determine what portion of each IDR is attributed to seismic response as opposed to response to the vertical gravity loads (e.g., for shear walls with an IDR of 1.50, the entire amount may be due to seismic loads where a concrete or steel frame column with the same IDR may have 0.60 due to gravity loads and 0.90 due to seismic loads).

(*e*) Multiply the seismic portion of the IDR by the ratio, R, previously calculated and add this to the gravity load portion of the IDR (e.g., for a given building, if R = 0.40, the same shear wall with an IDR of 1.50 for EQ-II would have an IDR of 0.60 (i.e., 0.40 × 1.50) for EQ-I while the above frame column would have an IDR of 0.96 (i.e., 0.60 + 0.40 × 0.90)).

(*f*) Unless adjustments are made for differences in EQ-I and EQ-II gravity load factors, none of the tolerances permitted for exceeding the flexural elastic capacity in paragraph 4–3*e*(1) of the SDG and paragraph (1) above will apply.

f. Comparative cost estimates. After it has been confirmed that each alternative concept is in compliance with the acceptance criteria, comparative cost estimates will be prepared to provide a basis for the selection of the recommended concept. Since the primary purpose of these estimates is to differentiate the relative costs of the concepts, a complete cost estimate is not required at this point. Only those principal items of cost that vary among the concepts need to be recognized. For

Table 6-1. Strengthening options for unreinforced concrete or masonry buildings. (Sheet 1 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Shear walls	(1) Inadequate shear capacity	(a) Add reinforced concrete to exterior face	Para. 6-4b(4)	Figure 6-7
		(b) Add reinforced concrete to interior face	Para. 6-4b(4)	Figure 6-10
		(c) Remove and replace with reinforced concrete	Para. 6-4c	Figure 6-22
b. Connecting beams	(1) Inadequate shear or flexural capacity	(a) Fill-in openings	Para. 6-4b(3)(a)	Figure 6-9
		(b) Remove and replace with reinforced concrete	Para. 6-4b(3)(a)	Figure 6-8
c. Concrete floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with new reinforced concrete slab	Para. 6-4b(7)(b)	Figure 6-16
	(2) Inadequate chord capacity	(a) Add new reinforced concrete chord	Para. 6-4b(7)(b)	Figs. 6-10, 6-16, 6-17
	(3) Inadequate wall anchorage	(a) Provide drilled-in wall anchors to new construction	Para. 6-4b(4)	Figs. 6-7, 6-8, 6-10, 6-16, 6-17
d. Timber floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with plywood	Para. 6-4b(7)(a)	Figure 6-14,
	(2) Inadequate chord capacity	(a) Add new continuous steel angle	Para. 6-4b(7)(a)	Figure 6-14
	(3) Inadequate wall anchorage	(a) Provide drilled-in wall anchors to new construction	Para. 6-4b(7)(a)	Figure 6-14

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Table 6-1. Strengthening options for unreinforced concrete or masonry buildings. (Sheet 2 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
e. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
f. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(b)	Figure 6-21

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example, if diagonal bracing and eccentric bracing are considered as alternate concepts to be installed in the same number of bays and both concepts required the same strengthening of floor and roof diaphragms and foundations, only the differential costs of the two bracing systems need to be identified.

g. Selection of recommended concept. The optimum concept will be the one that meets the acceptance criteria and best satisfies the general considerations of subparagraph a at a reasonable cost. This may not necessarily be the least expensive concept if justification can be provided for greater reliability, improved structural performance, functional advantages, or reduced maintenance of a better and more cost effective system.

6-4. Strengthening techniques

Techniques for strengthening or upgrading existing buildings will vary according to the nature and extent of the deficiency, the configuration of the structural system, and the structural materials of which it is comprised. It is not practicable within the scope of this manual to deal with every possible variation of all conditions. This paragraph will provide guidelines for the seismic upgrading of typical structural members or systems and guidance for structural engineers to utilize judgment and ingenuity to deal with specific situations. The strengthening or seismic upgrading of the building will generally fall into one or more of the following categories: rehabilitation of existing structural members; modification of existing structural members; replacement of existing deficient structural members; or the addition of new structural members or elements (i.e., shear walls, braced frames, etc.).

a. Rehabilitation of existing structural members. Seismic upgrading of existing buildings by strengthening or replacement of existing structural members and/or the addition of new structural members may also require rehabilitation to restore the initial capacity of existing structural members that have been subjected to damage or deterioration. Representative examples of feasible rehabilitation for typical structural members are described in appendix E. General deterioration of materials, such as corrosion of structural steel members or concrete reinforcement, or weathering of concrete brick or mortar, may not be readily repaired and such materials will be assigned a capacity reduction factor as indicated in appendix E.

b. Modification/strengthening of existing structural members. In some cases, the modification and/or strengthening of existing structural members could be the most cost effective method for

Table 6-2. Strengthening options for reinforced concrete or masonry shear wall buildings. (Sheet 1 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Shear walls	(1) Inadequate shear capacity	(a) Add reinforced concrete to exterior face	Para. 6-4b(3)(a)	Figure 6-7
		(b) Add reinforced concrete to interior face	Para. 6-4b(3)(a)	Figure 6-10
		(c) Fill-in openings	Para. 6-4b(3)(a)	Figure 6-9
		(d) Add new interior concrete shear walls	Para. 6-4d(2)(b)	Figure 6-24
		(e) Add new interior steel shear walls	Para. 6-4d(2)(b)	Figure 6-25
		(f) Add new exterior steel or concrete buttresses	Para. 6-4d(5)	Figure 6-28
		(g) Remove and replace with new construction	Paras. 6-4b(3)(a) & 6-4c	Figure 6-22
b. Connecting beams	(1) Inadequate shear or flexural capacity	(a) Fill-in openings in shear walls	Para. 6-4b(3)(a)	Figure 6-9
		(b) Remove and replace with reinforced concrete	Para. 6-4b(3)(a)	Figure 6-9
c. Concrete floor roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with new reinforced concrete slab	Para. 6-4b(7)(b)	Figure 6-16
	(2) Inadequate chord capacity	(a) Add new reinforced concrete chord	Para. 6-4b(7)(b)	Figs. 6-10, 6-16, 6-17
	(3) Inadequate wall anchorage	(a) Provided drilled-in wall anchors	Para. 6-4b(4)	Figs. 6-7, 6-8, 6-10, 6-16, 6-17

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Table 6-2. Strengthening options for reinforced concrete or maqsonry shear wall buildings. (Sheet 2 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
d. Timber floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with plywood	Para. 6-4b(7)(a)	Figure 6-14
	(2) Inadequate chord capacity	(a) Add new continuous steel angle	Para. 6-4b(7)(a)	Figure 6-14
	(3) Inadequate wall anchorage	(a) Provide drilled-in wall anchors	Para. 6-4b(7)(a)	Figure 6-14
e. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
f. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(b)	Figure 6-21

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the seismic upgrading of an existing building. Typical examples of the structural modification of existing structural members, in place, are provided in the following paragraphs. *Other cost-effective methods will be investigated for each condition illustrated in figures 6-2 through 6-28.*

(1) *Structural steel framing.*

(*a*) *Columns.* The capacity of columns is determined from interaction equations for axial loads and bending, thus the seismic capacity of a column can be upgraded, within reasonable limits, by increasing either or both its capacity for axial loads or for moment. The axial load capacity of steel columns can be upgraded by welding cover plates on the flanges or by "boxing" the column with plates between the tips of the flanges. Typical details are indicated on figure 6-2. These plates may also serve to increase the moment capacity of the columns at the base. Increasing the moment capacity of existing columns at the beam-column connection is usually not feasible because of the interference of the connecting beams. In some cases, it may be possible to increase the shear capacity of the column web with doubler plates as indicated in figure 6-2 provided that there is adequate clearance for the necessary welding.

(*b*) *Beams.* Strengthening of existing beams, in place, may be required to improve the moment capacity by an increase in the section modulus, S, or to reduce drift by an increase in the moment of inertia, I. The section modulus of a beam may be increased by welding cover plates to the top or bottom flanges. In many cases, it may not be feasible to provide cover plates on the top flange because of interference with the floor beams, slabs, or metal decking. (Note that for a bare steel beam, a cover plate on only the lower flange may not significantly reduce the stress in the upper flange.) However, if the floor slab or metal decking is adequately detailed for composite action at the end of the beam, it may be economically feasible to increase the moment capacity by providing cover plates at the lower flanges at each end of the beam as indicated in figure 6-3. The length of the cover plates, in this case, will be determined from the combined (DL + LL + EQ) demand moment diagram. The cover plates will be tapered as shown to avoid an abrupt change in section modulus beyond the point where the additional section modulus is required. Where frame drift governs, it may be feasible to increase the moment of inertia and thus reduce the drift by the addition of a cover plate to the lower flange of existing steel beams between the columns as also indicated in figure 6-3. It should be noted that beams with discontinuous cover plates must be treated as tapered or haunched sections and will have different carry-

Table 6-3. Strengthening options for reinforced concrete frame buildings.

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Frames	(1) Inadequate lateral load capacity	(a) Add new external steel frames	Para. 6-4d(1)(a)	Figure 6-23
		(b) Add new interior concrete shear walls	Para. 6-4d(2)(b)	Figure 6-24
		(c) Add new interior steel shear walls	Para. 6-4d(2)(b)	Figure 6-25
		(d) Add new exterior concrete or steel buttresses	Para. 6-4d(5)	Figure 6-28
		(e) Structural addition(s) to building	Para. 6-4d(6)	
		(f) Remove and replace with new construction	Para. 6-4b(2) & 6-4c	Figure 6-22
b. Concrete Floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with new reinforced concrete slab	Para. 6-4b(7)(b)	Figure 6-16
	(2) Inadequate chord capacity	(a) Add new reinforced concrete chord	Para. 6-4b(7)(b)	Figs. 6-16, 6-17
c. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
d. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(b)	Figure 6-21

Table 6-4. Strengthening options for steel moment-resisting frame buildings. (Sheet 1 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Steel frames	(1) Inadequate lateral load capacity	(a) Strengthen existing columns	Para. 6-4b(1)(a)	Figure 6-2
		(b) Strengthen existing beams	Para. 6-4b(1)(b)	Figure 6-3
		(c) Modify existing simple beam connections	Para. 6-4b(1)(d)	Figure 6-5
		(d) Add diagonal bracing	Para. 6-4d(3)	Sim. to Fig. 6-4
		(e) Add eccentric bracing	Para. 6-4d(3)	Figure 6-26
		(f) Add new interior or exterior concrete shear walls	Para. 6-4d(2)(a) & (b)	Figure 6-24
		(g) Structural addition(s) to building	Para. 6-4d(6)	
b. Concrete floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with new reinforced concrete slab	Para. 6-4b(7)(b)	Figs. 6-16, 6-18
		(b) Add new horizontal steel bracing	Para. 6-4d(4)	Figure 6-27
	(2) Inadequate shear transfer	(a) Add shear studs	Para. 6-4b(7)(b)	Sim. to Fig. 6-19
	(3) Inadequate diaphragm chord	(a) Modify existing spandrel connections	Para. 6-4b(1)(d)	Figure 6-6

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Table 6-4. Strengthening options for steel moment-resisting frame buildings. (Sheet 2 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
c. Steel deck floor or roof diaphragms	(1) Inadequate shear capacity	(a) Additional welding of deck to beams	Para. 6-4b(7)(c)	BDM (Figs. 5-19 & 5-20)
		(b) Add concrete fill and shear studs	Para. 6-4b(7)(c)	Figure 6-19
		(c) Add new horizontal steel bracing	Para. 6-4d(4)	Figure 6-27
d. Timber floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with plywood	Para. 6-4b(7)(a)	Figure 6-14
		(b) Add new horizontal steel bracing	Para. 6-4d(4)	Figure 6-27
	(2) Inadequate shear transfer	(a) Bolt timber decking to steel frame	Para. 6-4b(7)(a)	Figure 6-15
		(b) Bolt timber joists or blocking to steel frame	Para. 6-4b(7)(a)	Figure 6-15
e. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
f. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add new additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(b)	Figure 6-21

Sheet 2 of 2

over factors for moment distribution than prismatic members. This may tend to increase the beam moments over the values for the unmodified beams and should be carefully checked to avoid undesirable overstress at critical sections of the beam. The capacity of steel beams in rigid frames may, in some cases, be governed by lateral stability considerations. Although the upper flange may be supported for positive moments by the floor or roof system, the lower flange must be checked for compression stability in regions of negative moments in accordance with section 1.6.1.4 of the AISC Specification. Although properly designed secondary floor beam connections may provide adequate lateral support for those frame beams supporting these secondary beams, beams in frames that are parallel to the secondary beams may need lateral support for the lower flanges in compression due to negative moments. The necessary lateral support may be provided by diagonal braces to the floor system.

(*c*) *Bracing.* Strengthening of existing steel bracing, in place, is a viable alternative, provided that the connections, foundations, and other members of the bracing systems are adequate or can also be strengthened to provide the necessary additional capacity. Strengthening of beams and columns in bracing systems can be accomplished as discussed in paragraphs (*a*) and (*b*) above, and strengthening of bracing members that are designed to act only in tension can be accomplished by simply increasing the cross-sectional area of the brace. In strengthening bracing that will act in both tension and compression, it is desirable to strenghten the bracing in a manner that will improve the slenderness ratio, l/r, as well as increase the cross-sectional area. For existing single angle bracing, this may be done by adding an additional angle, back to back, to provide a double angle bracing system. For existing double angle bracing, an additional pair of angles may be added to provide a "starred" section. Typical strengthening details for bracing are shown in figure 6-4.

(*d*) *Connections.* Development of a feasible scheme for strengthening the existing connections may be the deciding factor as to whether it is practicable to strengthen existing deficient steel framing. Figure 6-5 indicates how an existing simple beam connection can be modified to resist moment. Spandrel beams in perimeter frames are sometimes required to provide the necessary tension or compression chords for floor or roof diaphragms. If these existing beams are framed to the columns with only simple connections, the flexibility of the connection in tension may result in excessive cracking of the diaphragm. Figure 6-6 indicates how an existing simple beam connection in a spandrel beam can be modified to provide positive chord action for diaphragm. Columns also can be modified to provide increased moment capacity at their base, but the capacity of the base detail needs to be investigated for resistance to the additional moment and horizontal shear resulting from these modifications. Assuming that the foundation is adequate (see paragraphs 6-4*b*(4) or 6-4*c*(4) for modification or replacement of existing footings), the maximum allowable bearing stress under the base plate or the tensile stresses in the anchor bolts may govern the moment capacity at the column base. These stresses are governed by the size of the base plate and the number and configuration of the anchor bolts. While it may be possible to strengthen the column and to stiffen the base plate against local bending, it is usually not practicable to increase the size of the base plate or the number of anchor bolts without removal and replacement of the base plate. The horizontal column shears may be transferred to the column footing by shear lugs between the base plate and the footing, and/or shear in the anchor bolts, and to the ground by passive pressure against the side of the footing. If the column base connection is embedded in a monolithic concrete slab, the slab may be considered for distribution of the shear to the ground by means of any additional existing footings that are connected to the slab.

(2) *Concrete frames.* Strengthening of existing concrete frames is not considered practicable because of the difficulty associated with providing the necessary confinement and shear reinforcement in the beams, columns, and the beam-column panel zones. When deficiencies are identified in these frames, the forces and displacements resisted by these frames can be reduced to acceptable limits by the addition of new structural members (e.g., new frames, shear walls, or bracing) as indicated in paragraph 6-4*d*.

(3) *Reinforced concrete or masonry walls or piers.*

(*a*) *Walls with openings.* Existing reinforced concrete or masonry walls with openings may exhibit deficiencies (e.g., excessive shear stresses) in the piers between the openings and/or in the connecting beams between the piers formed by the openings.

1. If the deficiency is in both the piers and the connecting beams, the wall may be strengthened by the addition of reinforced concrete on one or both sides of the existing wall as indicated in figure 6-7. Shallow, highly stressed connecting beams may have to be replaced with properly reinforced concrete as part of the additional wall section. The new concrete may be

Table 6-5. Strengthening opotions for steel braced frame buildings. (Sheet 1 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Steel bracing	(1) Inadequate axial load capacity	(a) Increase effective area of braces	Para. 6-4b(1)(c)	Figure 6-4
		(b) Remove and replace with larger bracing	Para. 6-4c	
		(c) Add additional braces	Para. 6-4d(3)	
		(d) Add exterior or interior concrete shear walls	Para. 6-4d(2)(a) and (b)	Figure 6-24
		(e) Structural addition(s) to building	Para. 6-4d(6)	
b. Concrete floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with new reinforced concrete slab	Para. 6-4b(7)(b)	Figs. 6-16, 6-18
		(b) Add new horizontal steel bracing	Para. 6-4d(4)	Figure 6-27
	(2) Inadequate shear transfer	(a) Add shear studs	Para. 6-4b(7)(b)	Sim. to Figure 6-19
	(3) Inadequate diaphragm chord	(a) Modify existing spandrel connections	Para. 6-4b(1)(d)	Figure 6-6
c. Steel deck floor or roof diaphragms	(1) Inadequate shear capacity	(a) Additional welding of deck to beams	Para. 6-4b(7)(c)	BDM (Figs. 5-19, 5-20)
		(b) Add concrete fill and shear studs	Para. 6-4b(7)(c)	Figure 6-19
		(c) Add new horizontal steel bracing	Para. 6-4d(4)	Figure 6-27

Table 6-5. Strengthening options for steel braced frame buildings. (Sheet 2 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
d. Timber floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with plywood	Para. 6-4b(7)(a)	Figure 6-14
		(b) Add new horizontal steel bracing	Para. 6-4d(4)	Figure 6-27
	(2) Inadequate shear transfer	(a) Bolt timber decking to steel frame	Para. 6-4b(7)(a)	Figure 6-15
		(b) Bolt timber joists or blocking to steel frame	Para. 6-4b(7)(a)	Figure 6-15
e. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
f. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(b)	Figure 6-21

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Table 6-6. Strengthening options for heavy timber frame buildings.

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Posts and beams	(1) Inadequate lateral load capacity	(a) Add diagonal bracing	Para. 6-4b(5)(a)	Figure 6-11
		(b) Add knee bracing	Para. 6-4b(5)(a)	Figure 6-12
b. Timber floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with plywood	Para. 6-4b(7)(a)	Figure 6-14
	(2) Inadequate shear transfer	(a) Provide blocking for nailing to diaphragm and frame members	Para. 6-4b(7)(a)	Figure 6-11
	(3) Inadequate chord capacity	(a) Provide continuous steel members for chord action	Para. 6-4b(7)(a)	Figure 6-14
		(b) Provide continuous timber members for chord action	Para. 6-4b(7)(a)	BDM (Figure 5-33)
c. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
d. Pile or drilled pier	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(b)	Figure 6-21

Table 6-7. *Strengthening options for wood stud framed buildings.*

Structural Element	Deficency	Strengthening Technique	Reference	Applicable Figure
a. Wood stud walls	(1) Inadequate lateral load capacity	(a) Add plywood sheathing	Para. 6-4b(5)(b)	BDM (Figs. 5-32, 5-34, 6-15, Table 5-6)
		(b) Add let-in bracing	Para. 6-4b(5)(b)	Figure 6-13
	(2) Inadequate tie-down capacity	(a) Add new steel tie-down straps	Para. 6-4b(5)(b)	BDM (Figure 6-16)
b. Timber floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with plywood	Para. 6-4b(7)(a)	Figure 6-14
	(2) Inadequate shear transfer	(a) Provide blocking and nailing, as necessary	Para. 6-4b(7)(a)	BDM (Figs. 5-33, 6-15)
	(3) Inadequate drag struts	(a) Provide new drag struts	Para. 6-4b(7)(a)	BDM (Figure 5-34)
c. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
d. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(a)	Figure 6-21

Table 6-8. Strengthening options for steel stud framed buildings.

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Steel studs	(1) Inadequate shear capacity	(a) Add new steel straps	Para. 6-4b(6)	BDM (Figure 6-17a)
b. Steel deck roof or floor diaphragms	(1) Inadequate shear capacity	(a) Provide additional welding	Para. 6-4b(7)(c)	BDM (Figs. 5-19 & 5-20)
		(b) Provide concrete fill and shear studs	Para. 6-4b(7)(c)	Figure 6-19
	(2) Inadequate shear transfer	(a) Provide new steel members and welding	Para. 6-4b(7)(c)	BDM (Figure 6-17b)
	(3) Inadequate chord capacity	(a) Provide new steel members and welding	Para. 6-4b(7)(c)	BDM (Figure 6-17b)
c. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
d. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(a)	Figure 6-21

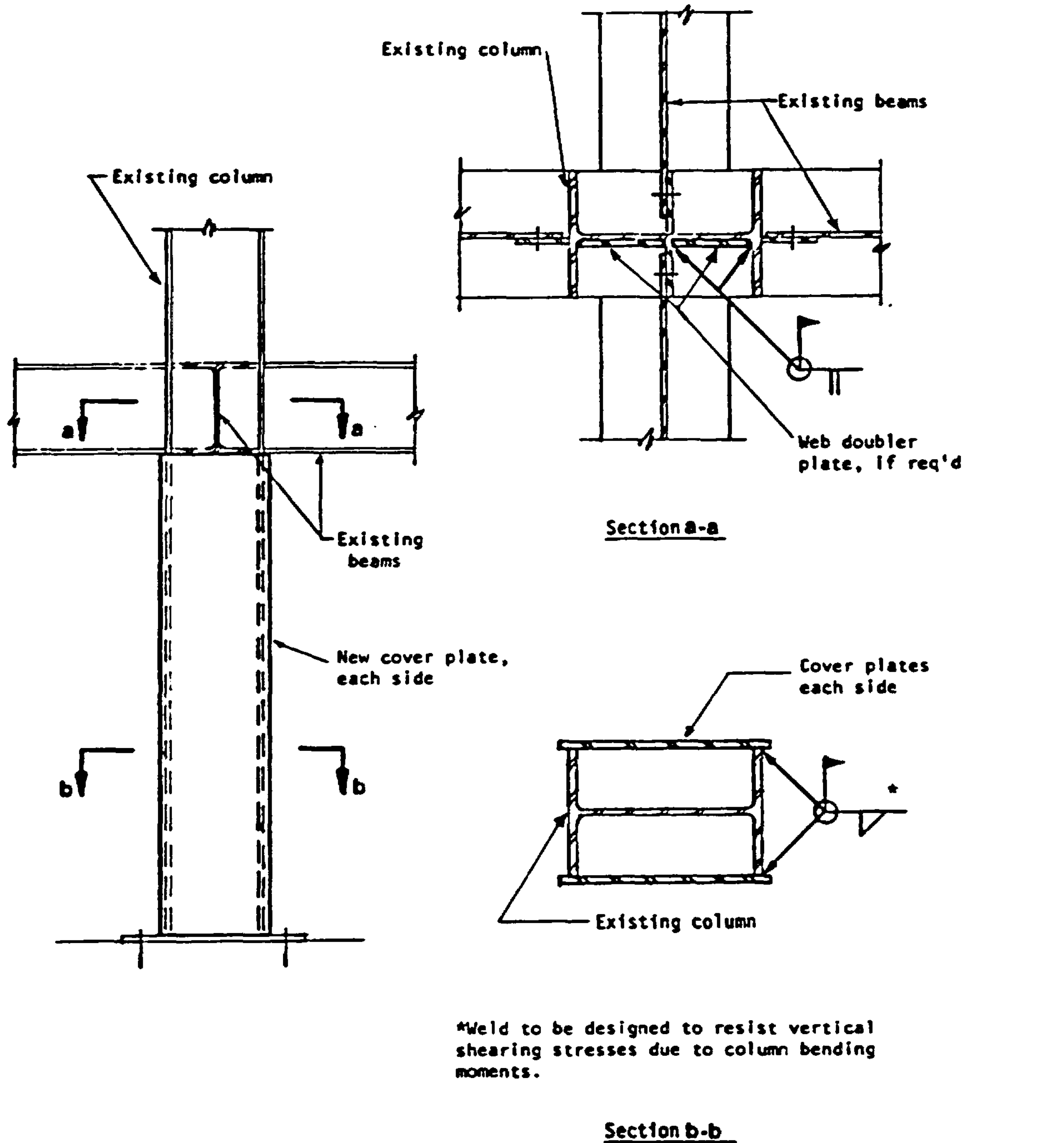


Figure 6–2. Strengthening of existing columns

formed and poured in place or may be placed by the pneumatic method. Paint or other finish on the existing concrete surfaces will be removed and the existing concrete lightly sandblasted to improve the bond of the new concrete.

2. If the identified deficiency exists only in the connecting beams, consideration will be given to acceptance of some minor damage in the form of cracking and/or spalling by repeating the structural evaluation with the deficient beams modeled as pin-ended links between the piers. If this condition is unacceptable, the beams will be removed and replaced with properly designed reinforced concrete as indicated in figure 6–8. An alternative to the above procedures is to fill in selected openings with reinforced concrete as indicated in figure 6–9. The number and location of the openings to be filled in with concrete will depend on functional and architectural as well as structural consideration. If none of the above alternatives are feasible or adequate to ensure the proper performance of the wall, the wall will either be removed and replaced with a new wall that complies with the criteria in chapter 5 or will be supplemented by the addition of new structural elements, as described in paragraph 6–4*d*, that

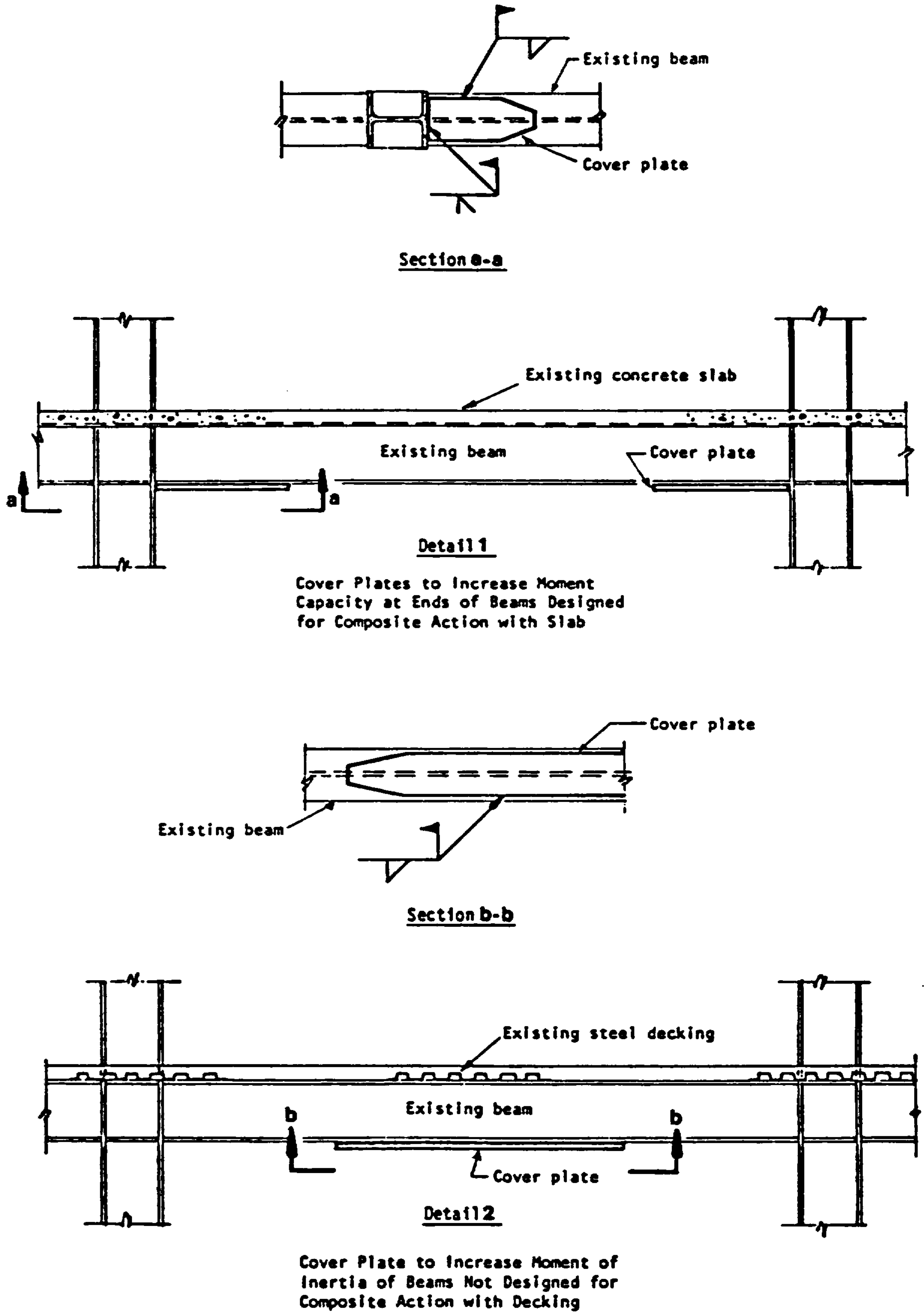
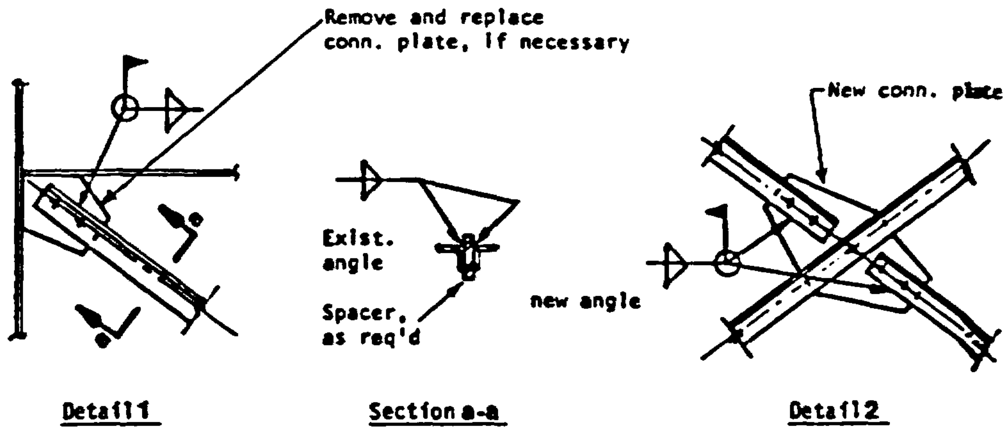
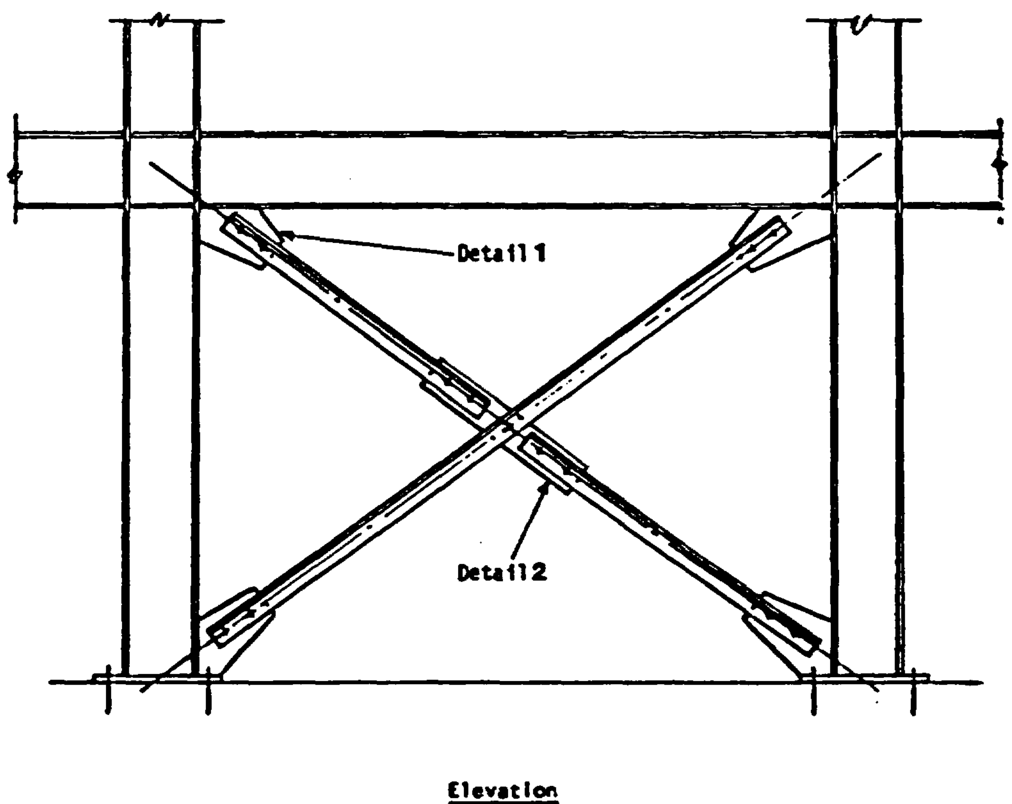


Figure 6–3. Strengthening of existing beams



Upgrading of Single Angle Bracing
to Double Angle Bracing



Existing Single Angle Bracing

Figure 6-4. Strengthening of existing bracing

will reduce the forces on the existing wall to acceptable limits.

(*b*) *Walls without openings.* Existing reinforced concrete walls or piers without openings can be strengthened by the addition of reinforced concrete on one or both sides as described above for walls with openings and as indicated in figure 6-7.

(4) *Unreinforced concrete or masonry walls or piers.* Unreinforced concrete or masonry walls or piers that do not comply with the acceptance criteria prescribed in chapter 5 will be strengthened or will have the seismic demand forces that they are to resist reduced by the addition of new

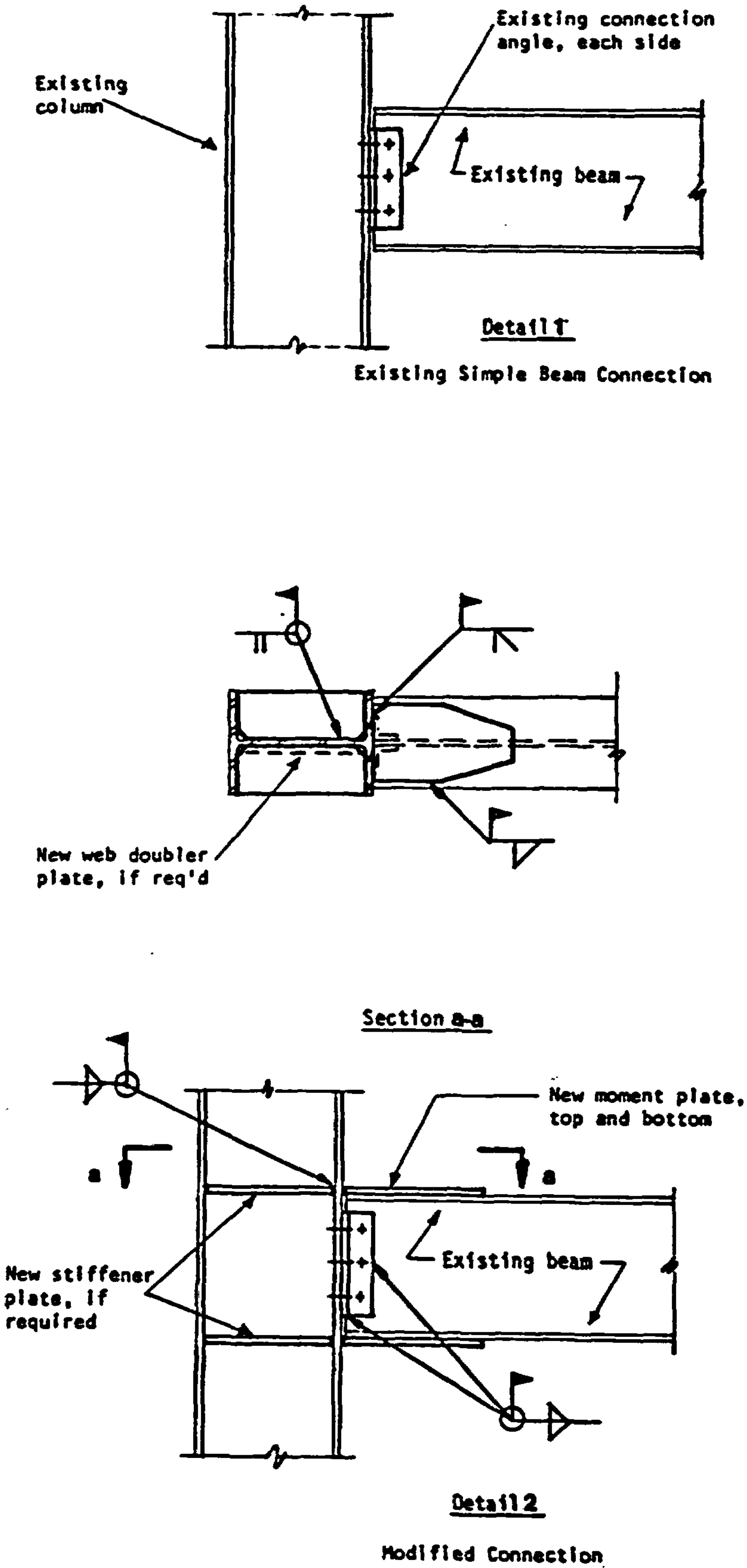


Figure 6-5. Modification of an existing simple beam connection to a moment connection

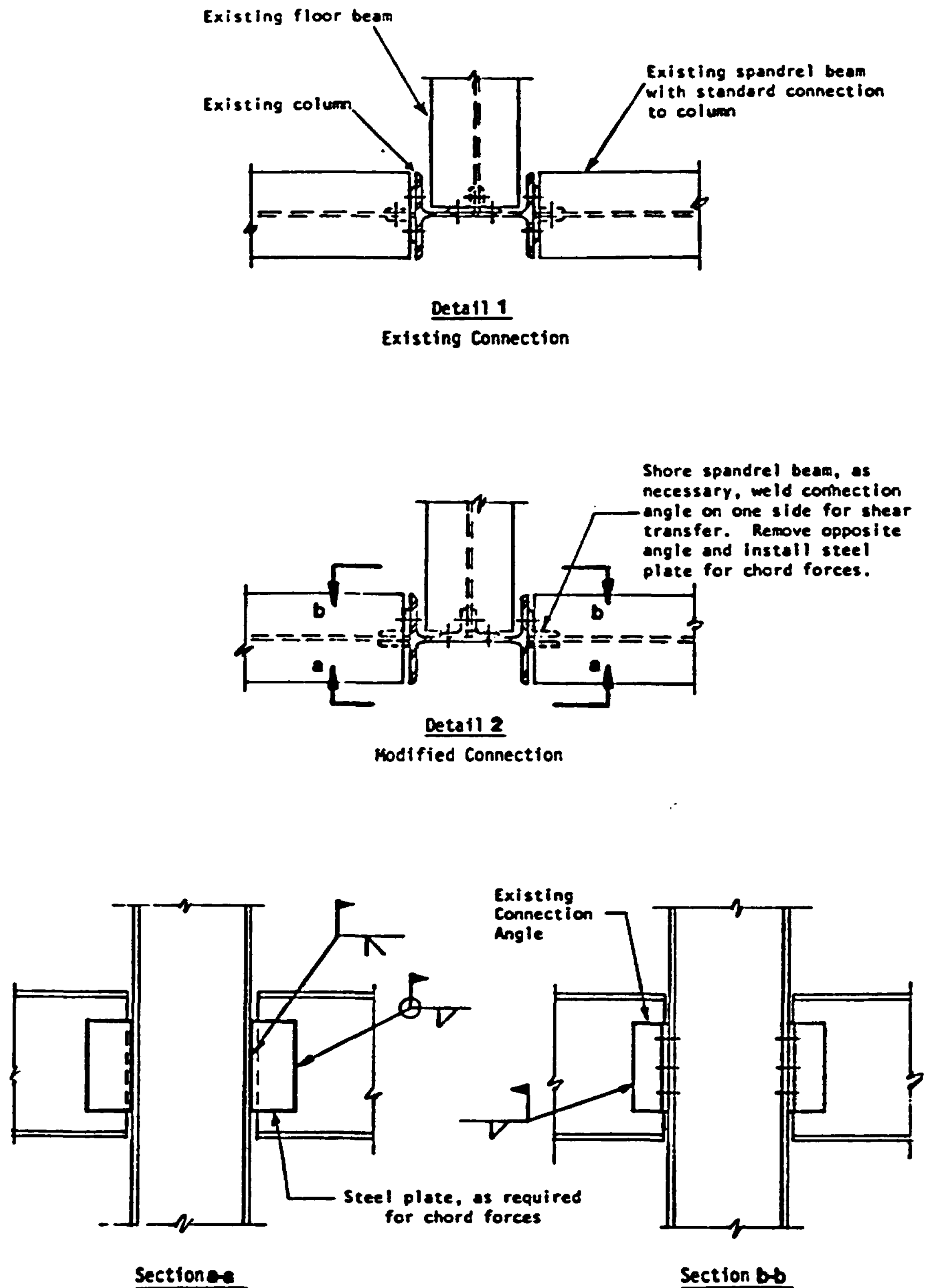


Figure 6–6. Modification of existing steel framing for diaphragm chord forces

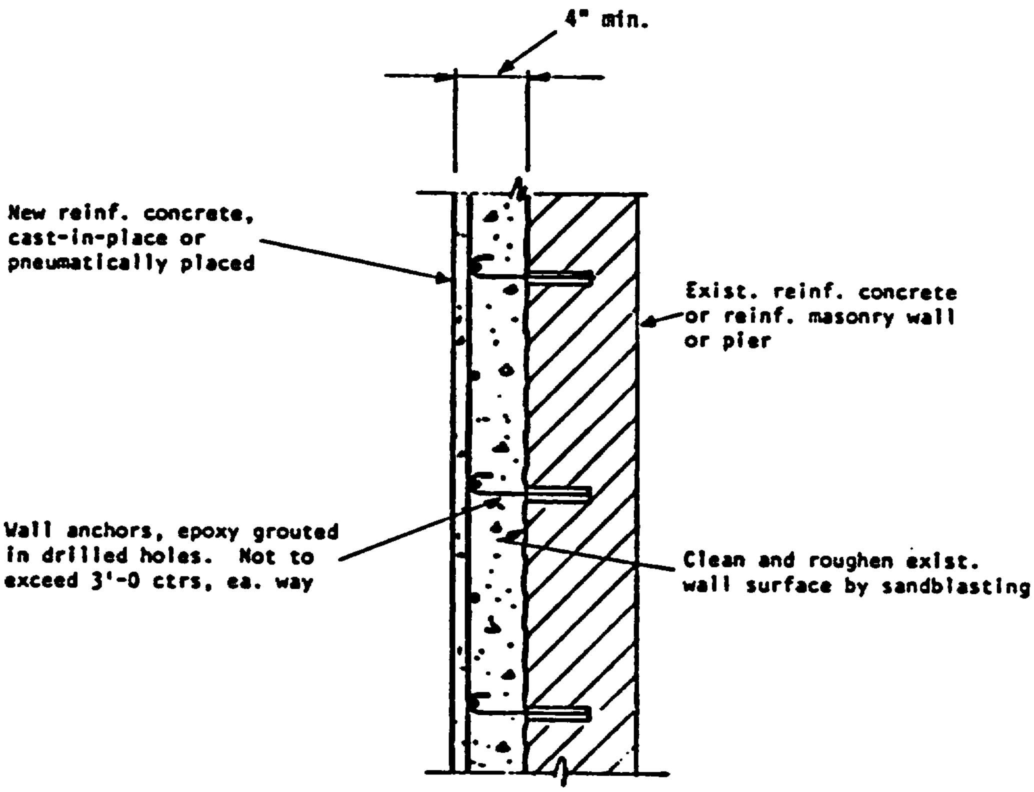


Figure 6–7. Strengthening of existing reinforced concrete or masonry walls or piers

structural elements as described in paragraph 6-6*d*. The strengthening procedure can be similar to that indicated in figure 6–7 for reinforced concrete or masonry. For unreinforced brick masonry walls consisting of four or more wythes of bricks (16 to 18 inches) in thickness, it may be advantageous to remove two or more wythes prior to the addition of the reinforced concrete section as indicated in figure 6–10. This procedure has the advantage of reducing the seismic mass as well as reducing the additional loads on the foundation. For perimeter walls, this procedure is most easily accomplished on the exterior face of the existing wall; however, if the building has historical significance or if the exterior face must be preserved, the procedure can be accomplished on the interior face. Strengthening the interior face of the existing wall will introduce complication because of slabs, beams, or other structural elements framing into the wall that may require temporary shoring, but it will facilitate anchorage of the wall and provide chords for the floor or roof diaphragms as indicated in figure 6–10.

(5) *Timber construction.*

(*a*) *Heavy timber construction.* The seismic capacity of existing heavy timber construction may be upgraded by the use of diagonal bracing or knee braces. The diagonal braces may be either timber or steel and are designed for both tension and compression forces. Figure 6–11 indicates typical details for timber bracing. Knee bracing of existing timber framing may be adequate for moderate seismic forces that can be resisted by the resulting knee braced frame with the columns assumed to be pin-ended. The resulting horizontal shear at the base of the columns must be investigated to determine the need for additional connections to transfer the shear to the floor diaphragm or foundation as indicated in figure 6–12.

(*b*) *Wood stud wall.* Existing wood stud walls in one- and two-story residential or similar construction can be upgraded with 1 x 6 or 2 x 6 let-in bracing as indicated in figure 6–13. The capacity of this bracing will generally be governed by the effective number of nails that have been driven with allowable spacing and end and edge distances. For heavier lateral loads, plywood sheathing of existing stud walls is an effective procedure for providing the necessary resistance. Figure 6–15 of the BDM provides allowable shear values for plywood shear walls. These values may be increased by the ratio of 1.70/1.33 for compliance with the acceptance criteria of chapter 5. Figure 6–13 from the BDM provides similar allowable shear values for various materials other than plywood. These values may also be increased by the above ratio. Other useful design data and typical connection details for wood stud shear walls are contained in chapter 6 of the BDM. Although the BDM data is applicable to code level

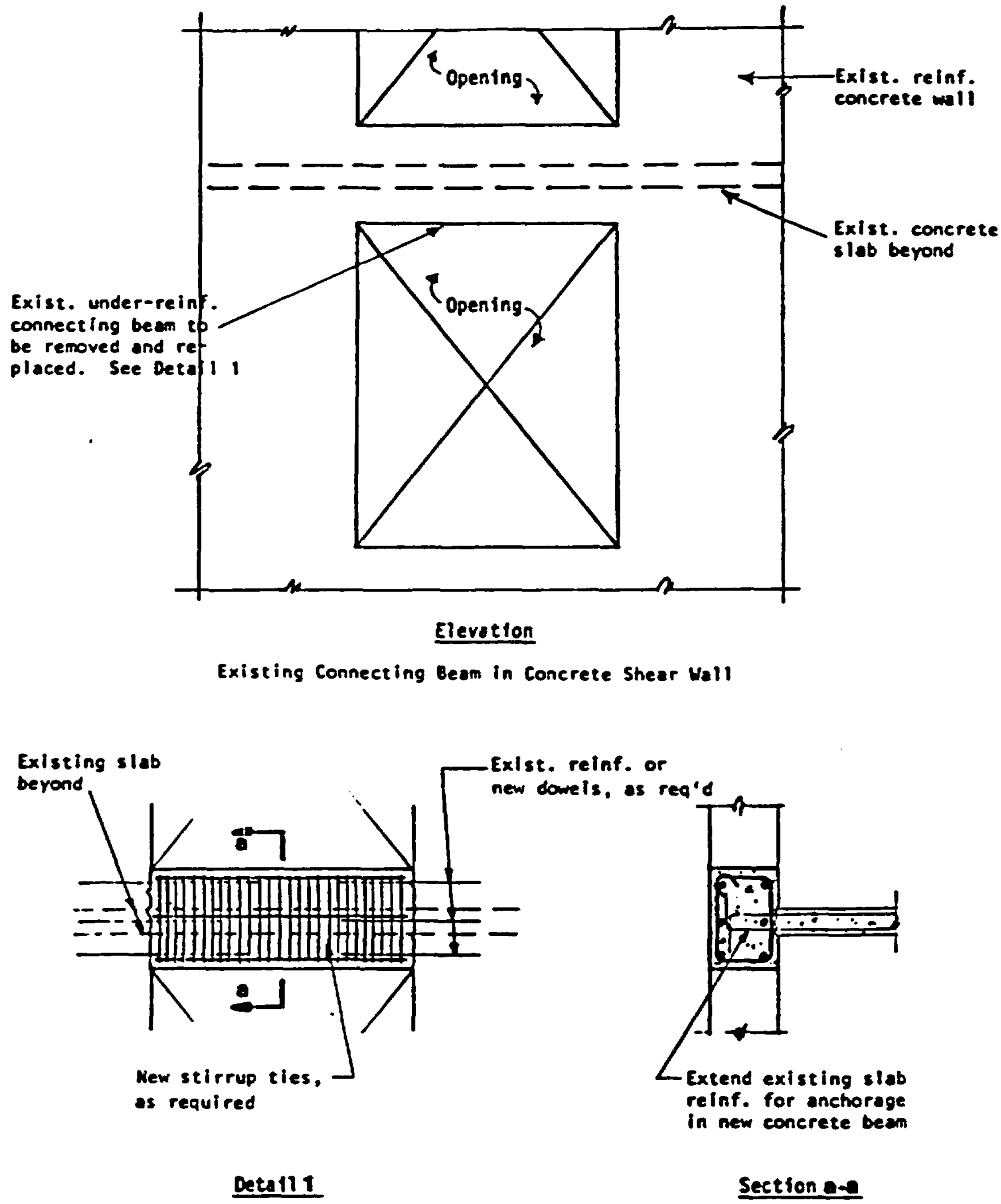


Figure 6–8. Strengthening of existing connecting beams in reinforced concrete or masonry walls

stresses for new construction, similar details with the increased stresses prescribed in the SDG are suitable for seismic upgrading of existing wood stud walls.

(6) *Steel stud walls.* Existing steel stud walls can be upgraded by bracing in the plane of the wall. Typical details are provided in figures 6–17a and 6–17b of the BDM. The indicated details provide a capacity of 1000 lbs for code level forces (allowable stress plus a one-third increase). This capacity may be increased by 1.70/1.33 for yield level capacity.

(7) *Diaphragms.*

(*a*) *Wood floor framing.* Typical details for wood diaphragms are provided in chapter 5 of the BDM. When the seismic evaluation indicates that the existing floor or roof system is inadequate for the necessary diaphragm action, the seismic capacity of the existing system can be upgraded by means of a plywood overlay. The existing floor or roof covering will be removed so that the plywood can be applied to the existing subfloor or roof

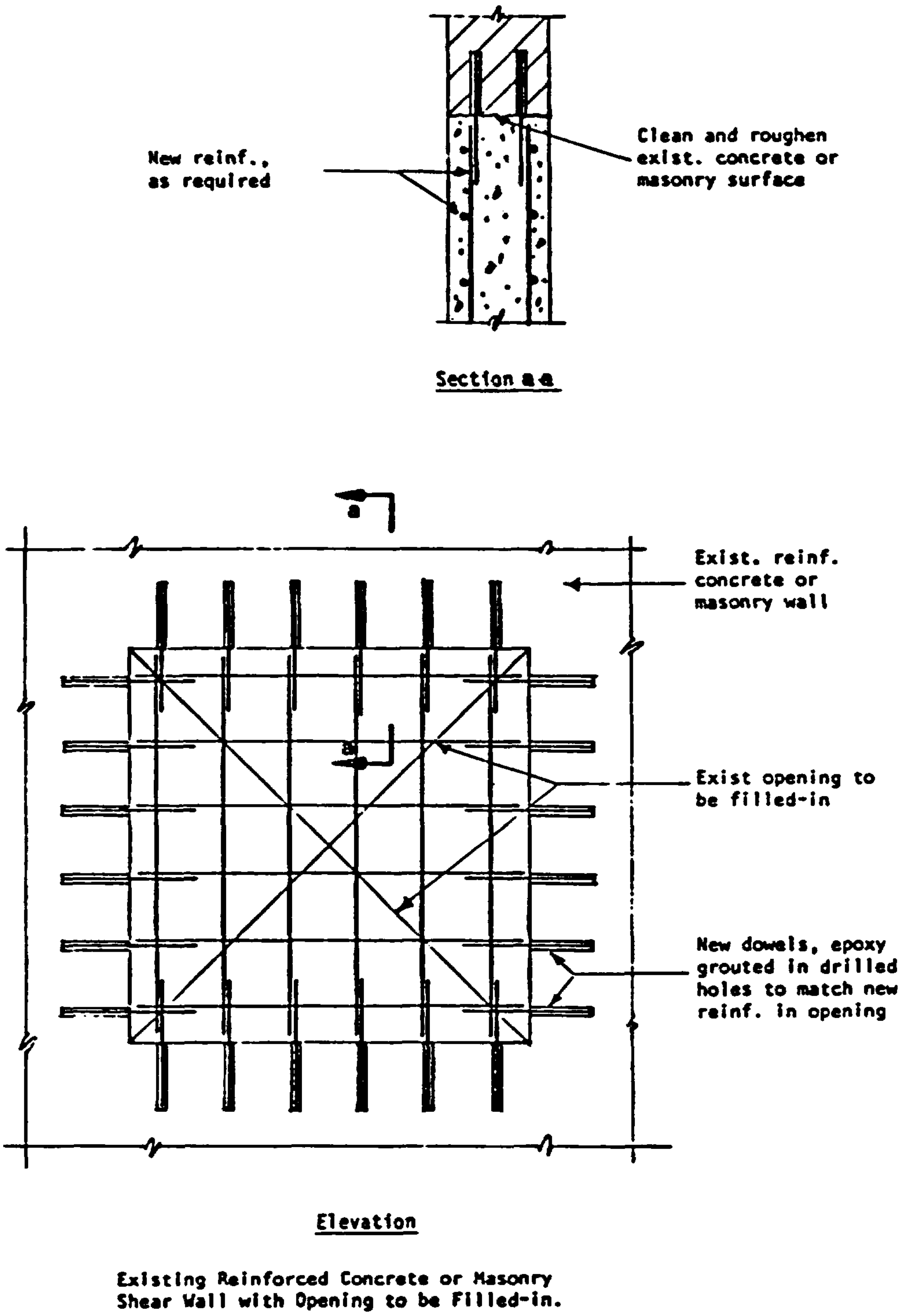


Figure 6-9. Strengthening of existing reinforced concrete or masonry walls by filling in of openings

sheathing. In some cases it may be desirable to remove the subfloor or sheathing to facilitate the installation of the necessary blocking and other connections similar to those shown in figures 5-33 and 5-34 of the BDM. The upgraded diaphragm needs to be evaluated for its capacity to distribute the seismic shear forces to the vertical resisting elements below and also for its capacity to provide resistance to the out-of-plane seismic forces on the exterior walls. The horizontal deflection of wood diaphragms will be checked, in accordance with the provisions of chapter 6 of the BDM, to preclude excessive out-of-plane deformation of masonry or concrete walls. Table 5-6 in the BDM provides allowable shear forces for horizontal plywood diaphragms. These values are to be increased by 1.70/1.33 for the determination of capacity at yield. The diaphragm must also be able to resist

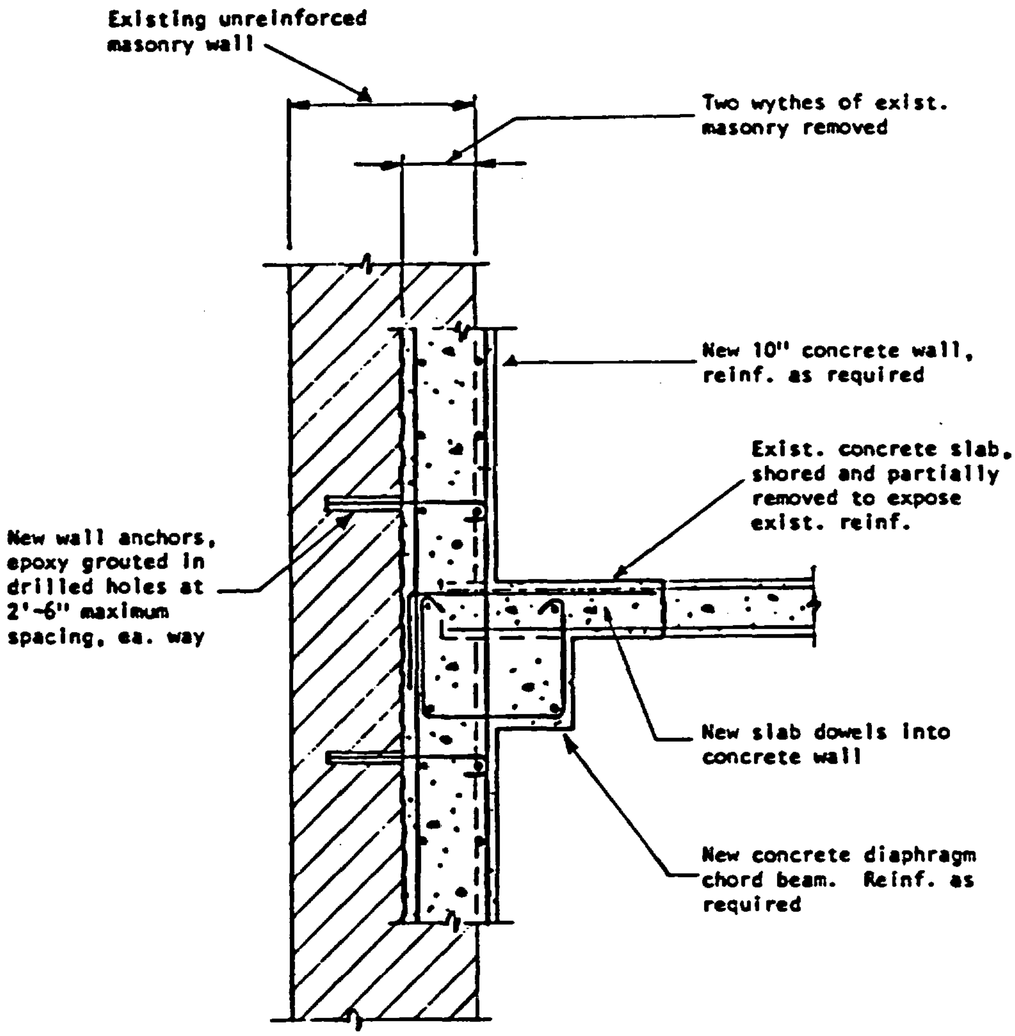


Figure 6-10. Strengthening of existing unreinforced masonry walls or piers

the chord stresses generated by the inertia loads on the diaphragm as it spans horizontally between the vertical resisting elements. Figure 6-14 indicates a detail whereby a continuous steel angle can provide all of the above functions for a wood diaphragm at a reinforced masonry wall. Figure 6-15 indicates details for strengthening of wood diaphragms in steel frame buildings.

(*b*) *Concrete floor or roof systems.* An existing concrete floor or roof system can be seismically upgraded for diaphragm action by the addition of a superimposed diaphragm slab as shown in figure 5-8 of the BDM. Figure 6-16 shows a detail of a superimposed diaphragm slab at an existing masonry wall. Diaphragm chords for upgraded concrete diaphragms can be provided or supplemented by continuous steel angles similar to the detail in figure 6-14. An alternative procedure is to form the chord with a new reinforced concrete beam as indicated in figures 6-16 and 6-17. Large openings in existing concrete diaphragms should be investigated to confirm that excessive shear or flexural stresses will not cause distress to the adjacent areas of the diaphragm. Figure 6-18 indicates how additional reinforcement in the superimposed diaphragm slab can be used to provide supplementary trim bars for openings. An alternative procedure for reinforcement of diaphragm openings is to form a reinforced concrete beam around the perimeter of the opening. An additional alternative is to protect the opening with welded structural sections forming a frame for the opening and anchored to the concrete. Inadequate shear transfer from an existing concrete diaphragm to steel framing can be upgraded with shear studs similar to those in figure 6-19 except that the studs would be welded and grouted in holes cored in the existing slab.

(*c*) *Steel decking.* The seismic capacity of existing steel decking may be updated, in place, only when it does not have a concrete fill slab or when the fill material can be readily removed. If the above condition is met, the deck can be upgraded

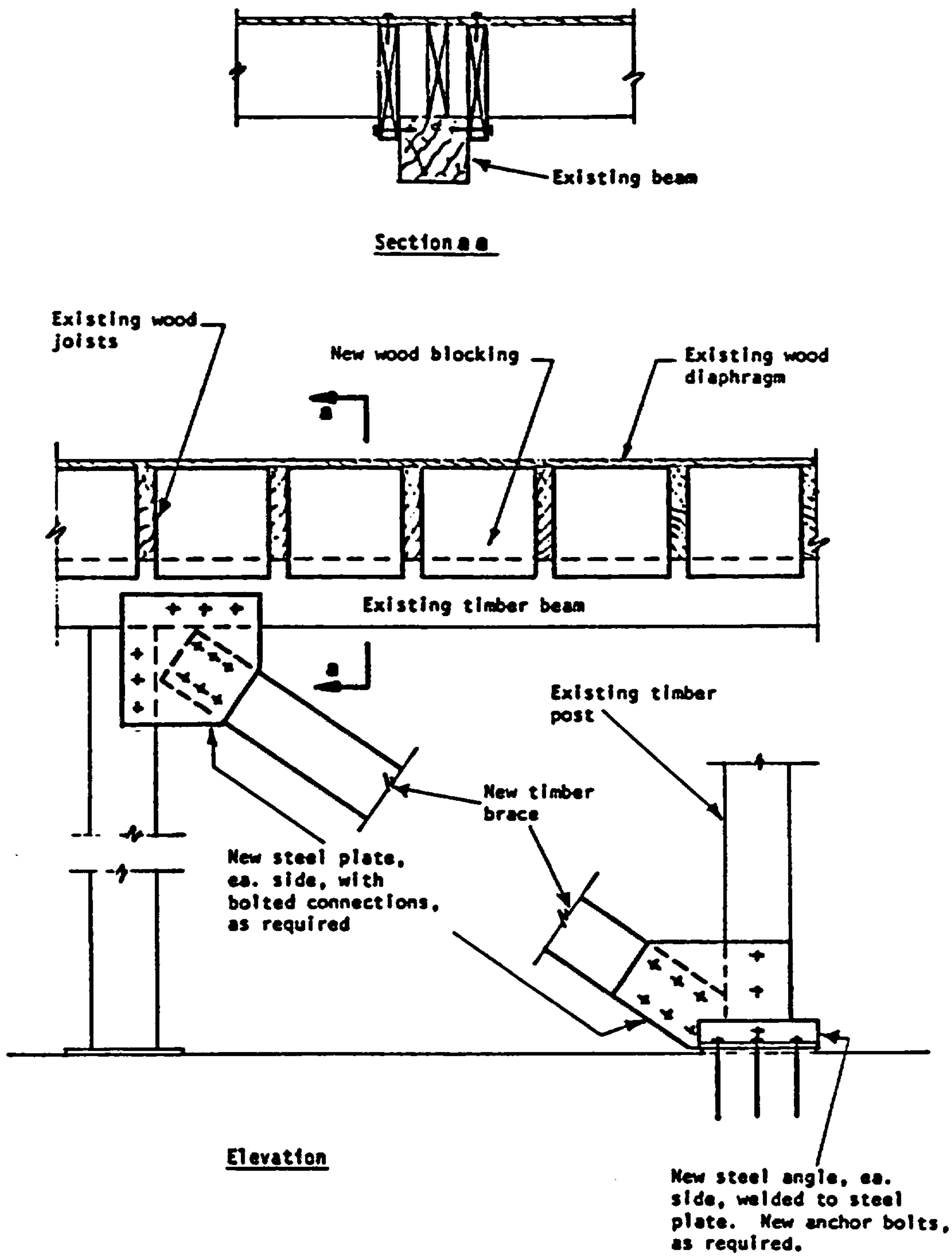


Figure 6–11. Bracing of heavy timber construction

to its maximum capacity by additional welding in accordance with the details indicated in chapter 5 of the BDM. Additional capacity, if required, can be provided by a reinforced concrete fill as indicated in figure 6–19. If the existing steel decking has a concrete fill, but the composite assembly does not have adequate capacity, additional capacity can be provided by a superimposed diaphragm as is indicated in figure 6–16 for an existing concrete slab.

(8) *Foundations.* Strengthening of existing foundations is generally an expensive and disruptive procedure. If the existing foundations are deficient because of the additional seismic loads required by the provisions of this manual, it will usually be more cost effective to reduce the seismic loads on the existing foundations by redistribution of the lateral loads to other new or strengthened resisting elements.

(*a*) *Spread or strip footings.* The bearing

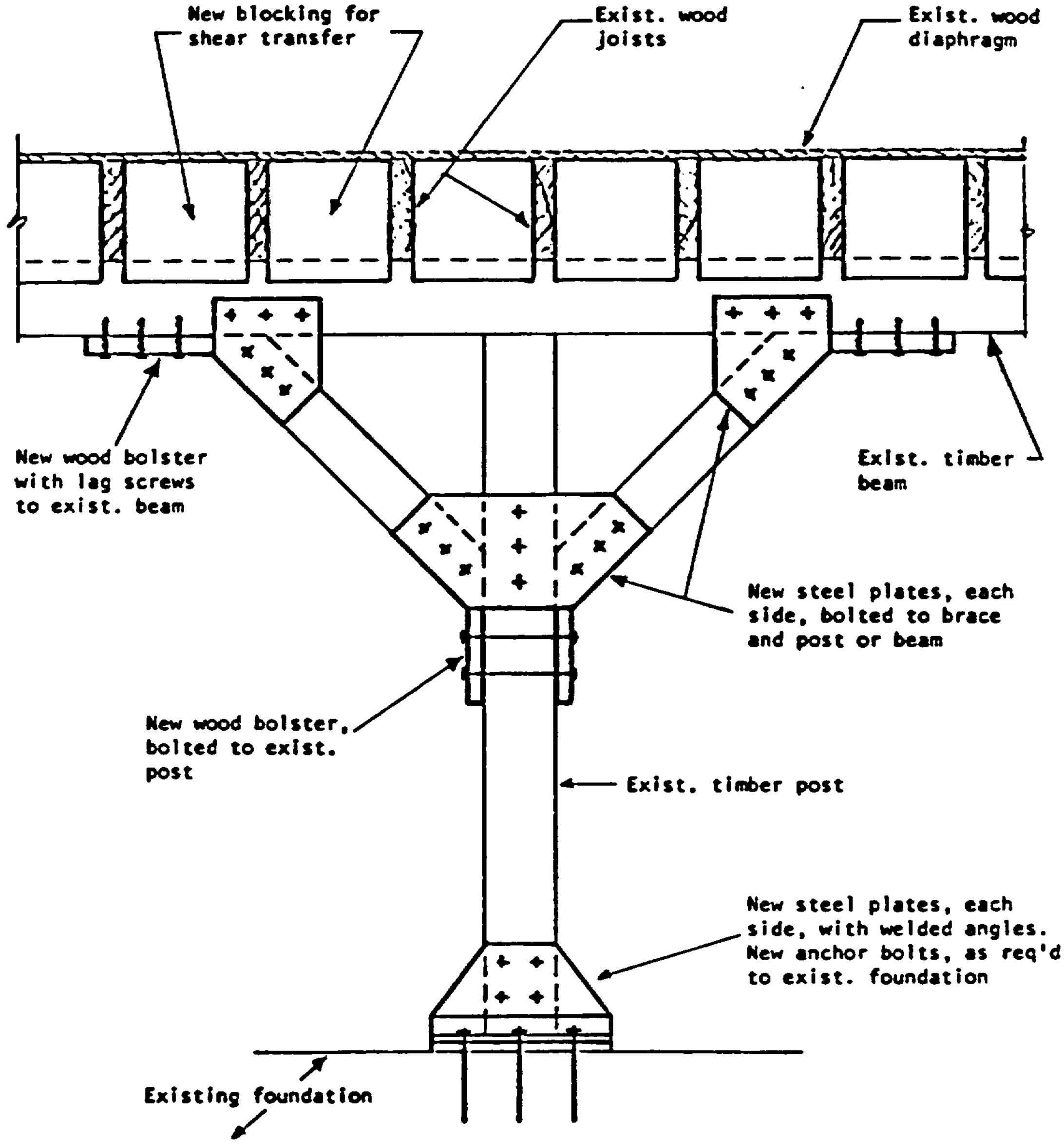


Figure 6-12. Knee bracing of heavy timber construction

capacity of existing strip footings can be upgraded by underpinning as indicated in figure 6-20. The underpinning is performed in spaced segments with the strip footing and wall above providing the rigidity to distribute the loads to the remaining portion of the foundation. As each section of the new foundation is completed and cured it is preloaded by jacking against the existing foundation. When all sections are complete, the space between the new and existing footing is drypacked with nonshrink grout and the jacks are removed and their accesses are also drypacked. Similar procedures can be used to upgrade the capacity of a spread footing. The procedure is only practicable for very large spread footings where a reasonable segment can be undermined and underpinned without significant impairment to the stability of the existing footing.

(*b*) *Piles or drilled piers.* The capacity of an existing pile or drilled pier foundation can be upgraded by the addition of additional piles or drilled piers. This will usually require removal and replacement of the existing pile or pier cap and temporary shoring of the column or other element supported by the foundation. Figure 6-21 indicates a typical detail for a pile foundation supporting a steel column.

c. Replacement of existing deficient structural members. Upgrading of deficient structural members by removal and replacement with larger or stiffer members or systems is a feasible, but not always cost effective, rehabilitation procedure. The replacement procedures are essentially the same as those described in paragraph 6-6*d*. The removal

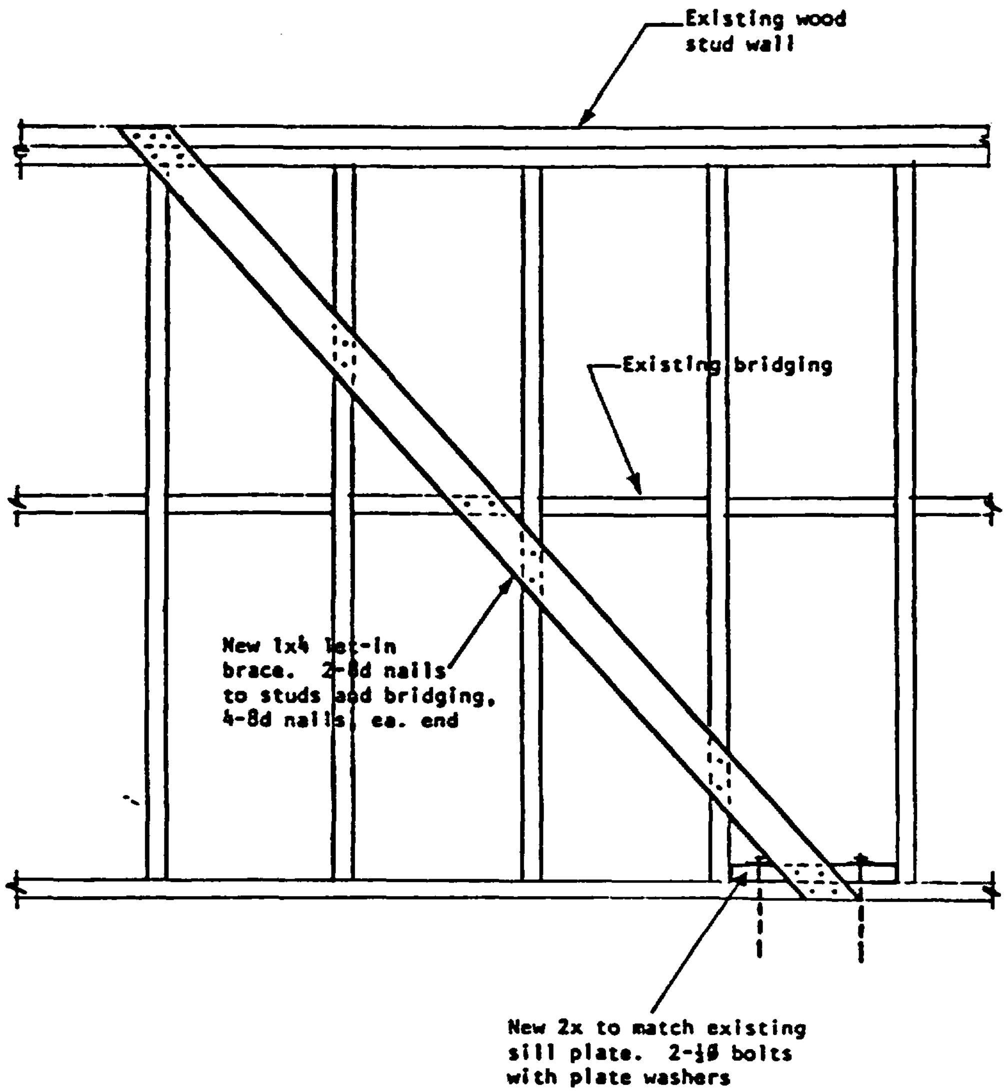


Figure 6–13. Strengthening existing wood stud walls with let-in bracing

procedures must be carefully planned and monitored so as to avoid disturbing construction which is to remain and to provide for proper connections to the replacement construction. This may require temporary shoring or other means of support for the existing construction to remain and careful sequencing of the removal and replacement of the deficient structural members. Figure 6–22 describes the removal of an existing unreinforced masonry wall and replacement with a reinforced concrete wall.

d. Addition of new structural members.

(1) *Rigid frames.*

(*a*) *External frames.* Existing buildings whose lateral force resisting system consists of moment frames that do not comply with the acceptance criteria of chapter 5 may be strengthened by the addition of supplementary external rigid frames of reinforced concrete or structural steel. The effectiveness of this procedure depends on providing new frames of adequate strength and rigidity to reduce the forces and deformation of the existing frames to acceptable limits. The costs associated with this strengthening technique may exceed those for more conventional procedures (e.g., addition of shear walls or bracing); however, a significant advantage of this procedure is that it minimizes disruption of the existing facility as most of the required construction is outside of the building. An adequate existing diaphragm is required, but the system can provide the necessary chord capacity as part of the new frame. Typical details for a supplementary steel frame are shown in figure 6–23.

(*b*) *Interior frames.* Supplementary interior frames may also be used to reduce the seismic forces and distortions of existing framing. The design and construction procedures for new interior frames in an existing building are similar to those for the removal and replacement of existing deficient structural members as described in paragraph 6–4*c*.

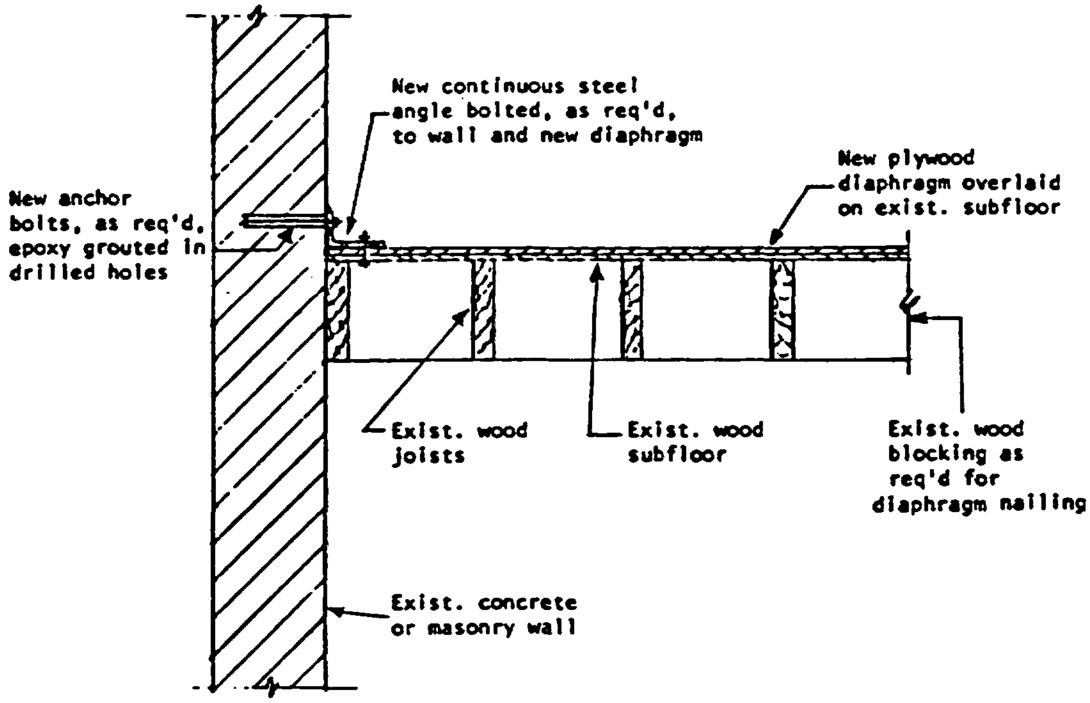


Figure 6-14. Strengthening of existing wood diaphragms in reinforced masonry buildings

(2) *Shear walls.*

(*a*) *Exterior shear walls.* The addition of new exterior shears to a seismically deficient existing building is often an attractive alternative because it minimizes disruption of the internal functions of the building. In wood framed buildings the shear walls could be wood stud shear walls or reinforced masonry shear walls depending on the magnitude of the seismic forces to be resisted and the relative costs of the walls and their attendant foundations and connections. In steel framed buildings or reinforced concrete or masonry buildings, the new shear walls will be either reinforced concrete or reinforced masonry. Figure 6-24 indicates the addition of a new reinforced concrete shear wall in an existing reinforced concrete frame building.

(*b*) *Interior shear wall.* The addition of new interior shear walls in an existing building may be seriously constrained by the need to maintain an essential function during the construction period. To minimize interference with the functional operations, it may be necessary to consider alternatives that may be more expensive in terms of material and installation costs, but which will minimize the construction time and the disruption of operation within the building. Steel shear walls have been successfully installed under such conditions as indicated in figure 6-25. Prefabrication of the walls is utilized to the maximum extent possible and temporary dust barriers are installed to protect adjacent functions in the building. In buildings that do not have the above constraints, more conventional alternatives, similar to those described in paragraph (*a*) above, may be more suitable. Although it is usually desirable to locate the new shear walls along frame lines (i.e., framing into existing columns and beams) to provide boundary members; to provide dead loads to help resist overturning forces; and to take advantage of existing column foundations, other considerations may dictate wall locations away from the existing frame lines. This condition is illustrated in detail 1 in figure 6-24. Except for very thick slabs, this detail will probably require supplementary members, such as the steel angles shown in the detail, to set as collector members extending beyond the shear wall to facilitate transfer of the diaphragm shear from the slab to the shear wall.

(3) *Vertical bracing.* The addition of new vertical steel bracing is usually a cost effective procedure for upgrading the seismic resistance of an existing steel frame building. The new bracing may be installed to supplement existing bracing or to reduce the seismic forces and displacements of existing moment frames. In low-rise buildings (less than about 6 stories) with average site conditions, the addition of bracing to a moment frame structure will usually mean an increase in the spectral seismic demand as the greater stiffness of the bracing will almost ensure that the building will respond at the maximum amplification of the response spectrum. Also, because of the greater relative rigidity of the bracing, it will usually be required to resist a larger share of the seismic forces than the moment frames. Concentric bracing may be diagonal bracing, x-bracing, or k-

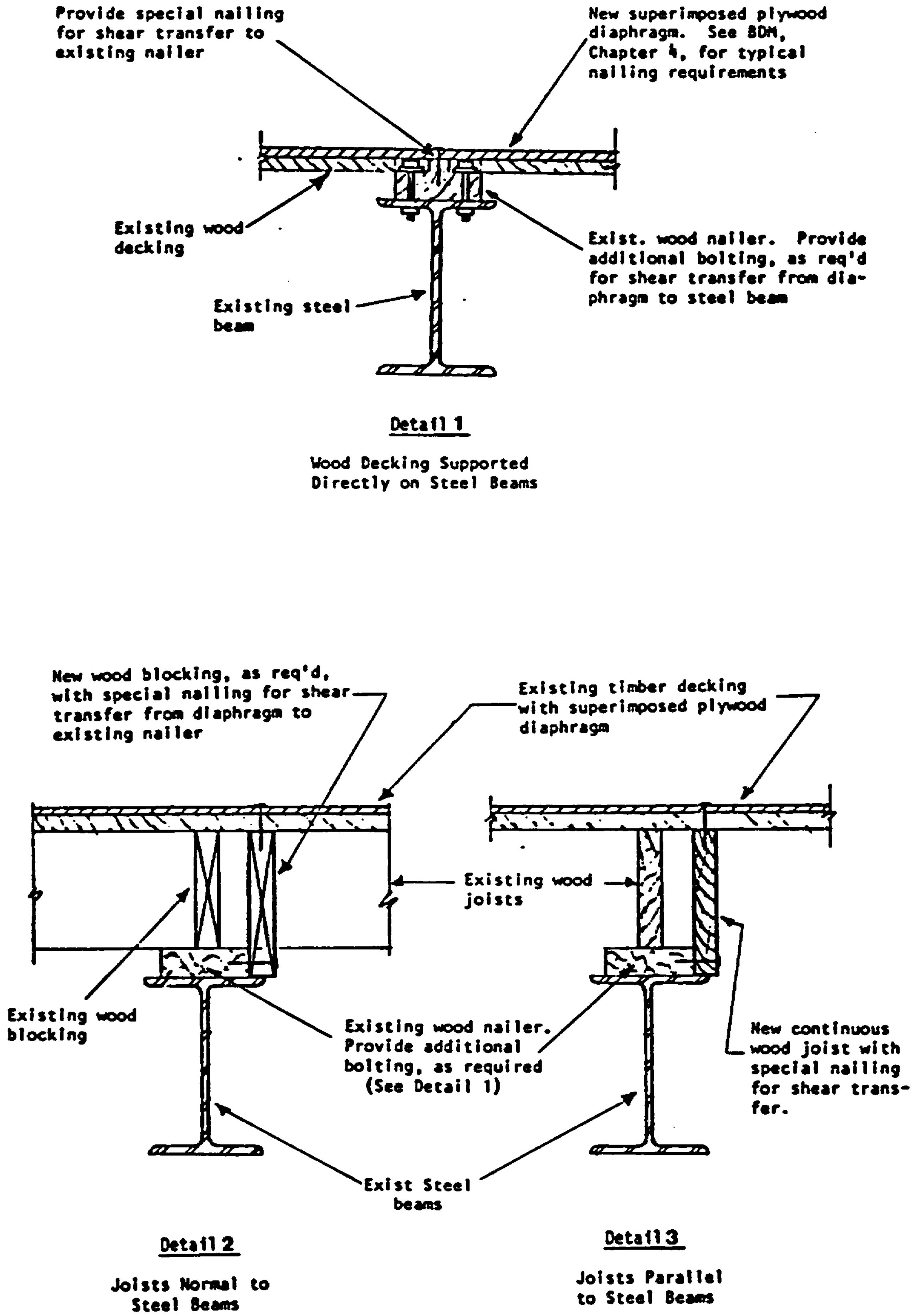


Figure 6-15. Strengthening of existing wood diaphragms in steel frame buildings

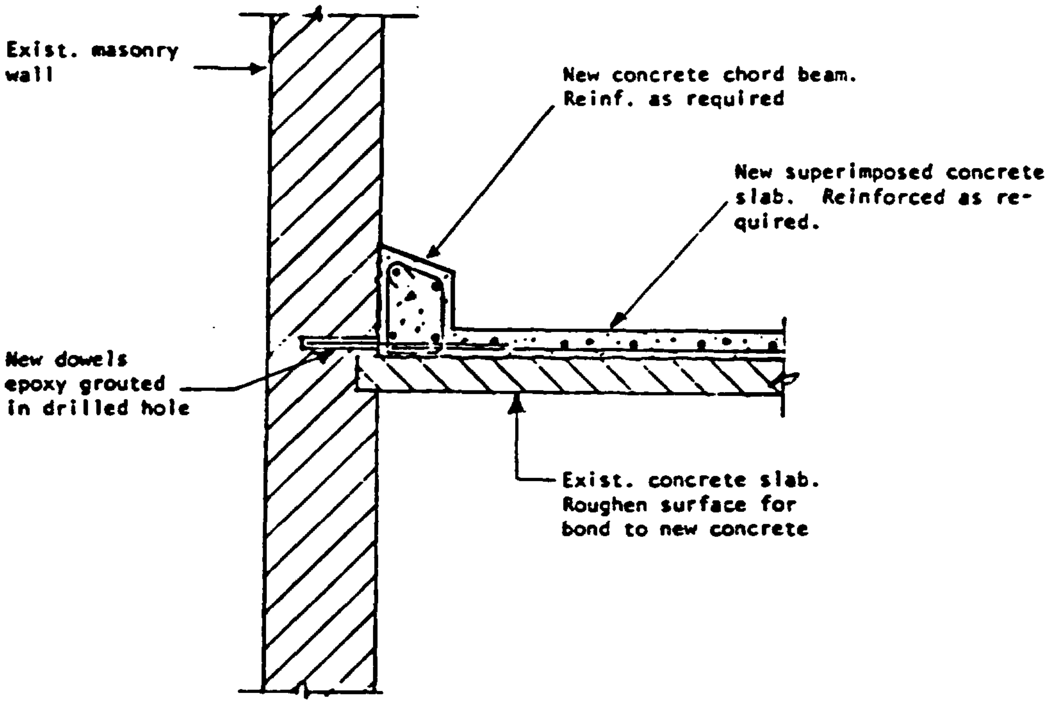


Figure 6-16. Superimposed diaphragm slab at an existing masonry wall

bracing and is usually designed to act both in tension and compression. In recent years eccentric bracing has been promulgated as a means to provide the strength and stiffness of bracing with much of the ductility associated with moment frames. Figure 6-26 indicates typical details for eccentric bracing. In the determination of the type, members, and location of additional braced bays in an existing building an important consideration is the cost associated with the required modification or strengthening of the columns, beams, and foundations as a result of the new bracing. Some of these costs may be significantly greater than the bracing itself so that the use of additional braced bays may be justified if the forces can be reduced so as to eliminate costly modification of the existing structural system.

(4) *Horizontal bracing.* A horizontal structural

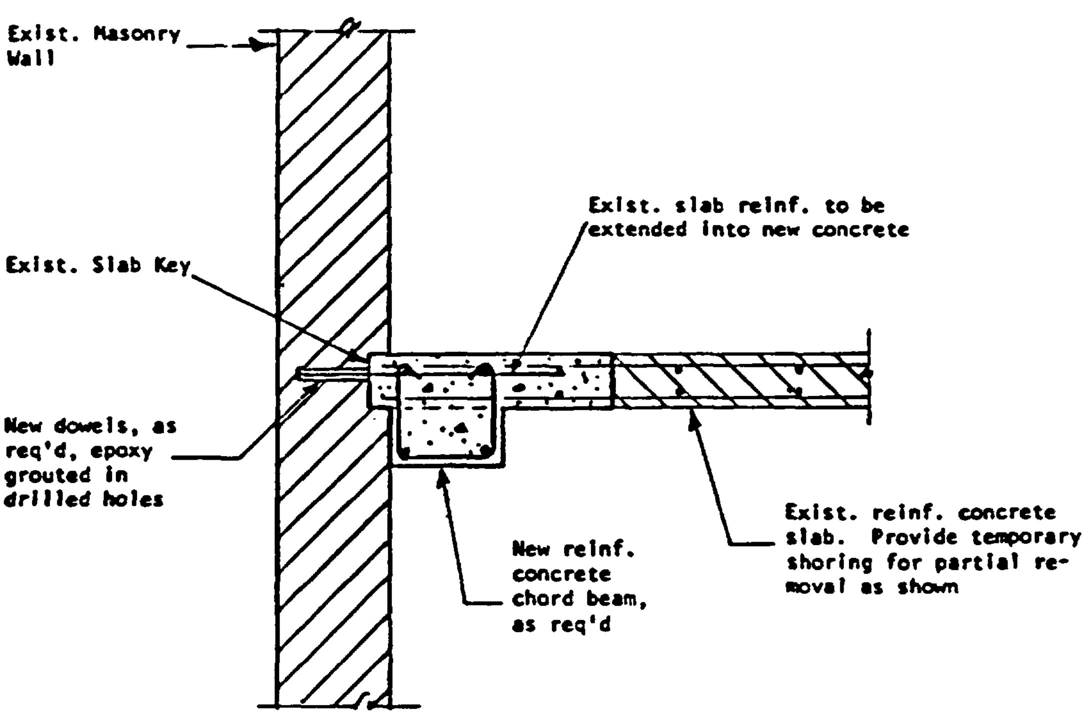


Figure 6-17. Diaphragm chord for existing concrete slab

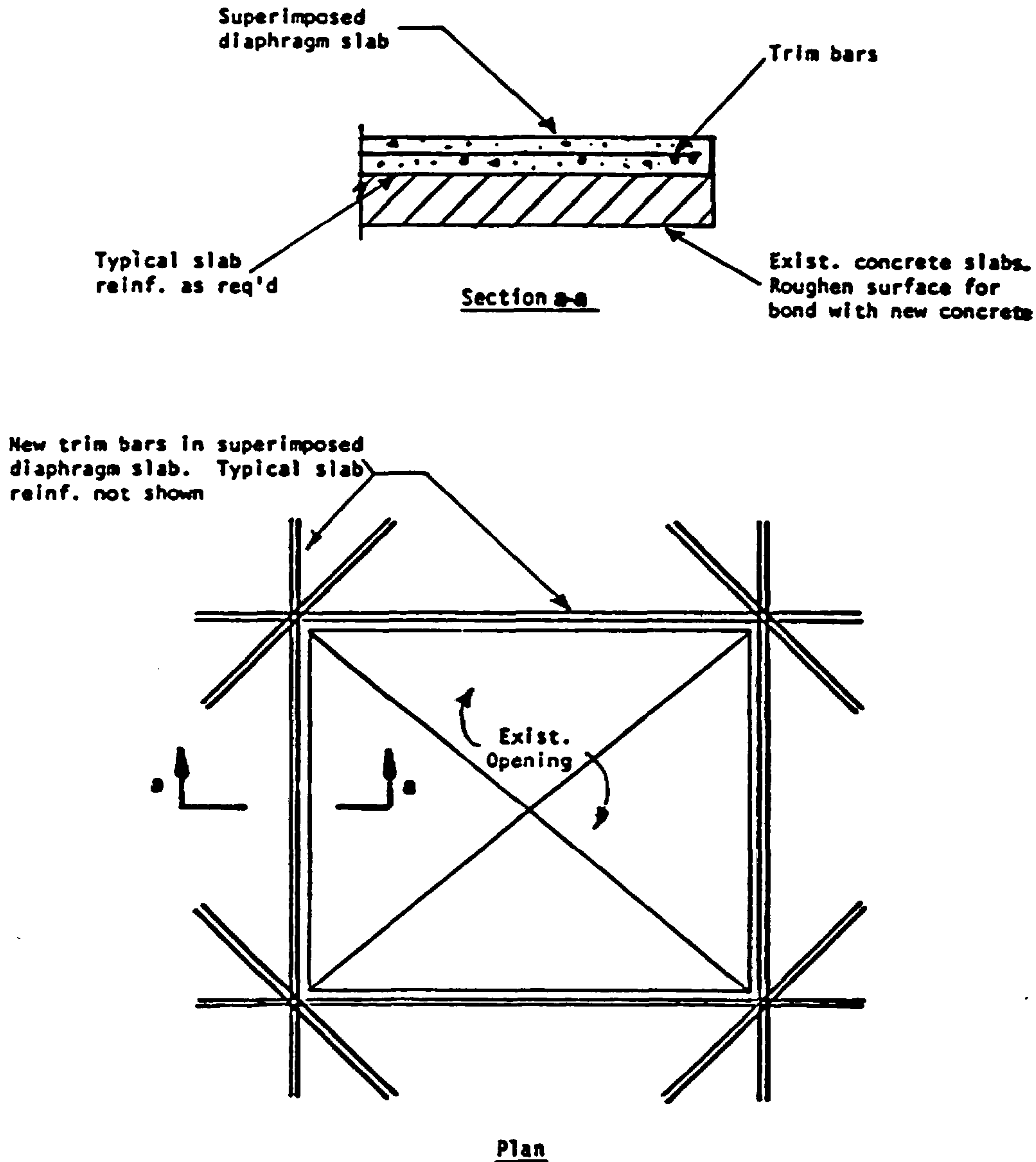


Figure 6-18. Strengthening of openings in a superimposed diaphragm

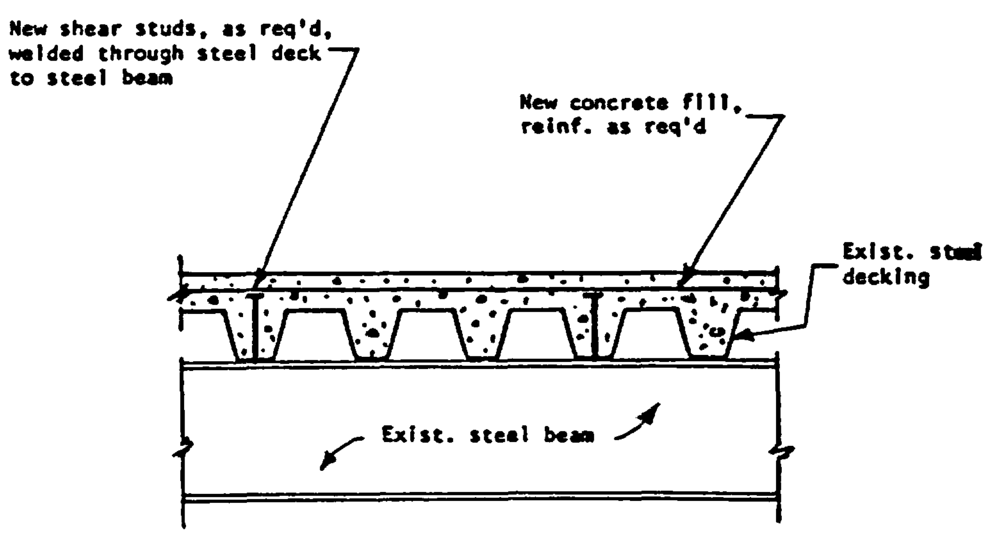


Figure 6-19. Strengthening of existing steel deck diaphragms

steel bracing system may be used as a diaphragm in existing buildings where the existing floor or roof system has not been designed for the required diaphragm action. This procedure will usually be limited to relatively flexible existing floor or roof systems such as timber or steel decking where the horizontal bracing can provide the necessary rigidity and transfer of the floor or roof shears to the vertical resisting elements. The bracing must be properly connected to the floor or roof system to pick up the inertia loads originating at that level and must include the necessary chords and connections to the vertical elements to provide the necessary out-of-plane support for these elements and to transfer the diaphragm shears to the resisting elements. Figure 6-27 indicates the use of horizontal bracing system in an existing steel frame building with concrete walls. It may not be necessary, in all cases, to provide crossbracing in all bays of the floor or roof system as shown in figure 6-27 if the existing system can be relied upon for some diaphragm action. The bracing configuration in figure 6-27 assumes negligible diaphragm capability (i.e., only adequate to transfer inertia forces between adjacent horizontal framing members) in the existing floor system.

(5) *External buttresses.* The use of external buttresses of reinforced concrete or braced structural steel could be feasible for strengthening an existing building. The buttresses may be designed to resist compressive forces only, in which case they must be located on both sides of the building in each direction to be braced. If the building has a structurally adequate diaphragm, the members and location of the buttresses will be determined to resist the calculated seismic forces and to minimize torsion. If the building has a flexible diaphragm, the buttresses must be designed for the tributary seismic forces and must be located where the existing framing can transfer the tributary loads by strut action. For example, in an existing steel frame building with a flexible diaphragm, the girders in the transverse direction may be assumed as receiving the seismic inertia loads from the secondary floor beams connected to the girders and buttresses would be located at each girder line. In the longitudinal direction, a similar assumption would be made regarding the secondary beams and buttresses would be located at every third or fourth beam assuming the existing floor system is capable of transferring the tributary floor loads. Buttresses that are designed to resist both tensile or compressive forces may be located on only one side of the building in each direction, but will require an adequate connection to be made to the existing building to transfer the tensile forces to the buttress. Figure 6-28 indicates the use of braced structural steel buttresses to strengthen an existing reinforced concrete building.

(6) *Structural additions.* An existing deficient building may be strengthened by a structural building addition that is designed to resist the seismic forces generated within the addition as well as all or a portion of the forces from the existing building. This alternative has the obvious advantage of generating additional useful space while upgrading the existing building. The design considerations are similar to those indicated in the above paragraph for buttresses except that the additional seismic forces from the new addition also have to be considered in the design of the resisting elements.

6-5. Upgrading of nonstructural elements

The evaluation of the adequacy of supports, anchorages, or bracing of nonstructural elements to resist the imposed seismic forces and displacements in existing buildings will be based on the results (story accelerations and interstory drifts) of the initial detailed structural analysis of the building or, if extensive structural modifications are required, on the subsequent reanalysis of the recommended concept. Analytical and acceptance criteria for these elements are provided in chapter 9 with typical details for seismic upgrading. Elements to be evaluated for upgrading will include:

a. Mechanical/electrical equipment (i.e., emergency motor generators, heating, air conditioning, and ventilation systems, electrical control panels, elevators, water and sewer lines, and sprinkler piping).

b. Suspended ceilings and light fixtures.

c. Nonstructural partitions.

d. Structural appendages (i.e., penthouses, parapets, canopies, and precast concrete curtain wall units).

e. Glazing (i.e., large glass panels).

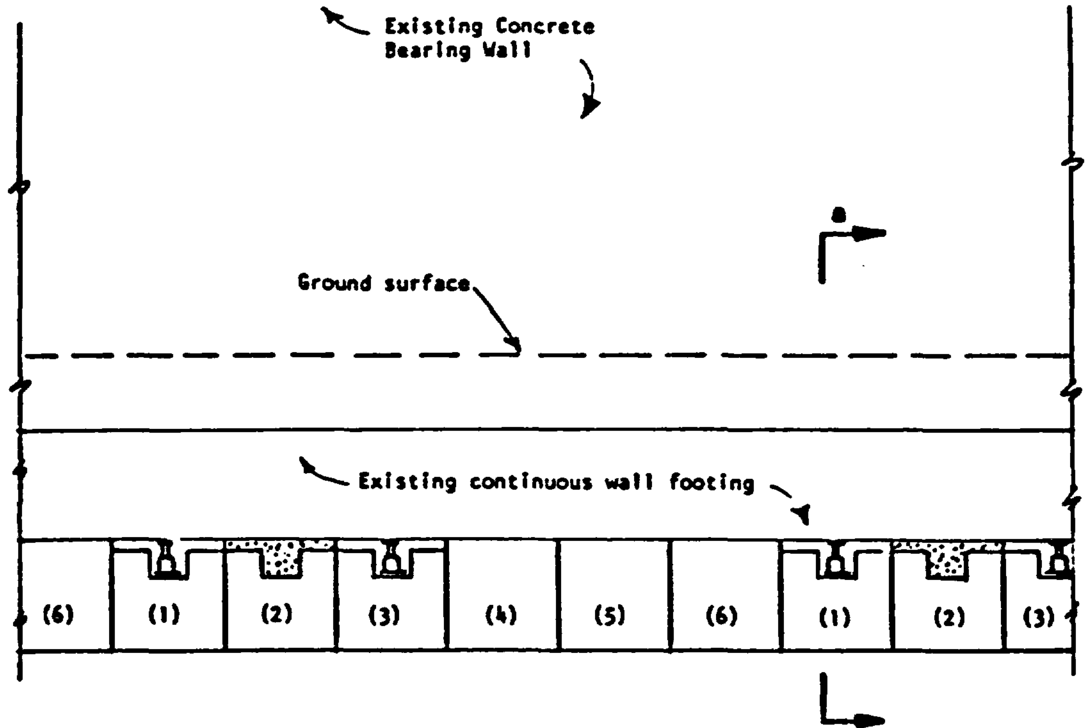
f. Miscellaneous (i.e., computer floors and free-standing storage units).

g. Storage shelving (i.e., supporting hazardous or essential items).

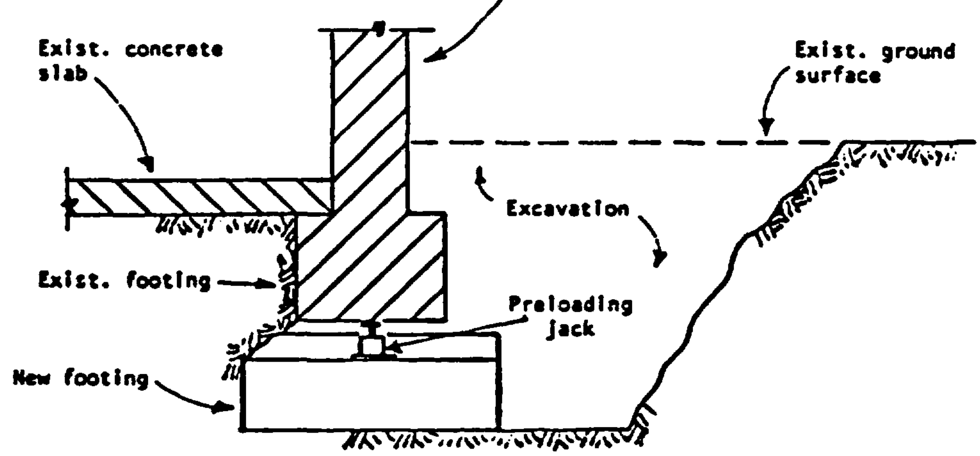
6-6. Concept submittal

A concept submittal will be prepared for review and approval by the approval authority. The submittal will comply with agency standards and will generally represent 25 to 35 percent of the effort required to complete the design of normal projects. The concept submittal will include the following elements:

a. Basis for design. This will include the acceptance and design criteria; a summary description



Elevation



Section a-a

Sheet 1 of 2

Figure 6-20. Underpinning of an existing footing. (Sheet 1 of 2)

Construction Sequence:

1. Perform excavation and underpinning sequentially in widely spaced segments as shown [e.g. segments designated (1)].
2. Pour new concrete footing with jacking pocket and gap for drypacking.
3. When new concrete in (1) has attained adequate strength, place jacks to preload footing.
4. Repeat procedure for segments (2) and (3).
5. When jacks have been placed and loaded for (1), (2), and (3), drypack (2) and remove jack.
6. Proceed with remaining segments until complete footing is underpinned.

Sheet 2 of 2

Figure 6-20. Underpinning of an existing footing. (Sheet 2 of 2)

of the deficiencies identified in the detailed structural analysis; a narrative description of the alternative upgrading concepts; and justification for the recommended concept.

b. Calculations. Edited, checked, and indexed calculations will be included in the submittal to support the design of the upgrading concepts.

c. Cost estimates. Comparative cost estimates for the alternative concepts and a complete preliminary cost estimate for the recommended concept.

d. Schematic drawings. Schematic drawings will be prepared for the recommended concept. The drawings must be adequate to describe the nature, extent, and location of work required and, as a minimum, will include foundation and framing plans, typical sections, and typical connection details.

e. Outline specifications. Outline specifications will be prepared to describe the type and grade of structural material and procedures by reference to standard or industry specifications.

f. Schedules. The concept submittal will include the estimated number of man-hours to complete the design and estimated schedules (calendar days) for the design submittals, reviews, bidding, contract award, and construction.

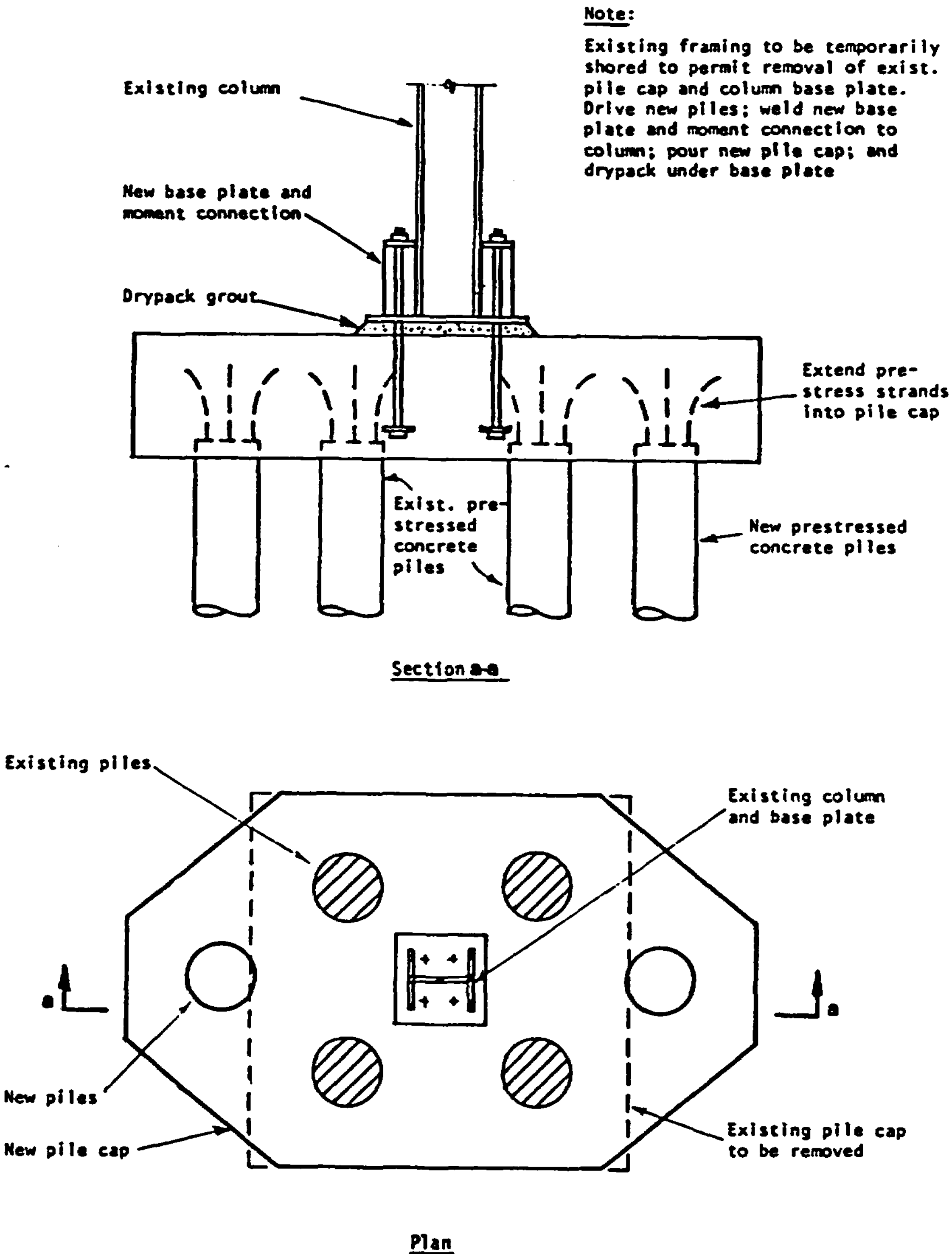


Figure 6-21. Upgrading of an existing pile foundation

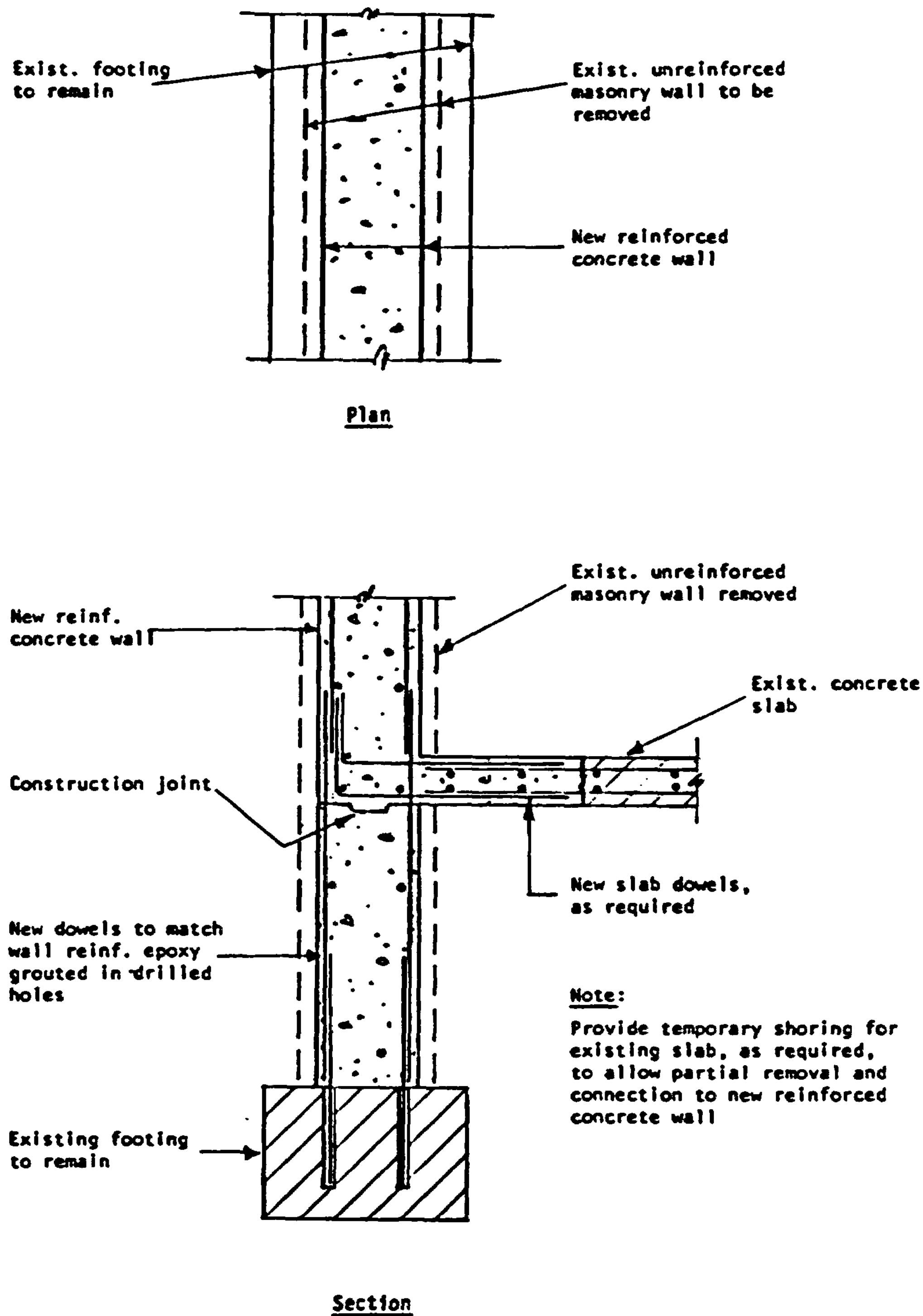
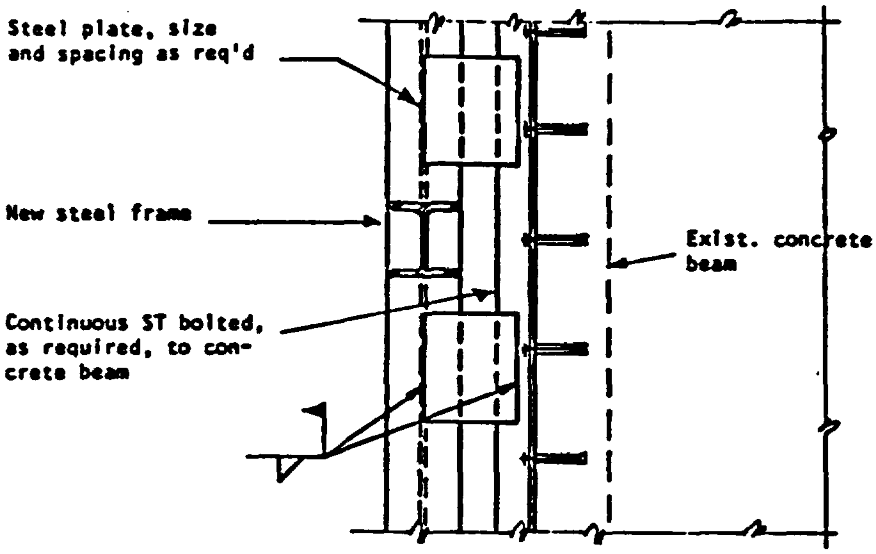
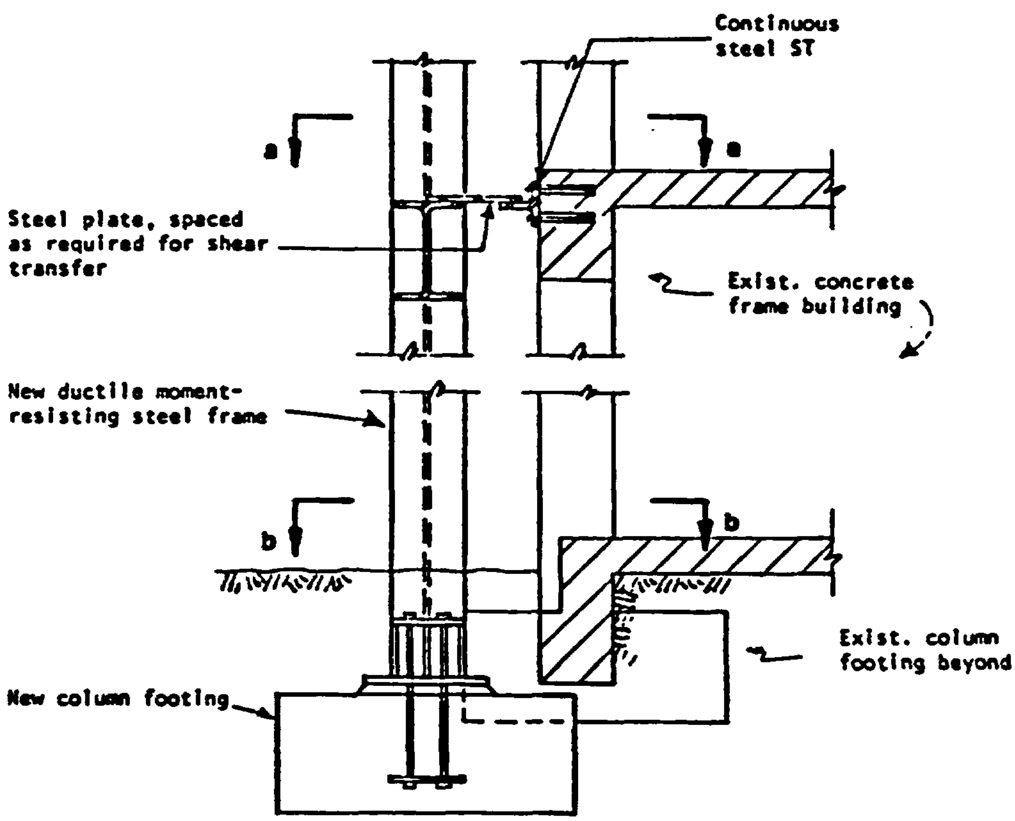


Figure 6-22. Removal and replacement of an unreinforced masonry wall



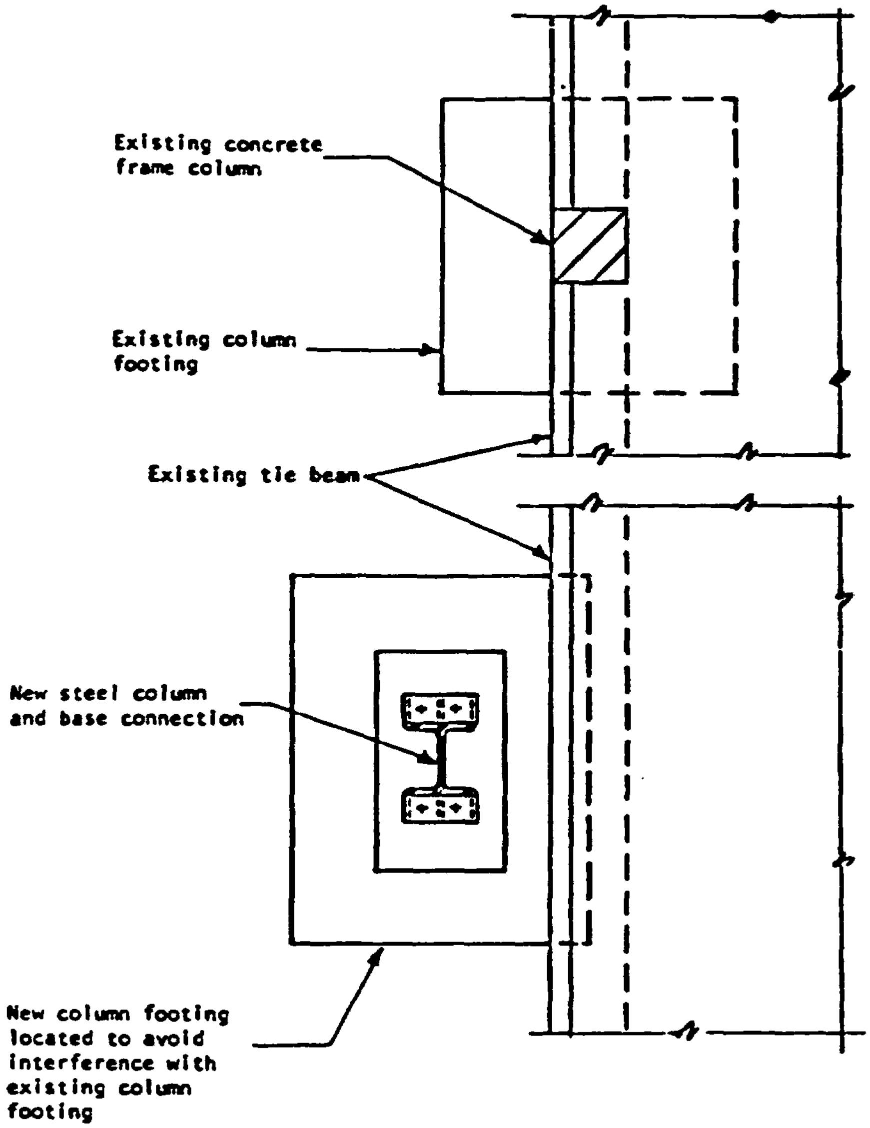
Section a-a



Section at Existing Wall

Sheet 1 of 2

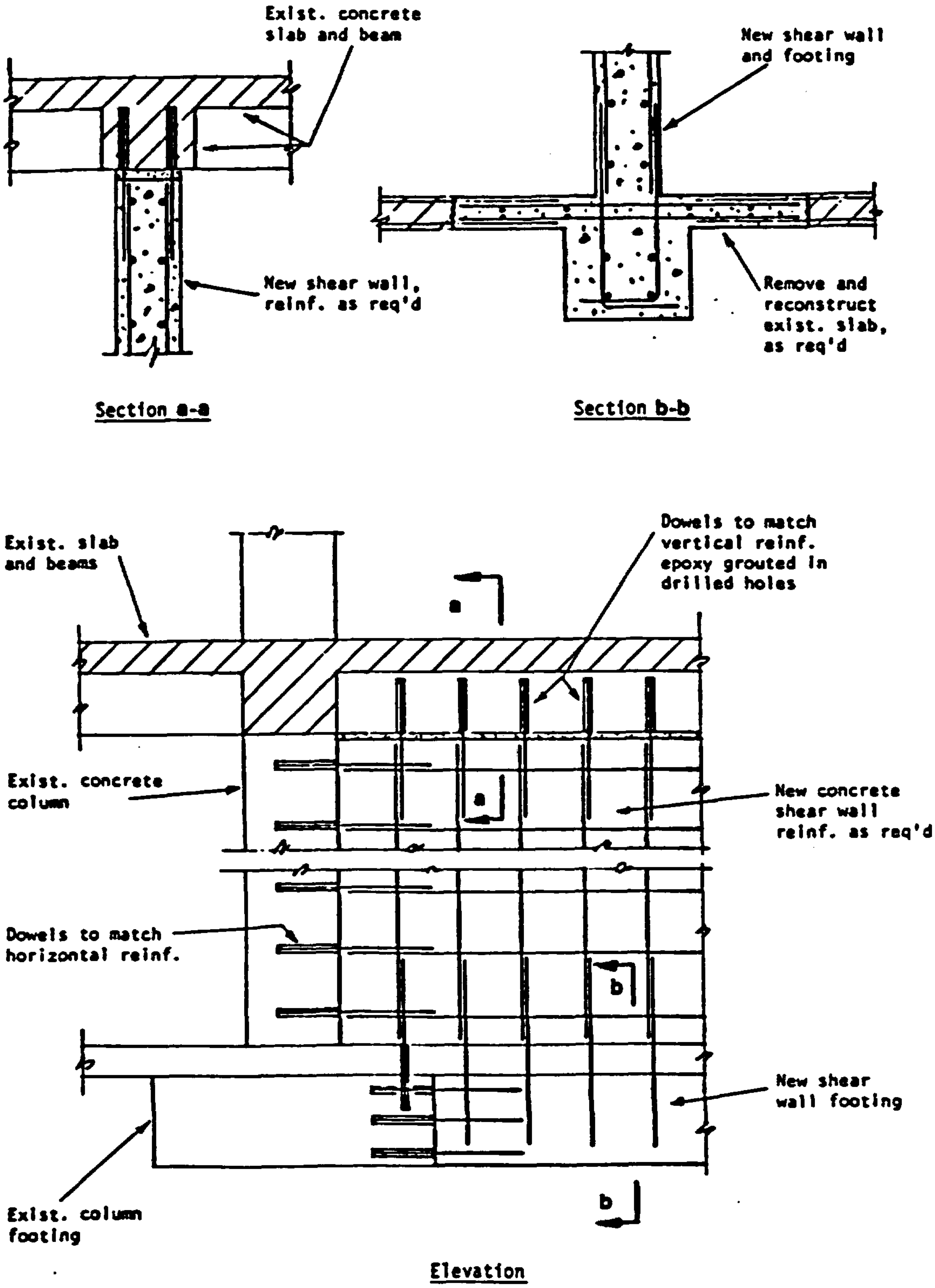
Figure 6–23. Upgrading an existing building with external frames. (Sheet 1 of 2)



Section b-b

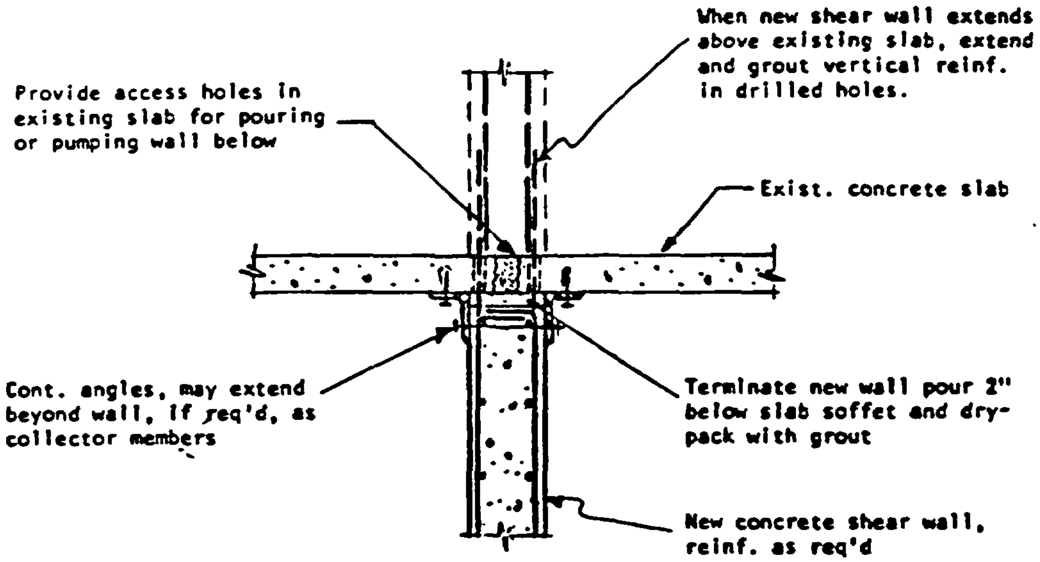
Sheet 2 of 2

Figure 6-23. Upgrading an existing building with external frames. (Sheet 2 of 2)



Sheet 1 of 2

Figure 6-24. Strengthening of an existing concrete frame building with a reinforced concrete shear wall. (Sheet 1 of 2)



Detail

New Concrete Shear Wall
at Existing Slab

Sheet 2 of 2

Figure 6-24. Strengthening of an existing concrete frame building with a reinforced concrete shear wall. (Sheet 2 of 2)

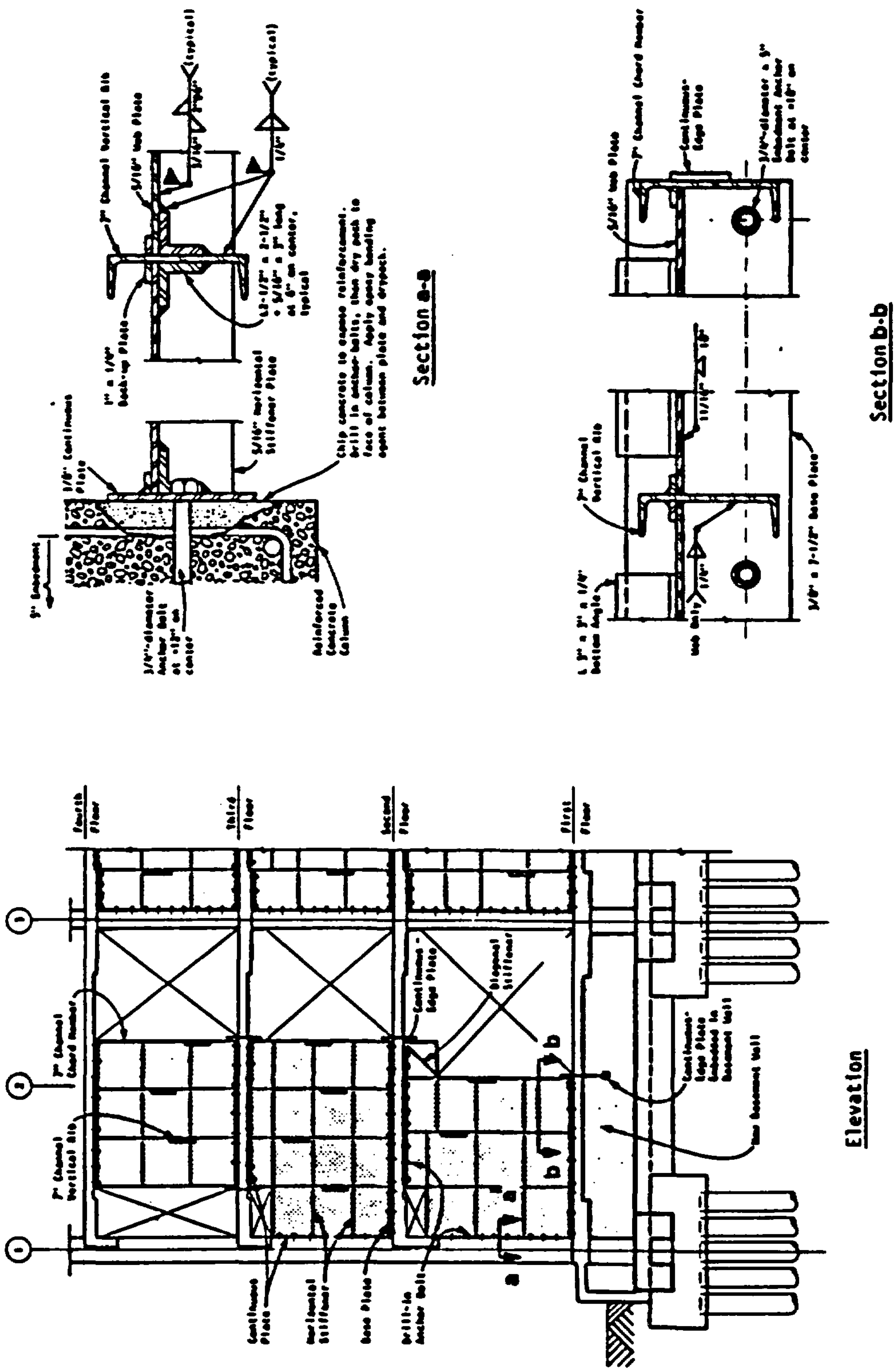
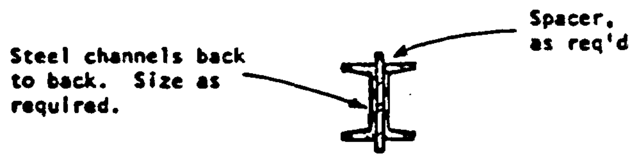
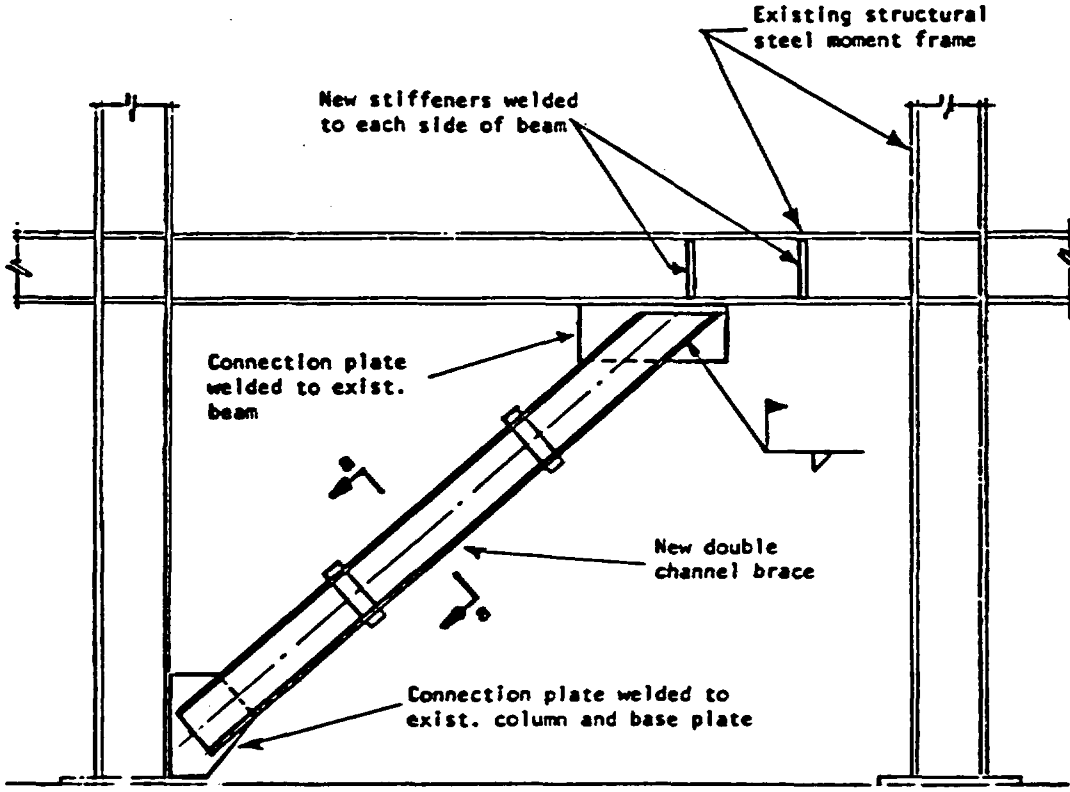


Figure 6-25. Steel shear wall in an existing building



Section a-a



Elevation

Figure 6-26. Strengthening of an existing building with eccentric bracing

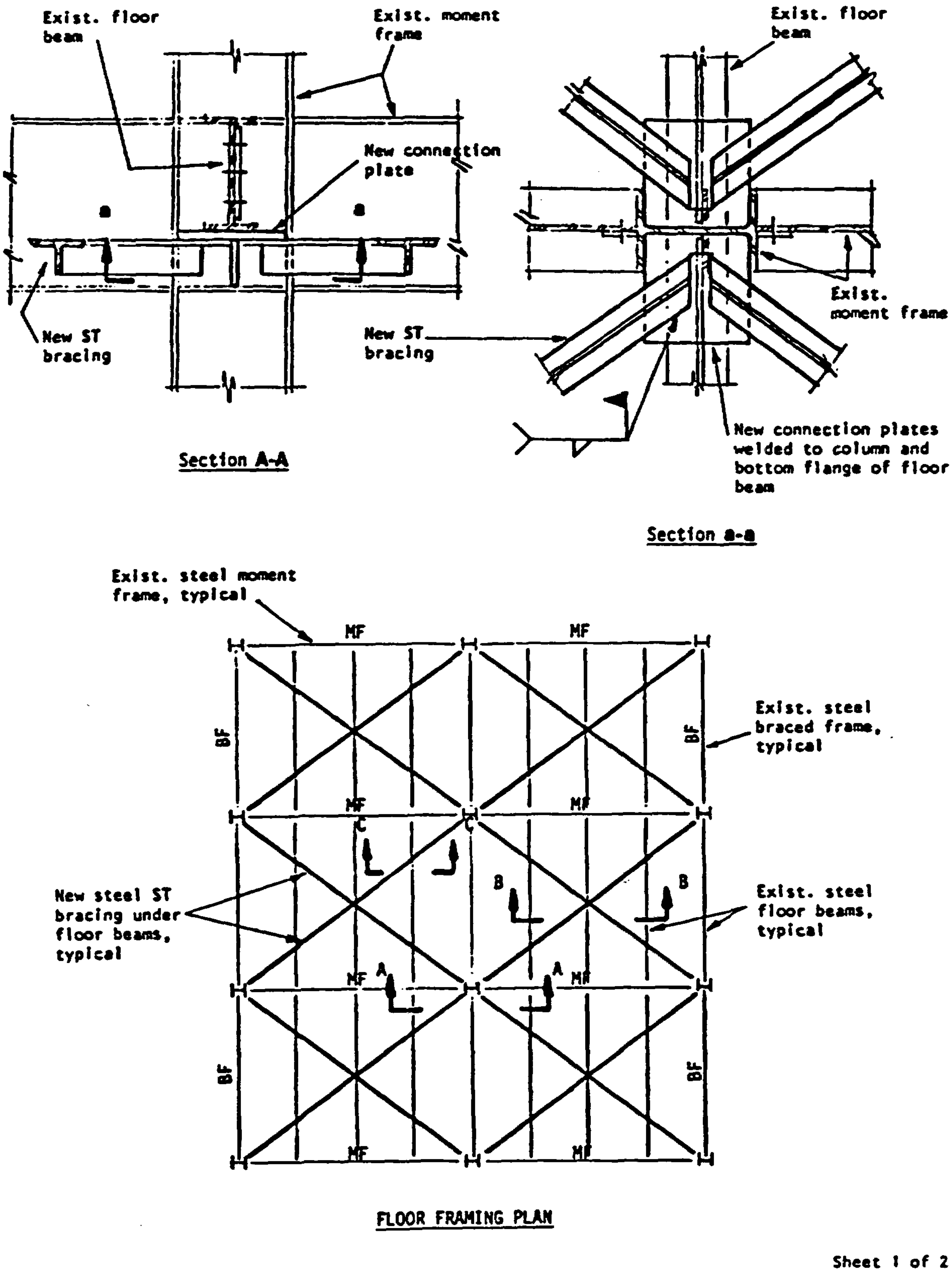
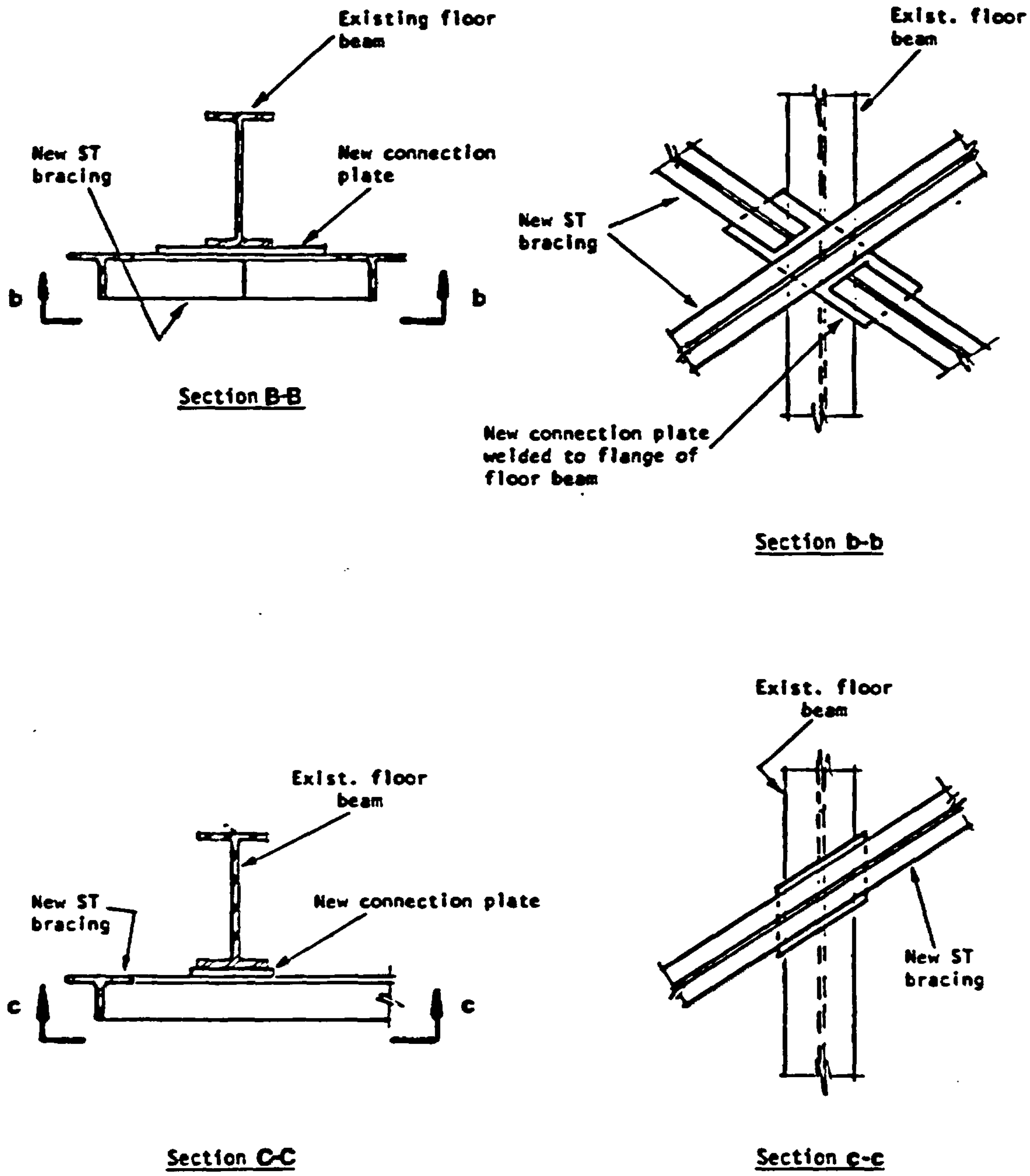


Figure 6-27. Strengthening of an existing steel frame building with horizontal bracing. (Sheet 1 of 2)



Sheet 2 of 2

Figure 6-27. Strengthening of an existing steel frame building with horizontal bracing. (Sheet 2 of 2)

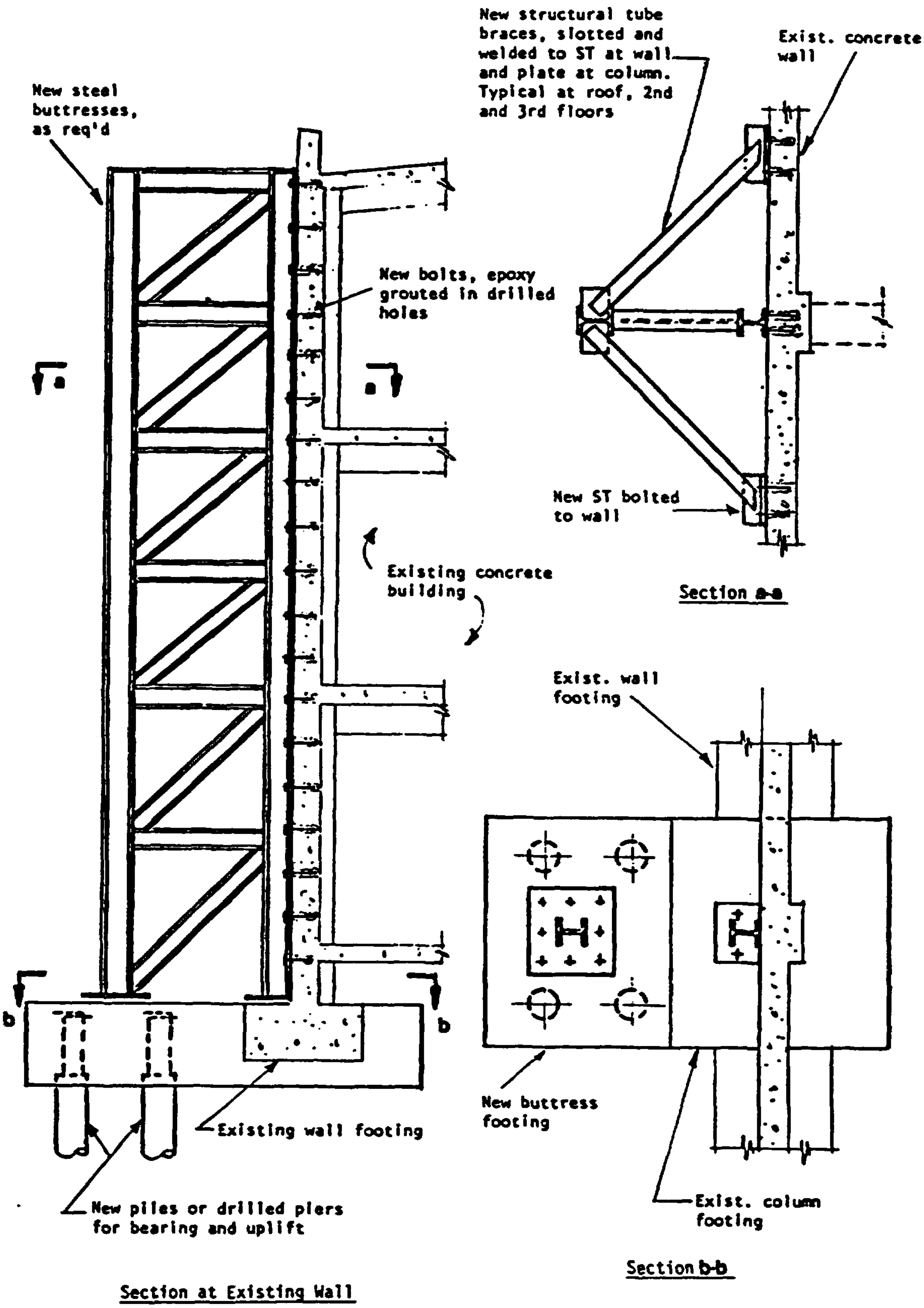


Figure 6-28. Braced structural steel buttresses to strengthen an existing reinforced concrete building

CHAPTER 7

COST BENEFIT ANALYSIS

7-1. Introduction

This chapter provides guidelines to evaluate the cost effectiveness of upgrading seismically deficient existing buildings on the basis of data obtained from the preliminary evaluation, the detailed structural analyses, and the development of design concepts. Criteria are provided to determine the cost effectiveness of taking no action (i.e., leave "as is"), upgrading or replacement of existing deficient buildings.

7-2. Earthquake risk

Methodology for the specification of ground motion is provided by chapter 3 of the SDG.

a. *Probability of occurrence.* On the basis of available data and state-of-the-art techniques, estimates can be made on the probability of occurrence of earthquake motion at a particular site

Table 7-1. Probabilities of exceedance of peak ground acceleration in 50 years, NAS Moffett Field, California

PROBABILITIES OF EXCEEDANCE OF PEAK GROUND ACCELERATION IN 50 YEARS, NAS MOFFETT FIELD, CALIFORNIA

PGA'S	Probability of Exceedance
0.0	1.0000
0.05	.9990
0.1	.9563
0.15	.7698
0.2	.5803
0.25	.4452
0.3	.3383
0.35	.2571
0.4	.1979
0.45	.1518
0.5	.1166
0.55	.0901
0.6	.0706
0.65	.0542
0.7	.0405
0.75	.0310
0.8	.0241
0.85	.0179
0.9	.0136
0.95	.0101
1.00	.0071

that is caused by a variety of earthquakes at different sources. The results of such a study can be summarized by a curve that plots number of occurrences per year that equal or exceed various levels of peak horizontal ground accelerations (PGA). An example of this relationship is shown in table 7-1.

b. Response spectra for selected seismic events. The various levels of earthquake ground motions that are postulated for a particular site during a selected period of time can be represented by a series of response spectra. For example, a set of response spectra, with appropriate damping values for elastic and post-yield responses, can be used to represent the earthquake demand for each of the PGA levels in table 7-1. The response spectra shown in figure 7-1 are normalized to 1.0g. These spectra can be used for any PGA level by selecting the appropriate damping levels and multiplying the spectral ordinates of the selected curves in the figure by the desired PGA level.

7-3. Damageability of the structure

The results of the preliminary evaluation as determined by the procedures in paragraph 4-2*d* will give an indication of the damageability of the structure. However, consideration should be given to the degree of accuracy used in the analysis of the structure. Due to the approximate nature of the damage estimate procedures, the results can sometimes be misleading. A review of the analysis should be made to determine if the amount of predicted damage has been overstated or understated. The results of the detailed analysis of chapter 5 can be used to more accurately describe the capacity of the structure, especially if method 2 was used.

a. Capacity to resist earthquake without damage. If the existing structure is able to conform to the acceptance criteria of paragraph 5-2, it is assumed that there will be no damage for EQ-I and little, if any, damage for EQ-II. The effects of EQ-I can be

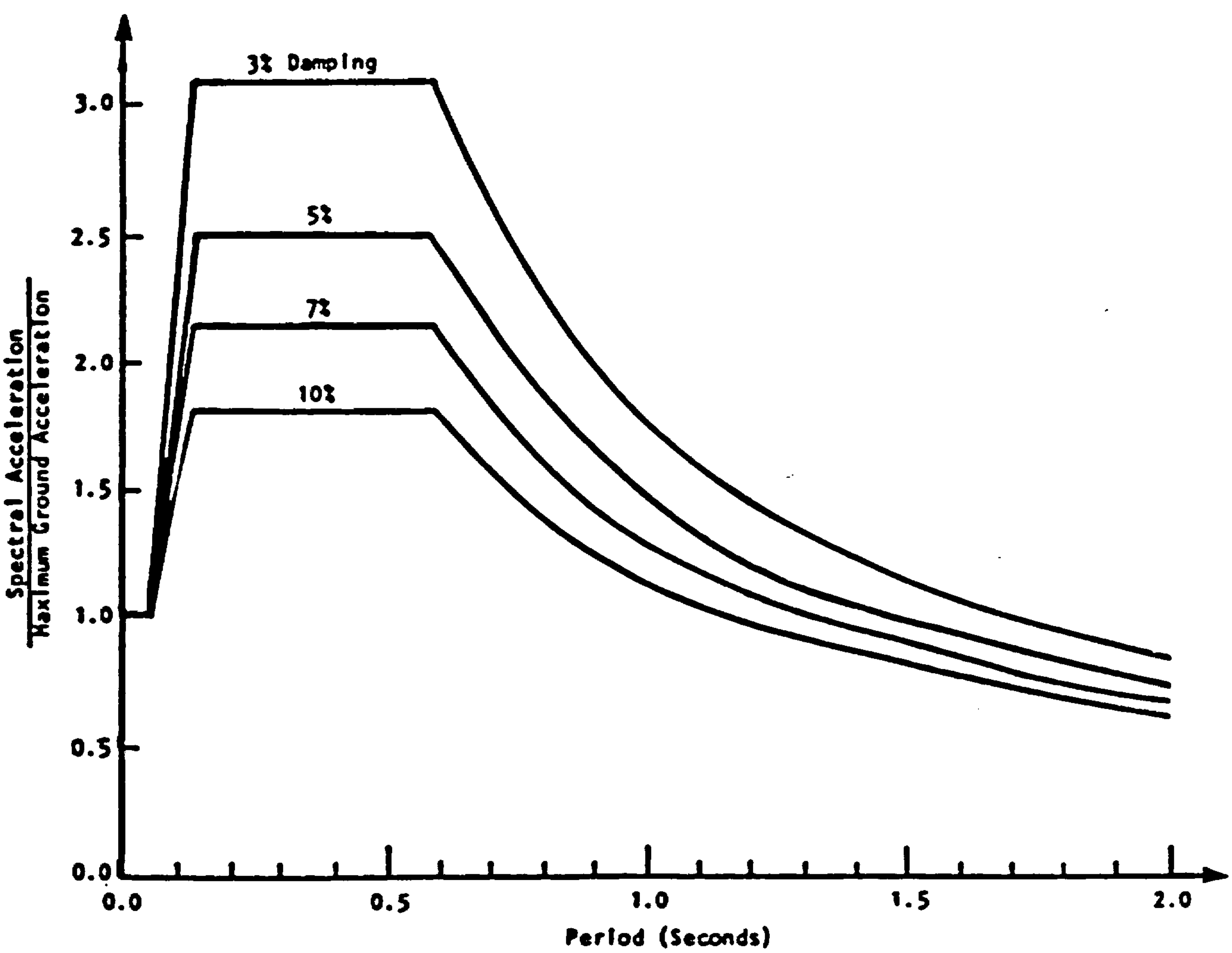


Figure 7-1. Acceleration response spectra normalized to 1.0g.

approximated by the damage control check procedure described in paragraph 6-3*e*.

b. Repair costs. If the results of the detailed analysis indicate that damage will occur in the event of EQ-II, estimates will be made to establish costs to repair for the damage associated with each of a series of earthquake response spectra such as discussed in paragraph 7-2*b*. The repair costs may be based on the damage estimate procedure described in paragraph 4-2 using the capacity determined in the detailed analysis described in paragraph 5-4. If sufficient data are available, it is preferable to make a more rational estimate based on an itemized list of repairs. The list might include such items as painting and cosmetic repairs, repairs to partitions, repair cracks in concrete elements, replacement of steel braces, repairs to diaphragms and connections, repair to shear walls, and replacement of major structural elements. The methods and degree of effort used to make the cost estimates will be dependent on the size, type, and function of the structure.

7-4. Annualized repair costs without seismic upgrading

Actual costs for damage predictions are difficult to define. In addition to technical evaluations, there are also social, economic, and administrative decisions that should be considered. For example, after an earthquake occurs that causes some damage to a structure, four general courses of action may be available: do nothing; do minimum repair and/or modifications; do moderate repair and/or modifications that may upgrade the capacity of the structure; or tear down the structure and rebuild. A decision on the course of action must be made after each earthquake. A solution based on this sort of approach would require the use of decision theory; however, development of a methodolgy using decision theory has been considered to be too complex for use in this manual. To simplify the procedure, one decision was made that governs all the structures for any size earthquake. If will be assumed that after each earthquake, the building will be repaired to restore it to the pre-earthquake condition. The cost of such repairs is the damage cost determined in paragraph 7-3, above. A procedure to estimate the annualized repair costs and determine their present value is outlined below:

a. Select a series of ground motion response spectra that represent all postulated earthquakes as described in paragraph 7-2*b*.

b. Determine the number of seismic events represented by each of the above earthquake spectra during the useful life of the building. The useful life of an existing building will be assumed to be not less than 25 years, unless otherwise directed by the approval authority.

c. Estimate the repair costs for each of the earthquake spectrum in *a*, above.

d. Multiply each of the repair costs, estimated in *c* above, by the number of seismic events associated with each spectrum as determined in *b* above.

e. Calculate total repair costs, R, by totaling the repair costs for all the postulated earthquakes.

f. Obtain the annualized repair costs by dividing the total repair costs, R, by the time span, n, used in *b*, above.

g. Calculate the present value of the annualized repair costs by:

$$PVR = \sum_{n=1}^{n} R/n \left(\frac{1+j}{1+i}\right)^{n-1} \qquad \text{(eq 7-1)}$$

where PVR = Present value of annualized repair costs

R = Total repair costs during the useful life of the building

i = Assumed average interest rate for next n years

j = Assumed inflation rate for next n years

n = Useful life of the building (i.e., not less than 25 years, unless otherwise directed).

7-5. Cost of seismic upgrading

The recommended concepts developed in accordance with the guidelines of paragraph 6-3 will be used for estimating the costs of seismic upgraded buildings.

7-6. Annualized repair costs after seismic upgrading

The acceptance criteria specified in paragraph 5-2 imply that essential buildings conforming to these criteria will be subjected to minor damage from the ground motion associated with EQ-II, but the buildings will be capable of performing their essential functions with little or no interruption of these functions. For high-risk and all other buildings, the criteria imply that structural collapse will be precluded, but that some structural damage is to be expected. These criteria also imply acceptance of the risk of additional damage from ground motion more severe than that associated with EQ-II. The repair costs associated with this seismic damage to the recommended concepts will be calculated as described in paragraph 7-4, above, for the existing building models but with the new structural capacities resulting from the upgrading modifications.

7-7. Cost versus benefits of a recommended upgrading concept

a. Cost of no action. The present value of the annualized repair costs to the existing building will be determined by the procedures outlined in paragraph 7-4.

b. Total costs of the recommended upgrading concept. The total cost associated with the recommended concept will be the upgrading construction cost, determined in paragraph 7-5, plus the present value of the annualized repair costs determined in paragraph 7-6.

c. Replacement costs. The cost of constructing a new building to replace the existing building will be estimated. Since this cost will be used only for comparison with the costs determined in paragraphs *a* and *b* above, the same degree of refinement will be used. In most cases, the replacement cost may be determined from the inventory of real property (see para 2-2) or estimated from representative costs per square foot of similar construction in the area adjusted, as necessary for size, inflation, and other factors.

d. Economic analysis. An economic analysis of the above costs will be made by comparison of the various costs outlined in paragraphs *a*, *b*, and *c* above.

e. Example. An example of a cost benefit analysis, taken from a recent study performed under the auspices of the Naval Civil Engineering Laboratory (NCEL), is summarized in tables 7-2 and 7-3 and is described below.

(1) *Description of building.* The building is a two-story reinforced concrete structure designated as unaccompanied enlisted personnel housing (UEPH). The lateral force resisting system is comprised of the concrete roof and floor diaphragms and the reinforced concrete shear walls. The elastic capacity of the longitudinal shear walls occurs at a PGA of 0.39g. In the transverse direction, the elastic capacity occurs at a PGA of 0.12g and is limited to flexural yielding of short interior shear walls and their connecting grade beams.

(2) *Earthquake demand.* The seismic hazard at this site (NAS Moffett Field, California) is represented in table 7-1. Corresponding values of EQ-I and EQ-II would be about 0.23g and 0.57g respectively. The site response spectra, shown in figure 7-1, are normalized to a PGA of 1.0g.

(3) *Number of seismic events.* The useful life of this building was assumed to be 50 years. The number of seismic events, that can be expected during this 50 year period, equal to or exceeding a given PGA value, was calculated for PGA increments of 0.05g from the data in table 7-1 using the Poisson probability relationships. The difference between the number of events for consecutive increments of PGA is designated as NEI in tables 7-2 and 7-3, and this difference represents the expected number of events of a severity bracketed by the two PGA levels (e.g., in table 7-2, the NEI value of 1.6616 represents the number of seismic events expected in 50 years with a PGA between 0.10g and 0.15g).

(4) *Damage estimates.* Estimates of damage for each PGA increment were calculated using procedures similar to method 2 as described in paragraph 5-3*f*. For purposes of this study, the total damage was defined as:

$$DE = \frac{D_1 + D_2}{2} + D_3 + D_4 \leq RC \qquad \text{(eq 7-2)}$$

where DE = average total damage for a seismic event corresponding to a given PGA level at the site

D_1 = total damage when full PGA is experienced in N/S direction and 0.75 PGA in E/W direction

D_2 = total damage when full PGA is experienced in E/W direction and 0.75 PGA in N/S direction

D_3 = damage to equipment which is uncoupled from building damage, except under collapse conditions, and related only to PGA

D_4 = damage to contents which is uncoupled from building damage, except under collapse conditions, and related only to PGA

RC = replacement cost of building and contents

(5) *Replacement and modification costs.* The building upon which this study was based contains 13,760 sq. ft. and has a current replacement cost of $1,106,000 plus contents valued at $97,600. A strengthening scheme was developed to increase the elastic capacity of the buiding in the transverse (north-south) direction to a level corresponding to a PGA of 0.20g. Strengthening is not required in the longitudinal (east-west) direction. The modified building would be substantially in compliance with the acceptance criteria of EQ-I and EQ-II and the estimated modification costs are $32,700.

(6) *Total cost of repairs.* The total damage, TD, in a PGA interval is defined as the average damage cost in the interval and is calculated by the number of events, NEI, multiplied by the average total damage per event, DE, for two successive PGA levels (e.g., in table 7-2, and TD value of $6,977 = 0.2791 x $\frac{15{,}000 + 35{,}000}{2}$). The averaging of the two DE values is required be-

Table 7-2. Summary of a cost benefit study for an existing building without seismic upgrading

PGA	NEI	D1	D2	D3	D4	DE	TD
.05	-	0.	0.	0.	0.	0.	0.
.10	3.7774	0.	0.	0.	0.	0.	0.
.15	1.6616	4000.	1000.	1000.	0.	3500.	2908.
.20	.6006	20000.	4000.	2000.	1000.	15000.	5555.
.25	.2791	37000.	17000.	4000.	4000.	35000.	6977.
.30	.1762	65000.	30000.	8000.	9000.	64500.	8766.
.35	.1157	83000.	47000.	14000.	14000.	93000.	9115.
.40	.0767	112000.	65000.	23000.	19000.	130500.	8568.
.45	.0559	141000.	113000.	32000.	23000.	182000.	8732.
.50	.0407	170000.	174000.	41000.	25000.	238000.	8539.
.55	.0296	224000.	236000.	50000.	27000.	307000.	8054.
.60	.0212	663000.	663000.	186000.	98000.	947000.	13295.
.65	.0175	663000.	663000.	186000.	98000.	947000.	16565.
.70	.0144	663000.	663000.	186000.	98000.	947000.	13619.
.75	.0099	663000.	663000.	186000.	98000.	947000.	9330.
.80	.0071	663000.	663000.	186000.	98000.	947000.	6719.
.85	.0063	663000.	663000.	186000.	98000.	947000.	5997.
.90	.0044	663000.	663000.	186000.	98000.	947000.	4137.
.95	.0035	663000.	663000.	186000.	98000.	947000.	3354.
1.00	.0030	663000.	663000.	186000.	98000.	947000.	2866.
						R =	143097.

DIFFERENTIAL RATE = 3.00%
INTEREST RATE = 8.00%
INFLATION RATE = 5.00%

CM = 0.
PVR = 77839.
TC = 77839.

NOTATION :

PGA = PEAK GROUND ACCELERATION IN G UNITS
NEI = NO. OF SEISMIC EVENTS BETWEEN PRIOR AND CURRENT PGA
D1 = STRUCTURAL & ARCHITECTURAL DAMAGE, N-S EARTHQUAKE
D2 = STRUCTURAL & ARCHITECTURAL DAMAGE, E-W EARTHQUAKE
D3 = ELECTRICAL & MECHANICAL EQUIPMENT DAMAGE
D4 = CONTENTS DAMAGE

DE = TOTAL DAMAGE PER EVENT
= (D1+D2)/2+D3+D4 WHERE (D1+D2)/2+D3+D4 < RC
TD = TOTAL DAMAGE IN THE PGA INTERVAL
= NEI*(PRIOR DE + CURRENT DE)/2
R = TOTAL COST OF FUTURE DAMAGE
RC = REPLACEMENT COST
CM = COST OF MODIFICATIONS
PVR = PRESENT VALUE OF COST OF FUTURE DAMAGE
ASSUMING DAMAGE SPREAD UNIFORMLY OVER 50 YEAR PERIOD
TC = TOTAL COST
= PVR + CM

Table 7-3. Summary of a cost benefit study of an existing building with seismic upgrading

PGA	NEI	D1	D2	D3	D4	DE	TD
.05	-	0.	0.	0.	0.	0.	0.
.10	3.7774	0.	0.	0.	0.	0.	0.
.15	1.6616	0.	0.	0.	0.	0.	0.
.20	.6006	0.	0.	0.	1000.	1000.	300.
.25	.2791	13000.	0.	1000.	4000.	11500.	1744.
.30	.1762	30000.	8000.	3000.	9000.	31000.	3744.
.35	.1157	60000.	19000.	7000.	14000.	60500.	5296.
.40	.0767	90000.	30000.	13000.	19000.	92000.	5846.
.45	.0559	120000.	88000.	22000.	23000.	149000.	6734.
.50	.0407	150000.	152000.	31000.	25000.	207000.	7238.
.55	.0296	194000.	216000.	40000.	27000.	272000.	7079.
.60	.0212	244000.	696000.	126000.	66000.	662000.	9903.
.65	.0175	696000.	696000.	186000.	98000.	980000.	14361.
.70	.0144	696000.	696000.	186000.	98000.	980000.	14094.
.75	.0099	696000.	696000.	186000.	98000.	980000.	9655.
.80	.0071	696000.	696000.	186000.	98000.	980000.	6954.
.85	.0063	696000.	696000.	186000.	98000.	980000.	6206.
.90	.0044	696000.	696000.	186000.	98000.	980000.	4281.
.95	.0035	696000.	696000.	186000.	98000.	980000.	3471.
1.00	.0030	696000.	696000.	186000.	98000.	980000.	2966.
						R =	109871.

DIFFERENTIAL RATE = 3.00%
INTEREST RATE = 8.00%
INFLATION RATE = 5.00%

CM = 32700.
PVR = 59766.
TC = 92466.

NOTATION :

PGA = PEAK GROUND ACCELERATION IN G UNITS
NEI = NO. OF SEISMIC EVENTS BETWEEN PRIOR AND CURRENT PGA
D1 = STRUCTURAL & ARCHITECTURAL DAMAGE, N-S EARTHQUAKE
D2 = STRUCTURAL & ARCHITECTURAL DAMAGE, E-W EARTHQUAKE
D3 = ELECTRICAL & MECHANICAL EQUIPMENT DAMAGE
D4 = CONTENTS DAMAGE

DE = TOTAL DAMAGE PER EVENT
= (D1+D2)/2+D3+D4 WHERE (D1+D2)/2+D3+D4 (RC
TD = TOTAL DAMAGE IN THE PGA INTERVAL
= NEI*(PRIOR DE + CURRENT DE)/2
R = TOTAL COST OF FUTURE DAMAGE
RC = REPLACEMENT COST
CM = COST OF MODIFICATIONS
PVR = PRESENT VALUE OF COST OF FUTURE DAMAGE
ASSUMING DAMAGE SPREAD UNIFORMLY OVER 50 YEAR PERIOD
TC = TOTAL COST
= PVR + CM

cause the NEI values represent all seismic events expected to occur between the two successive PGA values and the damage is assumed to vary linearly between the two PGA levels. The total cost of repairs, R, is defined as the sum of the total damage costs, TD, for all the PGA levels.

(7) *Economic analysis.* The economic analyses for this example were performed assuming an interest rate, i, of 8 percent and an inflation rate, j, of 5 percent.

(a) *Existing building.* Table 7-2 indicates the repair cost analysis for the existing (unmodified) building. The results of the analysis are as follows:

Total cost of repairs, R,	= $143,097
Present value of cost of repairs, PVR,	= $ 77,839

(b) *Upgraded building.* Table 7-3 contains a similar repair cost analysis for the modified building as follows:

Upgrading costs	= $ 32,700
Total cost of repairs, R,	= $109,871
Present value of cost of repairs, PVR,	= $ 59,766

(c) *Economic analysis.* The above roof analyses indicate that the present value of the anticipated repair costs of seismic damage to the building, over its useful life, will be $77,839 if the building is not upgraded. If the buiding is upgraded to compliance with the criteria of this manual, the present value of the anticipated repair costs will be reduced to $59,766, but the additional cost of $32,700, for the modification results in a total cost of $92,466. These total costs, with or without the upgrading modifications, are significantly less than the replacement costs of the building, equipment, and contents. Therefore, replacement need not be considered as an option.

(8) *Conclusions and recommendation.* The economic analysis indicates that upgrading this building is not cost-effective. The detailed structural analysis indicated that the existing building possessed adequate post-yield capacity to preclude collapse so that the life safety of the occupants is not in jeopardy. Therefore, unless there are other overriding considerations, seismic upgrading of this building should not be recommended.

7-8. Report

A report will be prepared for review by the approval authority and for formulating the decision as to whether the building should be upgraded, replaced, or left as is. In addition to the economic analysis, social, political, and administrative considerations will be addressed. These may include the impact of the potential seismic hazards on life safety of the occupants or to the public (e.g., collapse of a facility containing hazardous materials); current and future use of the building and its importance to the mission of the activity; costs associated with temporary interruptions of use during the upgrading and/or repair work; functionability of the existing building (e.g., are there functional problems that could be corrected during the upgrading work?); and the historic significance of the building. A discussion of these and other appropriate considerations will be included in the report with a qualitative evaluation applicable to each building in support of recommendations that will be made as to action to be taken for each building.

CHAPTER 8

FINAL DESIGN AND PREPARATION OF CONTRACT DOCUMENTS

8-1. General

Upon authorization of the approval authority, the final design of the approved concept will be implemented and the necessary project construction documents will be prepared in accordance with the requirements of the Basic Design Manual and the applicable provisions of the SDG and this manual.

8-2. Final Design

The final design will be done on the basis of the results from the detailed structural analysis, development of design concepts, and the cost benefit analysis as directed by the approval authority. The final design will include a complete analysis of the upgraded structure, completed drawings of all details for the project, and a detailed cost estimate. The final documents will be complete in themselves, without the need to refer to the previous analysis and development work.

8-3. Preparation of project documents

a. Design analysis. A design analysis conforming to agency standards will be provided with final plans. This analysis will include seismic design computations for the determination of earthquake forces on the building, for the structural evaluation of the existing building, and for the upgrading of the existing structure, including stresses in the lateral-force-resisting elements and their connections, and the resulting lateral deflections and interstory drifts. The first portion of the design analysis, called the Basis of Design, will contain the following specific information:

(1) A statement on the methodology used for determining the ground motion criteria and a description of the response spectra for which the existing building will be evaluated and the upgraded structure will be designed.

(2) A description of the existing structural system and the structural system selected for upgrading the building to resist lateral forces. Include a discussion of the reasons for its selection. If irregular conditions exist, a statement describing special analytical procedures to account for the irregularities will be submitted for review and approval by the approval authority.

(3) A statement regarding compliance with this manual, including the values selected for damping, the criteria used for capacities and deformations of structural elements, and the method to determine deformation compatibility between existing and new structural elements.

(4) Any possible assumed future expansion for which provisions are made.

b. Drawings. Preparation of drawings will conform to agency standards for ordinary construction with the following additional specific requirements for seismic construction:

(1) Preliminary construction drawings will contain a statement that seismic design will be incorporated in accordance with this manual. The Basis of Design submitted with these drawings will include the information outlined in paragraphs *a*(1) through *a*(4) above.

(2) Construction drawings for seismic areas will include the following additional special information:

(*a*) A statement on the seismic ground motion criteria including the design peak ground accelerations and related response spectra.

(*b*) A statement on the lateral-force design criteria including a tabulation of the periods of vibration and equivalent design lateral forces and other factors.

(*c*) Assumptions made for future extensions or additions.

c. Specifications. Preparation of specifications will conform to agency standards for ordinary construction with additional specific requirements that relate to seismic construction and to upgrading of existing construction.

d. Cost estimate. Prepare a detailed cost estimate for the upgrading project.

CHAPTER 9

SEISMIC UPGRADING OF NONSTRUCTURAL ELEMENTS

9-1. Introduction

This chapter prescribes guidelines for evaluating seismic resistance of nonstructural elements in existing buildings that must remain intact or functional after a major seismic disturbance. The provisions of this chapter consist of a qualitative evaluation based on available pertinent design and installation documents and on-site inspection, and a detailed analytical evaluation using either a dynamic approach prescribed in the Seismic Design Guidelines (SDG), chapter 6, or a static approach prescribed in the Basic Design Manual (BDM), chapters 9 and 10. This chapter also discusses modes of damage to nonstructural elements and their anchorages and suggested strengthening measures for correcting seismic deficiencies identified in the evaluation of existing structures.

9-2. Acceptance criteria

The acceptance criteria for the seismic resistance, details of anchorages, and performance of existing nonstructural elements will be essentially as prescribed in the BDM and/or SDG. However, if existing elements or their anchorages do not conform to the above criteria, a tolerance of 15 percent overstress (i.e., 15 percent understrength) is acceptable if required seismic upgrading would be an excessively expensive and disruptive process.

9-3. General considerations

a. Elements to be considered. Nonstructural elements, which are generally categorized as architectural, mechanical, or electrical, include items that are housed in or on the building, as well as portions of the building that are not part of the structural system. Some nonstructural elements are classified as essential systems. Essential systems include all elements that are needed for the performance of emergency services or that may, by their failure, cause life hazard or impair the performance of services. These systems are outlined in SDG, paragraph 6-7 and table 6-3.

b. Vulnerability to seismic damage. Damage can occur to nonstructural elements from two basic types of motion: large story accelerations that cause elements to topple, fail the anchor supports, or cause damage; and large interstory displacements that will cause damage to elements rigidly attached to the adjacent floor slabs. In addition, the presence of rigid nonstructural elements can affect the performance characteristics of the structural system. For example, the inclusion of an unreinforced masonry wall, tightly fitted within a structural steel framing system, will greatly increase the stiffness of the structure, will increase the force level due to the shortened period, and will change the distribution pattern of forces to the structural elements. While the nonstructual elements can generate unexpected forces in the structural system, so does the structural system create unexpected forces in the nonstructural system. Furthermore, the failure of one element may ultimately cause the failure of another element or an entire system.

c. Reasons for seismic upgrading. There are generally four basic issues in the decision-making process to decide whether nonstructural elements are in need of upgrading and to what extent the upgrading is needed.

(1) *Functional loss.* Nonstructual damage may cause serious postearthquake disruption in essential services and productivity.

(2) *Life safety.* Nonstructural damage may injure people.

(3) *Economic loss.* Nonstructural damage may be costly.

(4) *Structural interaction.* Nonstructural elements may interact with structural components and may change the overall structural behavior, which can be beneficial or harmful to the structure.

9-4. Qualitative evaluation

Because of the great number of nonstructural elements and systems in existing buildings, a qualitative evaluation is made prior to a detailed quantitative evaluation to determine the general susceptibility to damage of the nonstructural elements and systems under investigation. The qualitative evaluation includes an assessment of conformance with minimum design and installation requirements, the need for detailed quantitative evaluation as described in paragraph 9-5, and the requirements for seismic upgrading. The following factors should be considered in the decision-making process:

a. Classification. This factor is based on the classification of nonstructural elements in accordance with function, life safety, and economic requirements. The following is an order of priority of importance:

(1) All nonstructural elements that are housed

in an essential facility with special attention to essential systems required for life safety and postearthquake operations.

(2) Essential systems housed in a high-risk facility.

(3) Essential systems housed in other buildings.

(4) All nonstructural elements that are not covered above.

b. Building and site characteristics. The vulnerability of nonstructural elements are dependent on the amplitude of ground motion (e.g., response spectra of EQ-I and EQ-II) and the dynamic response characteristics of the building (e.g., periods of vibration and mode shapes). Data from the structural evaluation of the building will aid in the evaluation of nonstructural elements.

(1) Flexible equipment is susceptible to damage when its period of vibration is in tune with the natural periods of the building.

(2) Elements attached to adjacent floors are more susceptible to damage when located in flexible buildings.

c. Seismic vulnerability. A qualitative evaluation requires judgment and experience on the part of the engineer in assessing seismic vulnerability of nonstructural elements. Possible modes of failure must be anticipated in order to identify the elements most susceptible to damage from earthquakes. Table 9-1 compiles a listing of types of damage that should be considered in the evaluation. This is not presented as an all inclusive listing, but is presented as a guideline on the basis of experience and observations by others.

d. Field inspection. Many of the installation details of nonstructural elements are often omitted from drawings because of common construction practices that have left many of these decisions to product manufacturers and installers. Therefore, it is important that design and installation details of nonstructural elements, especially essential elements, be investigated thoroughly during onsite field inspection. Furthermore, the observed existing conditions should be compared with idealized construction and strengthening practices, as recommended in paragraph 9-6. Any observed deviation from these suggested measures would downgrade the seismic resistance capacity of the elements under investigation, and a further evaluation and/or a remedial action should be taken.

e. Rapid analysis. A minimum of engineering calculations, on the basis of approximate element weights and earthquake response accelerations, may be required to supplement the qualitative evaluation procedure. The force and deformation criteria will be in accordance with the provisions of the BDM and/or SDG.

9-5. Detailed evaluation

The elements and their anchorages that have been identified in the qualitative evaluation procedure to be susceptible to damage will be subjected to a detailed evaluation. The nonstructural elements and their anchorages will be checked to resist forces and deformations caused by earthquake motions prescribed for buildings in this manual. The effect of nonstructural elements on the performance of the building will also be considered. Either a dynamic approach prescribed in SDG, chapter 6, or a static approach prescribed in BDM, chapters 9 and 10, may be used, except when authorized the dynamic approach will be used in the evaluation of essential systems.

9-6. Representative upgrading techniques

Since structural quality and anchorage of nonstructural elements in existing buildings vary greatly, it is not feasible to present methods of strengthening all such elements in detail, especially for mechanical and electrical elements where there are many different types and models. Such equipment is usually designed and installed by manufacturers without consideration of seismic resistance. Care and engineering judgment should be exercised for determining the best feasible methods of upgrading. Economic feasibility must be weighted against seismic risk in strengthening nonstructural elements, as should be done for buildings as a whole. For example, loss of files, computer facilities, or communication systems in a medical facility can shut down the system. Some general upgrading measures are outlined in this paragraph.

a. Architectural elements.

(1) *Exterior walls, parapets, appendages, veneers, etc.*

(*a*) Reduce height of parapet or brace to the roof structural system, as necessary.

(*b*) Anchor appendages, veneers, and other potential falling objects or replace their anchorages, as needed.

(*c*) Refer to BDM, figure 9-2, for design of exterior precast elements.

(2) *Interior walls and partitions.*

(*a*) Remove clay-tile partitions.

(*b*) Provide lateral bracing for partitions.

(*c*) Separate partitions from structural elements with sufficient joints coordinated with anticipated interstory drifts.

(*d*) Refer to BDM, figure 9-1, for typical details of interior walls and partitions.

(3) *Ceilings: Suspended system and surfaced-applied system.*

Table 9-1. Guidelines for evaluating seismic performance of selected nonstructural elements. (Sheet 1 of 4)

System/Element	Potential Types of Damage
Architectural	
Appendages:	
Exterior P/C panels	Failure of connections due to prying action; bolts pull out.
Parapets	Cracks due to cantilever action.
Ornaments, veneers	Failure of anchorage.
Partitions:	
Permanent-masonry, tile, metal stud, gypsum board, plaster	Damage occurs in the anchorage to the supporting structure and in the cracking of brittle surfaces.
Demountable-metal, wood, metal/glass	Separation at top/bottom channels, overturning and compression failure, glass cracking.
Ceiling:	
Exposed tee bars and luminous systems	Tees deform or pulled away from the wall support; breakage of hangers.
Concealed spline system	Damage occurs at perimeter walls where supports bend and tiles tear and fall.
Gypsum board with tiles, plaster	Gypsum boards drop and loosen tiles at perimeter walls.
Wood joists with nail or tiles	Most earthquake resistant of all ceiling types.
Lighting Fixtures:	
Recessed	Separation of fixtures due to racking of suspended ceilings.
Surface mounted	Generally undamaged by earthquakes.

Sheet 1 of 4

Table 9-1. Guidelines for evaluating seismic performance of selected nonstructural elements. (Sheet 2 of 4)

System/Element	Potential Types of Damage
Pendant	Very susceptible to damage; failures at ceiling connections, in swivel joints, at fixture housings, in supporting stems or chains. Damage to suspended ceiling.
Building Contents:	
Shelving, cabinets, storage racks	Overturning, dislodging of stored items.
Computer equipment	Top-heavy tape transports fall over; impact damage caused by equipment hitting walls or other equipment.
2. Mechanical	
Rigidly Mounted Equipment:	
Equipment	Equipment without anchors cause secondary damage on connected pipes and electrical service connections.
Tanks	Unanchored tanks tip over, legs and support brackets collapse; secondary damage to connecting piping.
Equipment with Vibration Isolators:	More susceptible to damage than fixed mounted equipment; failure of isolators causes equipment to fall and damage to connecting piping and electrical service connections.
Piping:	Failures at elbows and bents due to excessive movement; screwed fittings are more vulnerable to damage than welded or brazed fittings; failures at building seismic joints due to differential movements; failure of hanger assembly.

Sheet 2 of 4

Table 9-1. Guidelines for evaluating seismic performance of selected nonstructural elements. (Sheet 3 of 4)

System/Element	Potential Types of Damage
Ducts, Diffusers, etc.:	Long runs of large ducts fail as a result of excessive motion by the earthquake; large diffusers drop from ceilings where they lack proper support.
3. Other Essential Systems	
Elevators: Traction	
Counterweight guide systems	Counterweights derail, allowing them to swing.
Cabs guide systems	Out of alignment.
Motor generator sets	Thrown off their unanchored isolation mounts.
Control panels	Topple over where not anchored to structural frame.
Control relays	Damaged when unlatched and hinged panels thrown open.
Elevators: Hydraulic	No specific experience on observed damage.
Emergency Power System:	
Transformers	Inadequately secured transformers fall from pedestals, causing major damage to bushings, radiators, internal parts, and interconnecting bus; pole transformers are more vulnerable to damage than pedestal-mounted.
Switchgears	Motion of unsecured switchgears damage connections to the equipment.
Motor and generators	Vibration isolators shear off; damage power, fuel, and cooling line connections.

Sheet 3 of 4

Table 9-1. Guidelines for evaluating seismic performance of selected nonstructural elements. (Sheet 4 of 4)

System/Element	Potential Types of Damage
Battery racks	Generally remain in place when strapped to walls.
Panel boards	Overturning of unsecured tall units; rigid conduit failure due to support failure.
Fuel storage tanks	Fracture of pipe connections due to excessive movement of unanchored support.
Fire Protection System:	
Sprinkler and stand pipes	Only minor damage has been observed.
Pumps and tanks	Fracture of pipe connections due to excessive movement of unanchored support.
Steel stairs	Yielding of welded connections.
Concrete stairs	Shear cracking if tied to the structure.
Doors and frames	Doors deform and jam; frames warp.
Corridors	Corridors blocked with debris.
Hazardous Materials:	
Storage tanks, bottles, cylinders, and pipes containing hazardous toxic materials	Expose chemicals due to rupture of containers; damage caused by excessive movement or failure of adjacent elements.
Communications:	
Intercom/PA system, telephone equipment, and switchboards	Loss of communications due to broken wires.

Sheet 4 of 4

(*a*) Brace ceiling grid at regular intervals against lateral and vertical movements.

(*b*) Fasten cross runners to the main runners with locking clips to prevent cross tees from pulling or twisting out of the main runners.

(*c*) Brace ductwork and piping systems in the ceiling space against lateral and vertical movements.

(*d*) Positively connect all elements together.

(*e*) Reinforce gypsum board ceiling at nail points, using large-head nails or steel nailing strip.

(*f*) Refer to BDM, figure 9-3, for recommended suspended ceiling system.

(4) *Lighting fixtures: Recessed fixtures, surface-mounted fixtures, and pendant fixtures.*

(*a*) Secure recessed fixtures directly to the main runners of the ceiling system.

(*b*) Provide recessed fixtures with independent secondary supports attached to the fixture housing and the building structures.

(*c*) Attach surface-mounted fixtures directly to the building structure and suspended ceiling system using positive locking devices.

(*d*) Separate pendant fixtures sufficiently so that sway arcs do not intersect.

(*e*) Provide sway bracing if swinging clearance of chain-hung fixture is not adequate.

(*f*) Refer to BDM, paragraph 10-6, for the design requirements of lighting fixtures and supports.

(5) *Building contents:* Shelving, storage racks, filing cabinets, computer equipment, etc.

(*a*) Anchor all storage racks at base and laterally brace at top or attach to walls.

(*b*) Provide safety bars for open shelving where practical.

(*c*) Brace computer floors and provide drop-in panels detailed to prevent displacement during an earthquake.

b. Mechanical systems:

(1) *Mechanical equipment:*

(*a*) Anchor all floor-mounted equipment to the structural slab.

(*b*) Provide isolation restraints for all hung equipment.

(*c*) Remove vibration isolators and bolt equipment to floor slab or add snubbers to limit excessive movement.

(2) *Distribution system:* Pipes, ducts, and conduit.

(*a*) Provide sway bracing in both longitudinal and transverse directions on all pipes 2½ inches or larger, based on intervals recommended in BDM, figures 10-4 to 10-7. Also refer to BDM, figure 10-8, for acceptable details of sway bracing.

(*b*) Provide flexible joints where pipes enter building, where rigidly supported pipes connect to equipment with vibration isolation, and where needed to accommodate large interstory drifts for large pipe risers rigidly mounted between floors. Refer to seismic details in BDM, figure 10-9.

(*c*) Provide pipe sleeves through walls or floors large enough to allow for relative movements.

(*d*) Provide sway bracing in both longitudinal and transverse directions on all ducts with a perimeter greater than 120 inches and for all ducts in boiler and equipment rooms.

(*e*) Diffusers, registers, and grillers should be positively attached to the ductwork.

(*f*) Positively tie flexible ducts to the ceiling, wall, or floor system.

c. Essential systems that are not covered above.

(1) *Elevators: Traction and hydraulic types.*

(*a*) Install additional rail support brackets and brace spreader beams (counterweight).

(*b*) Install safety shoes on roller guide.

(*c*) Strengthen the car guide rails on long spans by installing spacers between the back-to-back rails at midpoints between the spreader beams.

(*d*) Protect traveling cables to prevent them from being twisted and snarled or from jumping out of their sheaves or guides.

(*e*) Design more rigid structural frames around hoistways and door frames that can accommodate the anticipated interstory drifts.

(*f*) Anchor motor generators and control cabinets or provide restraints on vibration isolators under generators to prevent excessive movement.

(*g*) Install emergency stop gear.

(*h*) Refer to BDM, figure 10-13, for details of traction-type elevator.

(2) *Emergency power system.*

(*a*) Anchor or restrain transformers, switchgear, and control panels.

(*b*) Bolt generator directly to foundation.

(*c*) Brace cooling tower or install an auxiliary cooling system such as generator radiator system at grade level.

(*d*) Anchor fuel storage tank and install flex loops in fuel lines between the tank and the building and at the connection to the generator.

(*e*) Strap all batteries on racks; anchor and brace storage racks.

(3) *Fire protection system.*

(*a*) Install mounting brackets for hung and free-standing fire extinguishers.

(*b*) Brace standpipes.

(*c*) Brace the sprinkler system piping in accordance with NFPA No. 13 (refer to BDM, paragraph 10-7); fire pumps should be governed by NFPA No. 20.

(*d*) Provide slip joints at the top or bottom of each flight of stairs.

(*e*) Ensure that exitways will not become blocked after an earthquake.

(4) *Protection against hazardous materials.*

(*a*) Install seismic-activated shut-off valves at appropriate locations on supply lines for natural gas and other hazardous materials.

(*b*) Brace fuel lines, bottles of laboratory chemicals, lead storage safes for radioactive materials, liquid oxygen storage tanks, and similar containers and protect them from damage caused by movement or failure of adjacent elements.

(5) *Communications.*

(*a*) Secure and anchor emergency comunication equipment or relocate in a nonvulnerable portion of the facility, preferably the lower levels.

(*b*) Provide alternate internal and external communication systems.

APPENDIX A

SYMBOLS AND NOTATIONS

BDM = Basic Design Manual
C_b = base shear coefficient. Equivalent to ZIKCS coefficient in BDM, equation 3-1
D or DL = dead load
DMRSF = ductile moment resistant space frame as defined in BDM chapter 3
d_N = lateral displacement at level N
E or EQ = earthquake load
EQ-I = earthquake that has a 50-percent probability of being exceeded in 50 years
EQ-II = earthquake that has a 10-percent probability of being exceeded in 100 years
g = acceleration due to gravity
L or LL = live load
L/r or l/r = ratio of length (L or l) to radius of gyration (r)
N = number of stories above the base to level n
n = the level that is uppermost in the main portion of the structure (generally the roof)
PF_N = modal roof participation factor shown in table 4-1 (Refer to SDG 4-1)
PGA = peak ground acceleration
RSAP = Rapid Seismic Analysis Procedure summarized in appendix D
RSS = root-sum-squares, same as SRSS
S_a = response spectrum value for spectral acceleration, as a ratio of the acceleration of gravity (g)
S_d = response spectrum value for spectral displacement
S_v = response spectrum value for spectral velocity
SDG = Seismic Design Guidelines
SRSS = square-root-of-the-sum-of-the-squares
S_1,S_2,S_3 = soil types for developing ATC-3-06 response spectra (NBS 510)
t = time in seconds
T = fundamental period of vibration of the structure
T_a = period of vibration of equipment or architectural appendage
V = total lateral force
W = weight of a system or building
W_p = weight of a portion of a structure, equipment, or architectural appendage
w_x = weight at or assigned to level x
α = modal base shear participation ratio for fundamental mode as shown in table 4-1 (Refer to SDG eq 4-2)
δN = same as d_N

APPENDIX B

REFERENCES

B-1. Government Publications.

a. *Department of the Army.*

TM 5-838-2	Army Health Facility Design

b. *Department of the Navy.*

T.M. No. 51-78-02	Rapid Seismic Analysis Procedure Naval Civil Engineering Laboratory
T.M. No. 51-83-07	Modification for Enhancing the Rapid Seismic Analysis Procedure Naval Civil Engineering Laboratory

c. *Departments of the Army, Navy, and Air Force.*

TM 5-809-10/NAVFAC P-355/AFM 88-3, Chapter 13	Seismic Design for Buildings (BDM)
TM 5-809-10-1/NAVFAC P-355.1/AFM 88-3, Chapter 13, Sec A	Seismic Design Guidelines for Essential Buildings (SDG)

d. *National Bureau of Standards (NBS).*

National Technical Information Service, 5285 Port Royal Road, Springfield, VA, 22161
or
Superintendent of Documents, U.S. Government Printing Office, Washington, DC, 20402
Special Publication 510 (514 pages), Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC-3-06), 1978
Evaluation of Strength of Existing Masonry Walls for the Veterans Administration

B-2. Nongovernment Publications.

American Concrete Institute (ACI), Box 19150, Redford Station, Detroit, MI, 48219
ACI 318-83, Building Code Requirements for Reinforced Concrete
American Institute of Steel Construction (AISC), 400 N. Michigan Ave., 8th Floor, Chicago, IL, 60611
Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, November 1, 1978, with Commentary
American Society for Testing and Materials (ASTM), 1916 Race Street, Philadelphia, PA, 19103
ASTM Standards
International Conference of Building Officials, 5360 South Workman Mill Road, Whittier, CA, 90601
Uniform Building Code (UBC), 1985

APPENDIX C

STATIC CODE PROCEDURE

C–1. Introduction

This appendix prescribes the static code procedure for seismic evaluation analysis and upgrading/strengthening rquirements for existing buildings in low or moderate seismic regions. The static code provisions in TM 5–809–10/NAVFAC P–355/AFM 88–3, Chapter 13, Seismic Design for Buildings (BDM), are the basis for this procedure. The methodology for this procedure is indicated in figure C–1. This static code procedure will be performed on a project-by-project basis for seismic zone 1 and for nonessential buildings in seismic zones 2 and 3 as determined by approving authority.

C–2. Applicability of the static code procedure

Since the early 1970s, the static seismic provisions of the BDM have been utilized for the evaluation and upgrading of existing military buildings on a project-by-project basis. The static code procedure described in this appendix was used for the evaluation and seismic upgrading of buildings in seismic zone 1 and nonessential buildings in seismic zones 2 and 3. At the discretion of the approving authority, the procedure may also be used for selected high risk and essential buildings (importance factors I = 1.25 and 1.50) and also for buildings in higher seismic zones. The implementation of this procedure will be as authorized by the approving authority.

C–3. Preliminary structural evaluation

The purpose of the evaluation is to determine if the building is in compliance with the acceptance criteria; to identify any structural deficiencies; and to provide the basis for strengthening or upgrading. The preliminary evaluation will be made on the basis of structural analyses performed in accordance with the prescribed seismic forces and allowable stresses of the BDM.

a. Document review. The available "as built" drawings, design calculations, specifications, and other design documents obtained from the using agency will be reviewed by the engineer to identify the lateral force resisting system and other pertinent information. This initial study will compare wind lateral loads to seismic lateral loads on the structure. If the design wind load on the existing structure governs over the seismic load, no further investigation will be required, unless warranted by the irregular configuration and/or other characteristics of the building. If the seismic load governs, the data will be documented (i.e., the lateral load force resisting system) for the site inspection. Also, supplementary notes and/or sketches will be made of the lateral force resisting system, as necessary, to be confirmed by the site inspection.

b. Site inspection. A field examination of the building will be performed and the following observations will be noted for use in the structural evaluation and design of the seismic strengthening or upgrading.

(1) Confirm the structural data indicated on the drawings; particularly with respect to the lateral force resisting system. Note any structural additions or modifications not indicated on the drawings.

(2) Determine the general condition of the structural elements (e.g., corrosion of structural steel, shear cracks in concrete or masonry, and splitting or checking of timber). Note also any damaged or missing members or other deviations from the drawings.

(3) Establish the various load paths by which lateral forces are transferred from the roof or floor systems to the vertical resisting elements (i.e., frames or walls) and to the foundations. Note any discontinuities in the load paths, redundant paths or backup systems and the adequacy of support or anchorage of concrete and masonry walls, at each floor or roof level, for out-of-plane forces.

(4) Note extent and details of anchorage and/or bracing of architectural elements (i.e., partitions, suspended ceilings, curtain walls, parapets, and canopies) and mechanical and electrical equipment (i.e., emergency motor generators and pumps, boilers, cooling towers, critical piping, light fixtures).

c. Acceptance criteria. The basic acceptance criteria for the seismic resistance of existing buildings is based on the provisions of the BDM. However, if an existing building does not conform to the basic criteria, some tolerances are provided in the following paragraphs in recognition that seismic upgrading is an expensive and disruptive process and it may be cost-effective to accept an existing building that is marginally deficient rather than to enforce strict adherence to the criteria.

(1) Conforming systems and materials. When the lateral force resisting structural systems and materials are in compliance with the requirements of the BDM (Refer to BDM paragraph 3–6 for approved structural systems and to BDM chapters 3, 5, 6, 7, and 8 for material requirements), the earthquake demand represented by the lateral

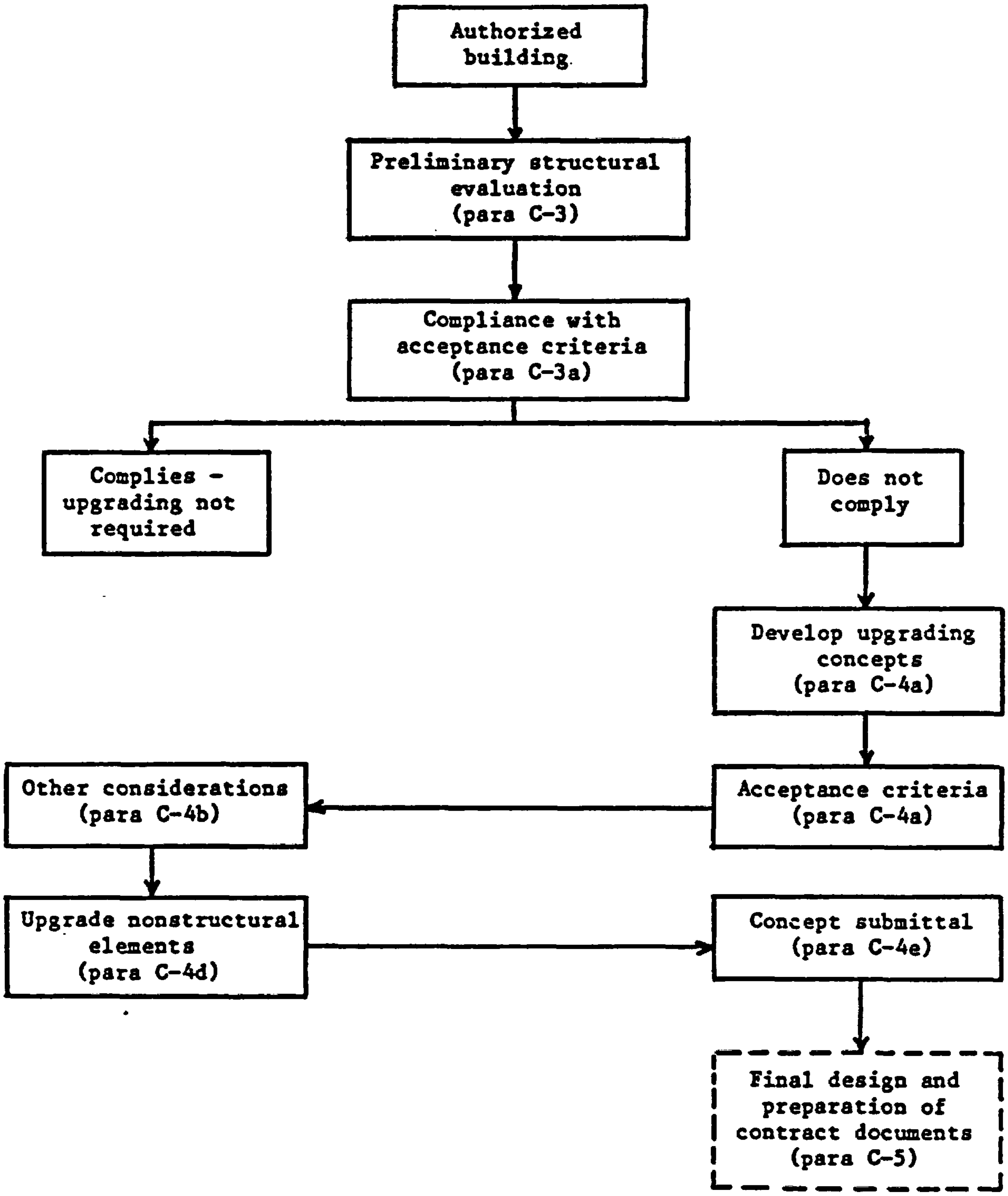


Figure C-1. Methodology for static code procedure

forces prescribed in paragraph 3-3 of the BDM may be reduced by a maximum of 20 percent (i.e., to 0.80 of the prescribed force) but the drift limitations will remain as prescribed in paragraph 3-3(H) of the BDM. This is the minimum acceptable level of safety for long-term (more than 5 years) use. If it is not feasible to meet this requirement, plans should be made to phase out the structure.

(2) Nonconforming systems and materials. When the lateral force resisting system or the structural materials do not conform to the approved systems and material specifications of the BDM, justification for acceptability of the existing systems and/or materials is required. Requirements for substantiated data are prescribed below.

Acceptance of the approval agency is also required.

(*a*) Structural systems not specified in the BDM (e.g., "nonductile" moment resistance reinforced concrete frames and unreinforced masonry shear walls) require an analytical evaluation report. The report will include data for establishing the capacity of the system to resist seismic loads and justification for the performance of the system satisfying the intent of the BDM provisions.

(*b*) Structural materials not satisfying the minimum requirements of the BDM require an evaluation report. Guidelines for evaluation of existing materials are provided in appendix E.

(*c*) The acceptance criteria for the substantiated noncomplying structural systems and materials are the same as prescribed in paragraph (1), above, except that the drift limitations will be reduced to 80 percent of those prescribed for conforming systems and materials.

d. Methodology for the evaluation. The structural analysis will consist in the application of the prescribed seismic forces to the lateral-force-resisting system of the building in the same manner as for new construction.

(1) In older existing buildings, particularly those not specifically designed for seismic forces, in addition to investigation of the primary structural elements (i.e., shear walls, frames, bracing), attention will be paid to the investigation of possible deficiencies in the design of floor and roof diaphragms, including necessary chords, drag struts, shear transfer to vertical resisting elements, and support or anchorage of concrete and masonry walls for out-of-plane forces (see chapter 5 of the BDM). The resulting stresses in the various structural elements will be combined with the dead and live load stresses as prescribed in the BDM and compared with the allowable stresses. Structural elements that are found to be overstressed will be evaluated as to their importance to the stability or integrity of the structure. For example, moderate overstress in flexural members of redundant systems (e.g., ductile steel or concrete frames) may not lead directly to structural failure until other mechanisms occur (e.g., buckling, P-delta instability, or shear failure). In a similar manner, shear overstress in a minor shear resisting element of a concrete building may not be of serious consequence if other shear resisting elements are available to resist the redistributed forces from the overstressed element.

(2) For an existing building with identified deficiencies (e.g., overstress in a primary shear wall, diaphragm, column, or brace), an overstress ratio will be calculated. This value is defined as the ratio of the calculated stress in the most overstressed primary structural elements to the allowable stress prescribed by the BDM for that element. The base shear capacity of these buildings shall be calculated by dividing the design base shear (i.e., the base shear from the BDM provisions as used in the evaluation) by the overstress ratio.

e. Report. A report will be prepared to summarize the results of the preliminary evaluation. The report will include the following items.

(1) Basic design data; i.e., design loads and properties of materials.

(2) Description of preliminary evaluation process.

(3) Method of analysis for each structural type.

(4) Description of each building analyzed including lateral force resisting system, assumed structural properties, etc.

(5) Design calculation with results of analyses, i.e., overstress ratios, and base shear capacities including a conclusion on the acceptance of the existing structure.

(6) Recommended upgrading design concepts and preliminary cost estimates.

C-4. Development of design concepts

Based on the results of the preliminary evaluation and the identified deficiencies with respect to the acceptance criteria of the various structural elements or systems, three alternative upgrading design concepts will be developed unless it is obvious that only one concept can be economically justified.

a. Acceptance criteria. The minimum design criteria for the development of concepts for seismic upgrading of existing buildings will be substantially in accordance with the applicable provisions of the BDM as required for new construction (exact compliance with all details is not required). Nonconforming structural systems or materials (e.g., unreinforced masonry and nonductile reinforced concrete frames) may be retained in the upgrading concept provided an evaluation analysis is submitted to demonstrate that the nonconforming elements are precluded from collapse and do not constitute a hazard to life safety when subjected to the BDM forces and deformations.

b. Other considerations. In addition to compliance with the acceptance criteria, the development of alternative concepts for seismic upgrading will address the general considerations prescribed in paragraph 6-3*a* of this manual. It will be recognized that it may not be feasible and/or cost effective to completely satisfy all of these considerations in the strengthening or upgrading of an existing building. However, in many cases, the engineer has the option of designing the structural modifications at little or no additional cost and the building is not only made stronger, but its response is also improved by reduction of torsional eccentricity or other undesirable characteristics.

c. Strengthening techniques and options. Generally, the strengthening options are simple and obvious (e.g., a braced steel frame building may need heavier or additional bracing or a concrete flat slab building may be strengthened with new shear walls with minimal impairment of the building function); however, in larger and more complex buildings (e.g., hospitals), the most cost effective solution may require detailed studies. All feasible options should be considered schematically and the three best alternatives selected for concept development. Chapter 6 of this manual presents representative strengthening techniques and options for various structural systems. Combinations or variations of these options may be developed to suit specific buildings.

d. Upgrading of nonstructural elements. Evaluation of the adequacy of supports, anchorages, or bracing of nonstructural elements will be performed for compliance with the requirements of chapters 9 and 10 of the BDM.

e. Concept submittal. A concept submittal will be prepared for review and approval by the approval authority. The submittal will comply with agency standards. The design effort represented in a concept submittal will generally represent 25 to 35 percent of the effort required to complete the design of normal projects, but this figure could be higher for structural modifications. The concept submittal will include the following elements:

(1) *Basis for design.* This will include the acceptance and design criteria; a summary description of the decifiencies identified in the structural analysis; a narrative description of the alternative upgrading concepts; and justification for the recommended concept including construction phasing when appropriate.

(2) *Concept drawings.* Drawings and/or sketches will be prepared to illustrate the recommended concept. The drawings must be adequate to describe the nature, extent, and location of work required and, as a minimum, will include foundation and framing plans, typical sections, and typical connection details.

(3) *Calculations.* Edited, checked, and indexed calculations will be included in the submittal to support the design of upgrading modifications.

(4) *Outline specifications.* Outline specifications will be prepared to describe the type and grade of structural material and procedures by reference to standard or industry specifications.

(5) *Cost estimates.* The concept submittal will include construction cost estimates for the alternative concepts as well as the recommended concept. These estimates shall be sufficiently accurate and detailed for budgeting and programming.

C-5. Final design and preparation of contract documents

Upon authorization of the approval authority, the final design of the approved concept will be implemented and the necessary project construction documents will be prepared in accordance with the requirements of the BDM.

a. Final design. The final design will be done on the basis of the results from the structural analysis and the development of design concepts as directed by the approval authority. The final design will include a complete analysis of the upgraded structure, completed drawings of all details for the project, and a detailed cost estimate. The final documents will be complete in themselves, without the need to refer to the previous analysis and development work.

b. Preparation of project documents.

(1) *Design analysis.* A design analysis, conforming to agency standards, will be provided with final plans. This analysis will include seismic design computations for the determination of earthquake forces on the building, for the structural evaluation of the existing building, and for the upgrading of the existing structure, including stresses in the lateral-force-resisting elements and their connections, and the resulting lateral deflections and interstory drifts. The first portion of the design analysis, called the Basis of Design, will contain assumptions made with regard to selection of dead, live, and seismic loads; allowable stresses for all original and new structural material; description of the existing structural system and the structural system selected for upgrading the building to resist lateral forces; and a discussion of the reasons for its selection. If irregular conditions exist, a statement describing special analytical procedures to account for the irregularities will be submitted for review and approval by the approval authority. The Basis of Design will also indicate any possible future expansion for which provisions are made.

(2) *Drawings.* Preparation of drawings will conform to agency standard.

(3) *Specifications.* Preparation of specifications will conform to agency standards for ordinary construction with additional specific requirements that relate to seismic construction and to upgrading of existing construction.

APPENDIX D

SUMMARY OF THE RAPID SEISMIC ANALYSIS PROCEDURE

D-1. Introduction

This appendix summarizes the rapid seismic analysis procedure (RSAP) developed by the Naval Civil Engineering Laboratory (NCEL) for the Naval Facilities Engineering Command (NAVFACENGCOM). The RSAP is preceded by computer and on-site screening at which time site hazards are identified. The RSAP is intended to identify buildings that are either liable to be severely damaged or only lightly damaged. It is a further screening tool. A complete description of this procedure is given in the NCEL Technical Memorandums TM No. 51-78-02 and TM No. 51-83-07. Examples showing the analysis of a steel and a concrete building are given in paragraph D-9.

D-2. Background

The RSAP was initially developed by John A. Blume & Associates in a pilot study of a relatively large number of buildings at Puget Sound Naval Shipyard in 1973. The procedure was formalized by NCEL.

a. Seismic investigation of an activity. The seismic investigation is divided into two phases. In Phase I the selected buildings at the activity are analyzed approximately by RSAP. Phase I parallels chapters 2, 3, and 4 of this manual. Those buildings found to be inadequate to Phase I are analyzed in detail in Phase II to determine the degree of strengthening required and to estimate costs of upgrading. Phase II parallels chapters 5, 6, and 7 of this manual.

b. RSAP. The main purpose of the RSAP is to identify those buildings that may be susceptible to severe damage. The major steps of the RSAP are shown in table D-1. The procedure has the same development roots as the procedures covered by chapters 2, 3, and 4 of this manual. The major modifications that NCEL made to the basic rapid analysis procedure follow:

(1) Systemization of the analysis of the facility inventory assets at a Naval installation.

(2) Development of the response spectra for the design earthquakes. This procedure has since been formalized by the Tri Services Committee and is covered by NAVFAC P-355.1 (e.g., SDG).

(3) Automation of computation of shear stiffnesses for concrete or masonry buildings, the first mode shape and natural period of multi-story buildings, and estimation of building damage from the response spectra.

(4) Enhance the RSAP with the following modifications:

(*a*) Criteria for field screening.

(*b*) Criteria for eliminating buildings from further investigation in the rapid analysis.

(*c*) Modified criteria for determining structural properties including damping values, natural periods and base shear capacities.

(*d*) Modified criteria for determining the site demand from the response spectra at the ultimate base shear capacity for certain systems.

(*e*) Criteria to aid the selection of buildings for detailed analysis.

(*f*) Criteria to aid in evaluating the adequacy of the lifeline utilities at a given Naval activity.

D-3. Selection of buildings

The selection procedures of the RSAP includes provisions for inventory reduction, field screening, gathering of structural drawings and calculations, a visual inspection of the selection buildings, and a cursory survey of the site geological hazards.

a. Inventory reduction. A procedure and criteria are presented in the RSAP references to facilitate the selection of the buildings for the visual screening. With the issue of this manual, the RSAP criteria are superseded by the screening procedure of paragraph 2-3 of this manual.

b. Field screening. The RSAP references recommend criteria for eliminating buildings from further investigation. These decisions are made after the brief survey to determine physical conditions and after a brief examination of construction drawings. The criteria are similar to those provided in paragraph 3-2 of this manual.

c. Visual inspection of selecting buildings. A final visit is made to verify that buildings are built as shown on the drawings, especially the lateral-force resisting elements. This step of the RSAP is similar to the first two steps of the preliminary evaluation described in paragraphs 4-2*a* and 4-2*b* of this manual.

d. Site geological hazards. During the site visits, a cursory survey should be made of the potential seismically-induced geological hazards based on the available geologic subsurface information. These hazards include faults and fault rupture, liquefaction, landslide and lateral spreading, ground cracking, compaction settlement, tsunami, and seiches.

Table D-1. Major steps of the Rapid Seismic Analysis Procedure (RSAP)

Preliminary

- Visual survey of the lifeline utility system.
- Screening.
- Selection of buildings.

RSAP

- Determination of the site elastic response spectra.
- Determination of the structural properties at yield and ultimate levels for the transverse and longitudinal directions.
- Estimation of damage from the structural capacities and demands from the response spectra.

Follow-Up

- Selection of buildings for detailed analysis.
- Follow-up investigation of site hazards.

D-4. Determination of response spectra

Site specific elastic response spectra for single degree-of-freedom systems are determined in accordance with the procedures given in the SDG, chapter 3, appendix C and appendix D. The NAVFAC ground motion criterion for the RSAP is a maximum ground acceleration having a 20 percent probability of exceedence in 50 years. (Note, this differs from the provisions in this manual, which specifies EQ-II. EQ-II has a 10 percent probability of not being exceeded in 100 years.)

a. Sample response spectra. Figure D-1 shows the resulting response spectra for an intermediate soil site with a maximum ground acceleration of 0.25g. The curves in the figure are used for determining the seismic demands (loading) on the buildings. These spectra are used for the examples of the RSAP given in paragraph D-9.

b. Acceptable capacities. Buildings with spectra acceleration capacities at ultimate that satisfy the site demands at ultimate according to the ground motion criterion are considered fully acceptable. Those buildings whose spectral acceleration capacities at ultimate are 75 percent of the demands at ultimate are considered marginal.

c. Variation in force levels. It is recommended that damage estimates be made for a few force levels below and above the 80 percent/50 year level. These estimates provide a profile of the expected seismic response of the building. This recommendation is similar to those in paragraph 4-2*d*(6) of this manual.

D-5. Determination of structural properties at yield and ultimate levels

The damping values, the natural periods, and the base shear capacities are determined for the transverse and longitudinal directions of the building.

a. Damping values. The assumed damping values used in the RSAP are given in table D-2.

Table D-2. Damping values

Type	Percent of Critical Yield	Percent of Critical Ultimate
Steel	5	10
Concrete	5	10
Wood	10	20
Masonry	5	10

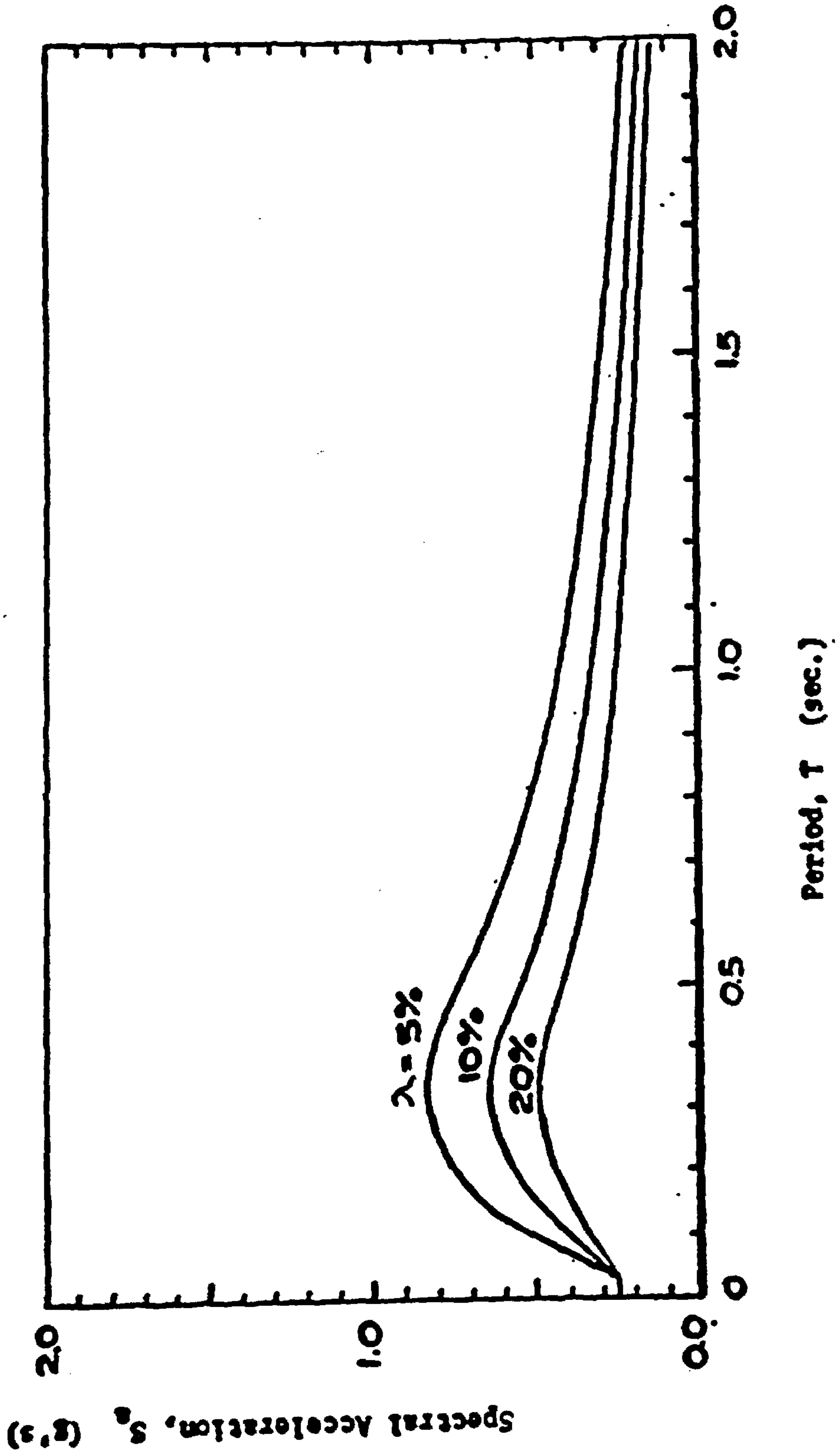


Figure D-1. Site response spectra for a Naval activity in southern California

(Note, these vary from the values given in table 4-2 of this manual.) The damping value increases from the yield to the ultimate level due to the inelastic deformation of the structural and non-structural elements of the building.

b. Natural periods. Natural periods of the building in the transverse and longitudinal directions are determined from the following equations:

(1) Yield Level:

$$\text{Empirical: } T_Y = C\left(\frac{0.05\,h_n}{\sqrt{D}}\right) \quad \text{(eq D-1)}$$

$$\text{Theoretical: } T_Y = 2\pi\sqrt{\frac{m}{k}} \quad \text{(eq D-2)}$$

$$T_Y = 2\pi\,\frac{\sum_{i=1}^{n} w_i\delta^2}{g\sum_{i=1}^{n} f_i\delta_i} \quad \text{(eq D-3)}$$

(2) Ultimate Level:

$$T_U = T_Y\sqrt{\mu\,\frac{S'_{aY}}{S'_{aU}}} \quad \text{(eq D-4)}$$

where
h_n = height of building (ft)
D = width of building in the direction considered (ft)
C = a constant between 0.75 and 1.5 to account for building mass and stiffness
m = seismic mass
k = stiffness of the building in the direction considered
w_i = weight of the building at level "i"
δ_i = elastic deformation at level "i" using the applied lateral forces f_i
f_i = approximate lateral force distribution consistent with the assumed fundamental mode shape
μ = ductility factor equal to ratio of maximum displacement to yield displacement
S_{aY} = spectral acceleration capacity of the building at yield level
S_{aU} = spectral acceleration capacity of the building at ultimate level

(3) Equation D-1 is obtained by multiplying equation 3-3A of NAVFAC P-355 (e.g., BDM) by the constant C to account for the different building masses and stiffnesses. Equation D-2 is the natural period for a single degree-of-freedom system.

(4) Equation D-3 is the Rayleigh equation 3-3 of the BDM. The weight of the building is approximated by assuming unit weights for the roof framing, floor framing, wall, actual live loads (if any), and other miscellaneous items.

(5) The natural periods of the building at the ultimate level, T_U, are computed from the periods at the yield level, T_Y, by using equation D-4. The range of the recommended ductility factors, μ, are given in table D-3.

Table D-3. Ductility factors

Type	μ
Steel	4-6
Concrete	3-4
Wood	3-4
Masonry	2-3

c. Base shear capacities. After reviewing the field survey notes and the construction drawings, rought sketches of typical plans and elevations of each building are made to determine the primary lateral-force resisting system or systems. The yield and ultimate base shear capacities of a building are computed by summing the contributions from the vertical lateral force-resisting elements of the building in the transverse and longitudinal directions and dividing the results by the seismic weight of the building. The horizontal lateral-force resisting elements such as beam, girders, floor and roof diaphragms are only considered indirectly in the analysis by examining the effectiveness of their connections to the vertical lateral-force resisting elements.

(1) *Yield capacity.* The yield capacity of a building is defined as the lateral-force required to cause the significant yielding of the most critical, not necessarily the most rigid, component of the lateral-force resisting system.

(2) *Ultimate capacity.* The ultimate capacity of a building is defined as the lateral-force required to cause yield initiation of the most flexible component of the lateral-force resisting system of the formation of a collapse mechanism.

(3) *Examples.*

(*a*) A steel building with a lateral-force resisting system consisting of infill brick walls and X-braces may behave as follows in resisting seismic forces. The brick wall and X-braces may act together in resisting the seismic forces until cracking of the brick wall is initiated. Then the X-bracing and columns (only after the yielding of the X-braces) will take more and more of the seismic loading until they fail.

(*b*) For a reinforced concrete building with shear walls, the shear walls will resist most of the seismic loading until they have started to crack. Thereafter, the frames will start to resist on increasing portion of the loading. For reinforced concrete frame and/or shear wall and reinforced masonry buildings, the ultimate base shear capacity, C_{BU}, is computed first. Then, the yield base shear capacity, C_{BY}, is obtained by dividing C_{BU} by a load factor 1.5.

(*c*) Wooden frame buildings with shear panels will behave like the concrete frame and shear wall buildings.

d. Spectral acceleration capacities.

(1) Before they can be used for estimating the earthquake damage, the base shear capacities C_{BY} and C_{BU} must be transformed to the spectral acceleration capacities S'_{aY} and S'_{aU} using the following equations:

$$S'_{aY} = \alpha\, C_{BY} \qquad \text{(eq D-5)}$$

$$S'_{aU} = \alpha\, C_{BU} \qquad \text{(eq D-6)}$$

(2) The constant α in the equations depends on the mode shape and mass distribution. The great majority of the Navy buildings are less than three stories high and can be classified as low-rise ($\leq$ 6-story). The α constant for low-rise buildings ranges between 1.05 and 1.18, with the larger value for the taller buildings. For conservatism and simplicity, α is assumed to be one in most cases. (Note, α as used in this appendix is the inverse of α used in the SDG and in table 4-1 of this manual.)

D-6. Estimate of damage

Earthquake damage is estimated from the demands of the response spectra using the damping values, natural periods, and spectral acceleration capacities of the building.

a. Damage assumption. Until yield capacity of the building is reached, damage is assumed to be equal to zero and ductility factor equal to one. When the ultimate capacity is reached, damage is assumed to be equal to 100 percent and ductility factor equal to the maximum value. For intermediate values of capacity, damage assessment is necessarily somewhat subjective and depends on many factors not amenable to analytical treatment. For the rapid analysis, damage is assumed to vary linearly between the yield capacity, S'_{aY}, and the ultimate capacity, S'_{aU}, as shown in figure D-2.

b. Damping assumption. Another assumption required for estimating damage is the amount of damping during the response of the building. Damping is assumed to be a constant up to the yield capacity. Above yield, the damping increases because of energy absorption and dissipation from inelastic response. The damping values used in the rapid analysis were given in table D-2. Furthermore, damping is assumed to vary linearly between the yield and ultimate capacities of the building.

c. Damage estimating procedure. The procedure for estimating damage is based on the reconciliation of the site demands, S_{aY} and S_{aU}, and the spectral acceleration capacities of the building, S'_{aY} and S'_{aU}. The procedure is illustrated graphically in figure D-2. The spectral acceleration capacities of the building are denoted by the open circles at the natural periods shown. The corresponding site demands are denoted by the black dots. The intersection of the two lines defined by the two sets of points determines the estimated damage of 60 percent. This procedure is essentially the same as the capacity spectrum method of the SDG that is described in paragraph 4-2*d* of this manual.

d. Modification to damage estimation procedure. After performing the rapid seismic analysis on a fairly large number of steel buildings and wooden buildings, comparisons of the RSAP damage estimates with damage observed in major earthquakes for buildings of similar construction indicated that the estimated damage were much higher than the observed. More realistic damage estimates were obtained by applying a reduction factor R_U to the ultimate site demands for steel, wooden, and reinforced concrete and reinforced masonry buildings with better-than-average reinforcement detailing.

(1) The reduction factor R_U is used to account for energy absorption and dissipation from inelastic seismic response of the building during actual earthquakes not accounted for by the lengthening of the natural periods and increase in damping from the yield to the ultimate level. The following R_U values are recommended:

(*a*) Steel Buildings: R_U = 5.0.

(*b*) Wooden Buildings: R_U = 5.0 for those buildings with a large number of interior partitions. For wooden warehouses and large-span wooden structures, R_U = 1.5.

(*c*) Reinforced Concrete and Masonry Buildings: R_U = 1.5 for those buildings with better-than-average detailing than required by code during their design. Otherwise, R_U = 1.0.

(2) An illustration of the effect of R_U on the estimated damage is shown in figure D-2. With R_U of 5.0, the estimated damage is reduced from 60 percent to 34.4 percent.

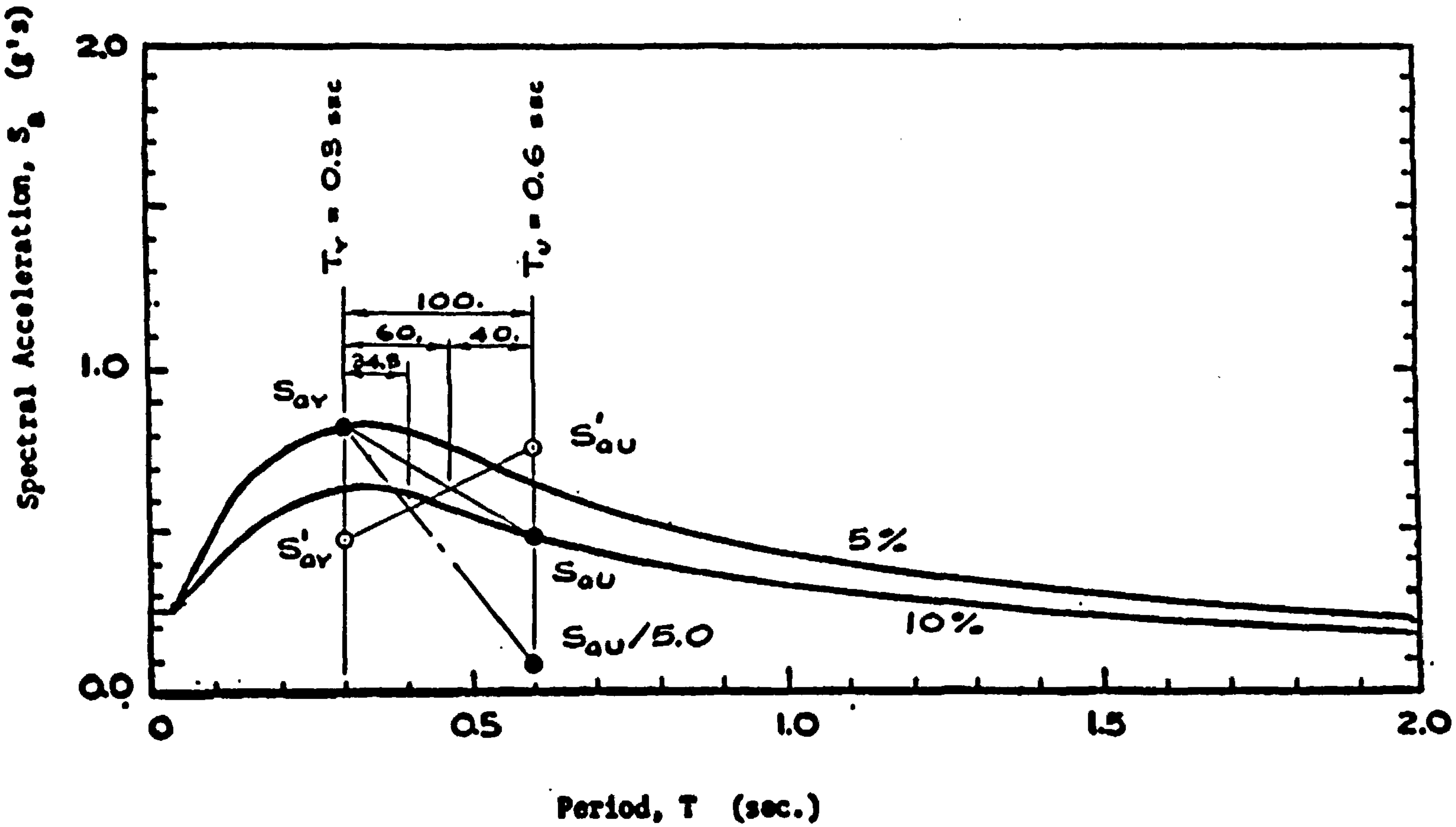


Figure D-2. Graphical illustration of damage estimation *e. Combined building damage estimate.* For each building, damage is computed for the transverse and longitudinal directions. To determine the combined damage for the building, it is assumed that one-third of the building depends on the lateral-force resisting system in each principal direction and one-third depends on both directions. That is, if a lateral-force resisting element required to provide seismic resistance in both directions is damaged by earthquake ground shaking in one direction, it is also damaged in the other direction. Combined damage for the building is obtained by taking two-thirds of the damage in the more critical direction and adding one-third of the damage in the other direction. For instance, if the damages are 60 percent and 30 percent in the transverse and longitudinal directions, the combined damage is 50 percent. (Note, this is essentially the same as paragraph 4-2*d*(5) of this manual.)

f. Computer aided procedure for damage estimates. When computing damage estimates for many buildings and/or at many different ground acceleration levels, the computation is best done by a computer program. NCEL has developed computer program CEL 9 to do the calculations. The site identification, maximum site ground acceleration, digitized site response spectra, building identification, damping values, natural periods, and spectral acceleration capacities at the yield and ultimate levels for the transverse and longitudinal directions, and the replacement cost are input into the computer. The program computes the estimates damage and cost for the building at the maximum site ground acceleration. The damage cost is obtained by multiplying the estimated percent damage by the replacement cost. In addition, the program computes damage estimates for maximum ground accelerations between 0.05 and 0.50g at 0.05g increments. A sample output from the program for a steel building is given in table D-4.

g. In general, the successful application of the rapid seismic analysis procedure demands experience in seismic design and construction and good engineering judgment.

D-7. Selection of buildings for detailed analysis

Based on the results from the rapid analysis, the following guidelines are used in selecting buildings for detail analysis:

a. Buildings with greater than or equal to 60 percent combined damage under the maximum site ground acceleration would definitely require detail analysis.

b. Buildings with greater than 30 percent combined damage may warrant detail analysis.

c. Buildings with relatively poor structural connections may require detail analysis, even if the combined damage is less than 30 percent.

d. Essential buildings and other structures that are required to remain functional during and after a major earthquake are analyzed in detail as for new buildings according to the criteria given in NAVFAC P-355.1 (e.g., SDG). Variance from the criteria is allowed only with the consent of the approving authority.

D-8. Visual survey of lifeline utilities

If an activity is to remain functional before and after an earthquake, the lifeline utility systems and the mechanical and electrical equipment must also remain functional. As a part of the rapid seismic analysis, a cursory survey is made of the lifeline utility system to determine its adequacy. The lifeline utility system at an activity includes:

- Energy
- Water
- Sewer
- Communication
- Transportation

a. Network of utility elements. The effects from the failure of an utility element of the lifeline utility system is different than the failure of a building in an activity with many buildings. The failure of a building generally has little or no effect on the surrounding buildings, except in case of fire. By contrast, the utility elements are part of a network. The failure of one element can have an immediate effect on the function of the whole network. A discussion of lifeline utility problems in past earthquakes and solutions is given in NCEL TM No. 51-83-07.

b. Administrative measures. The following administrative measures are recommended to minimize effects from earthquake damage to lifeline utilities on the mission of an activity:

(1) Analyze and strengthen inadequate structures.

(2) Provide adequate seismic bracing and/or anchorage to utility equipment and storage facilities (see chapters 3 and 10 of the BDM and chapter 6 of the SDG for examples).

(3) Provides standby emergency power, water, materials, storage facilities, and alternative utility routes to insure rapid restoration capacity.

(4) Develop disaster recovery strategies.

(5) Coordinate emergency planning with other military activities.

Table D-4. Sample output of damage estimate for a steel building from computer program CEL 9

DAMAGE ESTIMATES FOR VARIOUS LEVELS OF EARTHQUAKE

DAMAGE ESTIMATE FOR VARIOUS BUILDINGS AT NSY LONG BEACH

BLDG 132 MACHINE TOOL AND ELECTRO SHOP

BUILDING PROPERTIES AND DAMAGE ESTIMATE FOR A NOMINAL ACCELERATION OF $\boxed{\text{0.25 G}}$

	PERIOD (SEC)	DAMPING	SA STR CAPACITY (G)	SA SITE DEMAND (G)	R
TRANSVERSE DIRECTION					
YIELD LEVEL	2.430	0.05	0.130	0.169	
ULTIMATE LEVEL	3.640	0.10	0.160	0.002	5.000
LONGITUDINAL DIRECTION					
YIELD LEVEL	0.620	0.05	0.150	0.628	
ULTIMATE LEVEL	1.240	0.10	0.170	0.053	5.000

BUILDING REPLACEMENT COST $ 17260000.

ESTIMATED TOTAL DAMAGE TO BUILDING 60.1 PERCENT

ESTIMATED COST OF DAMAGE $ 10380682.

DAMAGE ESTIMATES FOR VARIOUS LEVELS OF MAXIMUM GROUND ACCELERATIONS

	TRANSVERSE DIRECTION			LONGITUDINAL DIRECTION				
	SPECTRAL ACCEL			SPECTRAL ACCEL				
MAX GRND ACCL. G	YIELD G	ULT. G	DAMAGE PCNT	YIELD G	ULT. G	DAMAGE PCNT	COMBINED DAMAGE PCNT	DAMAGE EST 1000 $
0.14	0.095	0.001	0.0	0.352	0.030	59.0	39.3	6785
0.19	0.128	0.001	0.0	0.477	0.040	71.6	47.7	8240
0.25	0.169	0.002	19.8	0.628	0.053	80.3	60.1	10380
0.05	0.034	0.000	0.0	0.126	0.011	0.0	0.0	0
0.10	0.068	0.001	0.0	0.251	0.021	40.5	27.0	4657
0.15	0.101	0.001	0.0	0.377	0.032	62.1	41.4	7149
0.20	0.135	0.001	3.2	0.502	0.042	73.4	50.0	8650
0.25	0.169	0.002	19.8	0.628	0.053	80.3	60.1	10380
0.30	0.203	0.002	31.5	0.754	0.064	85.0	67.2	11596
0.35	0.237	0.002	40.3	0.879	0.074	88.4	72.4	12490
0.40	0.270	0.003	47.1	1.005	0.085	90.9	76.3	13175
0.45	0.304	0.003	52.6	1.130	0.095	92.9	79.5	13717
0.50	0.338	0.003	57.0	1.256	0.106	94.5	82.0	14157

D-9. Examples of the RSAP

The RSAP is illustrated by means of two examples. One is a steel building and the other is a concrete building. Table D-5 gives the response spectra data for both examples. Table D-6 gives the damage estimates for the steel building and table D-7 gives the estimates for the concrete building. Figures D-3 and D-4 give the building descriptions and the RSAP calculations for the steel and concrete buildings, respectively.

Table D-5. Response spectra for steel building, example 1.

DAMAGE ESTIMATES FOR VARIOUS LEVELS OF EARTHQUAKE

DAMAGE ESTIMATE FOR VARIOUS BUILDINGS AT NSY LONG BEACH

DIGITIZED SITE RESPONSE SPECTRA FOR 0.25 G

PERIOD	PERCENT OF CRITICAL DAMPING				
	0 PCNT	2 PCNT	5 PCNT	10 PCNT	20 PCNT
0.00	0.25	0.25	0.25	0.25	0.25
0.04	0.25	0.25	0.25	0.25	0.25
0.05	0.45	0.33	0.30	0.29	0.26
0.10	1.15	0.64	0.49	0.38	0.32
0.15	1.52	0.89	0.65	0.44	0.39
0.20	1.66	0.99	0.75	0.55	0.42
0.25	1.75	1.06	0.79	0.60	0.46
0.30	1.80	1.09	0.82	0.63	0.48
0.35	1.81	1.10	0.83	0.63	0.49
0.40	1.79	1.08	0.81	0.62	0.47
0.45	1.75	1.05	0.77	0.58	0.44
0.50	1.69	1.00	0.74	0.54	0.42
0.55	1.60	0.94	0.69	0.51	0.39
0.60	1.51	0.88	0.64	0.48	0.37
0.65	1.40	0.82	0.61	0.45	0.34
0.70	1.29	0.77	0.57	0.43	0.32
0.75	1.21	0.72	0.54	0.41	0.30
0.80	1.12	0.68	0.51	0.39	0.29
0.88	1.01	0.63	0.48	0.35	0.26
0.96	0.93	0.59	0.44	0.34	0.05
1.04	0.86	0.54	0.41	0.31	0.24
1.12	0.79	0.50	0.39	0.29	0.23
1.20	0.73	0.47	0.36	0.27	0.22
1.28	0.68	0.44	0.34	0.26	0.21
1.36	0.64	0.42	0.32	0.25	0.20
1.44	0.61	0.40	0.30	0.23	0.19
1.52	0.58	0.37	0.28	0.22	0.18
1.60	0.55	0.35	0.27	0.21	0.17
1.68	0.53	0.34	0.26	0.20	0.17
1.76	0.51	0.32	0.24	0.19	0.16
1.84	0.48	0.31	0.23	0.18	0.15
1.92	0.46	0.30	0.22	0.18	0.14
2.00	0.44	0.28	0.21	0.17	0.13

Table D-6. Output for steel building, example 1.

DAMAGE ESTIMATES FOR VARIOUS LEVELS OF EARTHQUAKE

DAMAGE ESTIMATE FOR VARIOUS BUILDINGS AT NSY LONG BEACH

BLDG 131 PIPE AND COPPER SHOP

BUILDING PROPERTIES AND DAMAGE ESTIMATE FOR A NOMINAL ACCELERATION OF $\boxed{0.25\ G}$

	PERIOD (SEC)	DAMPING	SA STR CAPACITY (G)	SA SITE DEMAND (G)	R
TRANSVERSE DIRECTION					
YIELD LEVEL	0.410	0.05	0.510	0.802	
ULTIMATE LEVEL	0.750	0.10	0.619	0.082	5.000
LONGITUDINAL DIRECTION					
YIELD LEVEL	0.440	0.05	0.130	0.778	
ULTIMATE LEVEL	1.430	0.10	0.210	0.046	5.000

BUILDING REPLACEMENT COST $ 3428000.

ESTIMATED TOTAL DAMAGE TO BUILDING 65.1 PERCENT

ESTIMATED COST OF DAMAGE $ 2231705.

DAMAGE ESTIMATES FOR VARIOUS LEVELS OF MAXIMUM GROUND ACCELERATIONS

	TRANSVERSE DIRECTION			LONGITUDINAL DIRECTION				
	SPECTRAL ACCEL			SPECTRAL ACCEL				
MAX GRND ACCL. G	YIELD G	ULT. G	DAMAGE PCNT	YIELD G	ULT. G	DAMAGE PCNT	COMBINED DAMAGE PCNT	DAMAGE EST 1000 $
0.14	0.449	0.046	0.0	0.436	0.026	62.4	41.6	1426
0.19	0.610	0.062	15.4	0.591	0.035	72.5	53.5	1833
0.25	0.802	0.082	35.6	0.778	0.046	79.9	65.1	2231
0.05	0.160	0.016	0.0	0.156	0.009	11.3	7.5	258
0.10	0.321	0.033	0.0	0.311	0.019	44.6	32.4	1111
0.15	0.481	0.049	0.0	0.467	0.028	64.9	43.5	1493
0.20	0.642	0.066	19.5	0.622	0.037	74.0	55.4	1914
0.25	0.802	0.082	35.6	0.779	0.046	79.9	65.1	2231
0.30	0.962	0.098	46.3	0.934	0.056	83.9	71.6	2453
0.35	1.123	0.115	55.3	1.089	0.065	86.3	76.4	2617
0.40	1.283	0.131	61.8	1.245	0.074	89.2	80.0	2743
0.45	1.444	0.148	66.7	1.400	0.084	91.0	82.7	2842.
0.50	1.604	0.164	71.0	1.556	0.093	92.4	85.3	2923

Table D-7. Output for concrete building, example 2.

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DAMAGE ESTIMATES FOR VARIOUS LEVELS OF EARTHQUAKE

   DAMAGE ESTIMATE FOR VARIOUS BUILDINGS AT NSY LONG BEACH

      BLDG 129B  MARINE MACHINE

BUILDING PROPERTIES AND DAMAGE ESTIMATE FOR A NOMINAL ACCELERATION OF  0.25 G
```

	PERIOD (SEC)	DAMPING	SA STR CAPACITY (G)	SA SITE DEMAND (G)	R
TRANSVERSE DIRECTION					
YIELD LEVEL	0.080	0.05	0.320	0.409	
ULTIMATE LEVEL	0.160	0.10	0.480	0.502	1.000
LONGITUDINAL DIRECTION					
YIELD LEVEL	0.070	0.05	0.340	0.372	
ULTIMATE LEVEL	0.149	0.10	0.510	0.468	1.000

```
BUILDING REPLACEMENT COST          $  4455000.

ESTIMATED TOTAL DAMAGE TO BUILDING      81.1 PERCENT

ESTIMATED COST OF DAMAGE           $  3612159.
```

DAMAGE ESTIMATES FOR VARIOUS LEVELS OF MAXIMUM GROUND ACCELERATIONS

	TRANSVERSE DIRECTION			LONGITUDINAL DIRECTION				
	SPECTRAL ACCEL			SPECTRAL ACCEL				
MAX GRND ACCL. G	YIELD G	ULT. G	DAMAGE PCNT	YIELD G	ULT. G	DAMAGE PCNT	COMBINED DAMAGE PCNT	DAMAGE EST 1000 $
0.14	0.220	0.281	0.0	0.208	0.262	0.0	0.0	0
0.19	0.310	0.382	0.0	0.283	0.356	0.0	0.0	0
0.25	0.409	0.502	100.0	0.372	0.468	43.2	81.1	3612
0.05	0.082	0.100	0.0	0.074	0.094	0.0	0.0	0
0.10	0.163	0.201	0.0	0.149	0.187	0.0	0.0	0
0.15	0.245	0.301	0.0	0.223	0.281	0.0	0.0	0
0.20	0.326	0.402	7.5	0.298	0.374	0.0	5.0	224
0.25	0.408	0.502	100.0	0.372	0.468	43.2	81.1	3612
0.30	0.490	0.602	100.0	0.446	0.562	100.0	100.0	4454
0.35	0.571	0.703	100.0	0.521	0.655	100.0	100.0	4454
0.40	0.653	0.803	100.0	0.595	0.749	100.0	100.0	4454
0.45	0.734	0.904	100.0	0.670	0.842	100.0	100.0	4454.
0.50	0.816	1.004	100.0	0.744	0.936	100.0	100.0	4454.

Building 131 - Pipe and Copper Shop

Building Data

One-story steel frame building
Drawn in 1940
122 ft X 402 ft in plan X 31.5 ft high

Construction

Laterally braced steel frames
Composition roof supported by steel Trusses and bracing.
Rivet connections

Foundation consists of concrete slab supported on concrete piles.

Lateral force resisting system:

Steel frames with vertical bracing in the longitudinal direction

Beam-Girder System

Beams:

6 - W 21 X 59
27 - W 24 X 74

Crane girders:

20 - W 30 X 116

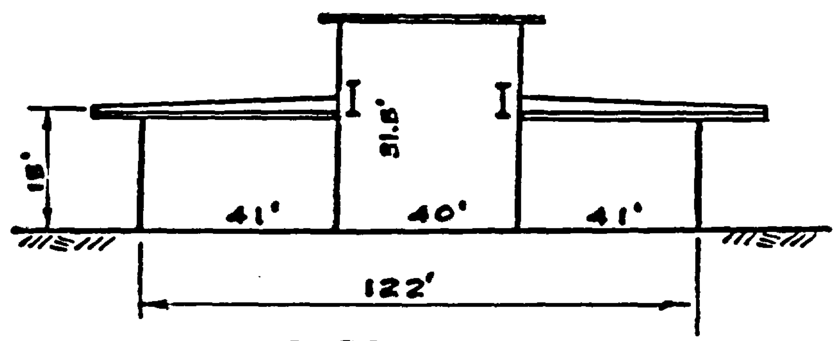
Columns

22 - W 10 X 33

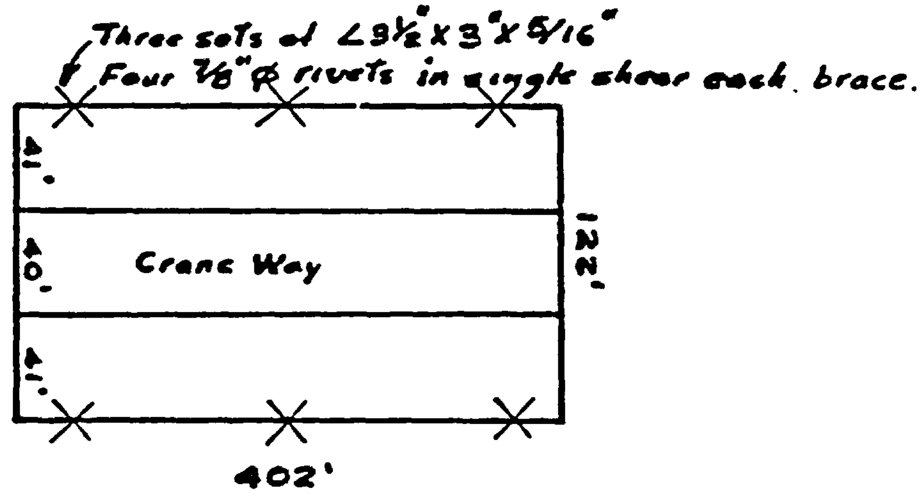
22 - W 18 X 45

Sheet 1 of 7

Figure D-3. Example of steel building. (Sheet 1 of 7)



CROSS - SECTION



PLAN

Sheet 2 of 7

Figure D-3. Example of steel building. (Sheet 2 of 7)

Weights

Roof & framing	25 psf	
Walls	10	
Misc	5	Area
	40 psf	$\times (122' \times 402') = \underline{\underline{1961.8^k}}$

Spectral Acceleration Capacities $\quad F_y = 36.\ ksi$

Columns $\quad h_{eff} = 15.'/2 = 7.5'$

W 10×33

$$v_x = \frac{F_y S_{xx}}{h_{eff}(12''/')} = \frac{36.\ ksi(35 in^3)}{7.5'(12''/')} = 14.^k/col.$$

$$v_y = \frac{F_y S_{yy}}{h_{eff}(12''/')} = 14.^k\left(\frac{9.16}{35.}\right) = 3.66^k/col$$

W 18×45

$$v_x = 14.^k/col\left(\frac{79.0}{35.0}\right) = 31.6^k/col.$$

$$v_y = 14.^k/col.\left(\frac{9.32}{35.}\right) = 3.73^k/col.$$

At yield:

Long.: $V_{YC_L} = 22 col.(3.66^k/col. + 3.73^k/col.) = 162.6^k$

Trans.: $V_{YC_t} = 22 col.(14.^k/col. + 31.6^k/col.) = 1003.2^k$

Sheet 3 of 7

Figure D-3. Example of steel building. (Sheet 3 of 7)

At ultimate:

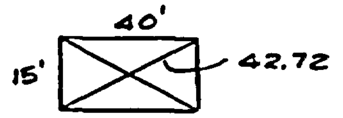
Long: $V_{uc_\ell} = 1.5\,V_{yc_\ell} = 1.5(162.6^k) = 243.9^k$

Trans.: $V_{uc_t} = 1.2\,V_{yc_t} = 1.2\,(1003.2^k) = 1203.8^k$

Diagonal Braces. Only effective in tension

Six sets of $\angle 3\tfrac{1}{2}'' \times 3'' \times 5/16''$.
Each set is connected by four $7/8''\phi$ rivets in single shear.

$A = 3.0\ in^2$



Brace:

$$P_\ell = f_y A \left(\frac{40.}{42.72}\right) = 36.\,ksi\,(3.0\,in^2)\left(\frac{40.}{42.72}\right)$$

$$= 101.1^k/_{brace}$$

Four $7/8''\phi$ rivets:

$$S_r = 4 A f_v \left(\frac{40.}{42.72}\right) = 4\left(\frac{\pi}{4}\right)(0.875)^2(18.75\,ksi)$$

$$\times\left(\frac{40.}{42.72}\right) = 42.2\,^k/_{brace}$$

Controls.

Strength of the six diagonal braces:

$$S_b = 6\,(42.2^k) = \underline{253.2}^k$$

Neglect contribution from siding.

Totals

At yield:

Long.: $S'_{ar_\ell} \div C_{Br_\ell} = \dfrac{253.2^k}{1961.8} = \underline{0.13}$

Sheet 4 of 7

Figure D-3. Example of steel building. (Sheet 4 of 7)

Trans. : $S'_{ay_t} = \dfrac{1003.2^k}{1961.8^k} = \underline{\underline{0.51}}$

At ultimate :

Long. : $S'_{au_l} = \dfrac{253.2^k + 162.6^k}{1961.8^k} = \underline{\underline{0.21}}$

Trans. : $S'_{au_t} = \dfrac{1203.8^k}{1961.8^k} = \underline{\underline{0.61}}$

Natural Periods

At yield

$$T_Y = 2\pi\sqrt{\frac{m}{k}}$$

$$m = \frac{W}{g} = \frac{1961.2^k}{32.2} = 60.9^k\text{-sec}^2/ft$$

Column stiffnesses :

$$k_c = \sum \frac{12EI}{L^3}$$

Long.: $k_{cl} = 22\left[\dfrac{12.(30.\times10^3)}{(15.)^3(144)}\right](36.5 + 34.8)$
$= 1,161.^k/ft$

Trons.: $k_{ct} = 22\left[\dfrac{12.(30.\times10^3)}{(15.)^3\,(144)}\right](171. + 706.)$
$= 14,289.^k/ft.$

Bracing stiffness :

$$k'_{dl} = \frac{AE}{L}\left(\frac{40.}{42.72}\right)^2 = \frac{3.(30.\times10^3)}{42.72}\left(\frac{40.}{42.72}\right)^2$$
$$= 1847.0^k/ft/\text{brace}$$

Sheet 5 of 7

Figure D-3. Example of steel building. (Sheet 5 of 7)

$$k_{d\ell} = 6\,(1847.0\,^{k}/ft) = 11{,}082\,^{k}/ft$$

Long. :

$$k_{\ell} = k_{d_\ell} + k_{c_\ell} = 11{,}082.\,^{k}/ft + 1{,}161.\,^{k}/ft$$
$$= 12{,}243\,^{k}/ft$$

$$T_{Y_\ell} = 2\pi \sqrt{\frac{60.9^{k}\text{-}sec^2/ft}{12{,}243.\,^{k}/ft}} = \underline{\underline{0.44}}\ sec$$

Trans. :

$$T_{Y_t} = 2\pi \sqrt{\frac{60.9^{k}\text{-}sec^2/ft}{14{,}289.\,^{k}/ft}} = \underline{\underline{0.41}}\ sec$$

<u>At ultimate</u> .

Long. : Only the column stiffnesses are effective

$$T_{U_\ell} = 0.44 \sqrt{\frac{1{,}161.}{12{,}243.}} = \underline{\underline{1.43}}\ sec$$

Trans. :

$$T_{U_t} = 0.41 \sqrt{\mu \left(\frac{S'_{aY_t}}{S'_{aU_t}}\right)} = 0.41 \sqrt{4/1.2}$$
$$= \underline{\underline{0.75}}\ sec$$

Sheet 6 of 7

Figure D-3. Example of steel building. (Sheet 6 of 7)

Summary

	T (sec.)	S_a' (g)
At Yield		
Long.	0.44	0.13
Trans.	0.41	0.51
At Ultimate		
Long.	1.43	0.21
Trans	0.75	0.61

The response spectra used to load the building is given in Table D-5.

Computer output for the building is shown in Table D-6. . The combined damage for the building at 0.25 g is 65.1%. Hence, the building requires strengthening. This can be accomplished by welding the existing diagonal brace connections and/or the installation of new diagonal braces.

Sheet 7 of 7

Figure D-3. Example of steel building. (Sheet 7 of 7)

Building 129 B - Marine Machine Shop

Building Data

One-story reinforced concrete building with two mezzanines
Drawn in 1944
172 ft X 275 ft in plan X 34 ft high

Construction

Reinforced concrete frames and shear walls.
Built-up roofing over concrete roof slab.
Concrete slab foundation

Lateral-force resisting system:

Reinforced concrete frames and shear walls.

Beams

	No.	Size (in)	
	24	14 X 16	
	24	14 X 28	
	24	18 X 42	
	36	18 X 33	
Total	= 108	16 X 30	= Average

Columns

	No.	Size (in.)	
	12	14 X 18	
	12	16 X 16	
	24	18 X 24	
	12	18 X 30	
Total	= 60	18 X 24	= Average

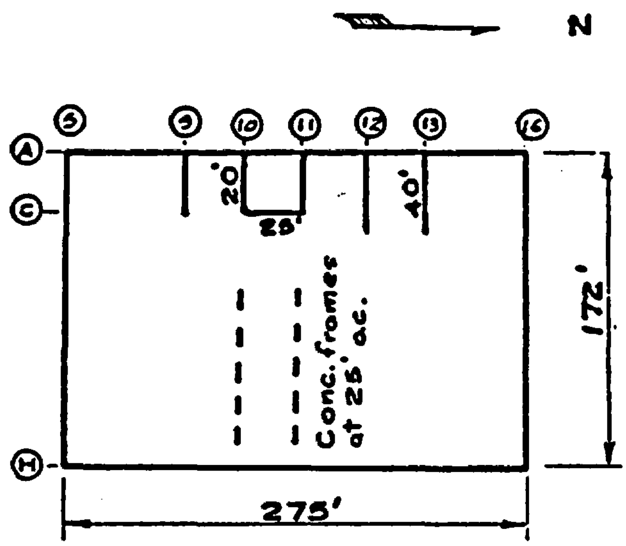
Sheet 1 of 6

Figure D-4. Example of concrete building. (Sheet 1 of 6)

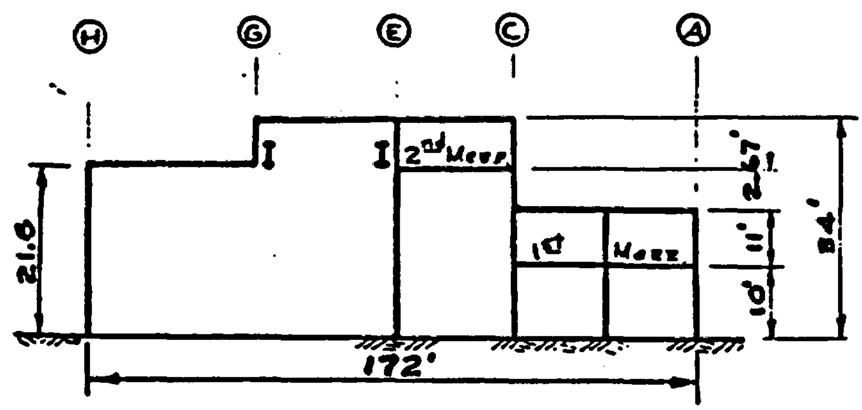
Shear Walls

Exterior : 8 in. thick with about 50% openings

Interior : 6 in. thick with negligible amount of openings.



PLAN



CROSS-SECTION

Sheet 2 of 6

Figure D-4. Example of concrete building. (Sheet 2 of 6)

Weights

Roof

Beams $\frac{16 \times 30}{144}(150pcf) = 500\ lb/ft$

$\frac{500\ lb/ft}{25\ ft\ spacing} = 20\ psf$

Slab $\frac{6}{12}(150pcf) = 75\ psf$

$95\ psf$

Use 100 psf

Walls

1050 linear foot of tributary 15' high 8 in. thick wall.

$15'(\frac{8}{12})(150pcf)(1059ft) = 158{,}850\ lb$

$\frac{158850\ lb}{(172')(275')} = 33.6\ psf$

Use 35 psf

Seismic Weight

Roof & framing	100 psf
Walls	35
Misc.	15
	150 psf

$Weight = 0.150psf\ (172')(275')$

$= 7095.^{k}$

Sheet 3 of 6

Figure D-4. Example of concrete building. (Sheet 3 of 6)

Spectral Acceleration Capacities

Assumptions :

1. Ultimate shear strength for concrete columns and shear walls = 100 psi
2. Only one-third of the column cross-sectional area is effective in resisting seismic shear forces.

Columns

60 - 16" X 24" (Average size) columns

$A = 16 \times 24 = 384.\ in.^2/col$

$v_{cu} = 384.\ in^2/col.\ (0.1 ksi)/3 = 12.8^k/col$

$V_{cu} = 60\ col.\ (12.8^k/col.) = \underline{\underline{768.}}^k$

Shear Walls

Long.: $(550\ ft)(12 in/ft)(8\ in.)(0.5)$
$+ (25 ft)(12 in/ft)(6 in)(1.0)$
$= 28,200 in.^2$
$\times 0.1 ksi$
$V_{swu_l} = \underline{\underline{2820}}^k$

Trans.: $(344 ft)(12. in/ft)(8.\ in)(0.5)$
$+ (140 ft)(12 in/ft)(6. in)(1.0)$
$= 26,529\ in.^2$
$\times 0.1 ksi$
$V_{swu_t} = \underline{\underline{2,659.}}^k$

Sheet 4 of 6

Figure D-4. Example of concrete building. (Sheet 4 of 6)

Total Capacities

At Ultimate

Long.: $S'_{au_\ell} \doteq C_{Bu_\ell} = \dfrac{2820^k + 768^k}{7095^k}$

$= \underline{\underline{0.51}}$

Trans.: $S'_{au_t} \doteq C_{Bu_t} = \dfrac{2659^k + 768^k}{7095^k}$

$= \underline{\underline{0.48}}$

At Yield

Long.: $S'_{ay_\ell} = \dfrac{0.51}{1.5} = \underline{\underline{0.34}}$

Trans.: $S'_{ay_t} = \dfrac{0.48}{1.5} = \underline{\underline{0.32}}$

Natural Periods

At Yield

Long.: $T_{Y_\ell} = \dfrac{0.05\,h_n}{\sqrt{D}} = \dfrac{0.05(22)}{\sqrt{275}}$

$= \underline{\underline{0.07}}$ sec

Trans.: $T_{Y_t} = \dfrac{0.05(22)}{\sqrt{172}} = \underline{\underline{0.08}}$ sec

At Ultimate

Long.: $T_{U_\ell} \doteq 2\,T_{Y_\ell} = 2(0.07) = \underline{\underline{0.14}}$ sec

Trans.: $T_{U_t} \doteq 2\,T_{Y_t} = 2(0.08) = \underline{\underline{0.16}}$ sec

Sheet 5 of 6

Figure D-4. Example of concrete building. (Sheet 5 of 6)

Summary

	T (sec.)	S_a' (g)
At Yield		
Long.	0.07	0.34
Trans.	0.08	0.32
At Ultimate		
Long.	0.14	0.51
Trans.	0.16	0.48

The response spectra used to load the building was given in Table D-5

The computer output for the building is shown in Table D-7. The damage threshold is at a maximum ground acceleration of about 0.20 g. The estimated combined damage

at 0.25 g acceleration is 81.1%. Thus, the building is inadequate and requires strengthening, particulary in the transverse direction. This can be accomplished by thickening the

existing shear walls with shotcrete and/or the addition of new interior shear walls. Understandably, load paths must be provide to the new shear walls to transfer seismic

forces to them, through them, and into the foundation soil.

Sheet 6 of 6

Figure D-4. Example of concrete building. (Sheet 6 of 6)

APPENDIX E

GUIDELINES FOR THE EVALUATION OF EXISTING MATERIALS

E-1. Introduction

Many existing buildings may have less strength in members and connections than they would have if they were constructed more recently. This is due to: poor quality control and detailing practices specified at the date of construction; damage or deterioration of structural materials with age or use; and uncertainties in the estimation of the material and section properties. This difference may be taken into account by assigning capacity reduction factors to members in buildings. These factors should be determined in accordance with the engineers' assessment of the existing conditions of the building under consideration, the confidence level of the estimating material properties, and the workmanship. Physical properties of existing materials are usually available in the original design drawings and construction documents. In the absence of existing data, field investigation and tests of sample members may be required as described in this appendix. The bending moment, shear, and axial load capacities of critical members in existing buildings will be determined assuming that they have the same yield strength as the new materials. Rehabilitation of some existing structural materials to remain in an upgraded building may be required in addition to the modification or strengthening of other materials. Where rehabilitation of damaged or deteriorated structural material is not feasible or cost effective, capacity reduction factors, as described above, may be assigned to the existing members if the member is capable of resisting loads, but at reduced capacity (e.g., a steel beam that has suffereed a measurable loss of section due to corrosion).

E-2. Physical properties of existing building materials

a. Structural steel. Physical properties of steel members and connections can be determined with reasonable confidence from the review of existing data and/or field inspection. If the age of the building is known, the physical properties of the steel members may be inferred by reference to manuals or specifications of that time period.

b. Concrete.

(1) *Reinforced concrete.* With reinforced concrete elements, it is essential to estimate the compressive stress (f_c') of concrete and the minimum yield stress (f_y) of reinforcing steel. In addition, the amount of reinforcement and connection details are important factors in evaluating the capacity of reinforced concrete members. Field tests described in paragraph E-3 may be required to verify the existing data available from the as-built drawings and construction documents.

(2) *Unreinforced concrete.* Although unreinforced concrete construction is permitted only in the design of pedestal or footing not on piles, in accordance with recent building codes, most unreinforced concrete components used as structural or nonstructural elements in older buildings have some structural capacity that should be considered in the capacity evaluation. The capacity criteria of unreinforced concrete elements in existing buildings may be determined considering $5\sqrt{f_c'}$ for flexural tension and $2\sqrt{f_c'}$ for shear. Capacity reduction factors should be assigned to account for uncertainties in material evaluation and workmanship of the construction.

c. Masonry.

(1) *Reinforced masonry.* The capacity evaluation of reinforced and unreinforced masonry elements in existing buildings is rather difficult. Because age and deterioration may affect the capacity of existing masonry elements; the type of masonry and the quality of mortar are generally unknown; construction details may be greatly different from current practices; testing is expensive; and interpretation of the test results may be difficult. Despite the above deficiencies, field and laboratory tests of sample members prescribed in paragraph E-3 are advisable. However, in some cases, it may be less expensive to assume a minimum compressive strength (f_m') consistent with the codes and construction practices at the date of construction rather than to perform extensive field and/or load tests.

(2) *Unreinforced masonry.* Unreinforced masonry construction is generally not permitted in design to resist seismic forces, in accordance with current building codes. However, most unreinforced masonry elements used either as nonstructural partitions or structural elements in existing buildings have some structural capacity and should be considered in the capacity evaluation of existing buildings. The yield strength criteria of existing unreinforced elements may be assumed 1.7 times working stresses specified in agency manuals for ordinary or nonseismic construction. Capacity reduction factors may be assigned to take into account uncertainties in material evaluation

and workmanship of the construction.

d. Timber. The physical properties of wood members and connections may be determined from field inspection and existing data shown in the original design and construction documents. When grade marks are available, appropriate physical properties may be determined by referring to recommended design values by the National Design Specification for Wood Construction or other relevant documents. Conversely, when grade marks are not available but such information is essential, field inspection and/or tests should be performed to evaluate the quality of the materials and their strength properties. However, in some cases, finishes or members must be removed for the field inspection. The decision to undertake extensive explorations involving the removal of finishes should be made by weighing the benefits gained against the costs of such exploration.

e. Foundations. Evaluation of the capacity of existing foundations requires the evaluation of the structural materials (i.e., concrete, piles, drilled piers, etc.) as well as the soil properties. Consultation by a qualified soils engineer and field investigations, including borings and soil tests, may be required to establish appropriate soil properties for the structural performance levels prescribed in this manual.

E-3. Testing criteria for existing materials

Determination of the physical properties of material may be made by in-place, nondestructive testing (NDT), removal of samples for destructive testing, or a combination of both. These two test procedures are described in this paragraph.

a. Nondestructive tests (NDT). The NDT approach has been used for many years for metallic and homogeneous materials. Because the direct determination of strength implies that a sample element must be loaded to failure, it becomes clear that the NDT methods cannot be expected to yield absolute values of strength and are limited in accuracy. The NDT methods for nonmetallic construction materials usually attempt to measure some other property of the material from which an estimate of its strength, its durability, and its elastic parameters are obtained. Some of the NDT methods described below are not truly "nondestructive." They are considered to be relatively nondestructive, in that they generally leave only minor surface damages that can be repaired. On the other hand, coring or cutting is usually considered to be a destructive test.

(1) *Surface hardness tests.*

(*a*) These tests are an indentation type and consist essentially of impacting the surface of concrete in a standard manner, using a given mass activated by a given energy level, and measuring the size of indentation. The three known methods employing the indentation principle are: Williams testing pistol, Frank spring hammer, and Einbeck pendulum hammer. The test methods are used only for estimating concrete strength.

(*b*) In-place Brinnel and Rockwell hardness testers are commonly used in the field to estimate the tensile strength and to establish the grade of structural steel or reinforcing steel. These two test methods are standardized in ASTM E 10 and ASTM E 18, respectively.

(2) *Rebound tests.* The Schmidt rebound hammer measures the elastic rebound of concrete and is primarily used for estimation of concrete strength and comparative investigation. The method provides an inexpensive, simple, and quick method for nondestructive testing of concrete, but has serious limitations. It should not be regarded as a substitute for standard compression tests, but as a method for determining the uniformity of concrete in structure and comparing one concrete against another. The Schmidt rebound hammer tests are standardized in ASTM C 805.

(3) *Pentration techniques.* These techniques include the use of the Simbi hammer, Spit pins, and the Windsor probe. They measure the penetration of concrete and are used for strength estimations and for determining the relative strength of concrete in the same structure. Like other hardness testers, these methods should not be expected to yield absolute values of strength of concrete in a structure. The Windsor probe system is standardized in ASTM C 803.

(4) *Ultrasonic pulse velocity method.* This method is used to evaluate uniformity of metallic or nonmetallic material and to estimate its strength and elastic properties. This method involves measurement of time of travel of electronically generated mechanical pulses through a medium, the time interval being measured by a digital meter and/or a cathode-ray oscilloscope. This method has gained considerable acceptance in quality contol operations. It has become a common method on construction sites when structural steel welding is involved. The tests can be carried out on both laboratory-sized test speciments and complete structures. The pulse velocity method is standardized in ASTM C 597.

(5) *Radioactive methods.* These methods include the X-ray and gamma ray penetration tests for the determination of rebar and strand location and size, voids in concrete and masonry walls, location of anchors in stone masonry, as well as the detection of weld flaws. The principle of these methods is to place the radiation source on one

side of the member to be inspected and the film on the other. The X-rays or gamma rays penetrate the member, but undergo attenuation in the process. The degree of attenuation depends on the kind of matter traversed, its thickness, and the wavelength (or energy) of the radiation. The maximum member thickness is limited to about two feet. The high initial cost and the immobility of testing equipment in the field, in the case of X-rays, have been the main limitations of these methods. The use of gamma rays has been more acceptable in construction testing because sources such as cobalt and iridium are more portable than X-ray equipment and are easier to use on in situ materials. Tu utilize these methods, both sides of a member must be accessible and very strict safety measures must be taken, as the radiation can be lethal.

(6) *Magnetic methods.*

(*a*) The Pachometer and cover meters are magnetic devices that can measure the depth of reinforcement cover in concrete and detect the position of reinforcement bars. The methods are based on the principle that the presence of steel affects the field of an electromagnet. The devices give satisfactory results if structural members are lightly reinforced. In heavily reinforced sections, the effect of secondary reinforcement may influence the dial reading, and the satisfactory determination of the cover to steel is practically impossible.

(*b*) The magnetic particle method is used primarily to locate surface cracks and to detect discontinuities of weld joints on or close to metal surfaces. In this method, an intense magnetic field is set up and magnetic particles are applied to the surface of a section under consideration. Particles will collect at lines of defects. Various colors of magnetic particles are available and can be selected on the basis of contrast with the material surface.

(7) *Nuclear methods.* The techniques include the neutron-scattering method for moisture-content determination and the neutron-activation method for cement-content determination. These methods are not suitable for determining the strength properties of concrete. The application of nuclear methods is still in the experimental stage. The equipment required is relatively sophisticated and expensive.

(8) *Electrical methods.* The application of electrical methods has been along the lines of: determination of moisture content of concrete by dielectric measurements, tracing of moisture permeation through concrete by electrical resistivity probes, and determination of thickness of concrete payments by electrical resistivity measurements. Because the development of electrical methods for concrete is limited to specialized applications only, these methods have received very limited acceptance by the concrete industry.

(9) *Microwave absorption techniques.* These techniques have been used to estimate the moisture content and thickness of concrete. Because of the electromagnetic nature of the microwaves, they can be reflected, diffracted, and absorbed. The absorption of these waves by water has led to the development of a method of determining the moisture content of concrete and brick. These techniques are still in the development stage and are not ready for much practical application.

(10) *Acoustic emission techniques.* These techniques have been used to study the initiation and growth of cracks in metals and concrete, but they are still in their infancy.

(11) *Load tests.* The gravity load testing of a structure or a segment is used to establish the factor of safety with respect to the simulated dead and live loads. Floor or roof flexural members are the most frequently tested. However, vertical elements can also be tested with similar techniques. American Concrete Institute Building Code Requirements (ACI 318-83), Chapter 20, and the Uniform Building Code (UBC), Section 2620, prescribe criteria for the acceptance of a test component. The applied load is specified as 85 percent of the sum of 1.4 times the dead load plus 1.7 times the live load. If the maximum deflection of a beam, floor, or roof exceeds the square of the span divided by the product of the member thickness and 20,000, the deflection recovery within 24 hours after the removal of the test load must be at least 75 percent of the maximum deflection. If the measured maximum deflection is less than this value, no deflection recovery requirements are imposed. The load tests are considered to be the most expedient method of establishing the safety of a structure with respect to gravity loading. With the exception of individual frames, it is generally not practical to load test for lateral forces.

(12) *Dynamic response testing.* The technique of artificially exciting a structure to determine its dynamic response characteristics is occasionally performed. The test results are useful to verify the adequacy and reliability of structural models developed during the analytical phase of building rehabilitation. Measurements of accelerations, displacements, and strains are often required. The structure can be excited by vibration generators or a low-amplitude pull-and-release technique.

(13) *Strain measurements.* Load, strain, and displacement measurements are commonly used in connection with static and dynamic load tests to

monitor the response of a structure. Electrical resistance strain gauges bonded to members at strategic locations are used to monitor changes in electrical resistance. Displacement and force measurements may be done remotely by electronic devices or by direct measurements such as dynamometers, potentiometers, or others.

b. Testing criteria of sample materials. Visual field inspection is probably the most important and least expensive method of quality assurance, but it is limited to surface evaluation. Other methods of nondestructive and destructive testing must be supplemented with visual inspection for full quality assurance. Sample materials should be selected objectively, so that sample elements are not weighted to be nonrepresentative. Furthermore, sample locations should be spread randomly or systematically over the structure in question. This paragraph prescribes the criteria for destructive testing of sample materials and summarizes some of the nondestructive testing methods described above which are commonly employed in field and laboratory tests for certain construction materials.

(1) *Structural steel.*

(*a*) *Destructive testing.* Material used in older buildings may no longer be in current use and, therefore, must be identified by reference to ASTM designations and specifications which were in effect at the time of construction. In the absence of existing data, the destructive testing of sample materials cut from sections of a structure should be made, along with nondestructive tests described below. The laboratory testing of tensile strength and other pertinent material properties should be performed in accordance with ASTM A 370. In taking sample elements, special care must be taken to not reduce the load-carrying capacity of the structure. When material is removed at critical sections, temporary supporting may be needed.

(*b*) *Nondestructive testing.*

1. Verification of dimensions. Visual measurement, size, thickness, and material uniformity, including possible corrosion, can be accurately and quickly determined by the ultrasonic pulse velocity method (ASTM C 597).

2. Determination of in-place tensile strength. In-place Rockwell (ASTM E 18) and Brinnell (ASTM E 10) hardness testers can be used to estimate the tensile strength and to establish the grade of steel.

3. Inspection of welds. The nondestructive testing of welds and weld-related material plays a very important part in quality assurance. In addition to visually determining size and apparent quality, nondestructive methods for flaw detection, such as the ultrasonic pulse velocity method, the radioactive method, the magnetic particle method, and the liquid penetrant method. The use of penetrants is especially useful in the detection of tight surface cracks which might not be detected easily by visual examination.

(*c*) *Load tests and dynamic response measurements.* Load tests may be used to establish the safety of a structure with respect to gravity load. Dynamic response measurements may be desirable when doubt arises concerning the adequacy and reliability of mathematical models developed during the analytical phase of building rehabilitation.

(2) *Reinforced concrete.*

(*a*) *Destructive testing.* Cores provide the best qualitative method for determining compressive strength, unit weight of concrete, Poisson's ratio, and modulus of elasticity of existing structures, which are essential for determining structural capacity of existing elements. A standard size of core is 6 in. by 12 in. (diameter by height); however, a 4-in.-diameter core may be acceptable. The ideal core will have a height-to-diameter ratio of 2.0, but not less than 1.0. In taking cores and in exposing and removing steel reinforcement, special care must be taken to not reduce the load-carrying capacity of the structure. Samples should be representative, and they should be done in a way to avoid rebars. The number of cores depends on the purpose of coring and the size of the structure. A minimum of three cores is recommended. The testing criteria have been standardized in ASTM C 42. The destructive core testing should be performed along with one or more of the nondestructive tests below.

(*b*) *Nondestructive testing.*

1. Uniformity of concrete. The Windsor probe system (ASTM C 803), the Schmidt hammer (ASTM C 805), and the ultrasonic pulse velocity method (ASTM C 597) can be used to determine the uniformity of field concrete.

2. Crack detection. Crack depth, size, direction, and propagation can usually be determined with the pulse velocity equipment (ASTM C 597).

3. Location and size of reinforcing bars. The Pachometer can be used to locate reinforcing steel, size, and depth of cover.

4. Strength of reinforcing bars. The chipping gun can be used to expose reinforcing steel. Access will provide the opportunity to establish the grade visually. However, an in-place Rockwell ASTM E 18 or Brinnell ASTM E 10 tester can be used to establish the grade of reinforcing steel if it cannot be determined visually. Alternatively, a laboratory tensile test (ASTM A 37) may be performed if more accurate tensile strength is desired.

(*c*) *Load tests and dynamic response mea-*

surements. Load tests prescribed in ACI 318-83, Chapter 20, and UBC, Section 2620, may be used to determine the safety of a structure with respect to the gravity loading. Dynamic response measurements may be useful in the development of realistic analytical models for seismic safety evaluation.

(3) *Masonry.*

(*a*) *Destructive testing.* The traditional method of determining shear strength of mortar is to cut an 8-in. core and test the core in the laboratory with the bed joint rotated to a position 15 degrees off vertical. Disadvantages are: the coring machine is cumbersome, water is required in cutting and is difficult to control, the sample is often damaged during cutting, and the resulting hole is difficult and expensive to repair. An alternate method is the in-place push test developed in conjunction with Division 68—Earthquake Hazard Reduction in Existing Buildings for the City of Los Angeles. In this method, a brick adjacent to the test brick is removed by drilling or sawing out the mortar joint. The head joint on the opposite end is also removed. A calibrated ram is placed in the space left by the removed brick and a load is applied until the test brick's bond is broken. This test is simple to perform and is nondestructive. It is easy to repair and relatively inexpensive. This test has the advantage of retaining the actual vertical load on the test brick, a condition that is difficult to achieve in laboratory testing. The tests are usually conducted at various heights to vary the actual dead load condition and at horizontal locations to minimize concentrations of load. The desirable frequency of tests is one test sample per 1,500 ft^2 of wall area with a minimum of two tests per wall. The following structural properties of masonry walls are usually of interest to the engineer in evaluating the structural capacities of masonry elements.

f'_m = compressive strength
f'_v = shear strength under diagonal compression, ASTM E 447
f'_t = tensile strength under out-of-place flexure and lateral loading, ASTM E 519
G = modulus of rigidity

The ASTM tests cited above are written for field-constructed test samples rather than drilled or sawn samples; however, the same criteria can be used with a few slight modifications.

Guidelines for selection of sample specimen may be consulted in the National Bureau of Standards study for the Veterans Administration titled "Evaluation of Strength of Existing Masonry Walls."

(*b*) *Nondestructive testing.*

1. Location and size of rebars. The radioactive methods (X-ray or gamma ray) are often used to determine the location and direction of rebars, depth below surface, and size of reinforcement when both sides of a wall are accessible. The Pachometer can also be used for the same purpose when one or both sides are exposed.

2. Uniformity of masonry. Voids and rock pocket areas of a double-wythe brick wall with a grout or concrete infill can be detected by the ultrasonic pulse velocity method.

(c) Load tests and dynamic response measurement. Load tests may be used to determine the safety of an element or a structure to resist the gravity design loads.

E-4. Rehabilitation of existing structural materials

Rehabilitation of existing damaged or deteriorated structural materials may be a significant factor in the seismic upgrading of some existing buildings, incidental to, or in addition to structural modifications and strengthening procedures. Following are representative examples of feasible rehabilitation for various structural members:

a. Structural steel. Moderate accidental damage, such as bent flanges, may be repaired by flame straightening and/or jacking or peening. Care must be taken to shore loaded steel members prior to heating. Corroded or otherwise deteriorated removable elements of steel framing, such as bolted bracing and fasteners, may be replaced with new elements. Scale and other corrosion byproducts shall be removed and the steel members lightly sandblasted in preparation for a rust preventative undercoat and painting. The loss of effective section can be evaluated after sandblasting and the assigned capacity reduction factor will be re-evaluated.

b. Reinforced concrete. Prior to undertaking the rehabilitation of existing concrete structures, the apparent cause of the damage must be ascertained.

(1) If cracking or other signs of distress can be related to differential settlement due to consolidation of the soil under the footings, soil investigations will be necessary to determine anticipated future consolidation and the cost effectiveness of rehabilitation.

(2) If the cracking is related to the shrinkage and heaving of expansive soils under the foundations, rehabilitation may be cost effective if supplementary measures are taken to restrict excessive changes in the moisture content of the soil. These measures may include removal of foundation planting and paving a strip to exclude moisture from the soil around the perimeter of the building. For buildings with exposed soil in crawl spaces under the first floor, a moisture barrier with a

sand or concrete cover may also be required.

(3) Cracks in concrete walls may also be due to initial drying shrinkage of the concrete or to temperature expansion and contraction. Hairline cracks are normal in concrete structures and have little or no detrimental effect on its strength. Evidence of rust stains at a concrete crack may indicate that moisture is intruding and corroding the reinforcement. If this is not corrected, the corrosion will progress and eventually spall the concrete surface. When this condition exists, the crack should be routed out to expose the reinforcement which should be thoroughly cleaned by wire-brushing prior to patching the crack with an epoxy mortar. Although it may not be possible to prevent the cracking due to temperature expansion and contraction, control joints are effective in limiting the location of these cracks. Vertical control joints can be sawed in the outside face of concrete walls at about 8 foot centers to a depth of about ¾-inch. The sawed joint is then filled with an elastomeric sealer to exclude water. Epoxy injection is an effective method for sealing concrete cracks and restoring shear strength. Epoxy injection requires special equipment and procedures and is best accomplished by an experienced specialty contractor.

(4) Spalling of concrete surfaces in cold climates is usually caused by the freezing and expansion of water intruding into the pore spaces of the concrete. This may be prevented by a suitable elastomeric coating to exclude the moisture.

c. Masonry. The various causative factors contributing to the cracking of concrete walls, and the mitigation of those factors, described in the preceding paragraph, also apply to masonry walls. The weakest element in older masonry is usually the mortar joint, particularly where significant amounts of lime was included in the mortar and subsequently leached out by exposure to the weather. For this reason, cracks in masonry walls will usually occur in the joints, although occasionally well-bonded masonry will crack through the masonry unit. Epoxy injection is the recommended procedure for sealing cracks and restoring shear strength for masonry walls with cracks in the joints or through solid masonry units. Where cracks occur through hollow masonry units, it may be feasible to pump mortar in the cracked units to restore shear strength prior to epoxy injection of the face shells. A common problem in masonry walls is the intrusion of moisture to the inside of the building through the joints where the mortar has cracked or where the drying shrinkage of the mortar or the units has formed a path for moisture to penetrate the wall. This condition can be remedied by routing out the mortar joints in the exterior face of the wall to a depth of about ½-inch and sealing the joint with an elastomeric joint sealer.

d. Timber. Common problems, requiring rehabilitation of timber structures, include termite attack, fungus ("dry rot" or "damp rot"), and warping, splitting or checking due to shrinkage or other causes.

(1) *Insect damage.* The subterraneous termites are the most common termite variety in the United States. These insects live in the ground and construct soil tubes to the timber members that they infest. These termites can be controlled by fumigants and toxic saturation of the soil. Preventative measures include concrete curbs or pedestals (at least 12 inches high) to remove the timber from close proximity to the ground. Sheet metal shields at the top of the concrete and the use of wood preservation for timber bearing on the concrete curb or pedestal are also common preventative measures. Dry wood termites and wood boring insects can also be controlled by fumigation and by painting of the exposed timbers with a suitable penetrating chemical preservative. Damaged portions of the timber structural members will be removed and replaced or supplemented with additional members if the infestation has been properly controlled.

(2) *Fungus.* Fungus damage to timber in buildings usually occurs where the timber is allowed to be saturated for long periods of time. Wood preservative is a good preventative measure, but in the presence of excess moisture, it will be leached out and become ineffective. The optimum solution is to exclude the moisture from the inside of the building (e.g., attic spaces with leaky roofs, crawl spaces with water leaks, etc.); provide good ventilation to the affected areas; and use wood preservative for timber members in contact with exterior masonry or concrete walls. Damaged structural members will be removed, replaced, or supplemented as described above for insect damage.

(3) *Warping, splitting, or checking.* These are common problems with older timber structures. If the distress can be attributed to the presence of excessive knots, or drying shrinkage of the wood, the timber members will be removed and replaced or supplemented with additional members to resist the applied loads. If, however, the distress is due to overstress, differential settlement, or improper connection details, then these conditions must be corrected before the individual members are repaired or replaced.

APPENDIX F

DESIGN EXAMPLES—STRUCTURES

F-1. Introduction

This appendix gives illustrative examples for evaluating and upgrading various types of lateral systems in accordance with the criteria and procedures of this manual.

F-2. Use of appendix

The design examples are purely advisory; they are not intended to place super-restrictions on the manual. This appendix is not a handbook for the inexperienced designer. Neither the manual nor the manual supplemented by the appendices can replace good engineering judgment in specific situations. Designers are urged to study the entire manual. Following is a listing of the design examples.

Fig. No.	*Description of Design Examples*
F-1	Sample screening and evaluation of a large military installation.
F-2	Brick building with concrete framing system. A 3-story concrete frame structure with brick exterior walls.
F-3	Building with steel ductile moment-resisting frames and steel braced frames. A 3-story building with transverse ductile moment-resisting frames and longitudinal frames with chevron bracing.
F-4	Building with concrete moment-resisting frames and shear walls. A 10-story building with reinforced concrete lateral force resisting frames in the longitudinal direction and shear walls in the transverse direction.

DESIGN EXAMPLE F-1

SAMPLE SCREENING AND EVALUATION OF A LARGE MILITARY INSTALLATION:

Purpose. This example is presented to illustrate the screening and preliminary evaluation procedures described in chapters 2, 3, and 4 of this manual. For purposes of this example it is assumed that an A/E firm has been contracted to perform the seismic vulnerability evaluation of a military installation. The A/E's contact at the installation are representatives of the Department of Public Works (DPW).

Description of Facility. Military installation with a large inventory of buildings. The data base inventory list includes over 100 structures ranging from flag-poles and gate houses to large warehouses and a regional medical center. The installation is located in the BDM seismic zone 3 and the SDG ground motion specification is equivalent to an ATC 3-06 spectra with $A_a = A_v = 0.30g$. The soil profile coefficient for the site is type S_2.

Inventory Reduction. A meeting of representatives of the A/E and the DPW is held to review the data base inventory list, to establish an inventory reduction procedure, and to visit the site for a general overall visual inspection of the installation. The data base inventory contains data on replacement costs, year of construction, size of building in square feet and number of stories, building identification by number and name, and general usage category. A computer program is able to reorder the data base files according to (1) largest to smallest replacement costs, (2) oldest to newest year built, and (3) largest to smallest building size in square feet.

a. A list of all buildings less than 500 square feet and all buildings with replacement costs less than $50,000 are reviewed to determine if any buildings on the list are categorized as essential or high-risk. Except for the essential and high-risk buildings, all other buildings on this list are removed from the overall inventory list.

b. A list of all buildings used for housing is reviewed. One- and two-family housing, two stories or less, are removed from the overall inventory list.

c. A list of all one-story buildings is reviewed for wood frame and pre-engineered metal construction. Construction type is not listed on the data base, therefore, a visual inspection of listed buildings is required to identify the wood frame and pre-engineered metal buildings for removal from the overall inventory list.

d. The visual inspection during the site visit is also used to list sheds and other low-risk buildings that are seldom occupied by persons (maximum occupancy less than 5 occupants). Unless these structures have an essential function, they are removed from the overall inventory list.

Sheet 1 of 12

Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 1 of 12)

e. A list of all buildings constructed since 1983 is reviewed. Essential buildings remain on the overall list and all others are removed from the list when it appears that 1982 BDM criteria or equivalent have been satisfied. When in doubt, structures were kept on the overall inventory list for review in phase I, preliminary screening.

f. A list of the remaining buildings is printed out with the available descriptive information contained in the data base in the order of their identification number for use in the preliminary screening procedure.

Preliminary Screening. By means of the inventory reduction process the number of structures requiring preliminary screening has been reduced from over 100 to 74. A meeting of the A/E representatives and using agency personnel is held to determine the classification (e.g., essential, high-risk, all others) of all structures on the reduced inventory list. The A/E is given access to available design data. This includes the data retrieval files that list available building drawings, storage files of original building drawings, and additional files for available calculations and specifications. Copies of the map of the installation, with building locations, are made available.

a. A table listing the 74 buildings is made with pertinent available data including (1) building classification (essential, high-risk, or all others), (2) construction category (steel, concrete, masonry, wood, and special structures), (3) size, (4) year constructed, and (5) location on the site.

b. All buildings are located on the installation map. The map is divided into 5 geographical zones that include no more than 20 buildings each. This is done in preparation for the field inspection survey. Preliminary screening forms are filled in with data available prior to the field inspection (sample shown on sheet 4). The field surveys are scheduled to cover one of the 5 geographical zones each day. Preliminary screening forms and other data are prepackaged to aid field inspection record taking.

c. During the field surveys the screening forms and additional notes are made to record pertinent observations or information received by the building supervisor or other building personnel (sample shown on sheet 5). One or two photographs of the exterior of the building are obtained, when possible, for inclusion in the report.

d. After the field surveys are completed, the observations are reviewed and compared to data available prior to the site visit. The files of available data are reevaluated to resolve conflicts or to clarify observations made during the field investigation.

e. On the basis of the collected data, the buildings are divided into two groups: (1) those buildings determined not requiring further analysis and (2) those recommended for preliminary evaluation.

(1) Buildings not required for further evaluation are listed in a summary report that includes reasons for making the decision and gives recommendations, if any, for further action.

Sheet 2 of 12

Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 2 of 12)

(2) Buildings recommended for preliminary evaluation are listed in a summary report that includes a description of the lateral force resisting system, general condition of the structure, observed hazards (if any), and additional comments.

Preliminary Evaluation. The inventory list for the preliminary evaluation has been reduced to 50 structures. The capacities of these structures are estimated by means of a rapid evaluation technique. The capacity of the building for an initial major yielding condition and for an ultimate load condition are estimated. By use of the capacity spectrum method, the capacity curve is reconciled with the demand curve of the EQ-II response spectrum. An example of the procedure are given in sheets 6 through 12. The results of the evaluation of all the structures are summarized in a report that includes capacities, percent damage, and damage costs.

Sheet 3 of 12

Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 3 of 12).

PRELIMINARY SCREENING
(PRE INSPECTION DATA)

BUILDING NO. 55 INSPECTED BY SAF DATE 1/15/86

DESCRIPTIVE TITLE (Current Use) HOSPITAL BUILDING

CLASSIFICATION ESSENTIAL

AVAILABILITY OF DESIGN DATA DRAWINGS AND CALCULATIONS ARE AVAILABLE

BUILDING DATA:

Number of Stories 3

Height 35' Plan (Show Dimensions) 48' x 192'

CONSTRUCTION:

Structural System Structural Steel Frame

Roof METAL DECK WITH LIGHTWEIGHT FILL

Intermediate Floors METAL DECK WITH CONC. FILL

Ground Floors SLAB ON GRADE

Foundations

Interior Walls

Exterior Walls

LATERAL FORCE RESISTING SYSTEM DMRSF TRANSV. BRACED FRAME LONGIT.

EVALUATION:

General Condition

Earthquake Damage Potential

DAMAGE OBSERVED:

COMMENTS:

Sheet 4 of 12

Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 4 of 12)

PRELIMINARY SCREENING

BUILDING NO. 105 INSPECTED BY SAF DATE 1/17/86

DESCRIPTIVE TITLE (Current Use) COMMUNICATIONS BUILDING
Built circa 1910

CLASSIFICATION ESSENTIAL

AVAILABILITY OF DESIGN DATA AS-BUILT DRAWINGS AVAILABLE

BUILDING DATA:

Number of Stories 3

Height 52'-6" Plan (Show Dimensions) 62' x 186'

CONSTRUCTION:

Structural System REINFORCED CONCRETE FRAME

Roof 6" R/C SLAB AND BEAMS

Intermediate Floors SAME

Ground Floors SLAB ON GRADE

Foundations R/C COLUMN FOOTINGS AND WALL FOOTINGS

Interior Walls NONE

Exterior Walls BRICK WALLS

LATERAL FORCE RESISTING SYSTEM UNREINFORCED BRICK MASONRY PARTIALLY CONFINED BY R/C FRAMES

EVALUATION:

General Condition GOOD

Earthquake Damage Potential HIGH

DAMAGE OBSERVED: NONE OBSERVED

COMMENTS:

Sheet 5 of 12

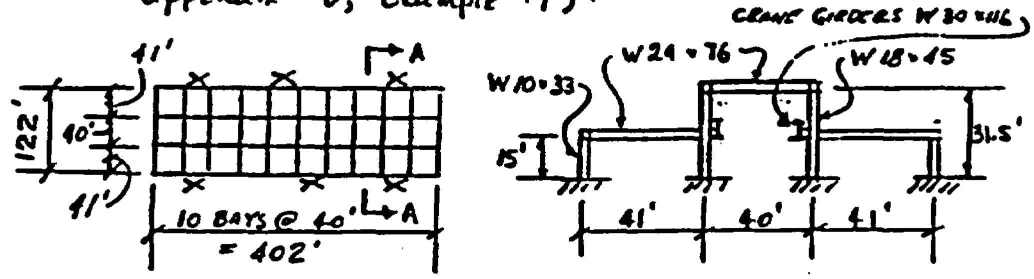
Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 5 of 12)

Building 131 - Pipe and Copper Shop (Circa 1940)

PRELIMINARY EVALUATION (para 4-2)

FIND: Capacity of the structure (para 4-2c) using a rapid evaluation technique.

GIVEN: One story steel frame building with a high crane bay (Note: Same building described in appendix D, example #1).



PLAN

SECTION A

TRANSVERSE DIRECTION:

Steel moment frames: Girders W 24×~~74~~ 76
Interior Col. W 18×45
Exterior Col. W 10×33

Crane girders W 30×116
Column base plate fixed with 4-1½" bolts

LONGITUDINAL DIRECTION:

Diagonal steel bracing: 3-bays on each side
∠ 8½ × 3 × 5/16
with 4-⅞" ϕ rivets ea. end, single shear

Note; upper portion of high bay also braced.

WEIGHTS: Roof and framing 25 psf
Walls (equiv) 10
Misc 5
} 40 psf × 122' × 402' = $\underline{\underline{1962}}^{k}$

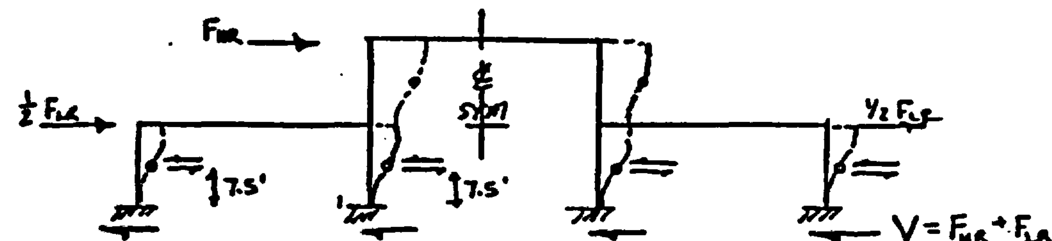
One 40' BAY: $W_B = \underline{\underline{195}}^{k}$

Sheet 6 of 12

Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 6 of 12)

Building 131 (CONTINUED)

CAPACITY IN TRANSVERSE DIRECTION



COL.	W 10 × 33	W 18 × 45
I	171 in⁴	706 in⁴
S	35 in³	79 in³
Z	39 in³	90 in³
Rel. k $\left(\frac{I}{L^3}\right)$	0.10	0.40
M_Y	105 k-'	237 k-'
M_P	117 k-'	270 k-'
M_u	146 k-'	338 k-'
V_Y	14 k	31.6 k
V_P	15.6 k	36.0
V_u	19.5 k	45.1

Assume inflection point at midheight
• GIRDER REL. RIGID
• BASE FIXED

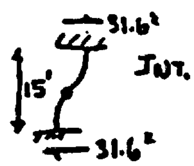
Moment at first yield = 36 ksi × S ÷ 12 (k-')
36 ksi × Z ÷ 12 - PLASTIC MOMENT
1.25 × M_P - ULTIMATE AT 125% of plastic

$V = \frac{M}{7.5'}$ at assumed inflection point.

First Yield	7.9 k	31.6 k
2ND Yield	14.0 k	36.0 k
ULT.	19.5 k	45.1 k

Interior Col. yields, exterior at $\frac{0.10}{0.40}$
$\Sigma V_Y = 2(7.9 + 31.6) = \underline{79}^k$
Interior at M_P, exterior at M_Y
$\Sigma V_2 = 2(14 + 36) = \underline{100}^k$
$\Sigma V_u = 2(19.5 + 41.1) = \underline{121}^k$

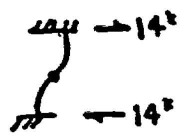
Deflection at first YIELD



INT. COL I = 706

$$\Delta = \frac{PL^3}{12EI} = \frac{31.6 \times 15^3 \times 12^3}{12 \times 29 \times 10^3 \times 706} = \underline{\underline{0.75''}}$$

Defl. when ext. col. yields



$$\Delta = \frac{14.0 \times 15^3 \times 12^3}{12 \times 29 \times 10^3 \times 171} = \underline{\underline{1.37''}}$$

Assumes Rigid Girder. Increase Values for joint rotation. Assume 20% increase.

Deflections at low roof: w/ est. for joint rotation
$\Delta_y \approx 0.75 \times 1.2 = 0.9''$
$\Delta_{y2} \approx 1.37 \times 1.2 = \underline{\underline{1.6}}''$

Sheet 7 of 12

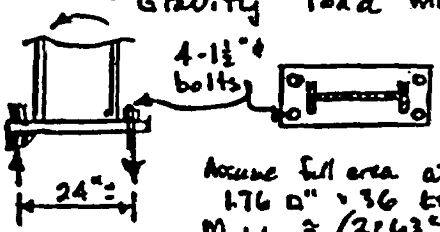
Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 7 of 12)

Building 131, Capacity Transverse Dir. (cont.)

High bay: $F_{HR} \simeq F_{LR}$ (twice acceleration, ½ mass)
$\therefore V_{HR} \approx ½ V$, ok by inspection

Check Following:
- Base plate capacity
- Girder capacities & Connections
- Gravity load moments

- Base plate



Assume full area at 36 ksi
1.76 □" × 36 ksi = 63^k/bolt
$M_{plate} \approx (2 \times 63^k) \times 2' = \underline{\underline{252}}^{k\text{-}'} > 237^{k\text{-}'}$

Bolts should hold until column yields
Conc. Foundation ok for loads.

- Girders: W 24×76: I = 2100, S = 176

$M_y = 36 \times 176 \div 12 = 528^{k\text{-}'}$ > Col. Capacities.
Beam-Column conn. ok by inspection

- Gravity loads

At 25 psf × 40' trib. width = 1000 #/' = 1 k/'
FEM at $wl^2/12$ = $1 \times 40^2/12$ = $133^{k\text{-}'}$
Moment will reduce due to moment distribution
ESTIMATE $M_{DL} \approx 90^{k\text{-}'}$
Beam: $M_y - M_{DL} = 528 - 90 = 438^{k\text{-}'}$ > Col.
Col.: Gravity moments will add to some columns and subtract from others - Capacities balance out.

TRANSVERSE CAPACITY SUMMARY (W = 195 k/BAY)

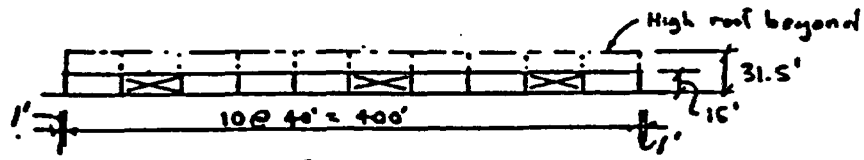
	V	C_S (V/W)	Δ_{LR}	Δ_{HR} ($2\times\Delta_{LR}$)	PFR (EST)	α (EST)	S_d $\Delta_{HR} \div PFR$	S_a $C_S \div \alpha$	T $2\pi\sqrt{\frac{S_d}{S_a g}}$
End YIELD	79	0.41	0.90"	1.8"	1.25	0.90	1.4"	0.46 g	0.56 s.
2ND YIELD	100	0.51	1.6"	3.2"	↓	↓	2.6"	0.57 g	0.68 s.
ULT w/. Δ=0.015H	121	0.62		5.7"			4.6"	0.69 g	0.82 s.

Sheet 8 of 12

Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 8 of 12)

Building 131 (CONT.)

CAPACITY IN LONGITUDINAL DIRECTION



ELEVATION

Assumptions based on field inspection:

1. High roof braced to low roof to transfer lateral forces
2. Low roof system horizontally braced to transfer from interior column lines to exterior columns
3. Interior longitudinal frame braced at upper level and by crane girder to develop frame action in W18×45 columns
4. Exterior columns fixed at top and base
5. Exterior facade negligible for lateral resistance.

Lateral force resistance :

CASE I All forces taken by 6 sets of X-braces (3 sets each side of building)

CASE II ~~Compression braces buckle, some participation by longitudinal frames~~ { VOID, NO COMPRESSION CAPACITY

CASE III Diagonal braces fail, resistance by longitudinal frames. (weak direction of columns).

DIAGONAL BRACES $\angle\, 3\tfrac{1}{2} \times 3 \times \tfrac{5}{16}$

$L = \sqrt{15^2 + 40^2} = 42.7$ feet ; Joined at center $L_x = 21.3'$

$r_z = 0.63''$, $r_x = 1.06$, $r_y = 0.983$, $A = 1.93\ \square''$

For r_z, $L = 21.3 - 2' = 19.3$; $L/r = 367$ NG no allow comp.

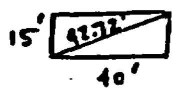
No COMPRESSION CAPACITY

Sheet 9 of 12

Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 9 of 12)

Building 131, Capacity Long. Dir. (cont)

CASES I & II   Braces effective in tension only



$F_D = \frac{40}{42.72} \times 36\text{ ksi} \times 1.93\,\square'' = \underline{\underline{65^k}}$

CHECK CONNECTIONS   4-7/8" rivets (Assume A502-1)

AISC table, Single shear allowable = 9.02 k/rivet

Yield Capacity ≈ 1.7 × Allowable = 15.3 k/rivet

$F_{D(conn)} = \frac{40}{42.72} \times 4 \times 15.3 = \underline{\underline{57.5}} < 65$

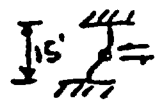
Conn. Governs

$V_{yield} = 6 \text{ braces} \times 57.5^k = \underline{\underline{345}}^k$

CASE III   Column bending action

$2 \times 11 - W10 \times 33 \quad I_{yy} = 36.5\text{ in}^4;\ S_{yy} = 9.16\text{ in}^3$
$2 \times 11 - W18 \times 45 \quad\quad = 34.8\text{ in}^4 \quad\quad = 9.32\text{ in}^3$ } Close use avg.

Use 44 col with $I = 35.6\text{ in}^4$ and $S = 9.24\text{ in}^3$

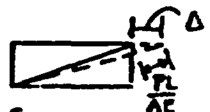


$V_{col} = \frac{M}{7.5'} = \frac{36\text{ ksi} \times S}{7.5 \times 12} = \frac{36 \times 9.24}{7.5 \times 12} = \underline{\underline{3.7^k/col}}$

V_{yield} for 44 columns $= 44 \times 3.7 = \underline{163}^k$

DEFLECTIONS

Brace:



$\frac{PL}{AE} = \frac{(4 \times 15.3)\,42.7 \times 12}{1.93 \times 29 \times 10^3} = \underline{0.56}''$

$\Delta_B = \left[(42.720' + .56/12)^2 - 15^2\right] - 40' = 0.050' = \underline{\underline{0.60}}''$ CASE I/II

Columns: $\Delta_c = \frac{PL^3}{12EI} = \frac{3.7 \times 15^3 \times 12^3}{12 \times 29000 \times 35.6} = \underline{\underline{1.74}}''$ CASE III

Sheet 10 of 12

Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 10 of 12)

Building 131, Capacity Long. Dir (CONT.)

Relative rigidities - Braces and Columns

BRACES: $\frac{345^k}{0.60''} = 575\ ^k/_{''}$

COLUMNS: $163^k/1.74'' = 94\ ^k/_{''}$

ratio = 6.1 to 1

First Yield When braces are taking 345^k at $\Delta = 0.60''$

Columns are resisting $\frac{0.60}{1.74} \times 163^k = 56^k$ at $\Delta = 0.60''$

Total Shear, $V_y = 401^k$ @ $\Delta = 0.6''$ L.R

Braces Fail Assume 1/3 fail, remainder incr. capacity by 15%

Braces: $2/3\,(345)\,1.15 = 264^k$ @ $\Delta = \frac{3}{2}\,0.6 = 0.9''$

Columns $(0.90/1.74)\,163 \quad 84^k$

$V_{u\text{-}1} = 348^k$ @ $\Delta = 0.9''$ L.R

ULTIMATE ALL BRACES FAIL

Columns Yield at 163^k @ $\Delta_y = 1.74''$

$\Delta_{max} = 0.015 \times H = 0.015 \times 15 \times 12 = 2.7''$ L.R

$\Delta_{max}/\Delta_y = 2.7/1.74 = 1.55$

Assume 5% capacity increase: $163^k \times 1.05 = 171^k$ @ $2.7''$

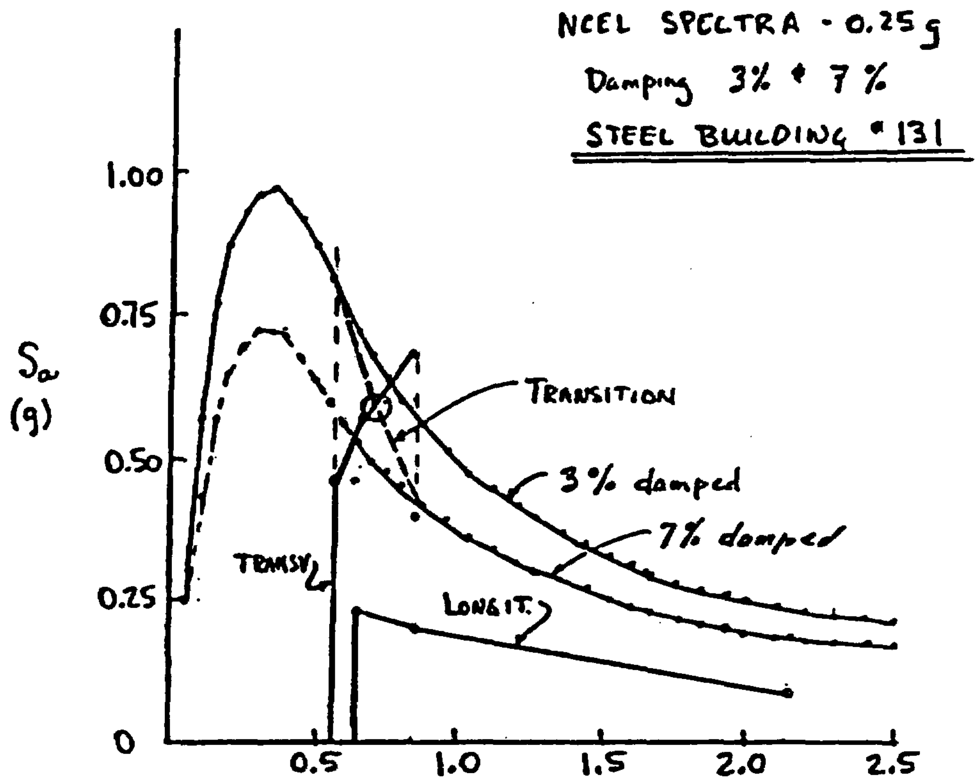
LONGITUDINAL CAPACITY SUMMARY ($W = 1962^k$)

	V	C_B (V/W)	Δ_{LR}	Δ_{NR} $2 \times \Delta_{LR}$	PFR (est)	α (est)	S_d $\Delta_{NR} \div PFR$	S_a $C_B \div \alpha$	T $2\pi\sqrt{\frac{S_d}{S_a g}}$
First Yield	401^k	0.204	0.6"	1.2	1.25	0.90	0.96"	0.23 g	0.65 s.
First Fail.	348^k	0.177	0.9	1.8	1.25	0.90	1.44"	0.20 g	0.85 s.
ULT @ Δ=0.015H	171^k	0.087	2.7	5.4	1.25	0.90	4.32"	0.097 g	2.13 s

Sheet 11 of 12

Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 11 of 12)

CAPACITY SPECTRA		TRANSVERSE			LONGITUDINAL		
	S_a	0.46	0.57	0.69	0.23	0.20	0.097
	T	0.56	0.68	0.82	0.65	0.85	2.13



Damage, percent

MAX G	TRANSV	LONGIT.	COMBINED
0.25	52%	>100%	84
0.125	0	50% *	33
0.063	0	0	0

* Capacity curve follows demand curve, assume 50%

Sheet 12 of 12

Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 12 of 12)

DESIGN EXAMPLE F-2

BRICK BUILDING WITH CONCRETE FRAMING SYSTEM

Description of Structure. A 3-story communications building built circa 1905 with vertical load carrying concrete frames and exterior unreinforced brick masonry walls. The floor and roof construction is comprised of reinforced concrete slabs and beams. The building is supported on concrete pile caps and timber piles. The structural design concepts are illustrated on sheets 2, 3, and 4.

Construction Outline.

Roof:
- Built-up roofing.
- Reinforced concrete slabs, beams, and girders.

2nd and 3rd Floors:
- Reinforced concrete slabs, beams, and girders.

1st Floor:
- Reinforced concrete slab-on-grade.

Foundation:
- Reinforced concrete tie beams and pile caps supported on timber piles.

Exterior Walls:
- Unreinforced brick masonry with terra cotta facade.

Partitions:
- Clay tile walls and wood stud walls with gypsum board sheathing.

Background. As a result of the inventory reduction and preliminary screening process the building was included in the list of buildings requiring a preliminary evaluation. On the basis of the preliminary evaluation (sheets 6, 8, and 9), upgrading concepts will be developed. A summary of the Acceptance Criteria and the determination of the Site Response Spectra are shown in the sheets 5, 6, and 7.

Sheet 1 of 25

Figure F-2. Brick building with concrete framing system. (Sheet 1 of 25)

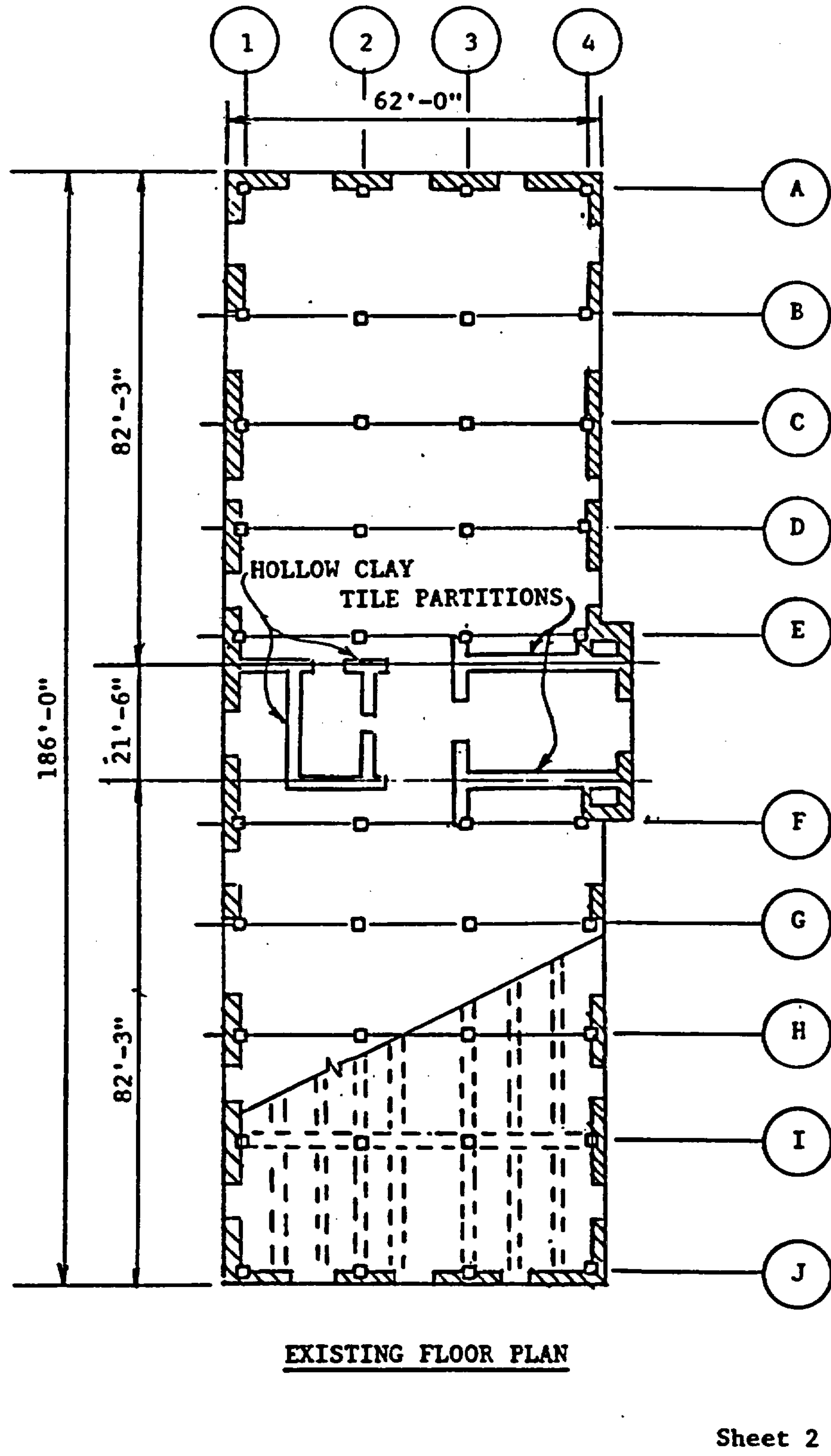
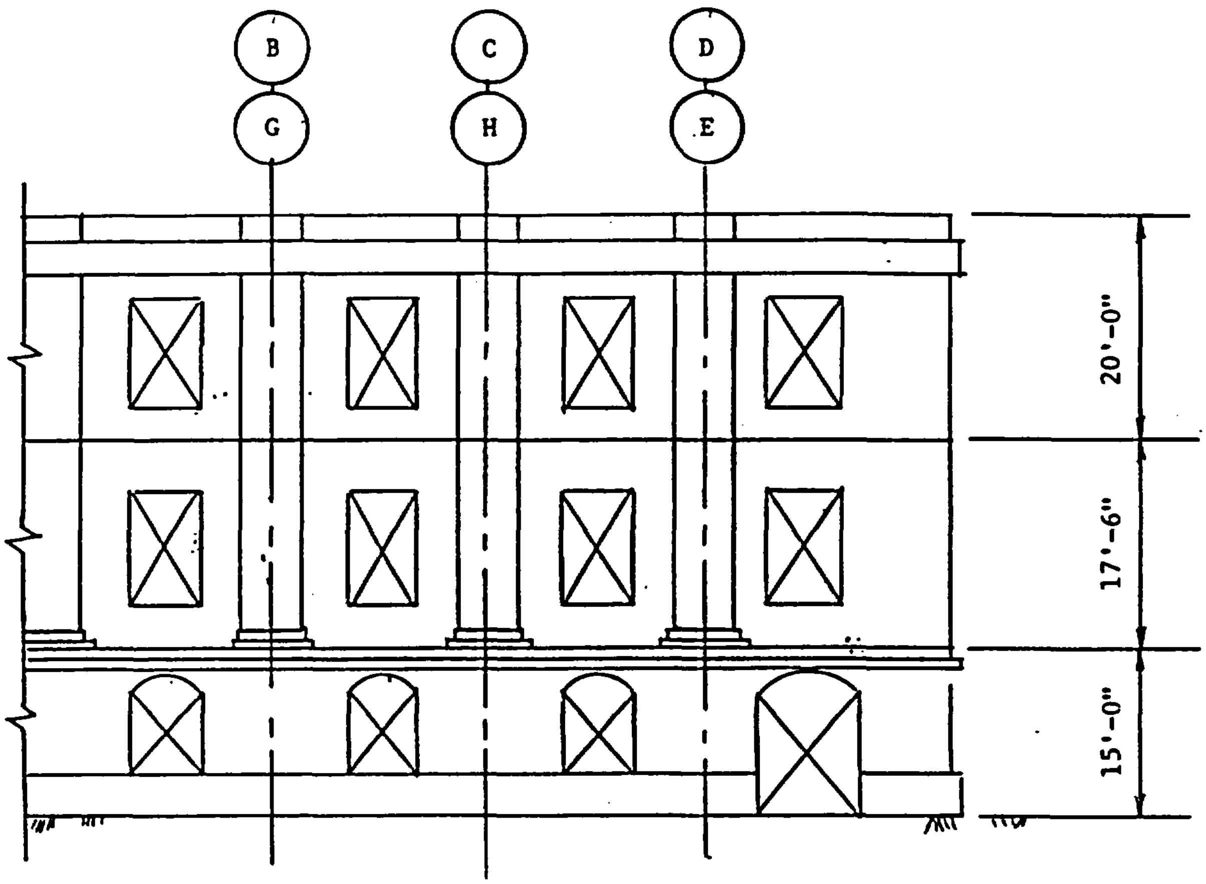


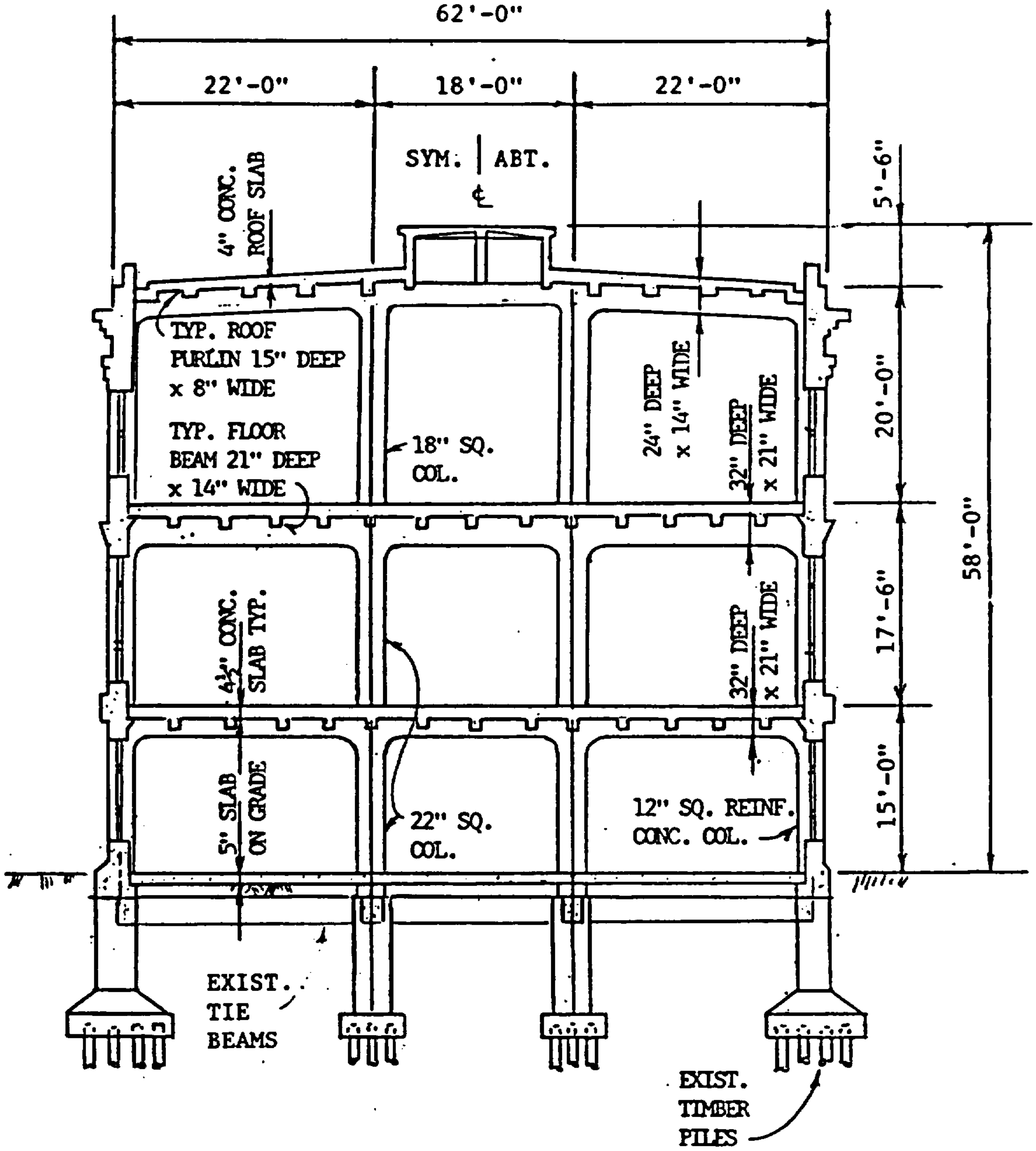
Figure F-2. Brick building with concrete framing system. (Sheet 2 of 25)



EXISTING EXTERIOR ELEVATION

Sheet 3 of 25

Figure F-2. Brick building with concrete framing system. (Sheet 3 of 25)



TYPICAL TRANSVERSE BUILDING SECTION

Sheet 4 of 25

Figure F-2. Brick building with concrete framing system. (Sheet 4 of 25)

The Acceptance Criteria for the seismic resistance is that presented for the post yield analysis for EQ-II, Method 1 (refer to SDG paras 4-4 and 5-5).

Classification:	Essential
Loading Combination:	DL + 0.25LL + EQ
Ultimate Strength Capacities:	ACI 318 Strength Design
Inelastic Demand Ratios:	
Nonductile Conc. Frames	
Columns	1.00
Beams	1.25
Reinf. Conc. Shear Walls	
Single Curtain Reinf.	Shear-1.10, Flexure-1.5
Double Curtain Reinf.	Shear-1.25, Flexure-2.0
Material Properties:	
Concrete	f'_c = 4000 psi(New)
	f'_c = 3000 psi(Exist)
Reinforcement	F_y = 60 ksi(New)
	F_y = 33 ksi(Exist)
Unreinf. brick masonry	E_m = 1000 ksi(Exist)
Story Drift Limitation:	0.006 x Story Height

Sheet 5 of 25

Figure F-2. Brick building with concrete framing system. (Sheet 5 of 25)

Site Response Spectra. The site response spectra are developed in accordance with the procedure in Chapter 3 of the SDG:

Building Classification: Essential Facility

Ground Motion Spectra: ATC 3-06 Map Contour Level, $A_a = A_v = 0.10$

Soil Classification: $S_i = 1.5$ (Type S3)

Earthquake I
- Damping = 5%, D.F. = 1.00 (SDG table 3-7)
- $A_a = A_v = 0.04g$ (Design Ground Motion, SDG table 3-4)
- S_a = D.F. $(1.22A_vS_i)/T = 0.073g/T$ less than D.F.$(2.5)A_a = 0.10g$ max

Earthquake II
- Damping = 10%, D.F. = 0.80
- $A_a = A_v = 0.12g$
- S_a = D.F. $(1.22A_vS_i)/T = 0.176g/T$ less than D.F.$(2.5)A_a = 0.24g$ max

EQ-II/EQ-I = 0.24/0.10 = 2.40

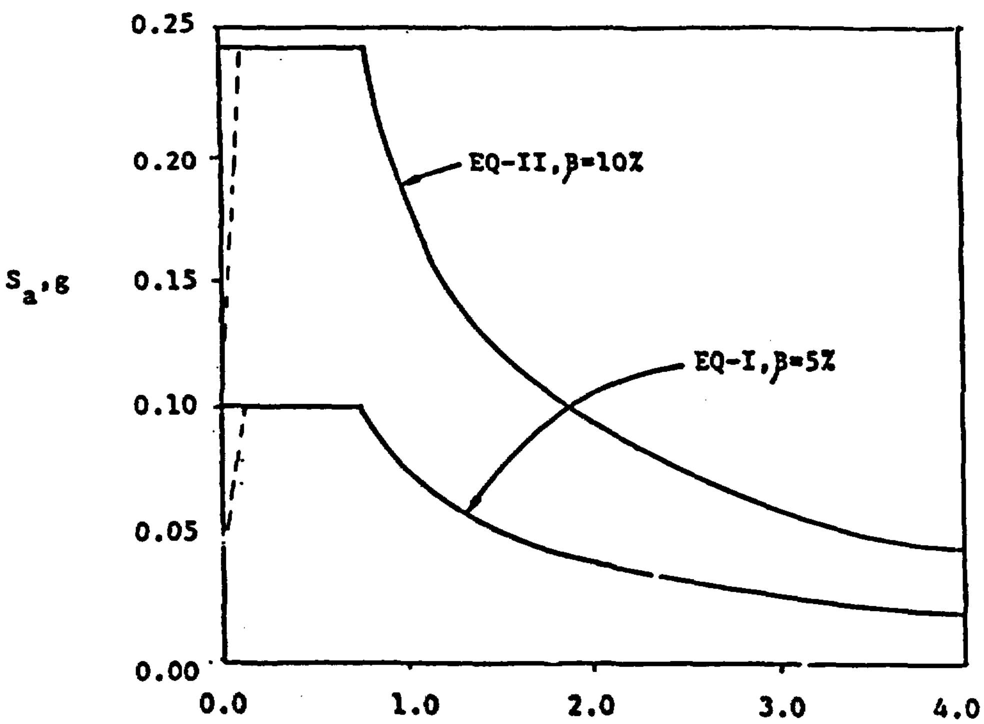
The resulting spectra are shown on sheet 7.

Preliminary Evaluation. The lateral force resisting system primarily consists of unreinforced brick masonry piers and walls, partially confined by the concrete frames. The concrete frames are capable of only a minimal amount of lateral force resistance as there is very little continuity in the reinforcement at the column-beam joints. Since the existing walls would be required to resist most of the seismic force by relative rigidity, the existing concrete frames will be ignored in the preliminary evaluation. A rapid approximation of the seismic demand is made by assuming that the demand spectral acceleration (S_a) for the first mode is 0.24g (i.e., T less than 0.7 sec) and that the base shear coefficient (C_B) $= 0.86S_a = 0.21g$ ($\alpha = 0.86$ per SDG para 5-3a(2)(c)). The seismic forces will be distributed to the various stories in accordance to the static design provisions of the BDM. The capacity of the existing structure will be approximated by calculating the average shear stress (story shear divided by the total net wall area in each direction) for each story. See sheets 8 and 9 for this preliminary evaluation. The structural deficiencies identified were the non-conforming moment-resisting concrete frames and the unreinforced brick masonry walls in both shear and flexure. Results show conclusively that the building does not satisfy the acceptance criteria. Therefore, a detailed structural analysis of the existing as-is building is not required. Upgrading concepts will be developed and the acceptance criteria of the upgraded structure will be confirmed by a detailed structural analysis.

Sheet 6 of 25

Figure F-2. Brick building with concrete framing system. (Sheet 6 of 25)

DESIGN RESPONSE SPECTRA FOR EQ-I AND EQ-II



EQ	β		PERIOD							
			0.0	.730	1.0	1.25	1.5	2.0	3.0	4.0
I	5%	S_a, g	.04	.10	.073	.058	.049	.037	.024	.018
II	10%	S_a, g	.12	.24	.176	.141	.117	.088	.058	.044
		S_d, in	0	1.25	1.72	2.16	2.58	3.45	5.11	6.89

* SPECTRAL DISPLACEMENT $S_d = S_a(T/2\pi)^2 g$

Sheet 7 of 25

Figure F-2. Brick building with concrete framing system. (Sheet 7 of 25)

RAPID EVALUATION

Longitudinal Direction

Lateral force resisted by two exterior walls.

Total length of walls = 2 x 186' = 372'.

Windows reduce effective length by 1/3.

Effective Length = 2/3 x 372' = 248'

Assume 18" brick wall at 15 psi shear strength
at 50 psi shear ultimate
(to be confirmed by tests)

At 15 psi $V = 248' \times 12 \times 18'' \times 0.015 = 803^k$
At 50 psi $V = 248' \times 12 \times 18'' \times 0.050 = 2680^k$

Calculated Weight: $10{,}000^k$

Equivalent to 17#/cu ft
or 290#/sq. ft

Estimate Capacity

$$C_B \text{ - yield?} \simeq \frac{803}{10{,}000} = \underline{0.08g}$$

$$\text{- ultimate} \simeq \frac{2680}{10{,}000} = \underline{0.27g}$$

NOTE: Typical pier width/height ratio = 1.20, therefore assume shear governs.

DEMAND of earthquake: $S_a = 0.24g$ (Sheet 6)
$C_B \simeq 0.86\ S_a = 0.21g$

Requires about 40 psi capacity (e.g., (0.21g/0.27g)x50).

If strength is confirmed by tests, the longitudinal direction could work if connections are acceptable.

NOW CHECK THE TRANSVERSE DIRECTION

Sheet 8 of 25

Figure F-2. Brick building with concrete framing system. (Sheet 8 of 25)

RAPID EVALUATION (continued)

Transverse Direction

Lateral Force Resistance

Two exterior brick walls: 44' x 2 = 88' effective length

V_{ext} = 88' x 12 x 18" x 50 psi = 950^k

Two interior hollow clay tile

V_{int} = 2 x 50'± x 12 x 8" x 20 psi = 192^k

Total resistance ≃ $1{,}142^{kips}$

Estimated capacity: $V/W = \frac{1142}{10{,}000} = 0.114$

is less than the demand C_B = 0.21 (Sheet 8).

Conclude: Weak in Transverse Direction

Even with liberal allowances for material strength, resistance about 1/2 demand.

Sheet 9 of 25

Figure F-2. Brick building with concrete framing system. (Sheet 9 of 25)

Development of Seismic Upgrade.

Structural Upgrading Concepts. Three concepts were considered:

1. Install reinforced concrete gunite against the interior faces of the exterior unreinforced brick walls and add new interior cast-in-place reinforced concrete walls.

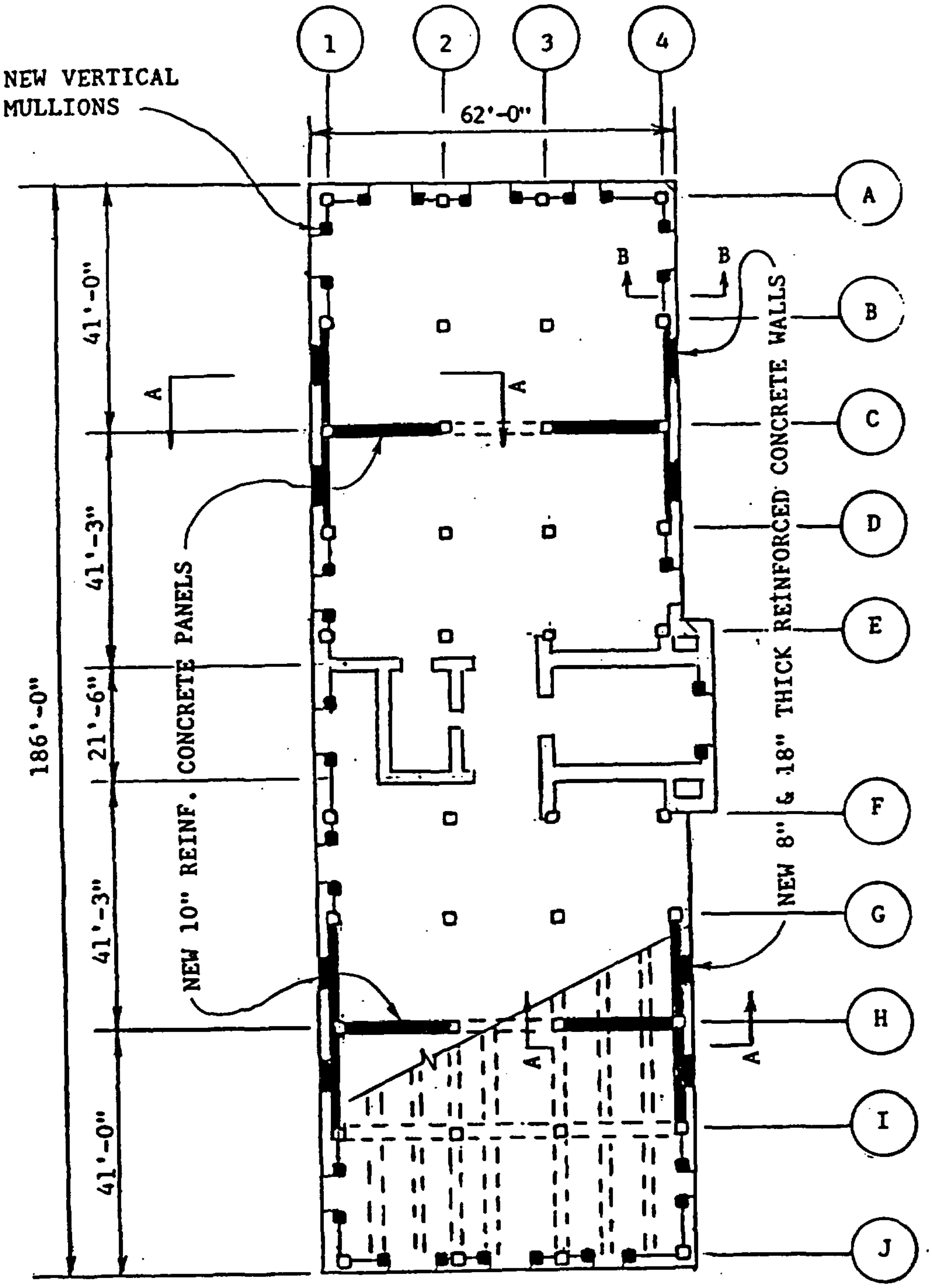
2. Install vertical structural steel plate panels as infill walls within existing transverse interior concrete framing and use diagonal steel braces for the longitudinal direction.

3. Construct exterior buttresses to give lateral support to the existing building.

No. 1, above, was selected as the recommended concept. Plans, elevations, and details are shown on sheets 11 through 17 and a discussion on the analysis is contained on sheet 18. For concept No. 2, steel walls and bracing, it was considered to be difficult to obtain satisfactory connections between steel and concrete because of the high force levels. For concept No. 3, exterior buttresses, a preliminary cost comparison indicated that it would not be as cost effective as concept No. 1. Also, concept No. 3 would distract from the historic significance of the building. A disadvantage of concept No. 1 was the blocking out of existing windows; however, it was determined that this would not be detrimental to the planned use of the building. It should be noted that, if it were mandatory to minimize on-going operations in this building, then the additional costs of concepts Nos. 2 or 3 might be justified.

Sheet 10 of 25

Figure F-2. Brick building with concrete framing system. (Sheet 10 of 25)



FLOOR FRAMING PLAN

Sheet 11 of 25

Figure F-2. Brick building with concrete framing system. (Sheet 11 of 25)

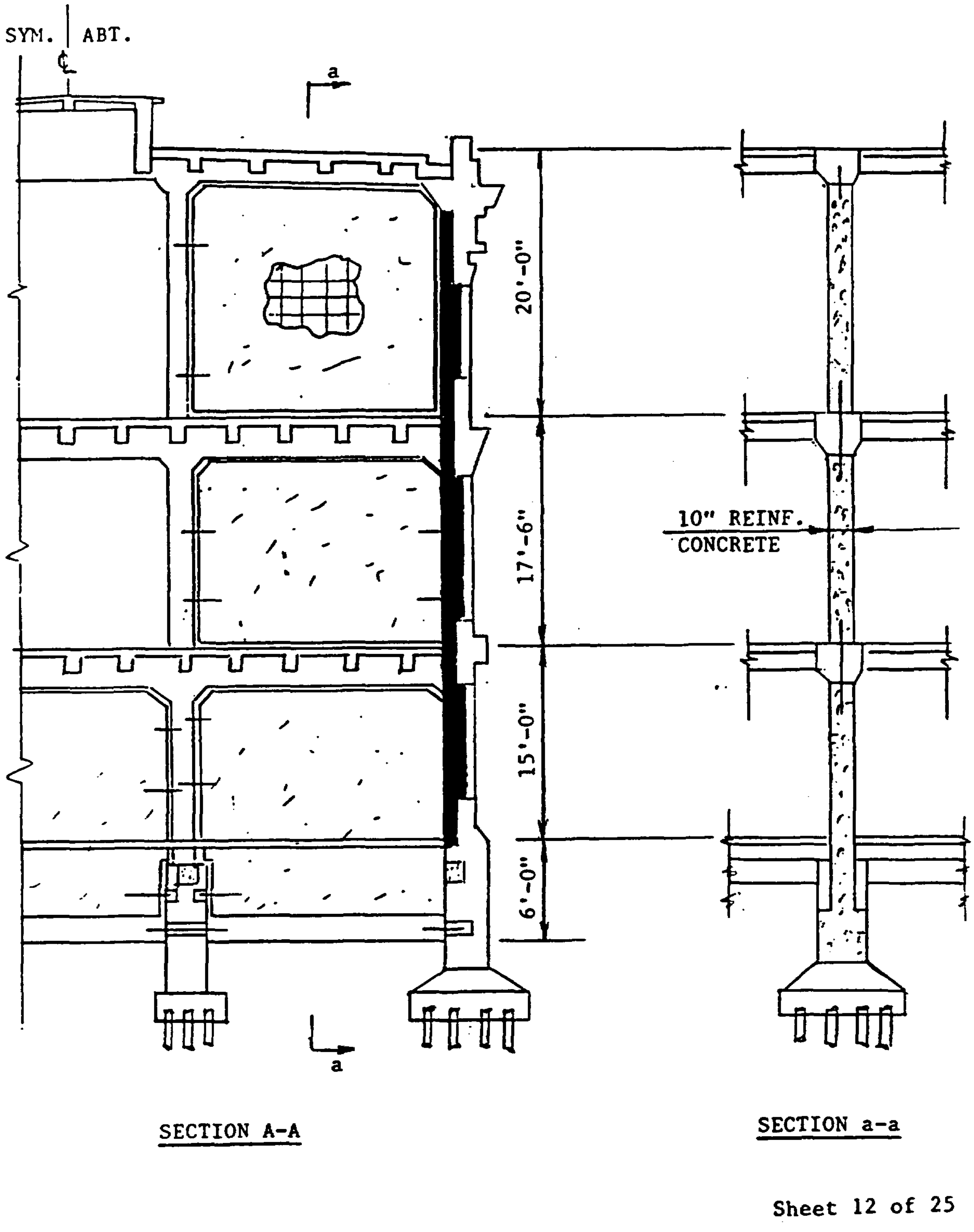
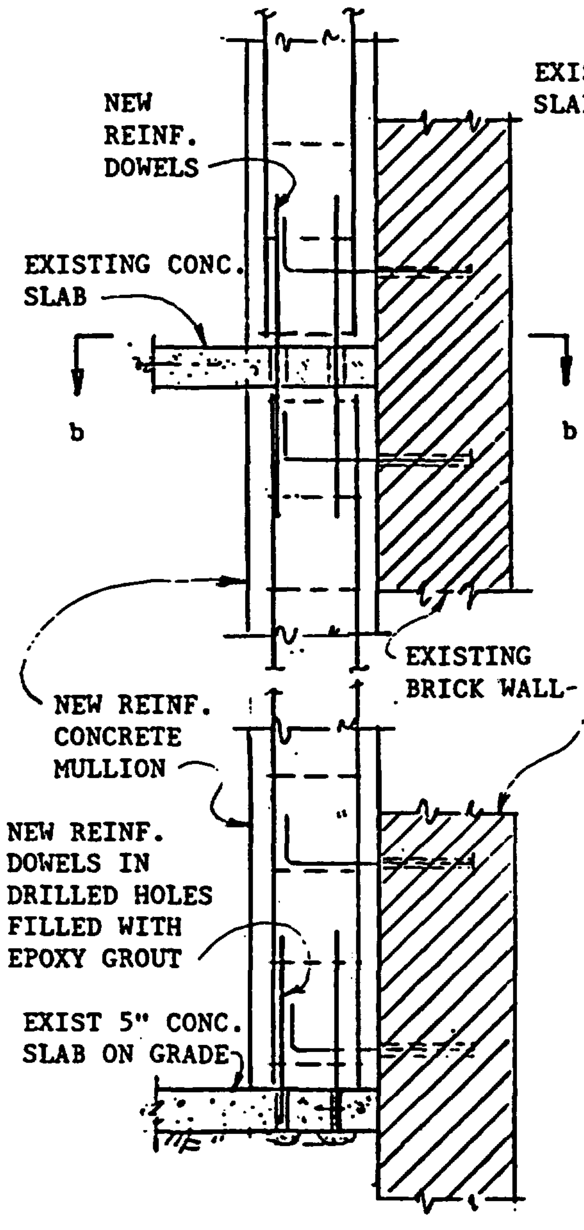
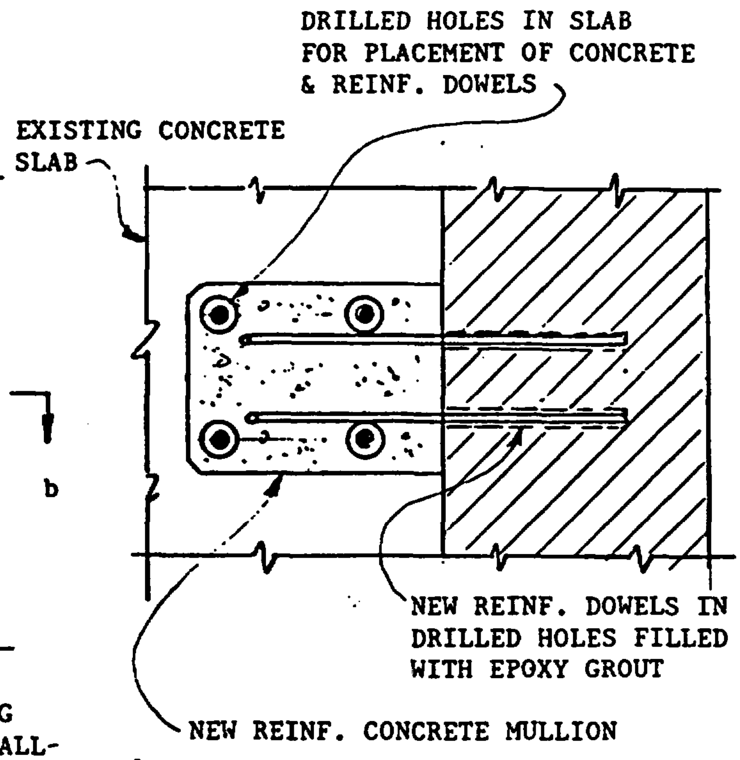


Figure F-2. Brick building with concrete framing system. (Sheet 12 of 25)



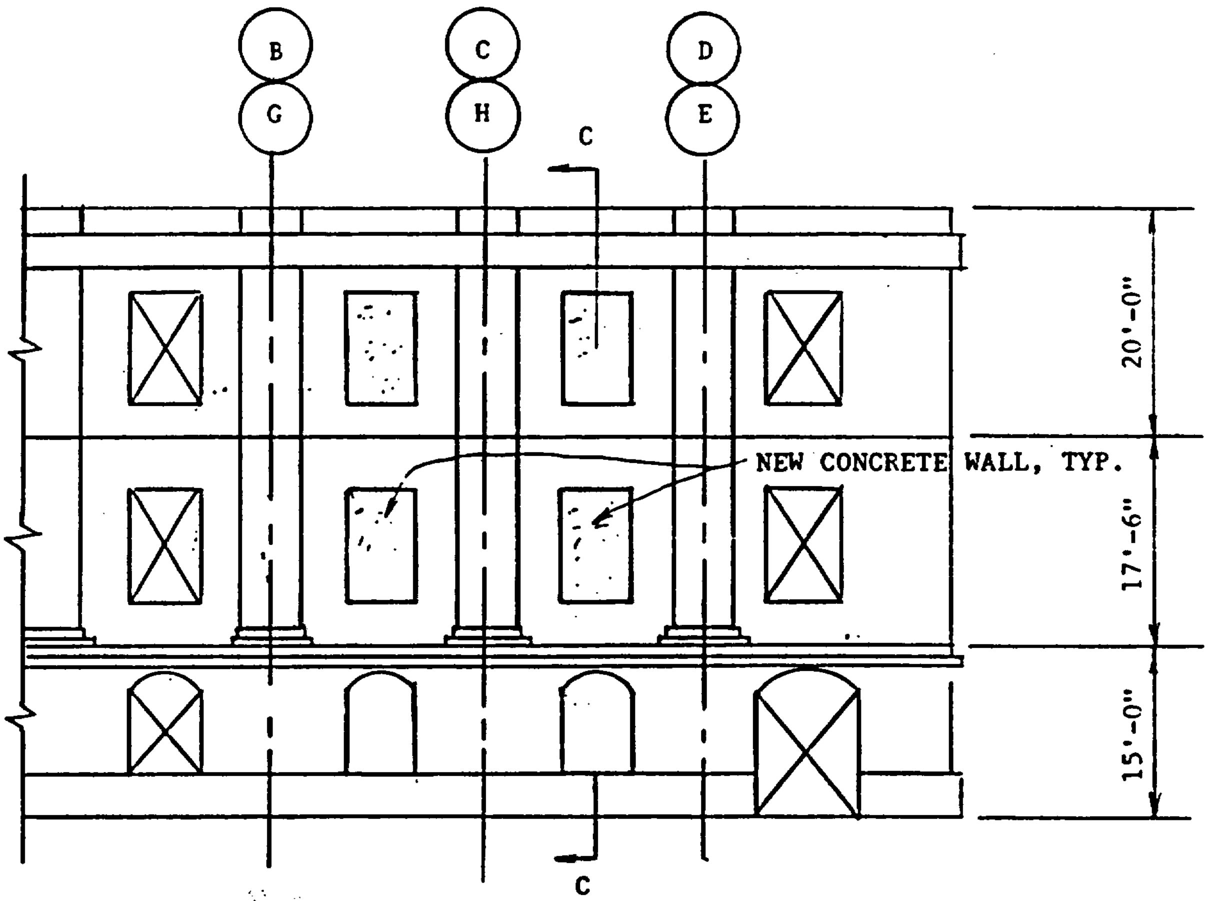
SECTION B-B



SECTION b-b

Sheet 13 of 25

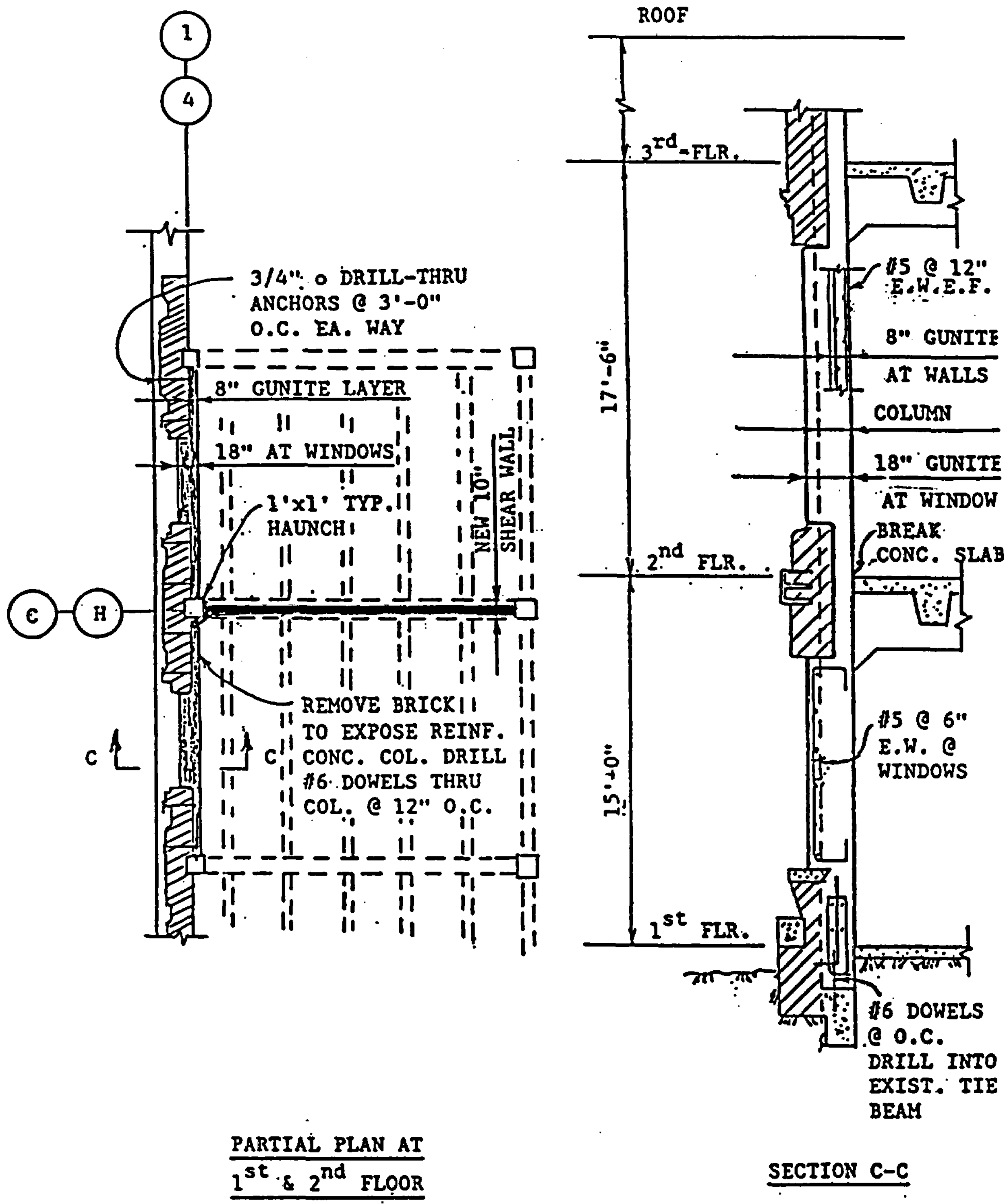
Figure F-2. Brick building with concrete framing system. (Sheet 13 of 25)



LINES 1 & 4

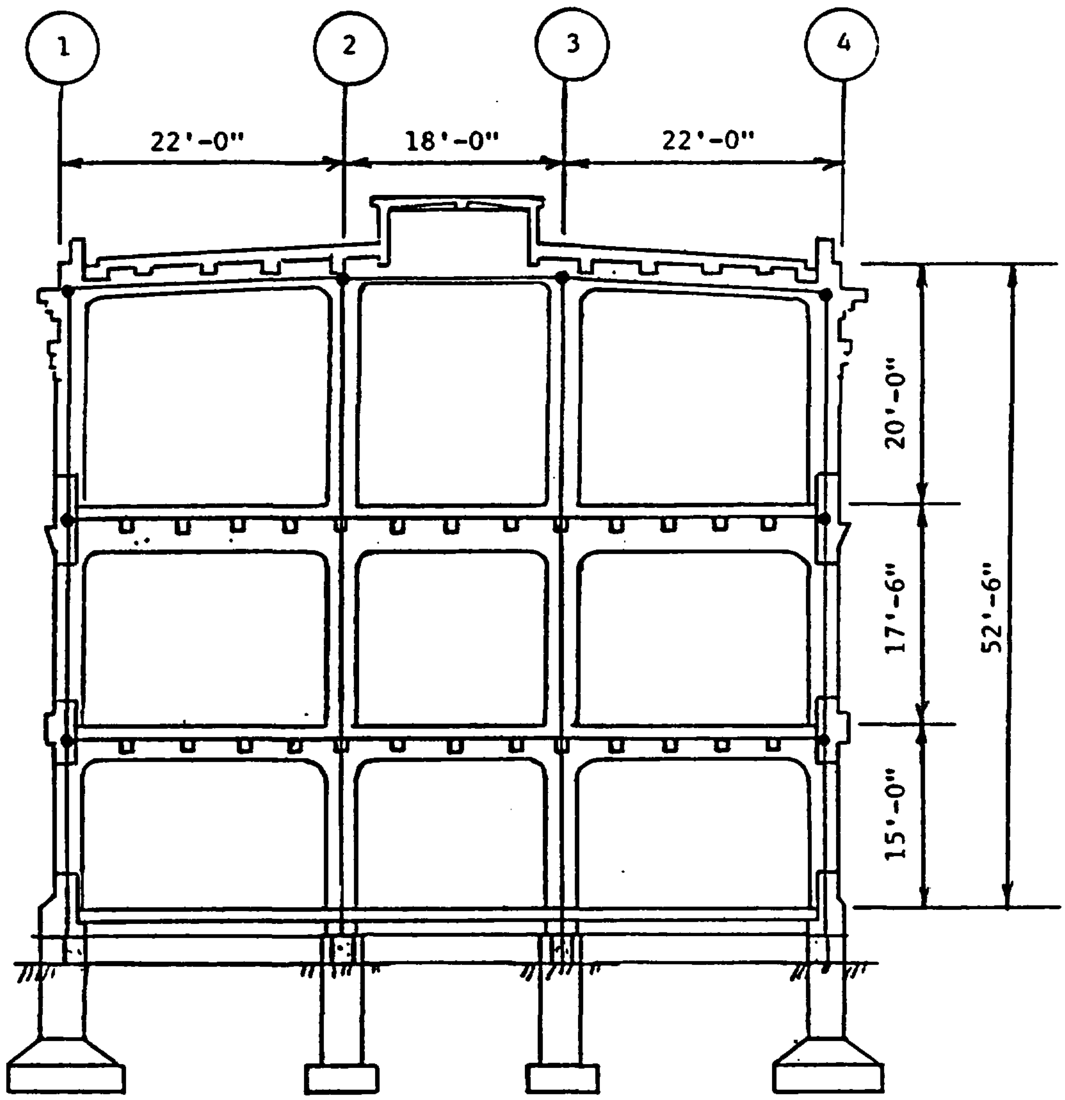
Sheet 14 of 25

Figure F-2. Brick building with concrete framing system (Sheet 14 of 25)



Sheet 15 of 25

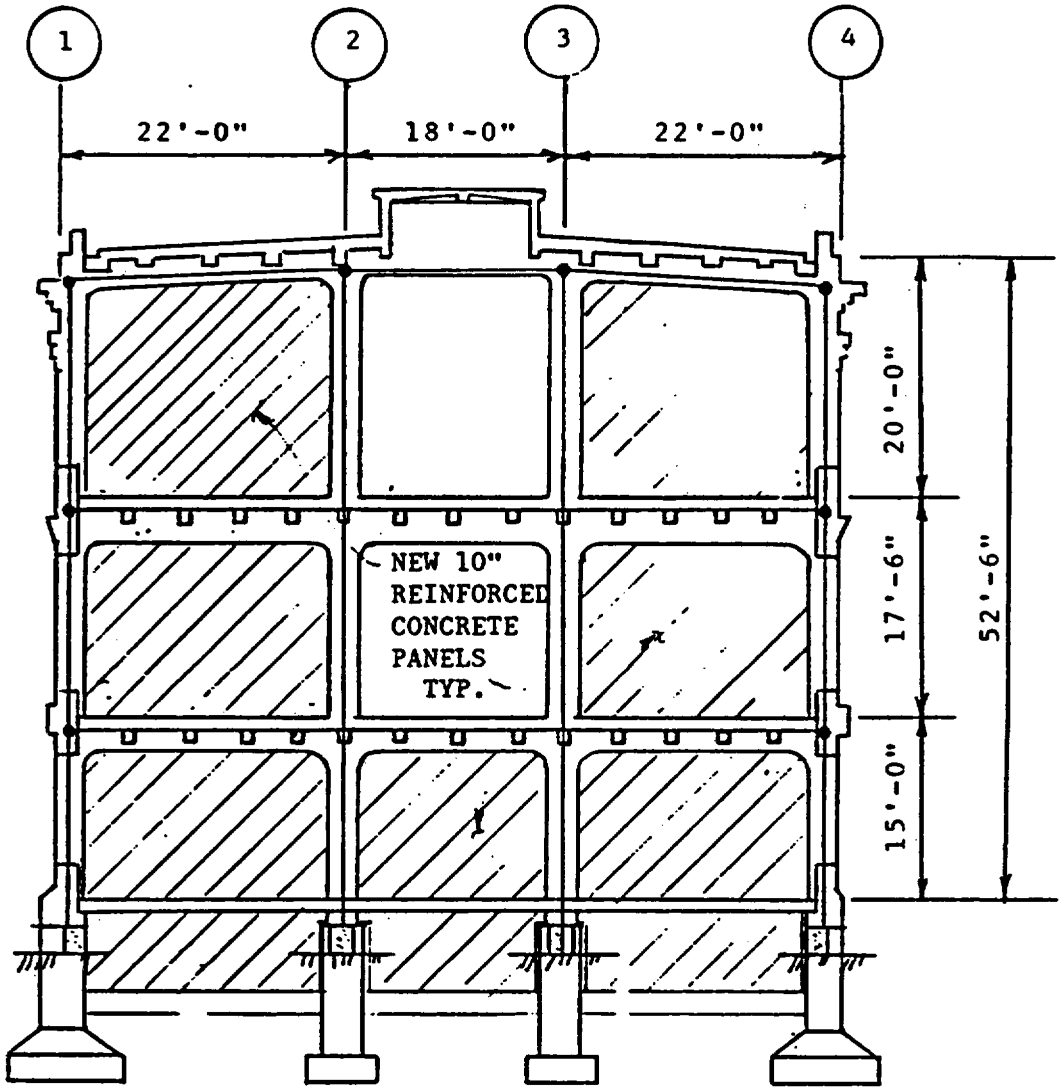
Figure F-2. Brick building with concrete framing system (Sheet 15 of 25)



LINES A, B, D, E, F, G & J

Sheet 16 of 25

Figure F-2. Brick building with concrete framing system (Sheet 16 of 25)



LINES C & H

Sheet 17 of 25

Figure F-2. Brick building with concrete framing system (Sheet 17 of 25)

Detailed Structural Analysis to Confirm Concept. A detailed structural analysis was not necessary for the existing structure because of the negative results from the preliminary evaluation. However, a detailed analysis is now required to determine if the recommended concept will satisfy the acceptance criteria outlined on sheet 5. A modal analysis of the modified structure was made with the aid of a general computer program for static and dynamic analyses of frame and shear wall three-dimensional buildings for both the transverse and longitudinal directions. The program assumes rigid diaphragms and the roof and floor diaphragms of this modified structure essentially met this assumption. Sheets 20 and 21 indicate the SRSS of the dynamic modal responses from the computer output. Sheet 22 indicates the evaluation of the SRSS response of some representative structural elements and sheets 23 and 24 contain stress checks of selected elements for compliance with the criteria.

Torsion Forces. Due to the symmetry of the structure lateral load resisting system there is no "calculated torsion." The "accidental torsion" is the story force times the nominal eccentricity of 5 percent of the maximum building dimension. The forces due to torsion were calculated by applying a torsional moment in each story equal to the seismic (SRSS) story shear times the "accidental" eccentricity (0.05 x 186 feet). The resulting member responses from this analysis were added to the translational member responses (SRSS) of the dynamic analyses.

Foundation Ties. The BDM (para. 4-8a) requires that pile, caisson, and deep pier footings in seismic zones 2, 3, and 4 be interconnected by ties. In this building, the existing foundation ties are near the top of the large piers (see sheet 4 of 24) and provide questionable restraint to the timber piles. The seismic upgrading modification provides a good tie, in the plane of the new walls, for the piles on lines C and H. The significant cost and disruption of the existing building required to install new tie beams throughout the building may not be justified if it can be demonstrated that the seismic forces from EQ-II can be transmitted to the ground with the existing tie system or by passive soil pressure on the existing piers.

Sheet 18 of 25

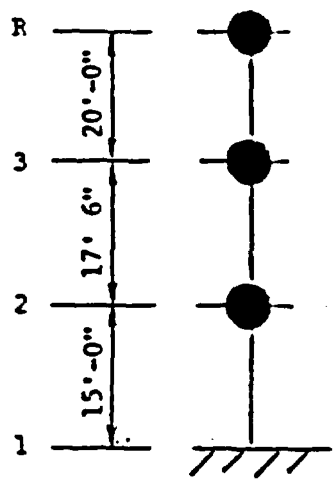
Figure F-2. Brick building with concrete framing system (Sheet 18 of 25)

Results of the Confirmation Analysis. The modified structure meets all the acceptance criteria requirements for EQ-II forces except for possibly the capacity of the timber piles to support the additional loads from the new concrete walls. The capacities of the timber piles to meet the requirements for the new dead load plus live load loading criteria will need to be re-evaluated. As a result of the detailed analysis it was determined that the unreinforced brick masonry walls that were not being reinforced with gunite were deficient for seismic forces normal to the walls. These walls will either be anchored to the new concrete walls or will be provided with new vertical concrete or steel mullions between existing concrete columns for additional lateral support to meet the EQ-II acceptance criteria. Shear and flexural stresses for seismic forces parallel to the walls were found to be within the Acceptance Criteria after strengthening. An alternative modification concept was studied that provided for the anchoring of all exterior unreinforced brick masonry end walls to new reinforced concrete gunite walls placed against their interior faces in lieu of constructing the new concrete walls on Lines C and H and the additional vertical concrete mullions. This concept was rejected because it resulted in unacceptable shears in the floor and roof diaphragms and excessive overturning forces for the end walls in the transverse direction. The recommended concept could have been implemented for the entire length of the longitudinal walls thus eliminating the vertical mullions, but it is more cost effective to provide new gunite walls as required for shear resistance and new concrete mullions in the remaining portions of the existing longitudinal walls.

Sheet 19 of 25

Figure F-2. Brick building with concrete framing system (Sheet 19 of 25)

MATHEMATICAL MODEL - TRANSVERSE DIRECTION



M_R = 75.36 Kip-Sec^2/Ft.

M_3 = 114.64 Kip-Sec^2/Ft.

M_2 = 119.88 Kip-Sec^2/Ft.

T_1 = 0.208 Sec., T_2 = 0.076 Sec., T_3 = 0.045 Sec.

EQ II - STRUCTURAL RESPONSE, TRANSVERSE DIRECTION - SRSS

LEVEL No.	STORY LOAD** F_x - Kips	STORY SHEAR V - Kips	DISPLACEMENT δ - Feet	STORY DRIFT** Δ_x - Feet
3	875	875	0.012	0.006 0.120*
2	809	1449	0.005	0.004 0.105*
1	565	1733***	0.002	0.002 0.090*

* MAXIMUM ALLOWABLE EQ II STORY DRIFT = 0.006H

** $F_x = \left[\Sigma(F_{xm})^2\right]^{1/2}$

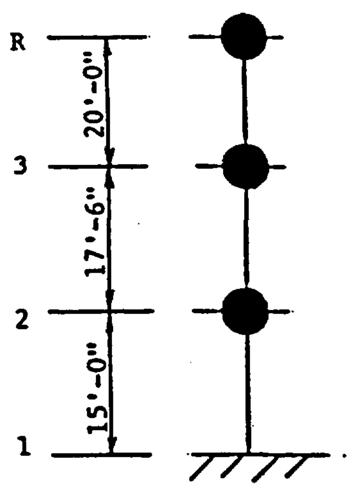
$\Delta_x = \left[\Sigma(\Delta_{xm})^2\right]^{1/2}$

*** $C_B = V \div \Sigma W = 0.174$

Sheet 20 of 25

Figure F-2. Brick building with concrete framing system (Sheet 20 of 25)

MATHEMATICAL MODEL - LONGITUDINAL DIRECTION



M_R = 75.36 Kip-Sec^2/Ft.

M_3 = 114.84 Kip-Sec^2/Ft.

M_2 = 119.88 Kip-Sec^2/Ft.

T_1 = 0.222 Sec., T_2 = 0.068 Sec., T_3 = 0.040 Sec.

EQ II - STRUCTURAL RESPONSE, LONG. DIRECTION - SRSS

LEVEL No.	STORY LOAD** F_x - Kips	STORY SHEAR V - Kips	DISPLACEMENT δ - Feet	STORY DRIFT** Δ_x - Feet
3	873	873	0.014	0.006 0.120*
2	804	1524	0.008	0.004 0.105*
1	577	1859***	0.003	0.003 0.090*

* MAXIMUM ALLOWABLE EQ II STORY DRIFT = 0.006H

** $F_x = \left[\Sigma(F_{xm})^2\right]^{\frac{1}{2}}$

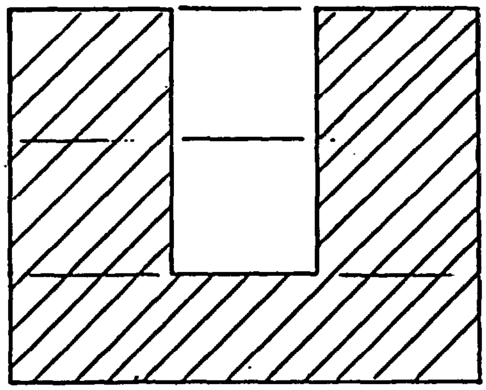
$\Delta_x = \left[\Sigma(\Delta_{xm})^2\right]^{\frac{1}{2}}$

*** $C_B = V \div \Sigma W = 0.186$

Sheet 21 of 25

Figure F-2. Brick building with concrete framing system (Sheet 21 of 25)

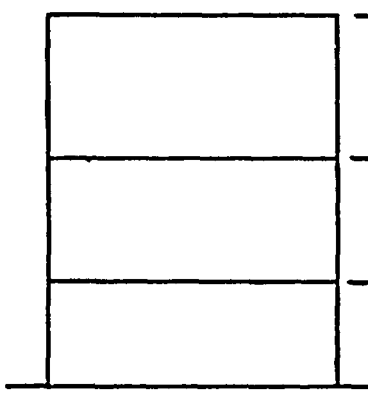
EQ II SRSS ELEMENT FORCES - TRANSVERSE DIRECTION



V (Kips)	M (Ft-Kips)
422	7880
690	19,680
843	31,572

WALLS C & H

EQ II SRSS ELEMENT FORCES - LONGITUDINAL DIRECTION



V (Kips)	M (Ft-Kips)
218	4365
381	10,812
465	17,422

WALLS ON LINES 1 & 4
(2 EACH LINE)

Sheet 22 of 25

Figure F-2. Brick building with concrete framing system (Sheet 22 of 25)

"ACCIDENTAL" TORSION FORCES

The "accidental" torsion is the story shear, V_x, times the nominal eccentricity of 5% of the maximum building dimension:

$$M_t = V_x \times 0.05 \times 186' = 9.3\ V_x$$

The story relative rigidity (K) of each wall line is obtained from the computer analysis.

$$\text{Torsion Shear} = \frac{Kd}{\Sigma Kd^2} \times 9.3\ V_x$$

Distribution of Forces (Frames neglected)

Roof Level

WALL LINE	REL. K	d	Kd	Kd^2	DIRECT SHEAR	TORSIONAL SHEAR
C	0.94	52	48.9	2542	$0.50V_T$	$0.067V_T$
H	0.94	52	48.9	2542	$0.50V_T$	$0.067V_T$
1	1.00	29	29.0	841	$0.50V_L$	$0.040V_L$
4	1.00	29	29.0	841	$0.50V_L$	$0.040V_L$
			$\Sigma =$	6766		

3rd Floor Level

WALL LINE	REL K	d	Kd	Kd^2	DIRECT SHEAR	TORSIONAL SHEAR
C	1.06	52	55.1	2866	$0.50V_T$	$0.069V_T$
H	1.06	52	55.1	2866	$0.50V_T$	$0.069V_T$
1	1.00	29	29.0	841	$0.50V_L$	$0.036V_L$
4	1.00	29	29.0	841	$0.50V_L$	$0.036V_L$
			$\Sigma =$	7414		

2nd Floor Level

WALL LINE	REL K	d	Kd	Kd^2	DIRECT SHEAR	TORSIONAL SHEAR
C	1.44	52	74.9	3894	$0.50V_T$	$0.074V_T$
H	1.44	52	74.9	3894	$0.50V_T$	$0.074V_T$
1	1.00	29	29.0	841	$0.50V_L$	$0.028V_L$
4	1.00	29	29.0	841	$0.50V_L$	$0.028V_L$
			$\Sigma =$	9470		

Sheet 23 of 25

Figure F-2. Brick building with concrete framing system (Sheet 23 of 25)

ELEMENT STRESS CHECK

Wall on Lines C & H at 2nd floor level

Moment

M_D:

EQ II Forces		19,680 ft-kips
Accidental Torsion:	19,680 x 0.069/0.50 =	2,716 " "
		22,396 ft-kips

M_u: 12,503 ft-kips

IDR: $22,396 \div 12,503 = 1.79 < 2.00$

Shear

V_D:

EQ II Forces:		690 kips
Accidental Torsion:	690 x 0.069/0.50 =	95 "
		785 kips

V_u: 1243 kips

IDR: $785 \div 1243 = 0.63 < 1.25$

Wall on Lines 1 & 4 at first floor level

Moment

M_D:

EQ II Forces		17,422 ft-kips
Accidental Torsion:	17,422 x 0.074/0.50 =	2,579 " "
		20,001 ft-kips

M_u: 10,088 ft-kips

IDR: $20,001 \div 10,088 = 1.98 < 2.00$

Shear

V_D:

EQ II Forces		465 kips
Accidental Torsion:	465 x 0.074/0.50 =	69 kips
		534 kips

V_u: 802 kips

IDR: $584 \div 802 = 0.67 < 1.25$

Roof Diaphragm

Moment

$M_D = (875 \text{ kips} \div 186 \text{ ft}) \times 41^2 \div 2 = 3950$ ft-kips

$M_u = 4348$ ft-kips

$IDR = 3950 \div 4348 = 0.91 < 1.50$

Shear

$V_D = (875 \text{ kips} \div 186 \text{ ft}) \times 104 \div 2 = 245$ kips

$V_u = 292$ kips

$IDR = 245 \div 292 = 0.84 < 1.10$

Sheet 24 of 25

Figure F-2. Brick building with concrete framing system (Sheet 24 of 25)

ELEMENT STRESS CHECK

Unreinforced Brick Masonry Wall (12" Min. Thick, $S_a = 0.36$; Max Span = 7')

Moment

M_D: (120 psf x 0.36) x $7^2 \div 8$ = 263 ft-lbs/ft

M_u: (1.6 x 7.5 psi) x 288 $in^3/12$ = 288 ft-lbs/ft

IDR: $263 \div 288 = 0.91 < 1.00$

Shear

V_D: (120 psf x 0.36) x 1.15 x 7' $\div$ 2 = 173 lbs/ft

V_u: (1.6 x 7.5 psi) x 144 in^2 = 1,728 lbs/ft

IDR: 173 1728 = $0.10 < 1.00$

Sheet 25 of 25

Figure F-2. Brick building with concrete framing system (Sheet 25 of 25)

DESIGN EXAMPLE F-3

BUILDING WITH DUCTILE STEEL MOMENT FRAMES AND STEEL BRACED FRAMES

Description of Structure. A 3-story hospital building with transverse ductile moment-resisting frames and longitudinal braced frames in structural steel. The building is the same as described in BDM design example A-3 and SDG example E-3 and shown on sheets 2 and 3.

Construction Outline.

Roof:
- Built-up 5 ply.
- Metal decking with insulation board.
- Suspended ceiling.

2nd & 3rd Floors:
- Metal decking with concrete fill.
- Asphalt tile.
- Suspended ceiling.

1st Floor:
- Concrete slab-on-grade.

Exterior Walls:
- Non-bearing, non shear, insulated metal panels.

Partitions:
- Non-structural removable drywall.

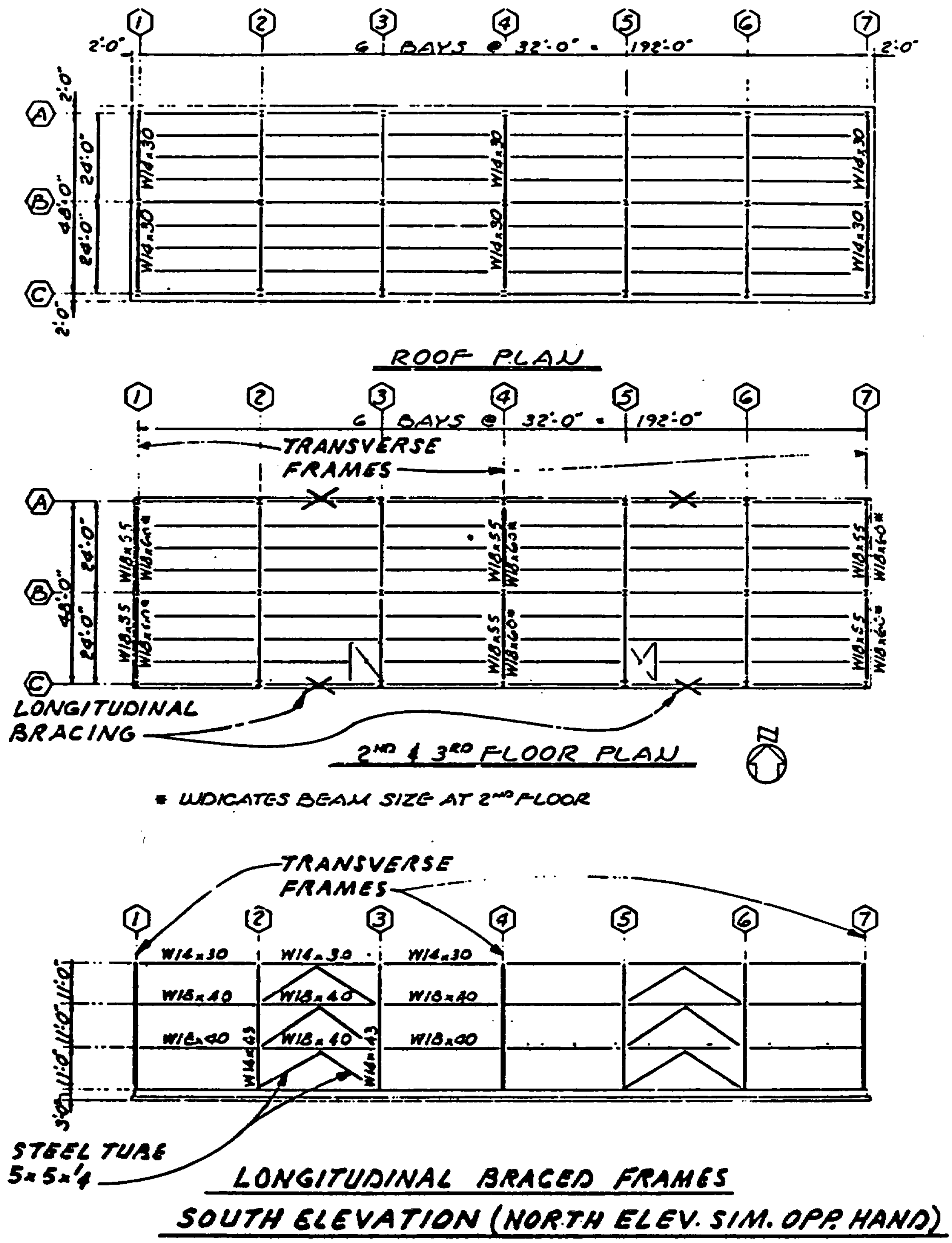
Original Design. The structure was designed in accordance with the February 1982 edition of the BDM. It is an essential building (I=1.5) in Seismic Zone 3 (Z=3/4) as designated in the SDG, Figure E-3, Sheet 1. A summary of the seismic design force coefficients follow.

	Transverse	Longitudinal
Seismic Zone 3	Z = 3/4	Z = 3/4
Hospital building	I = 1.5	I = 1.5
Ductile frame/braced frame	K = 0.67	K = 1.0
Soil period	T_s = 1.0 sec	T_s = 1.0 sec
Building period	T = 0.69 sec	T = 0.3 sec
	CS = 0.116	CS = 0.140
	ZIKCS = 0.087	ZIKCS = 0.157

Background. The building is classified as an essential building and was designed in accordance with the current BDM. Thus, it was decided that the preliminary screening and the preliminary evaluation would not be necessary because the building would be placed on a high priority listing for a detailed structural analysis. The detailed structural analysis for this building, which is covered in Figure E-3 of the SDG, is summarized on sheets 7 and 8.

Sheet 1 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 1 of 23)



Sheet 2 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 2 of 23)

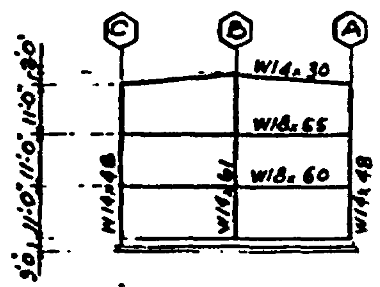
LOADS.

ROOF:

6-PLY ROOFING	=	6.0 P.S.F.
1" INSULATION	=	1.5
STEEL DECK	=	2.3
STEEL PURLINS	=	3.7
STEEL GIRDERS	=	1.2
CEILING	=	10.0
MISCELLANEOUS	=	1.0
DEAD LOAD	=	25.7 P.S.F
ADD FOR SEISMIC:		
PARTITIONS	=	10.0
TOTAL FOR SEISMIC	=	35.7 P.S.F.

2ND & 3RD FLOORS:

FINISH	=	1.0 P.S.F.
STEEL DECK	=	3.1
CONCRETE FILL	=	32.0
STEEL BEAMS	=	5.9
STEEL GIRDERS & COLUMNS	=	1.5
PARTITIONS	=	20.0
CEILING	=	10.0
MISCELLANEOUS	=	1.0
DEAD LOAD	=	74.5 P.S.F.
LIVE LOAD	=	50.0 P.S.F.



LINES 1, 4, & 7

TRANSVERSE DUCTILE MOMENT RESISTING FRAMES

Sheet 3 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 3 of 23)

Acceptance Criteria (para 5-2). The acceptance criteria for existing buildings are those presented for the post-yield analysis for EQ-II in the SDG with latitude allowed under certain conditions.

Conforming Systems and Materials. The systems and materials are in compliance with the requirements of the BDM; therefore, the EQ-II response spectra may be reduced by as much as 15 percent (para 5-2a).

Method 1. Elastic Analysis Procedure (Refer to SDG paras 4-4c and 5-5a).

Classification: Essential Building

Loading Combination: DL + 0.25LL + EQ

Plastic Member Capacities: 1.7 AISC allowable stresses, typ. (1.4 for shear stresses)

Inelastic Demand Ratios:	DMRSF beams	2.00
	" columns	1.25
	Braced Steel Frame beams	1.50
	" " " columns	1.25
	" " " dia. br.	1.25
	" " " K-braces	1.00
	" " " connector	1.00
	Metal Deck Diaphragm	1.25

Story Drift Limitation: 0.010 x Story Height

Method 2. Capacity Spectrum Method (Refer to SDG paras 4-4d and 5-5b). If the acceptance criteria of Method 1 are not satisfied, the structure will be analyzed in accordance with Method 2 prior to developing a seismic upgrading concept.

Sheet 4 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 4 of 23)

Site Response Spectra. The response spectra for the site, which are used for both the detailed structural analysis and the upgrade concept, were developed in accordance with procedures in Chapter 3 of the SDG and are the same as those used for SDG Figure E-3.

Building Classification: Essential Facility.

Ground Motion Spectra: ATC 3-06 Map Contour Level, $A_a = A_v = 0.30$

Soil Classification: $S_i = 1.2$ (Type S2)

Earthquake I

Damping = 3%, D.F. = 1.17 (SDG table 3-7)

$A_a = A_v = 0.14$ (Design Ground Motion, SDG table 3-4)

$S_a = \text{D.F.}\ (1.22 A_v S_i)/T = 0.24g/T$ less than $\text{D.F.}\ (2.5)A_a = 0.41g$ max

Earthquake II

Damping = 7%, D.F. = 0.9

$A_a = A_v = 0.35$

$S_a = \text{D.F.}\ (1.22 A_v S_i)/T = 0.46g/T$ less than $\text{D.F.}\ (2.5)A_a = 0.79g$ max

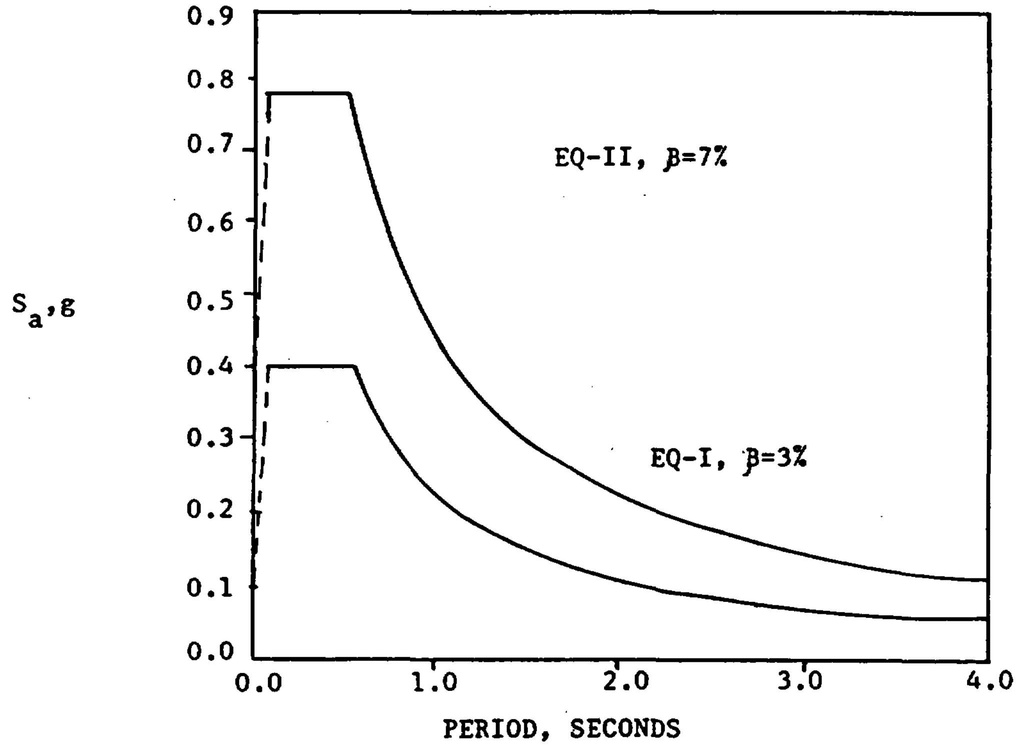
EQ-II/EQ-I = 0.79/0.41 = 1.93

The resulting spectra are shown on sheet 6.

Sheet 5 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 5 of 23)

DESIGN RESPONCE SPECTRA FOR EQ-I AND EQ-II



EQ	β		PERIOD							
			0.0	.568	.80	1.0	1.5	2.0	3.0	4.0
I	3%	S_a,g	.14	.41	.300	.240	.160	.120	.080	.060
II	7%	S_a,g	.35	.79	.576	.461	.307	.231	.154	.115
		S ,in	0	2.66	3.61	4.51	6.76	9.04	13.57	18.01

* SPECTRAL DISPLACEMENT $S_d = S_a(T/2\pi)^2 g$

Sheet 6 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 6 of 23)

Detailed Structural Analysis. The structural evaluation of the existing structure was covered by the EQ-II analysis of SDG Figure E-3. The results are summarized as follows:

Transverse Direction (Moment Frames):

Method 1. The inelastic demand ratios for the first floor columns were over 2.0. This is greater than the 1.25 that is allowed. Even with the 15 percent reduction in demand allowed for existing buildings, the allowable limits are exceeded. Drift limits were also exceeded by more than 15 percent.

Method 2. The capacity spectrum method was also used in the evaluation. The results are shown on sheet 8 for the full value of the EQ-II spectrum. Even with the 15 percent reduction in the spectrum the structure does not satisfy the acceptance criteria.

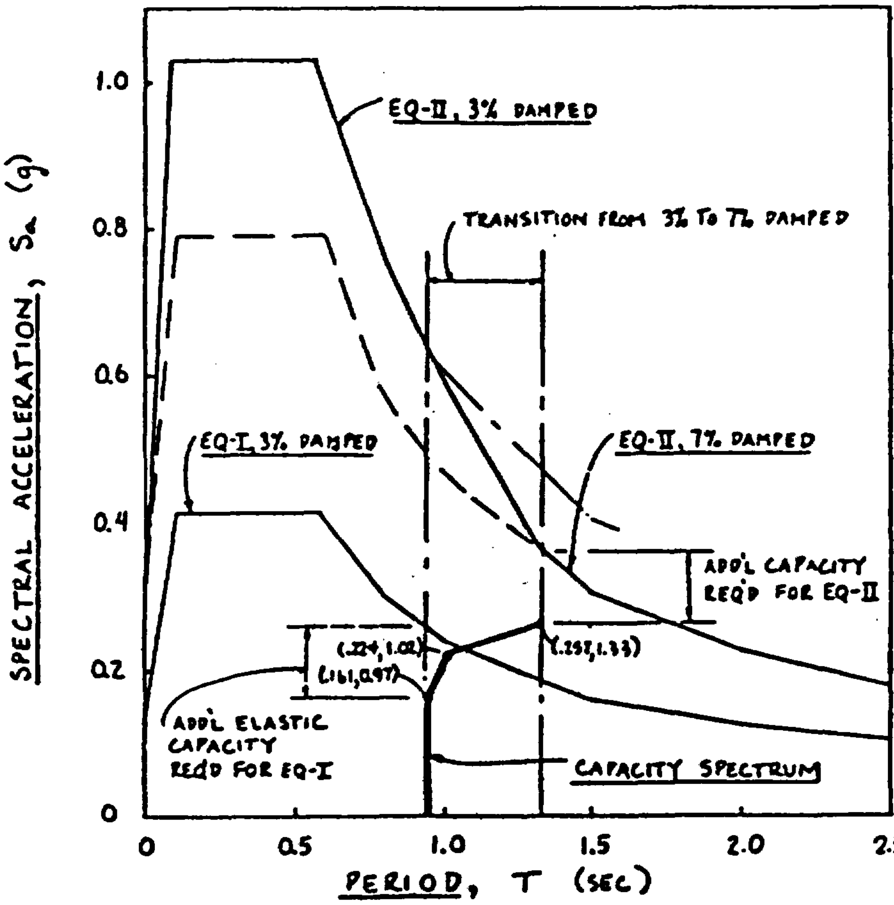
Longitudinal Direction (Braced Frames).

Method 1. The diagonal members in the chevron (K-braced) braced frames were overstressed by almost a factor of 3 for EQ-II; thus it will not satisfy the acceptance criteria.

Conclusions. The conclusion of the detailed analysis is that neither the moment frames in the transverse direction nor the braced frames in the longitudinal direction have the capacity to resist EQ-II within the acceptance criteria. It is therefore recommended that seismic upgrade concepts be developed.

Sheet 7 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 7 of 23)



REFER TO SHEET 20 FOR CAPACITY SPECTRUM VALUES
SHEET 4 FOR RESPONSE SPECTRA

CAPACITY SPECTRUM METHOD

Sheet 8 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 8 of 23)

Development of Seismic Upgrade.

Structural Upgrading Concepts. Three alternative concepts were considered for the transverse direction. For the longitudinal direction only one concept was considered.

Transverse Direction:
Concept 1. Modification of the existing four non moment-resisting frames to ductile moment-resisting frames to match the strength of the existing three ductile moment-resisting frames. This would require the strengthening of the members of the frames as well as the beam-column joint with doubler web plates and horizontal stiffener plates. This work would be difficult to accomplish especially when the facility is in operation.
Concept 2. A transverse lateral load resisting system consisting of the existing three ductile moment-resisting frames modified with eccentric steel bracing. In this concept the roof metal deck horizontal diaphragm would be overstressed for EQ-II forces and would require strengthening by adding a concrete fill to the metal deck. This would also require strengthening of the existing roof purlins to support this additional load as well as removal of the existing roofing and re-roofing. Additionally the column anchor bolts would need strengthening to resist the higher overturning forces.
Concept 3. A transverse lateral load resisting system consisting of the existing three ductile moment-resisting frames and the existing four non moment-resisting frames modified with eccentric steel bracing.

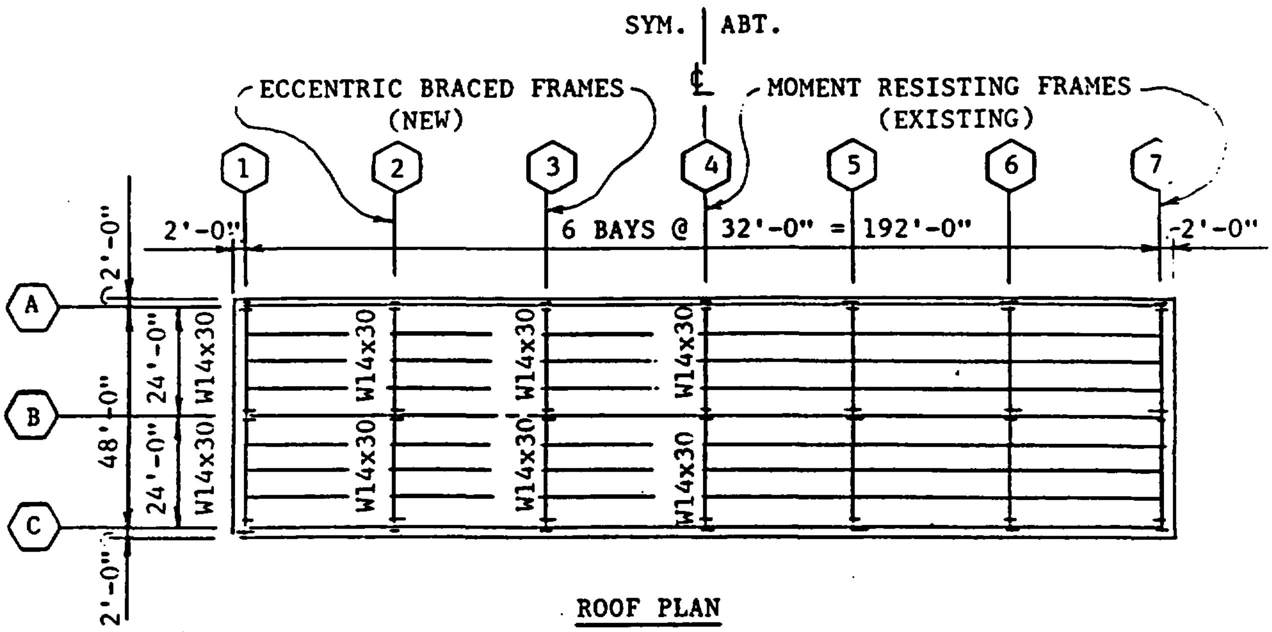
Longitudinal Direction: For the upgrading in the longitudinal direction, the number of braced bays are increased to meet the acceptance criteria for EQ-II forces. The modification indicated in Design Example E-3 of the SDG (sheet 34 of 34) proposed increasing the size and changing the configuration of the existing braced bays. This concept resulted in unacceptable uplift loads on the columns and was discarded in favor of increasing the number of braced bays.

Recommended Concept.

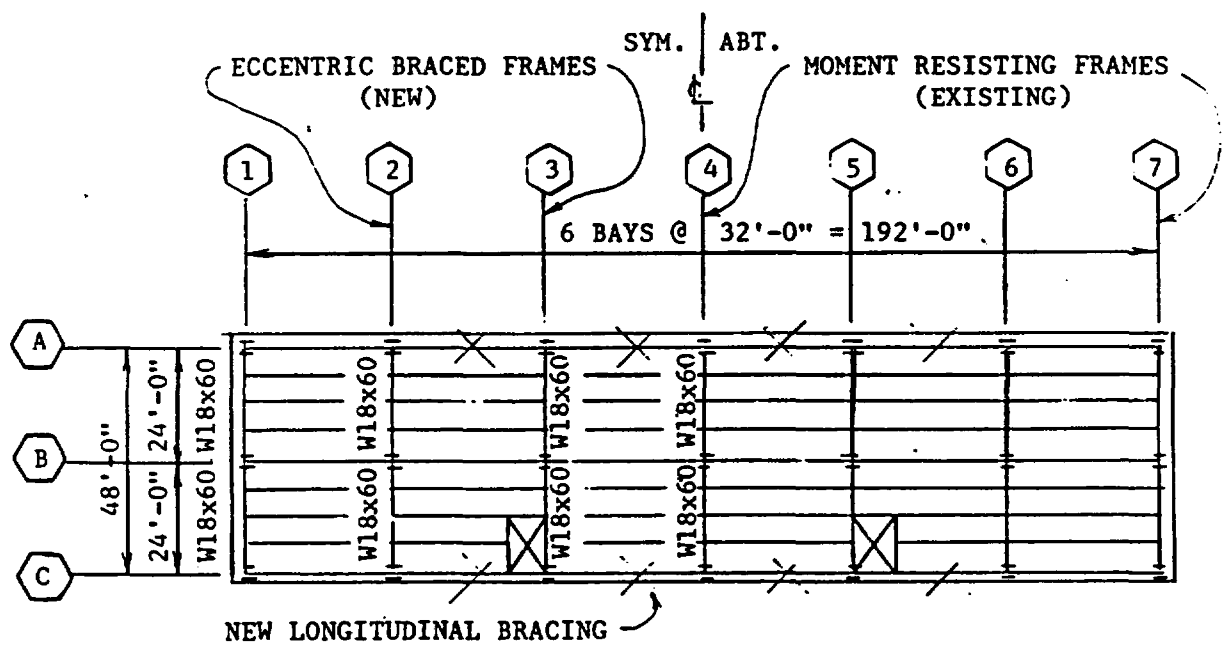
Transverse Direction. Concept 3 is the recommended transverse lateral load resisting system as it is considered to be the least expensive to construct and causes the least interference to the operation of the facility. Sheets 10 and 11 indicate the structural modifications of this concept and the basis for this design example.

Sheet 9 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 9 of 23)



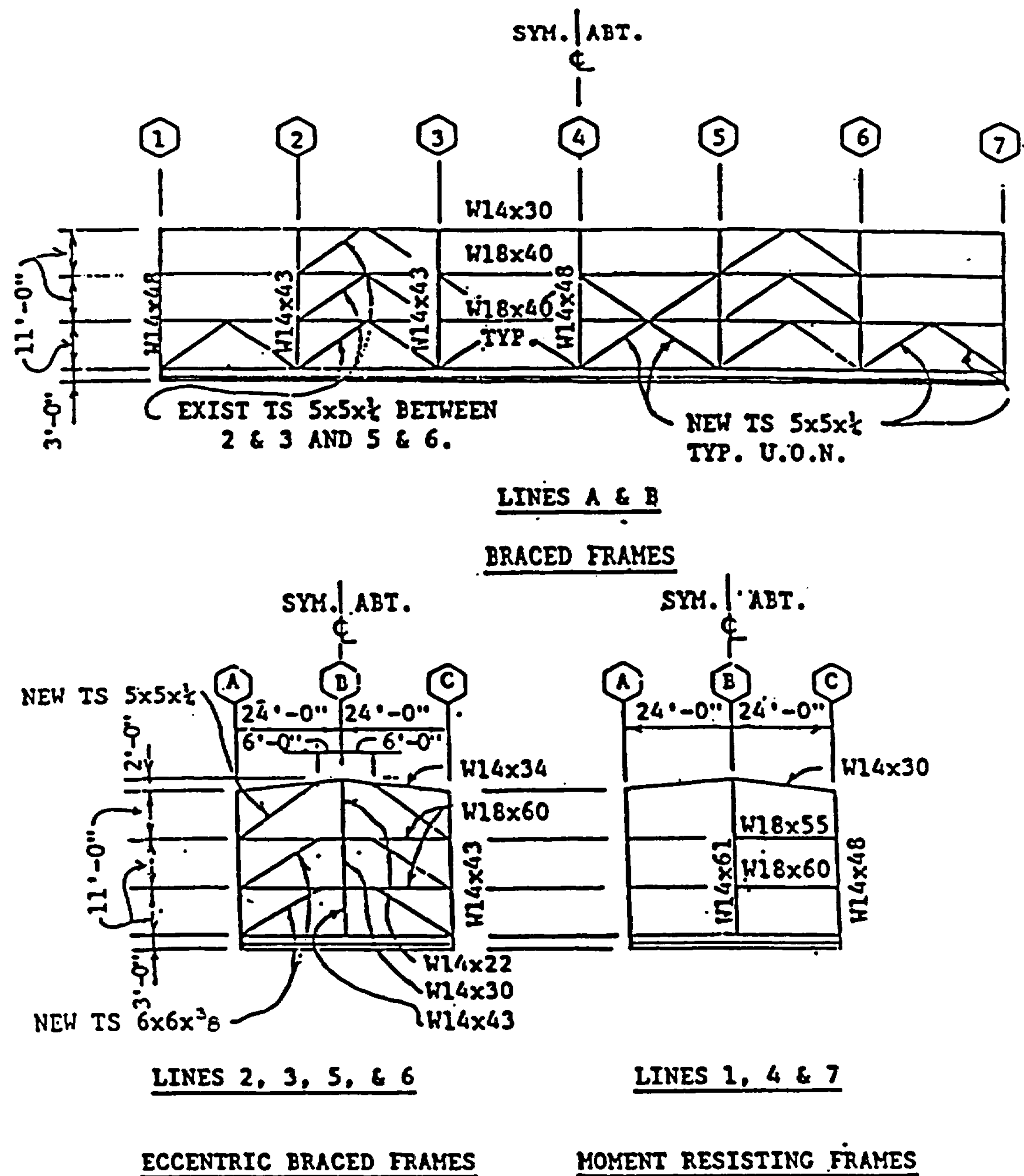
ROOF PLAN



2nd FLOOR PLAN (3rd FLOOR PLAN SIMILAR)

Sheet 10 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 10 of 23)



Sheet 11 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 11 of 23)

Longitudinal Direction. Two additional bays of bracing (column lines 3 to 5) on each exterior wall (column lines A and C) are recommended for upgrading the 2nd story in the longitudinal direction, and 4 additional bays are recommended for the 1st story. An elevation showing the new brace layout is shown on sheet 10.

Confirmation Analysis. The modal analysis to confirm compliance of the modified building was made with the aid of a general computer program for static and dynamic analyses of frame and shear wall three-dimensional buildings (TABS 4.0) for both transverse and longitudinal directions. The SRSS of the dynamic modal responses are indicated on sheets 14 and 15.

Flexibility of the Roof Diaphragm. The computer program used for the dynamic analyses assumes rigid diaphragms. This assumption is essentially valid for the existing structure except for the roof diaphragm in the transverse analysis. The horizontal roof load distribution to the moment resisting and eccentric braced frames of the transverse dynamic analysis was reviewed based on the actual flexibility of the metal roof diaphragm and the rigidities of the lateral resisting frames using a static computer program. The results from this analysis indicate that at the roof level the exterior moment resisting frames on lines 1 and 7 resist approximately 10 percent less load and the eccentric braced frames on lines 3 and 5 resist approximately 5 percent more load than that from the rigid diaphragm dynamic analysis.

Torsional Forces. Due to the symmetry of the structure lateral load resisting system there is no "calculated torsion." The "accidental torsion" is the story force times the nominal eccentricity of 5 percent of the maximum building dimension. The torsional forces for each story are distributed to the lateral force resisting frames in accordance with the method illustrated in the BDM Example A-3 and added to the forces from the dynamic analysis.

Structural Member Responses:

EQ-II Seismic Forces. Sheets 16 and 18 indicate the SRSS member responses from the dynamic computer analyses for EQ-II. For Frame 4 (moment frame) and Frame 3 (eccentric braced frame) the responses are given for the rigid diaphragm analysis and also the adjustment to the response for torsion. The torsional responses were obtained by applying a torsional moment, at each story, equal to the story mass times 5 percent of the maximum building dimension (i.e., 196 feet). The effect of the flexible roof diaphragm was ignored. (Ignoring the 10 percent decrease in Frames 1 and 7 is conservative. Ignoring the 5 percent increase in the eccentric braced frames on Frames 3 and 5 may be slightly unconservative, but this effect occurs only at the roof level and these frames have the reserve capacity for this additional load.)

Sheet 12 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 12 of 23)

Dead Load and Live Load Forces. The dead and live load member responses are indicated on sheets 16 and 18. Note that for the eccentric braced frames, the bracing does not participate in resistance to dead loads (assumed to be resisted by the existing framing before the braces are installed), but does participate in resisting the live loads subsequent to the installation of the braces.

Combined Responses. Sheets 16 and 18 indicate the combined (DL + 0.25LL + EQ-II) responses for Frames 3 and 4. The results of the calculations of the demand versus the allowable inelastic demand ratio (IDR) for representative structural members are shown on sheets 17 and 19.

Roof and Floor Diaphragms. The addition of the four eccentric braced frames in the transverse direction greatly reduced the diaphragms shears to well within the Acceptance Criteria requirements.

Overturning Forces. A check of the overturning forces due to EQ-II resulted in no uplift on any of the columns of the lateral load resisting frames. Soil pressures on the column footings due DL + 0.25LL + EQ-II were less than 1.5 times the soil pressures for the dead load plus live loads.

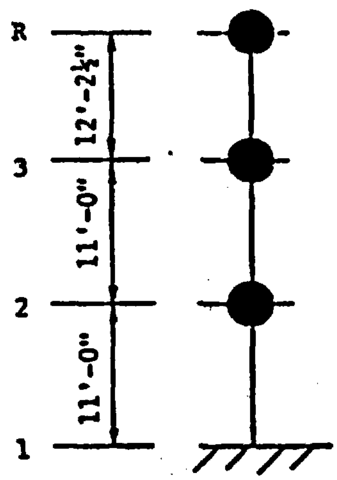
Sliding Resistance. Resistance to sliding forces of EQ-II are developed through friction on the foundations and the passive soil pressures on the footings and the foundation grade beams perpendicular to the lateral forces.

Conclusions. The modified structure meets all the acceptance criteria requirements for EQ-II forces. The member element stresses were well within the acceptance criteria requirements. The primary reason for the placement of the K-braces on lines A and C in every bay was to prevent uplift on the columns of all the lateral load resisting frames.

Sheet 13 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 13 of 23)

MATHEMATICAL MODEL - TRANSVERSE DIRECTION



M_R = 11.64 Kip-Sec^2/Ft.

M_3 = 21.96 Kip-Sec^2/Ft.

M_2 = 21.96 Kip-Sec^2/Ft.

T_1 = 0.365 Sec., T_2 = 0.174 Sec., T_3 = 0.100 Sec.

TOTAL BUILDING: 3 MOMENT FRAMES PLUS 4 ECCENTRIC BRACED FRAMES

EQ II STRUCTURAL RESPONSE - TRANSVERSE DIRECTION - SRSS

LEVEL No.	STORY LOAD** F_x - Kips	STORY SHEAR V - Kips	DISPLACEMENT δ - Feet	STORY DRIFT** Δ_x - Feet
3	456	456	0.125	0.051 0.122*
2	563	931	0.077	0.040 0.110*
1	331	1186***	0.038	0.038 0.110*

* MAXIMUM ALLOWABLE EQ II STORY DRIFT = 0.010H

** $F_x = \left[\Sigma(F_{xm})^2\right]^{\frac{1}{2}}$

$\Delta_x = \left[\Sigma(\Delta_{xm})^2\right]^{\frac{1}{2}}$

*** $C_B = V \div \Sigma W = 0.663$

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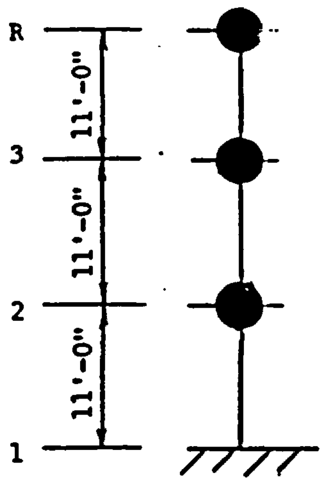
Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 14 of 23)

MATHEMATICAL MODEL - LONGITUDINAL DIRECTION, FRAME A



M_R = 5.82 Kip-Sec^2/Ft.

M_3 = 10.98 Kip-Sec^2/Ft.

M_2 = 10.98 Kip-Sec^2/Ft.

T_1 = 0.190 Sec., T_2 = 0.089 Sec., T_3 = 0.058 Sec.

FRAME A REPRESENTS ½ OF THE BUILDING

EQ II - STRUCTURAL RESPONSE, LONG. DIRECTION, FRAME A - SRSS

LEVEL No.	STORY LOAD** F_x - Kips	STORY SHEAR V - Kips	DISPLACEMENT δ - Feet	STORY DRIFT** Δ_x - Feet
3	223	223	0.033	0.012 0.110*
2	264	469	0.021	0.012 0.110*
1	163	588***	0.010	0.010 0.110*

* MAXIMUM ALLOWABLE EQ II STORY DRIFT = 0.010H

** $F_x = \left[\Sigma(F_{xm})^2\right]^{½}$

$\Delta_x = \left[\Sigma(\Delta_{xm})^2\right]^{½}$

*** $C_B = V_B \div \Sigma W = 0.657$

Sheet 15 of 23

Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 15 of 23)

"ACCIDENTAL" TORSION FORCES

Since the roof diaphragm is relatively flexible, the accidental torsional forces are applied only to the 2nd & 3rd floor levels. Due to the symmetry there is no "calculated" torsion. The accidental torsion is the story shear, V_x, times the nominal eccentricity of 5% of the maximum building dimension.

$$M_t = V_x \times 0.05 \times 196' = 9.8\ V_x$$

The story relative rigidity (K) of each frame line is obtained from the computer analysis.

$$\text{Direct Shear} = \frac{K}{\Sigma K} \times V_x$$

$$\text{Torsional Shear} = \frac{Kd}{\Sigma Kd^2} \times 9.8\ V_x$$

3rd Floor Level

FRAME LINE	REL K	d	Kd	Kd2	DIRECT SHEAR	TORSIONAL SHEAR
1	1.0	96	96	9216	$0.045V_T$	$0.009V_T$
2	4.75	64	304	19456	$0.216V_T$	$0.027V_T$
3	4.75	32	152	4864	$0.216V_T$	$0.014V_T$
4	1.0	0	0	0	$0.045V_T$	$0V_T$
5	4.75	32	152	4864	$0.216V_T$	$0.014V_T$
6	4.75	64	304	19456	$0.216V_T$	$0.027V_T$
7	1.0	96	96	9,216	$0.045V_T$	$0.009V_T$
	Σ 22.00					
A	36.43	24	874	20,984	$0.50V_L$	$0.079V_L$
C	36.43	24	874	20,984	$0.50V_L$	$0.079V_L$
				Σ 109,016		

2nd Floor Level

FRAME LINE	REL K	d	Kd	Kd2	DIRECT SHEAR	TORSIONAL SHEAR
1	1.00	96	96	9,216	$0.074V_T$	$0.012V_T$
2	2.61	64	167	10,691	$0.194V_T$	$0.022V_T$
3	2.61	32	84	2,673	$0.194V_T$	$0.011V_T$
4	1.00	0	0	0	$0.074V_T$	$0V_T$
5	2.61	32	84	2,673	$0.194V_T$	$0.011V_T$
6	2.61	64	167	10,691	$0.194V_T$	$0.022V_T$
7	1.00	96	96	9,216	$0.074V_T$	$0.012V_T$
	Σ 13.44					
A	26.66	24	640	15,356	$0.50V_L$	$0.083V_L$
C	26.66	24	640	15,356	$0.50V_L$	$0.083V_L$
				Σ 75,872		

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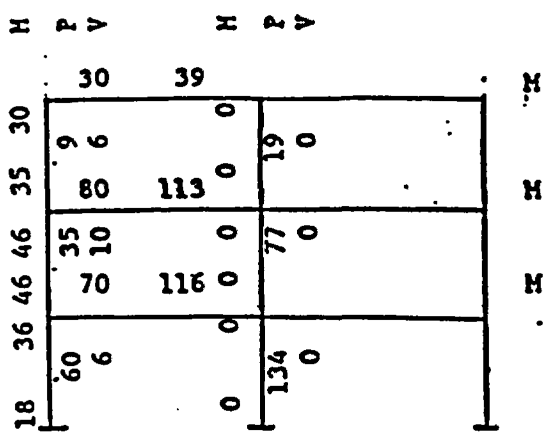
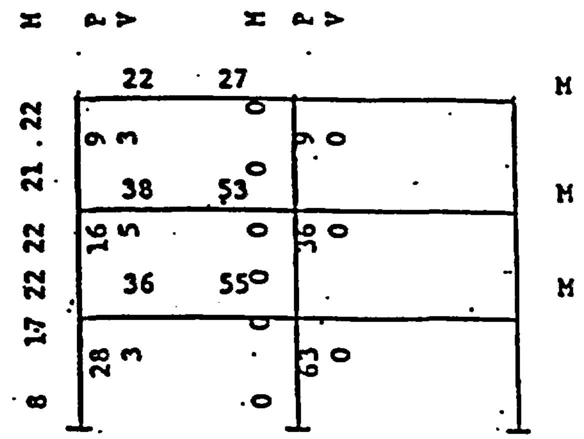
Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 16 of 23)

EQ-II ELEMENT FORCES : TRANSVERSE (N-S) DIRECTION - FRAME 4

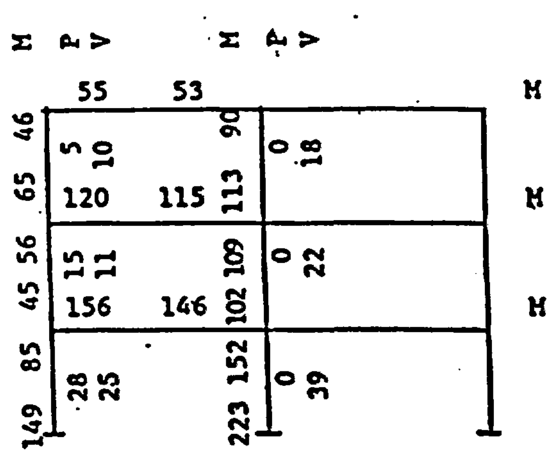
DL & LL RESULTS FROM DESIGN MANUAL EXAMPLE A-3, SHEET 18 OF 34.
SEISMIC RESULTS FROM COMPUTER ANALYSIS.
ALL END MOMENTS AND SHEARS GIVEN AT FACE OF SUPPORT.
UNITS ARE K, K-Ft.



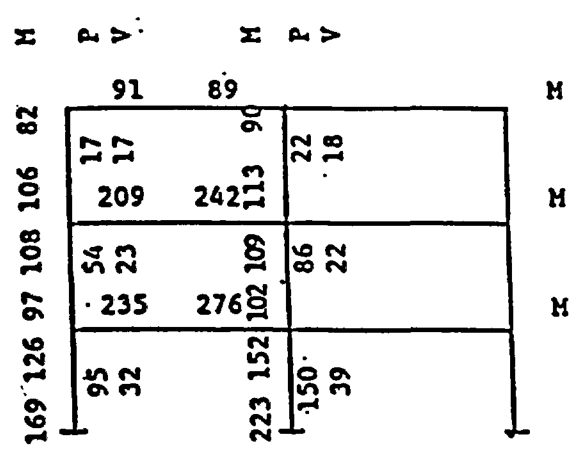
DEAD LOAD



LIVE LOAD



SEISMIC - EQ-II, SRSS



TOTAL = 1.0D + 0.25L + 1.0E

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Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 17 of 23)

EQ II ELEMENT STRESSES - FRAME 4 (DMRSF)

BEAM ELEMENTS : EQ-II UC $\geq$ 1.0D + 0.25L + 1.0E
F_y = 36 ksi

LEVEL	SIZE	Z_x (in^3)	M_D (K-Ft.)	M_c * (K-Ft.)	$\frac{M_D}{M_c}$	IDR**
ROOF	W14x30	47.3	91	142	0.64	2.0
3	W18x55	112	242	336	0.72	2.0
2	W18x60	123	276	369	0.75	2.0

* USE $M_c = M_{px} = Z_x F_y$

** INELASTIC DEMAND RATIO

COLUMN ELEMENTS : EQ-II UC $\geq$ 1.0D + 0.25L + 1.0E
F_y = 36 ksi

SIZE	A (in^2)	Z_x (in^3)	P_D (kips)	M_D (K-Ft)	M_D (K-in)	P_y (kips)	P_{cr} (kips)	M_{px} (K-in)	M_{pcx} (K-in)	$\frac{M_x}{M_{pcx}}$ +	IDR
W14x48	14.1	78.4	95	169	2028	507.6	412.3	2822	2707	0.75	1.25
W14x61	17.9	102	150	223	2676	644.4	559.9	3672	3324	0.81	1.25

+ SEE FIGURE 4-2, $M_x/M_{cpx} < \mu$ (μ = IDR)

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Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 18 of 23)

EQ-II ELEMENT FORCES : TRANSVERSE (N-S) DIRECTION - FRAME 3

DL & LL RESULTS BASED ON SIMPLE SUPPORTED ROOF & FLOOR GIRDERS.
SEISMIC RESULTS FROM COMPUTER ANALYSIS.
ALL END MOMENTS AND SHEARS AT FACE OF SUPPORT.
UNITS ARE K, K-Ft.

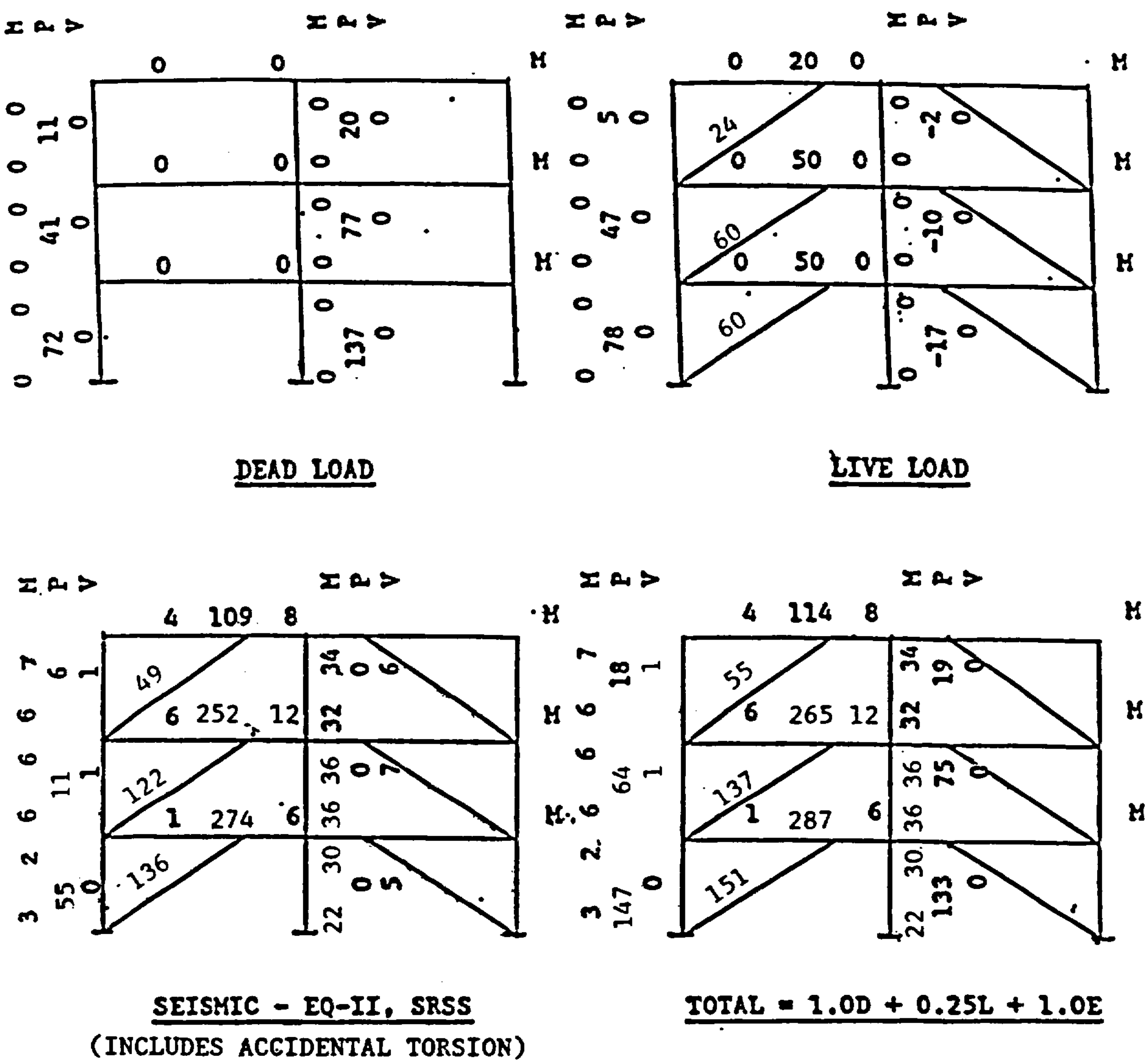

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Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 19 of 23)

EQ II ELEMENT STRESSES - FRAME 3 (ECCENTRIC BRACED FRAME)

BEAM ELEMENTS : EQ-II UC ≥ 1.0D + 0.25L + 1.0E
F_y = 36 ksi

SIZE	A (in^2)	Zx (in^3)	P_D (kips)	M_D (K-Ft.)	P_y (kips)	M_{pcx} (K-Ft.)	M_D/M_{pcx} *	IDR
W14x30	8.83	47.3	47	114	318	142	0.80	1.5
W18x55	16.2	112	111	265	583	321	0.83	1.5
W18x60	17.7	123	123	287	637	351	0.82	1.5

* M_{pcx}

COLUMN ELEMENTS : EQ-II UC ≥ 1.0D + 0.25L + 1.0E
F_y = 36 ksi

SIZE	A (in^2)	Zx (in^3)	P_D (kips)	M_D (K-Ft.)	M_D (K-in)	P_y (kips)	P_{cr} (kips)	M_{px} (K-in)	M_{pcx} (K-in)	M_D/M_{pcx} *	IDR
W14x43	12.6	69.7	147	3	36	453.6	352.0	2509	2020	0.02	1.25
W14x61	17.9	102	133	30	360	644.4	548.0	3672	3459	0.10	1.25

$M_D / M_{pcx} < \mu$ (μ = IDR)

K-BRACE ELEMENTS : EQ-II UC ≥ 1.0D + 0.25L + 1.0E
F_y = 36 ksi

LEVEL	SIZE	A (in^2)	r (in)	kl/r	Fa (ksi)	P_{cr} $1.7 F_a A$ (kips)	P_D (kips)	P_D/P_{cr}	IDR
3 - R	TS 5x5x¼	4.54	1.91	133	10.28	65	55	0.85	1.0
1 - 2	TS 6x6x⅜	7.95	2.25	225	12.98	154	151	0.98	1.0

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Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 20 of 23)

EQ II ELEMENT FORCES : LONGITUDINAL (E-W) DIRECTION - FRAME A

DL & LL RESULTS BASED ON SIMPLE SUPPORTED ROOF & FLOOR GIRDERS.
SEISMIC RESULTS FROM COMPUTER ANALYSIS.
UNITS ARE K, K-Ft.

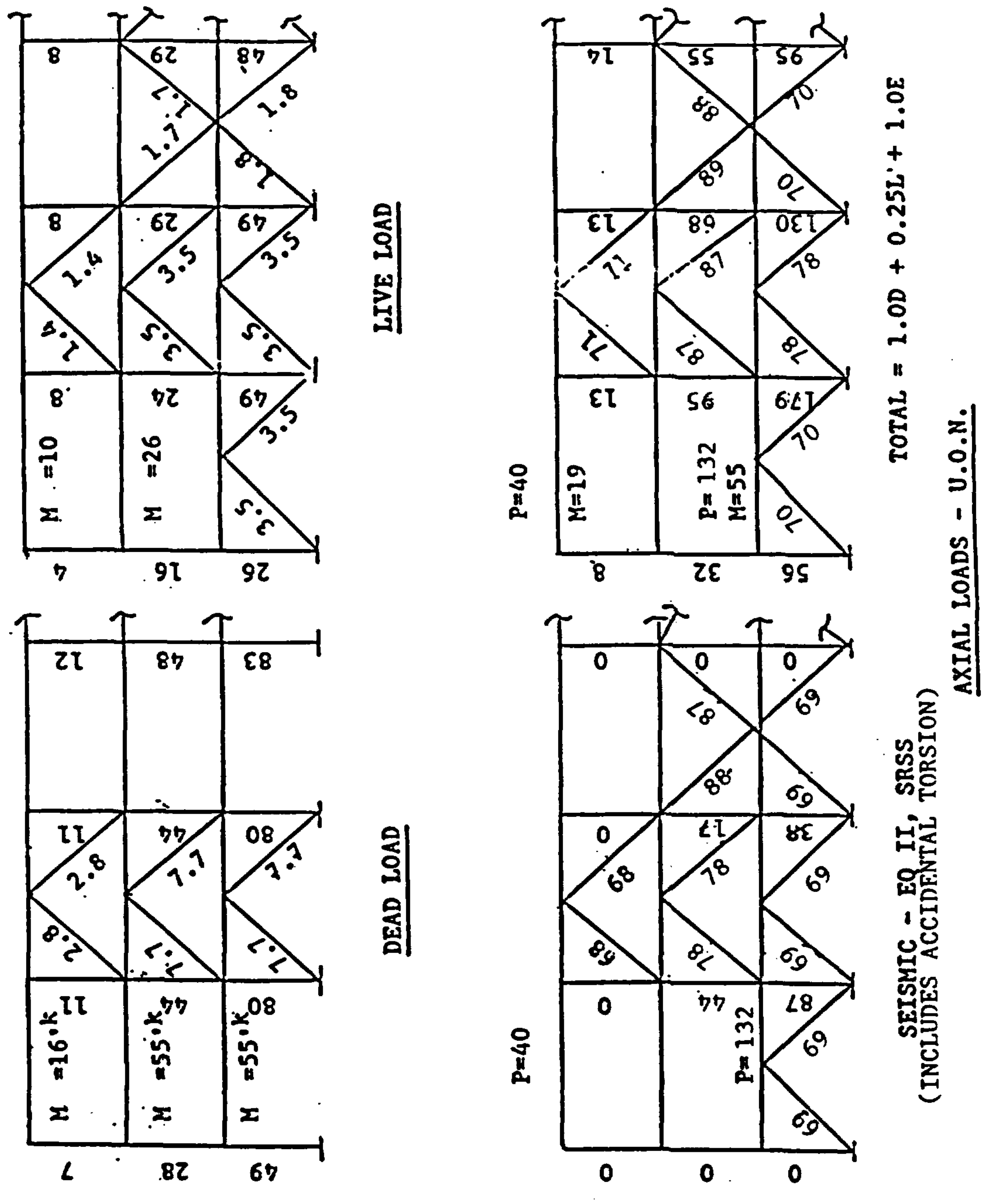

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Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 21 of 23)

EQ II ELEMENT STRESSES - FRAME A (BRACED FRAME)

K-BRACES, 2nd STORY (GOVERNS) EQ II UC $\geq$ 1.0D + 0.25L + 1.0E
F_y = 36 ksi

STEEL	A (in^2)	r (in)	kl/r*	F_a (ksi)	$P_{cr} = 1.7F_a A$ (kips)	P_D (kips)	$\frac{P_D}{P_{cr}}$	IDR
5x5x¼	4.54	1.91	105.4	12.28	94.8	89	0.94	1.0

* kl = 1.0 [19.42' - (1.50 x 19.42 / 11)] 12"/' = 201.3"

COLUMNS, 1st STORY EQ II UC $\geq$ 1.0D + 0.25L + 1.0E
F_y = 36 ksi

COL SIZE	P_D (kips)	M_D (kips-in)	P_y (kips)	P_{cr} (kips)	M_{px} (kips-in)	M_{pcx} (kips-in)	$\frac{M_D}{M_{pcx}}$	IDR
W14x43	179	-	453.6	366.3	-	-	0.49*	1.00
W14x48	95	483	507.6	412.3	2822	2707	0.18**	1.50

*P_D/P_{cr}
** SEE FIGURE 4-2 M_D/M_{pcx} < u (u = IDR)

BEAM ELEMENTS EQ II UC 1.0D + 0.25L + 1.0E
F_y = 36 ksi

BEAM SIZE	P_D (kips)	M_D (kips-in)	P_y (kips)	P_{cr} (kips)	M_{px} (kips-in)	M_{pcx} (kips-in)	$\frac{M_D}{M_{pcx}}$*	IDR
ROOF W14x30	40	228	318.6	252.0	1703	1757	0.13	1.50
FLOOR W18x40	132	660	425	405.2	2822	2445	0.27	1.50

* SEE FIGURE 4-2 M_D/M_{pcx} u (u = IDR)

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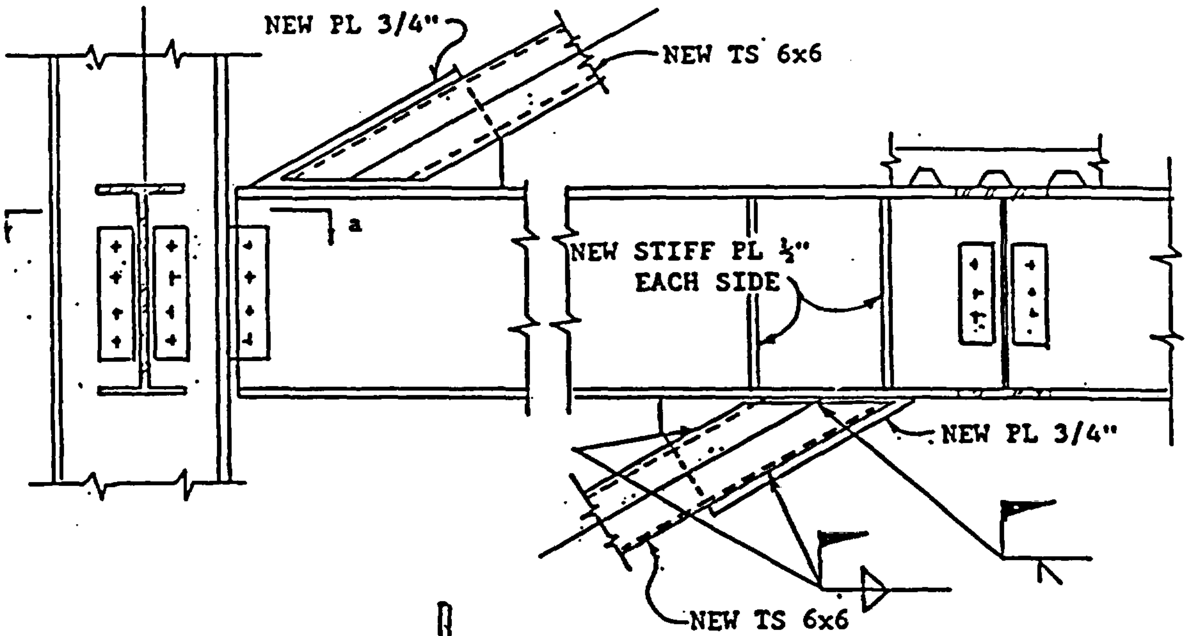
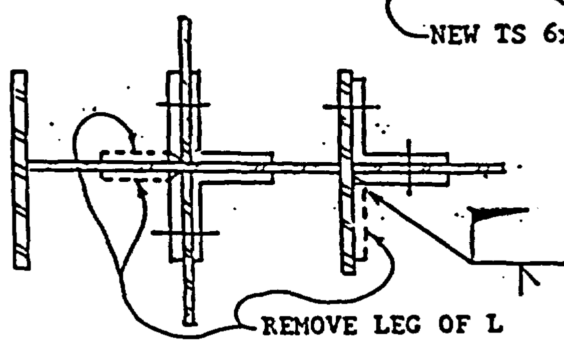
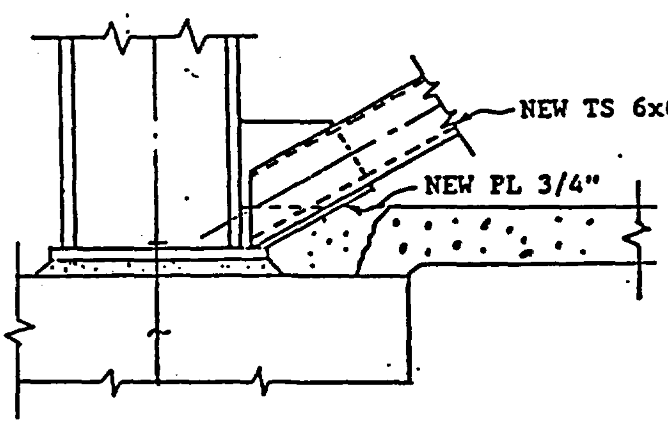
Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 22 of 23)





SECTION a-a



NEW BRACING DETAIL

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Figure F-3. Building with steel ductile moment-resisting frames and steel braced frames. (Sheet 23 of 23)

DESIGN EXAMPLE F-4

TEN-STORY CONCRETE FRAME AND SHEAR WALL BUILDING

Purpose. This example is presented to illustrate a procedure to evaluate an existing reinforced concrete structure, determine if it satisfies the acceptance criteria, and develop an upgrading concept for resistance to seismic forces.

Description of Structure. A 10-story office building (plus basement) with lateral force resisting systems consisting of reinforced concrete moment-resisting frames in the longitudinal direction and reinforced concrete shear walls in the transverse direction. .The building was designed and built in the late 1960's in accordance with the provisions of the 1964 Uniform Building Code (UBC). The earthquake design provisions are essentially identical to "Seismic Design for Building" (BDM) dated 15 March 1966 (TM 5-809-10/NAVDOCKS P-355/AFM 88-3, Chapter 13). These design provisions had not yet provided for concrete ductile moment-resisting space frames. However, the designer had provided some of the ductility requirements later adopted by the UBC and included in the April 1973 edition of the BDM. The ductility was provided using the concepts developed by Blume, Newmark, and Corning in "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions," Portland Cement Association, Skokie, Illinois, 1961. The structural design concepts are illustrated on sheets 2 through 5.

Construction Outline.

Roof:
- Built-up roofing.
- Reinforced lightweight concrete slabs, joists, and girders.
- Suspended ceiling.

Typical Floors:
- Reinforced lightweight concrete slabs, joists, and girders.
- Asphalt tile.
- Suspended ceiling.

Basement Floor:
- Reinforced concrete slab-on-grade.
- Asphalt tile.
- Suspended ceiling.

Foundation:
- Reinforced concrete mat.

Columns:
- Reinforced lightweight concrete.

Exterior Walls:
- Reinforced concrete.

Sheet 1 of 32

Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 1 of 32)

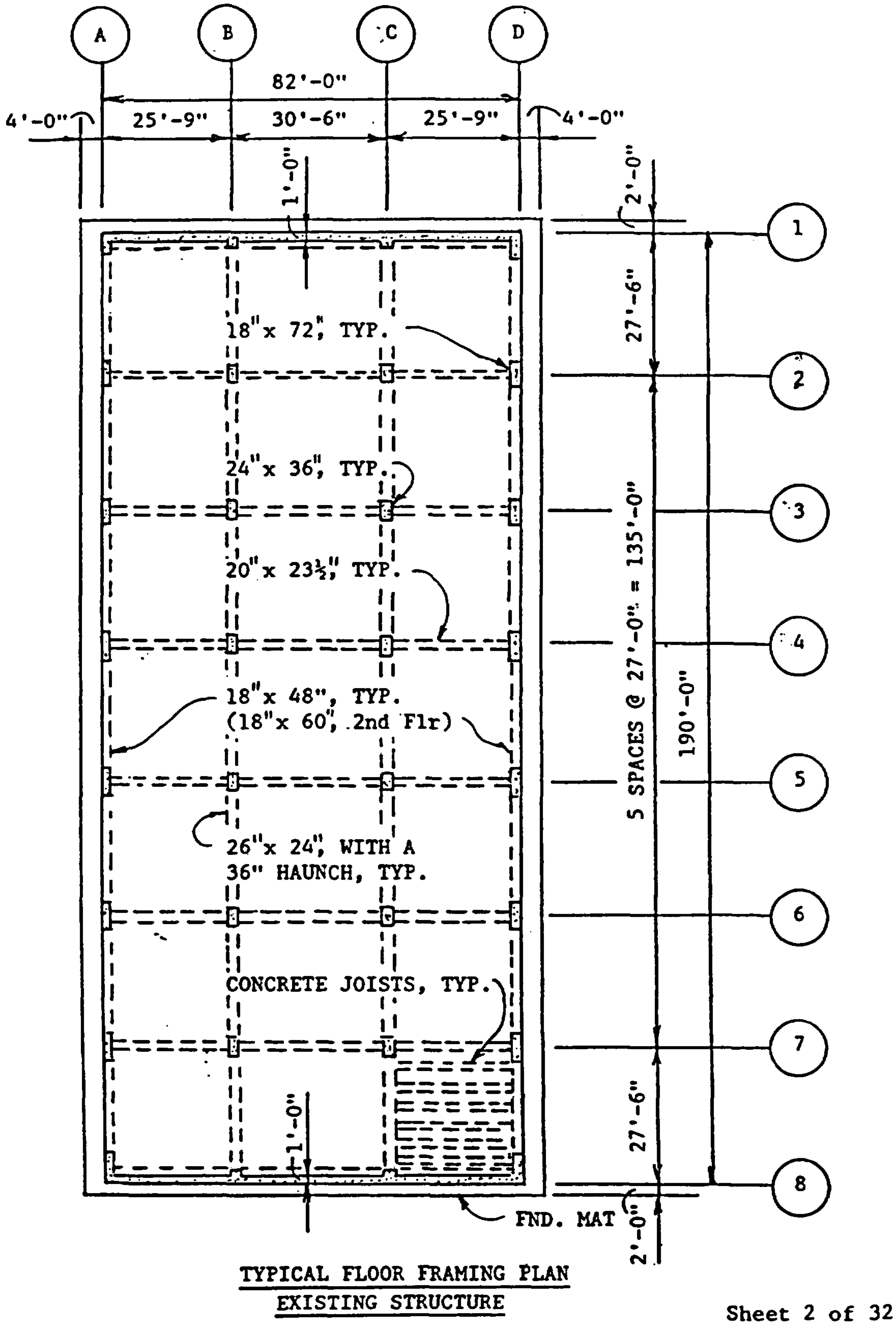
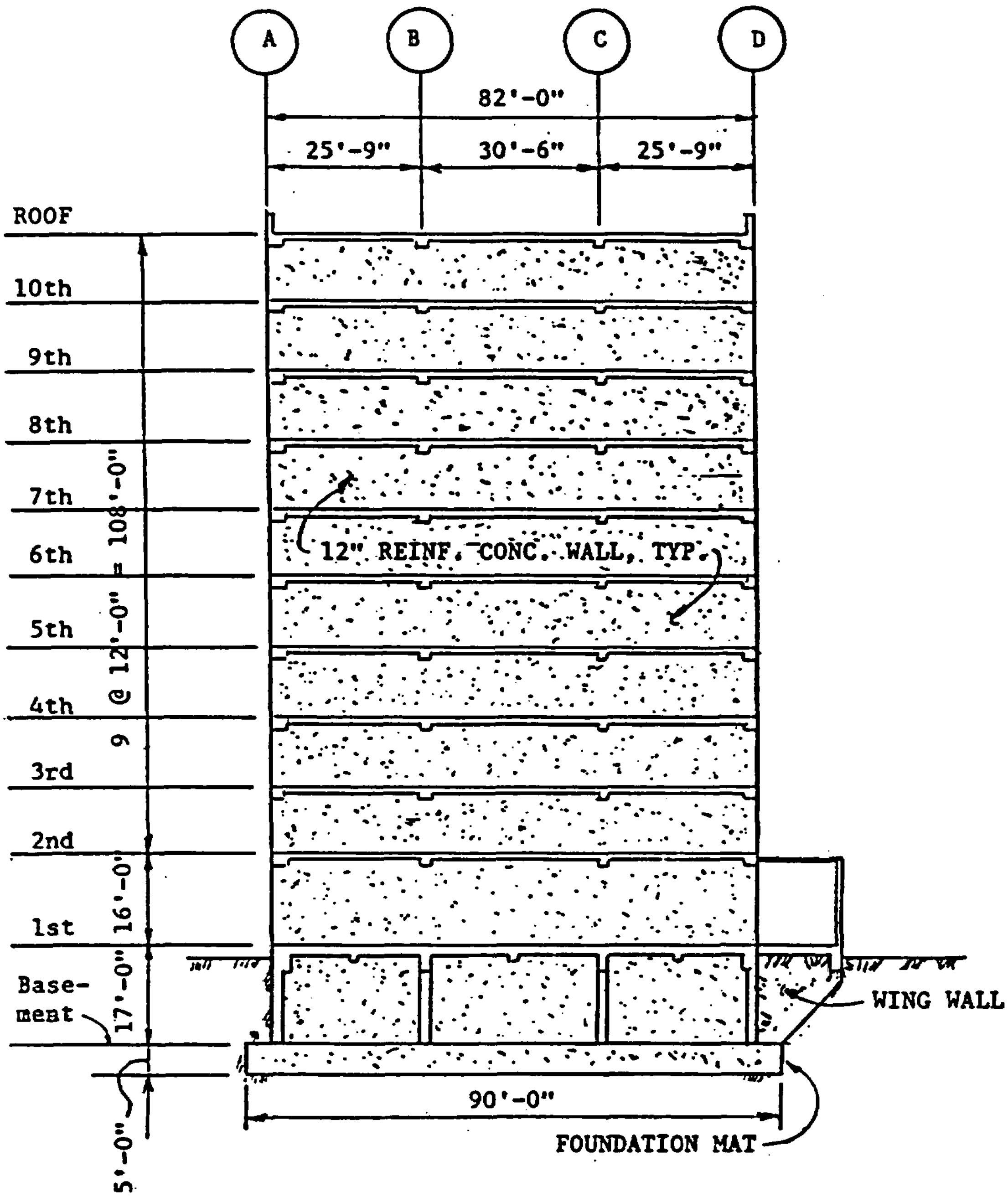


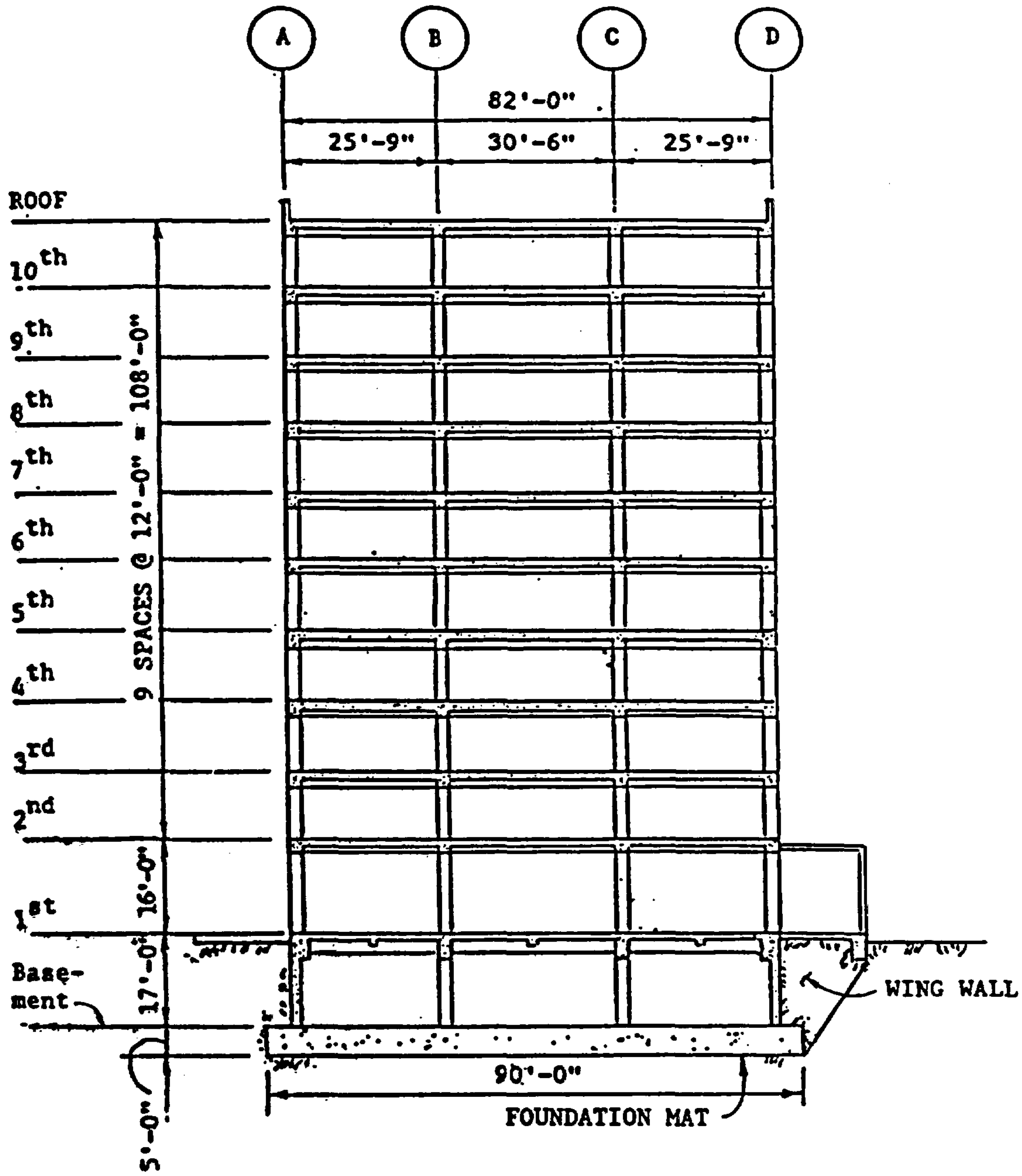
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 2 of 32)



LINES 1 & 8 (EXISTING)

Sheet 3 of 32

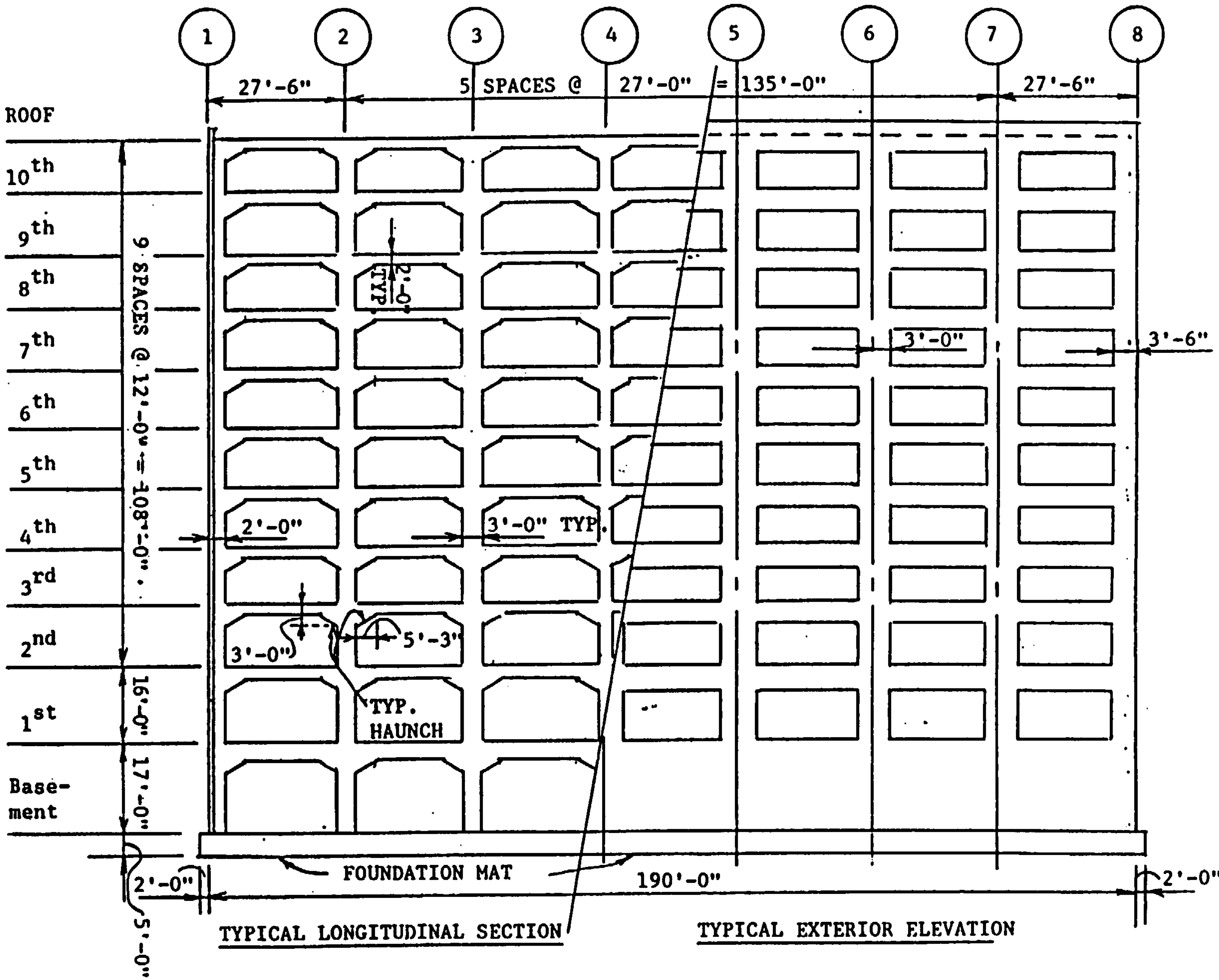
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 3 of 32)



FRAME LINES 2, 3, 4, 5, 6 & 7
(EXISTING)

Sheet 4 of 32

Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 4 of 32)



Sheet 5 of 32

Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 5 of 32)

Original Design. The original design for earthquake forces was based on the 1964 UBC (similar to 1966 BDM). The base shear was determined as follows:

$$V = ZKCW$$

where $Z = 1.0$ (seismic zone coefficient)
$K = 1.0$ (building systems coefficient)
$C = 0.05/T^{1/3}$

In the transverse direction, $T = 0.05\ h/D^{1/2} = 0.68$ sec
$C = 0.057$
In the longitudinal direction, $T = 0.1\ N = 1.0$ sec
$C = 0.05$

The weight $W = 32{,}600^k$ on the basis of regular weight concrete. Reinforced concrete design criteria were based on working stress design (WSD).

Design base shear:

Transverse = $1x1x0.057x32{,}600 = 1860^k$
Longitudinal = $1x1x0.05x32{,}600 = 1630^k$

Note: Due to "fast-tracking" of this building, the foundations were designed and under construction prior to completion of superstructure design. Because the above building weight would have overloaded the foundation soils, it was decided to use lightweight concrete for the frames and floors but not for the shear walls. This reduced the weight to $27{,}040^k$ and increased the effective base shear coefficients to:
$V/W = 0.069$ transverse
$V/W = 0.060$ longitudinal

In addition to the minimum requirements of the code, the engineer decided to supply additional detailing to provide ductility in accordance with the concepts developed by Blume, Newmark, and Corning. This included additional column ties (or hoops) in the column and in the beam-column joint zone to provide for confinement.

Sheet 6 of 32

Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 6 of 32)

Site Response Spectra. Site response spectra, which are used for the preliminary evaluation, the detailed analysis, and the upgrade concept, were developed in accordance with the procedure in chapter 3 of the SDG:

Building Classification: Others

Ground Motion Spectra: ATC 3-06 Map Contour Level
$A_a = A_v = 0.30$

Soil Classification: $S_i = 1.0$ (Type S1)

Earthquake I
Damping = 5%, D.F. = 1.00 (SDG table 3-7)
$A_a = A_v = 0.14g$ (Design Ground Motion, SDG table 3-4)
$S_a = D.F.\ (1.22A_vS_i)/T = 0.171g/T$ less than $D.F.\ (2.5)A_a = 0.35g$ max

Earthquake II
Damping = 10%, D.F. = 0.80
$A_a = A_v = 0.35g$
$S_a = D.F.\ (1.22A_vS_i)/T = 0.342g/T$ less than $D.F.\ (2.5)A_a = 0.70g$ max

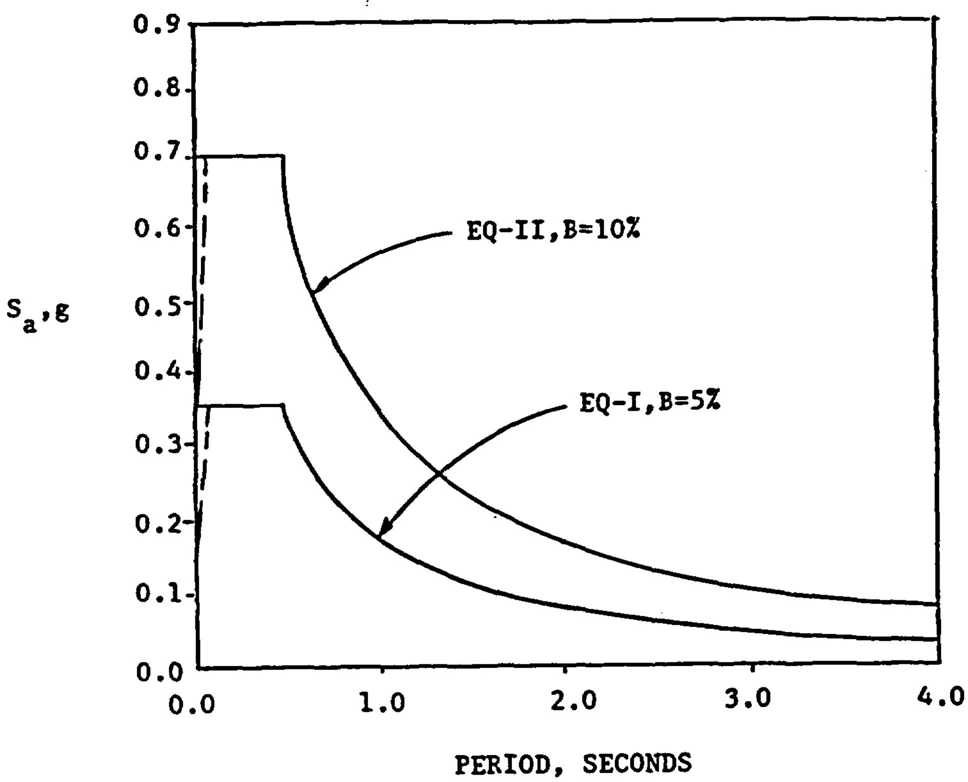
EQ-II/EQ-I = 0.70/0.35 = 2.0

The resulting spectra are shown on sheet 8.

Sheet 7 of 32

Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 7 of 32)

DESIGN RESPONSE SPECTRA FOR EQ-I AND EQ-II



EQ	B		PERIOD 0.0	.488	.80	1.0	1.5	2.0	3.0	4.0
I	5%	S_a,g	.14	.35	.214	.171	.114	.085	.057	.043
II	10%	S_a,g	.35	.70	.427	.342	.228	.171	.114	.086
		S_d,in	0	1.63	2.68	3.35	5.02	6.70	10.04	13.47

* SPECTRAL DISPLACEMENT $S_d = S_a(T/2\pi)^2 g$

Sheet 8 of 32

Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 8 of 32)

Preliminary Evaluation. A rapid evaluation of the structure was made using available data. For the longitudinal direction, the capacity was approximated by using the design base shear and assuming yield was at two times design. For the transverse direction, the capacity was approximated from the strength and area of the shear walls. Calculations are shown on sheets 10 and 11. The capacity spectrum method (sheet 12) was used to approximate damage. Over 100 percent for transverse, 70 percent for longitudinal, and 99 percent for combined (total) damage due to EQ-II. The results of the preliminary evaluation indicate that the structure will be substantially damaged by EQ-II; however, for a smaller earthquake (e.g., EQ-I) the results of the evaluation indicate that the structure would remain essentially elastic. Because of the size and value of the building, it was decided that a detailed analysis would be warranted to more accurately determine how the structure would perform under EQ-II loading.

Sheet 9 of 32

Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 9 of 32)

RAPID EVALUATION PROCEDURE

<u>Longitudinal Direction</u>: Moment frame

Effective design base shear coefficient at WSD
$V/W = 0.067$

<u>Yield Capacity</u>

Assume yield at 2 x design
$C_B = 2 \times 0.067 = 0.134$

Estimate period: $T = 0.1N = \underline{1.0}$ sec

Estimate $\alpha = C_B/S_a = 0.80$ (SDG para 5-3a(2)(c))

$S_a = C_B/0.80 = \underline{0.168g}$

$S_d = S_a \times g \times \left(\frac{T}{2\pi}\right)^2 = \underline{1.64}$ inches

Estimate roof displacement, Δ_R
$\Delta_R = P.F._{ROOF} \times S_d \simeq 1.3S_d = 2.1$ inches

Average interstory drift ratio
$\Delta_R/H = 2.1/124 \times 12 = 0.0014$

<u>Ultimate Capacity</u>

Assume: ULTIMATE CAPACITY = 1.5 x YIELD
ULTIMATE DISPLACEMENT = 4 x YIELD

$S_{aU} = 1.5 \times 0.168g = \underline{0.252}g$

S_{dU} 4 x 2.1 inches = $\underline{8.4}$ inches

$T_U = 2\pi\sqrt{S_d/S_a g} = \underline{1.84}$ sec

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 10 of 32)

RAPID EVALUATION PROCEDURE (continued)

Transverse Direction: Shear Walls

Effective design base shear at WSD: $V = 1860^k$; $V/W = 0.076$
Area Shear Walls: (82' x 1')2 x 144 = 23,616 square inches
Wall Capacity: 12" wall w/#4 at 12"ef,ew

$$2\sqrt{f'_c} = 2\sqrt{3000} = 110 \text{ psi}^*$$
$$pf_y = \frac{2x0.20}{12x12} \times 40{,}000 = \underline{111} \text{ psi}$$
$$\text{Total} \qquad 221 \text{ psi}$$

$$V_{CAPACITY} = 0.221 \times 23{,}616 = 5220^k$$

*May be 5000 psi concrete, see detailed analysis

Yield Capacity

$$C_B = V_{CAP}/W = 5220/24500 = 0.213$$

Est. Roof Displ.: Shear $\Delta = \frac{VH}{AG} \simeq 150 \text{ psi (avg)} \times \frac{124x12}{1.2x10^6} = \underline{0.2"}$

$$\text{Bending: } \Delta = \frac{PL^3}{3EI} \simeq \frac{2/3(5220)x124^3x12}{3(3x10^3x144)x46000x2} = \underline{0.66"}$$

$$\Delta_R \simeq \underline{0.9"} \pm$$

If $T = 0.68$ sec (sheet 6) and $S_a = \frac{C_B}{0.8} = \underline{0.27}g$:
$S_d = \left(\frac{T}{2\pi}\right)^2 S_a \times g = 1.22"$; $\Delta_R \simeq 1.3S_d = \underline{1.5}"$

Assume $\Delta_R = 1.0"$ (includes rocking added to 0.9")
$S_d = 1.0 \div 1.3 = \underline{0.77}"$ and $S_a = \underline{0.27}g$
$T = 2\pi\sqrt{S_d/S_a x g} = \underline{0.62}$ sec

Ultimate Capacity

Assume $S_{aU} = 1.25 \times 0.27 = \underline{0.34}g$
$S_{dU} = 4 \times 0.77 = \underline{3}"$
$T = \underline{0.95}$ sec

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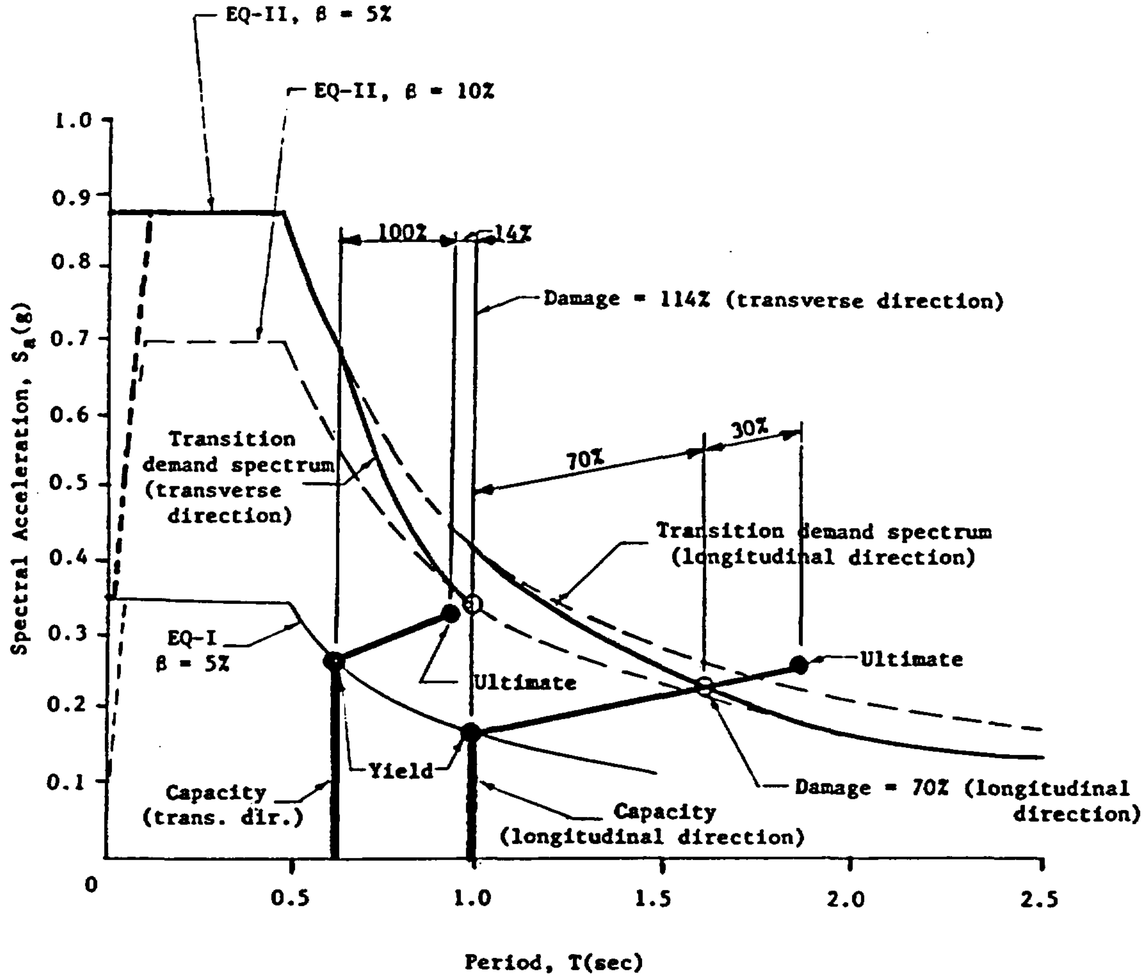
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 11 of 32)



Rapid Evaluation

Transverse direction:	114% damage
Longitudindal direction:	70% damage
Total damage:	2/3(114) + 1/3(70) = 99%

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 12 of 32)

Acceptance Criteria (para 5-2). The acceptance criteria for existing buildings are those presented for the post yield analysis for EQ-II in the SDG with latitude allowed under certain conditions.

Conforming/Nonconforming Systems and Materials. The reinforced concrete moment frames do not strictly conform to current standards; however, some ductility provisions were incorporated into the design and the current condition of the building is good. Therefore, the structure will be considered essentially conforming with some latitude allowed in the acceptance criteria.

Method 1. Elastic Analysis Procedure (Refer to SDG paras 4-4c and 5-5a).

Classification: Other Buildings

Loading Combination: DL + 0.25LL + 1.0 EQ

Ultimate Strength Capacities: ACI 318 Strength Design

Inelastic Demand Ratios: (table 5-1)

	Nonduct.	Ductile	Avg.
Reinf. Conc. Frames			
Columns	1.25	1.75	1.5
Beams	1.75	3.00	2.4
Reinf. Conc. Shear Walls			
Single Curtain Reinf.	Shear-1.50, Flexure-2.0		
Double Curtain Reinf.	Shear-1.75, Flexure-3.0		
Reinf. Conc. Diaphragms	Shear-1.75, Flexure-2.0		

Material Properties

Lightweight Concrete	$f'_c = 3750$ psi
Regular Weight Concrete	$f'_c = 4000$ psi
Reinforcement	$F_y = 40$ ksi

Story Drift Limitation: 0.015 x Story Height

Method 2. Capacity Spectrum Method (Refer to SDG paras 4-4d and 5-5b). If the acceptance criteria of Method 1 are not satisfied, the structure will be analyzed in accordance with Method 2 prior to developing a seismic upgrading concept.

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 13 of 32)

Detailed Structural Analysis.

Method 1. The existing structure was analyzed with the aid of a computer. Gross concrete section properties of the girders and columns were used for the moment frame. For simplicity the haunches were neglected. Also, the stiffening effects of the floor system were ignored in the mathematical model. It is assumed that the contribution of these items to stiffness are relatively small and are balanced out by neglecting the reduced stiffness effects of nominal "cracked" section properties.

The mathematical model was subjected to an elastic modal analysis using the design response spectrum for EQ-II, 10 percent damped, shown on sheet 8. The results of the analysis gave the following:

	Transverse	Longitudinal
Fundamental Period (sec)	0.46	0.80
Base Shear, 1st Mode (kips)	13,980	9,520
Base Shear, RSS (3 modes)	14,485	9,764
Roof Displacement (ft)	0.172	0.292
Roof Acceleration, 1st mode	1.00g	0.556g
Roof Acceleration, RSS (3 modes)	1.10g	0.656g

The results indicate that the structure is relatively stiff, such that the calculated periods are shorter than the empirical periods used in the original design (sheet 6). The EQ-II shear forces are 7.5 times design in the transverse direction and 5.8 times in the longitudinal direction.

Sample IDR*'s of the most critical elements follow:

	Calculated	Allowable
Transverse Shear Walls	IDR = 2.94	1.75 N.G.
Longitudinal frame, girder bending	IDR = 2.3	2.4 O.K.
Longitudinal frame, column bending	IDR = 2.0	1.5 N.G.

*IDR's are calculated by dividing the computer calculated force by the strength capacity for each element.

The conclusions of the Method 1 detailed evaluation indicate that the existing building does not conform to the acceptance criteria. However, the results are based on a gross concrete section model. With large overstresses it is likely that the period will lengthen (due to cracked concrete) and reduce the effective earthquake forces on the building. It should also be noted that as some elements yield, additional load will be distributed to other members. In the elastic model, the transverse interior frames only take about 3 percent of the lateral forces. However, if the shear walls yield, the frames can

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 14 of 32)

contribute some backup resistance. In order to get a better feel for the inelastic response of the building a Method 2 analysis was done.

Method 2. The Capacity Spectrum Method uses a step-by-step, pseudo-inelastic approach to approximate the inelastic capacity of the structure. This capacity is compared by means of a graphical procedure to the demands of the EQ-II response spectrum. Guidelines for this procedure are presented in the SDG, para 5-5.

For this example, the pseudo-inelastic analysis consisted of consecutive elastic analyses of an initial mathematical model of the structure that was modified in an iterative fashion to include the results of the previous analyses and loaded incrementally. The process began by defining the initial 2-D model as is typically done for any computerized elastic analysis (e.g., the analysis used in Method 1, sheet 14). In addition, beam yield strengths for positive and negative bending, beam shear capacities, and beam and column gravity induced forces were computed. For beams to be subjected to negative seismic bending, a seismic reserve capacity equal to beam negative yield strength less gravity moment at the face of support was computed. For beams to be subjected to positive seismic bending, the seismic reserve capacity equals the beam positive yield strength plus the gravity moment at the face of support. For columns, P-M interaction diagrams were used to aid in identifying load capacities as shown on sheet 17.

The incremental loading regimen commenced with the application of the EQ-II Spectrum (sheet 8) loading to the 2-D mathematical model of the initial structure. Seismic member forces derived from this analysis were compared to member seismic reserve capacities to identify the first set of plastic hinges to form and to obtain the maximum member overstress factor. The initial loading, S_{aII}, divided by this overstress factor defines the load, S_{aY}, at first yielding as well as the seismic member forces associated with first yielding.

For the second step, the mathematical model was altered to include pinned member ends which reflected the first set of plastic hinge locations. This model was subjected to a small, monotonic, incremental load, S_{ai}, and reanalyzed using the same elastic computer program. The new set of seismic member forces obtained from this was added to those corresponding to first yielding and this sum was again compared to the member seismic reserve capacities; thus a second set of plastic hinges could be identified. Subsequent analyses were performed identically, each time including the new set of plastic hinges in the previous model and comparing the summation of the member forces of previous analyses to the initial member seismic reserve capacities. The method of superposition of the incremental loads are illustrated in sheets 19 and 20 of Figure E-3 of the SDG.

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 15 of 32)

Longitudinal Direction. The results for the longitudinal direction are shown on sheets 18, 19, and 20. Sheet 18 shows the sequence of plastic hinges. Sheet 19 shows the relationship between V, Δ_R, C_B, S_a, S_d, and T, and plots the capacity curves. Sheet 20 shows the graphical solution for the capacity spectrum method.

From sheet 20 it appears that the structure, in the long direction, can survive EQ-II without collapse and that it will remain essentially elastic for EQ-I. The capacity curve crosses the demand curve (EQ-II) at approximately $S_a = 0.244g$ and $T = 1.44$ sec.

$$S_d = (T/2\pi)^2 S_a g = 4.95''$$

$$\Delta_R = 1.30 S_d = 6.42''$$
$$\Delta_R/H = 6/124 \times 12 = 0.0043, \text{ Avg. drift ratio}$$
If worst story = 2 x Avg
Max. story drift ratio = 0.008 < 0.015 O.K.

Although these analytical results are encouraging, the "survival" of the building against collapse for EQ-II should be considered marginal. More conservatism in modeling, application of the modal story force, or consideration of possible beam/column deterioration due to repetitive cycling of the inelastic rotation would tend to depress the capacity curve of sheet 20 below the demand spectrum of EQ-II.

Transverse Direction. The detailed evaluation for the transverse direction was not in the scope of this example. Because the calculated period of the structure is shorter than the one obtained by the empirical formula, it appears that the performance of the structure will be worse than approximated in the rapid evaluation. However, it should be noted that a detailed evaluation of the shear wall energy absorbing capabilities after initial yielding may show that the performance characteristics of the transverse direction are better than anticipated.

Results of Detailed Structural Analysis. Although the results indicate that the building may be severely damaged if subjected to the EQ-II earthquake, the overall performance characteristics are relatively good considering the age (pre-1973) and type of construction (reinforced concrete frame). It appears that the building will perform in an essentially elastic manner for EQ-I but compliance with the acceptance criteria for EQ-II may be marginal. It is therefore recommended that upgrade concepts be developed and that a cost-benefit study be made to determine priorities for upgrading.

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 16 of 32)

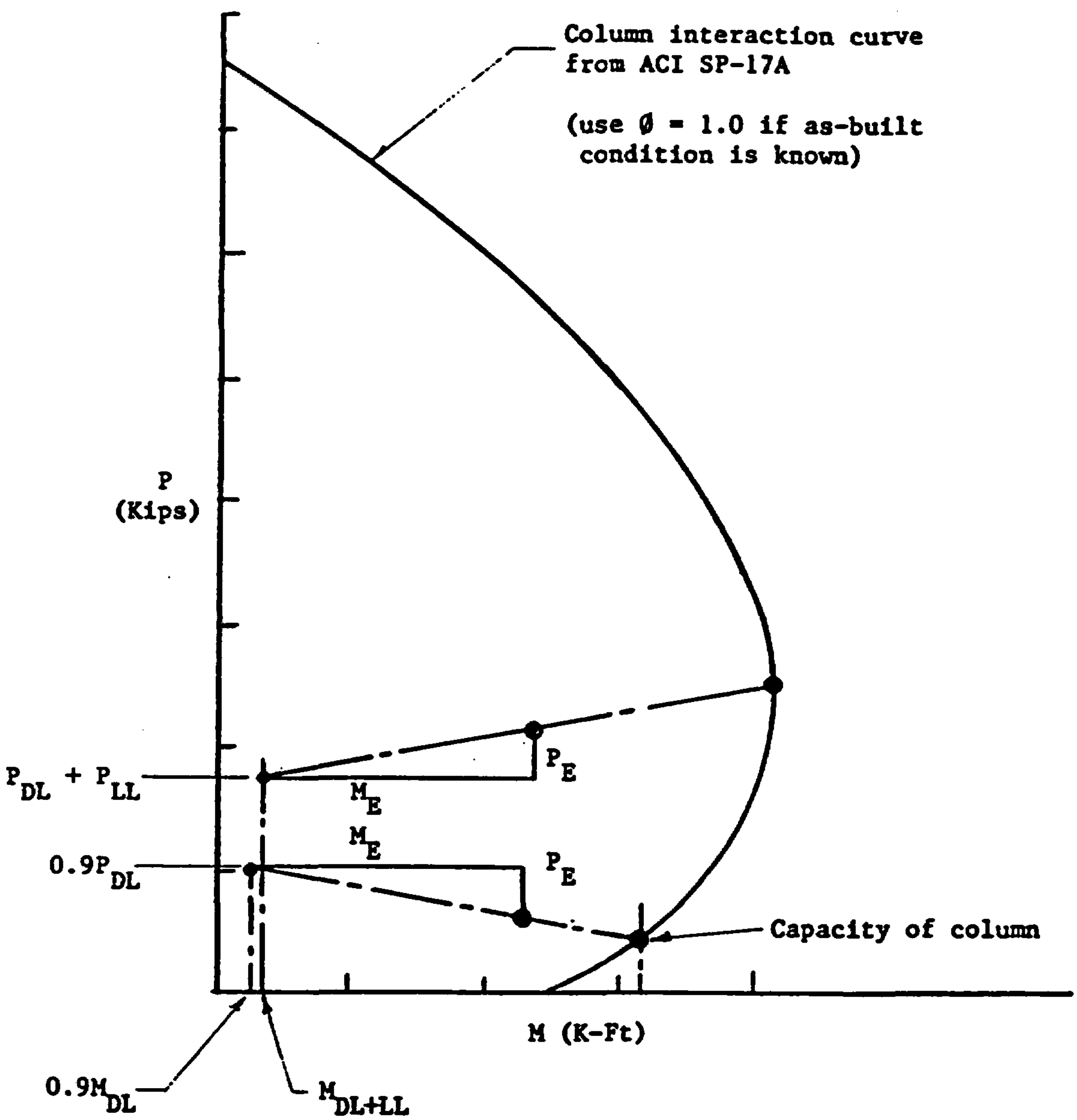

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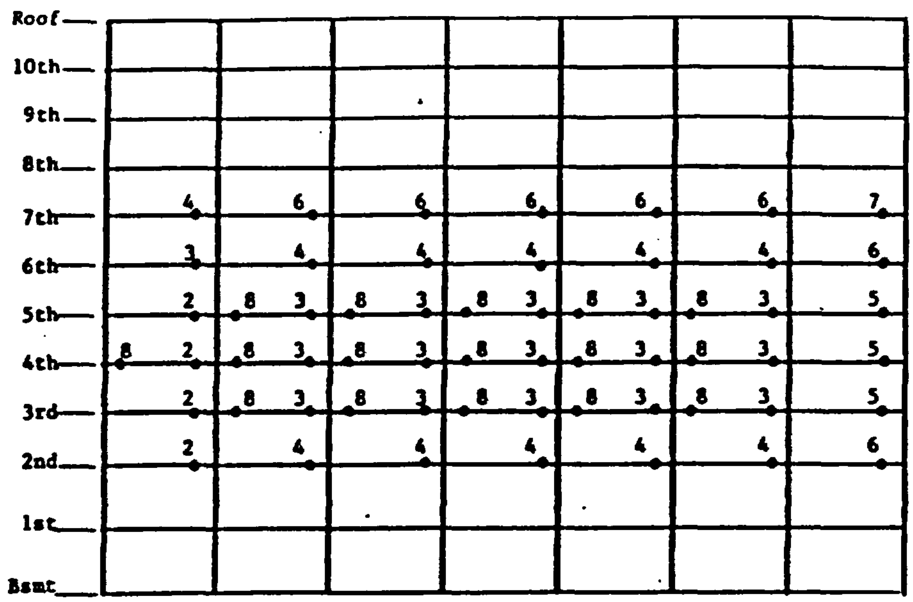
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 17 of 32)

FRAMES B & C



Load cycle	1	2	3	4	5	6	7	8	9
S_a*	.144g	.03g	.01g	.01g	.01g	.01g	.01g	.01g	.01g

* Load increment

FRAMES A & D

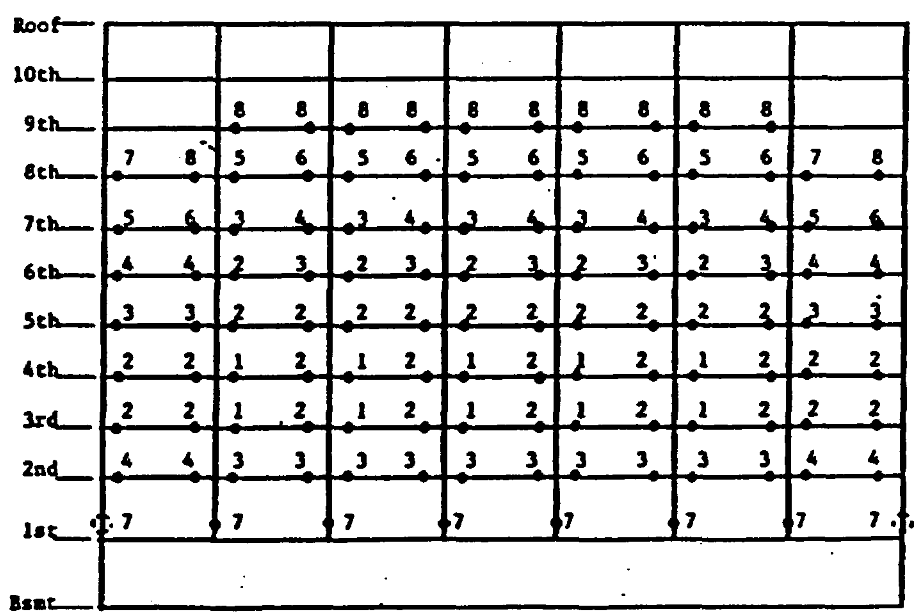


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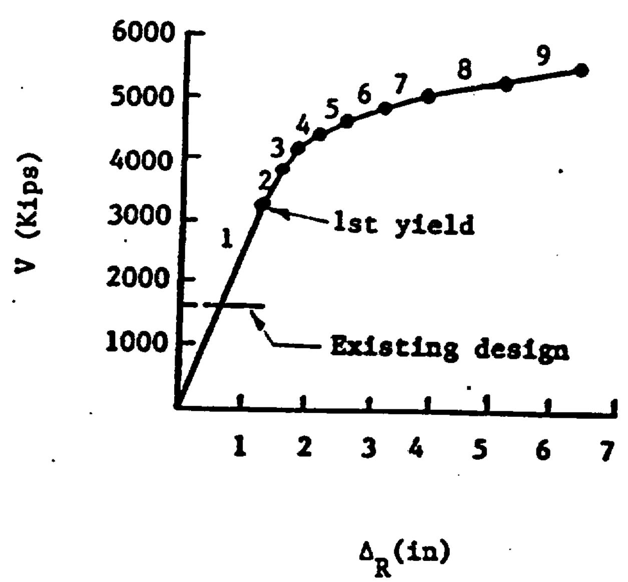
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 18 of 32)

EQ-II: CAPACITY SPECTRUM ($W = 27{,}000^K$)

INCR.	S_{ai}	ΣS_a	S_{di}	ΣS_d	Δ_{Ri}	$\Sigma\Delta_R$	V	C_B	T	$\frac{\Sigma\Delta_R}{\Sigma S_d}$	$\frac{C_B}{\Sigma S_a}$
1	0.144g	0.144g	0.94"	0.94"	1.25	1.25	3256	0.120	0.817	1.33	0.83
2	0.03	0.174	0.24	1.18	0.31	1.56	3931	0.147	0.832	1.32	0.84
3	0.01	0.184	0.14	1.32	0.17	1.73	4154	0.153	0.856	1.31	0.83
4	0.01	0.194	0.27	1.59	0.32	2.05	4379	0.162	0.915	1.29	0.84
5	0.01	0.204	0.41	2.00	0.50	2.55	4606	0.170	1.001	1.28	0.83
6	0.01	0.214	0.48	2.48	0.60	3.15	4830	0.179	1.088	1.27	0.84
7	0.01	0.224	0.54	3.02	0.69	3.84	5051	0.187	1.174	1.27	0.83
8	0.01	0.234	0.92	3.94	1.23	5.07	5285	0.196	1.312	1.29	0.84
9	0.01	0.244	1.01	4.95	1.85	6.42	5517	0.204	1.440	1.30	0.84



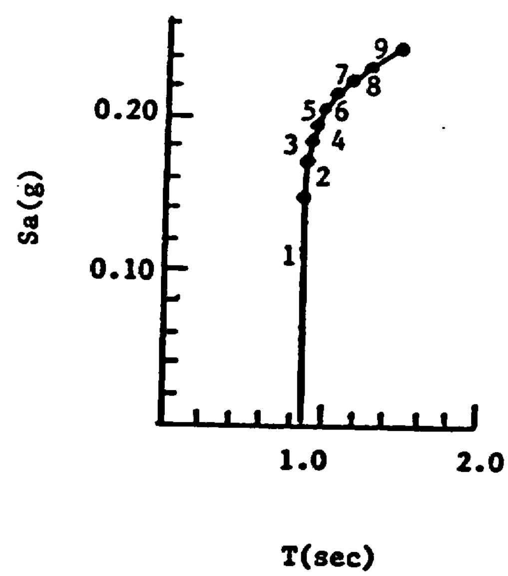


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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 19 of 32)

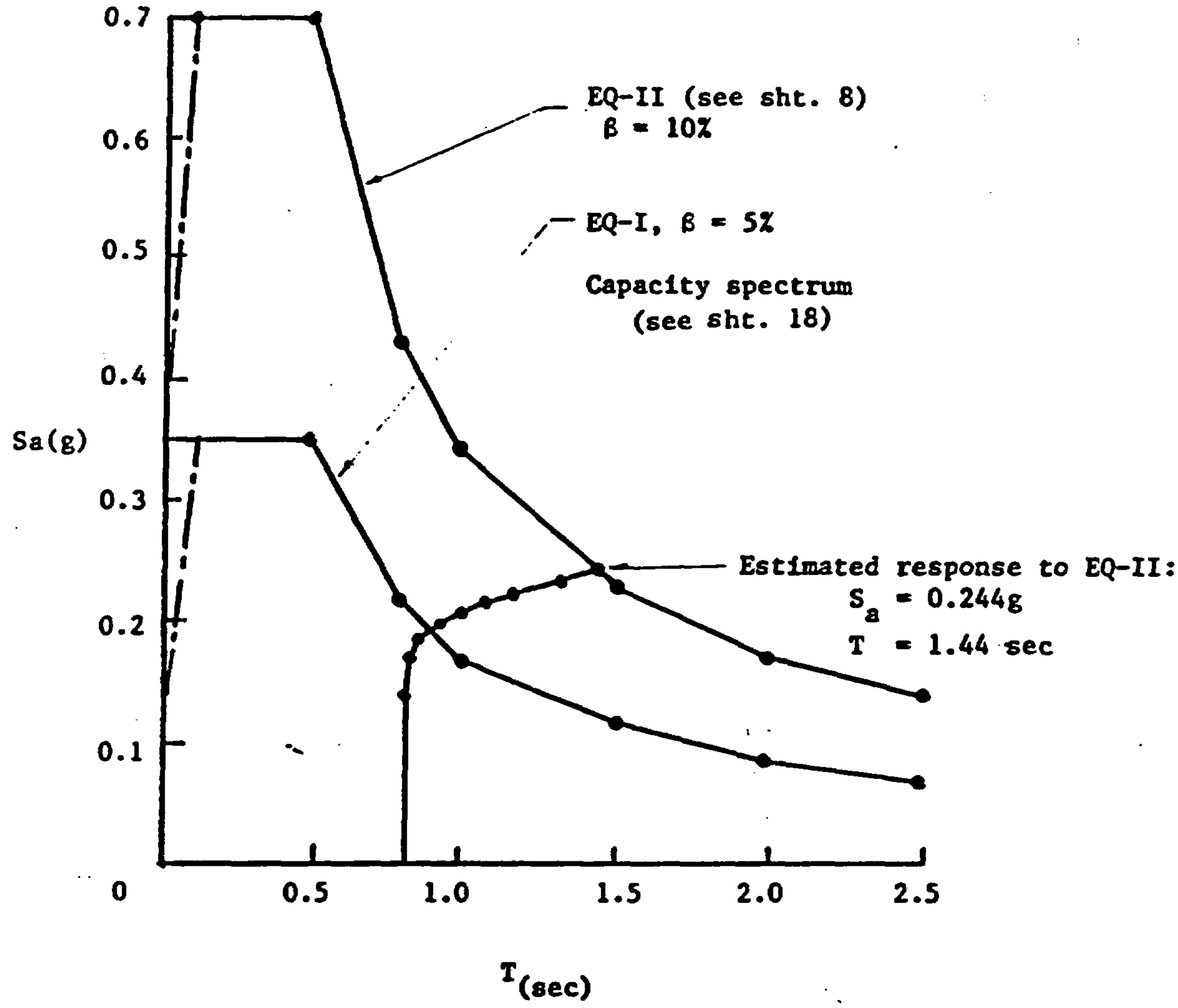

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 20 of 32)

Development of Seismic Upgrade.

Structural Upgrading Concept. The recommended upgrading concepts include the addition of interior cast-in-place reinforced concrete walls, to resist the transverse seismic forces and reduce the diaphragm stresses, and the placement of cast-in-place reinforced concrete panels in alternate window openings in the exterior concrete frames to resist the seismic forces in the longitudinal direction. For plans and elevations of the upgrade concept see sheets 22, 23, and 24.

Confirmation Analyses. A modal analysis of the modified structure was made with the aid of a general computer program for the static and dynamic analyses of frame and shear wall three-dimensional buildings for both the transverse and longitudinal directions. The program assumes rigid diaphragms and the roof and the floor diaphragms of this modified structure essentially meet the requirements of this assumption. The mathematical model was assumed fixed at the first floor level. The dynamic modal responses are indicated on sheets 25 and 26.

Structural Member Responses. Sheets 27 and 28 indicate the SRSS of modal responses for representative structural members in the transverse and longitudinal directions. The accidental torsion responses were calculated as described for design example F-2 and are given on sheet 29. A check of selected structural elements for compliance with the acceptance criteria is given on sheets 30 and 31.

Torsional Forces. Due to the symmetry of the structure lateral load resisting system there is no "calculated torsion." The "accidental torsion" is the story shear times the nominal eccentricity of 5 percent of the maximum building dimension. The torsional forces for the roof and the floors are distributed to the lateral force resisting elements in accordance with the method illustrated in the BDM Example A-3 and added to the forces from the dynamic analysis.

Overturning Forces. A check of the overturning forces due to EQ-II resulted in no instability of the structure as a whole. The soil pressure at the toe of the foundation mat due to DL + 0.25 LL + EQ-II forces in the transverse direction exceeds more than twice of the allowable design soil pressure when based on a triangular distribution of the soil pressure. Soil pressure under a rectangular distribution assumption results in a soil pressure less than twice the allowable design pressure. A Soil Engineering firm should be consulted to re-evaluate the allowable soil pressure and the shape of the soil distribution pressure under dynamic loadings.

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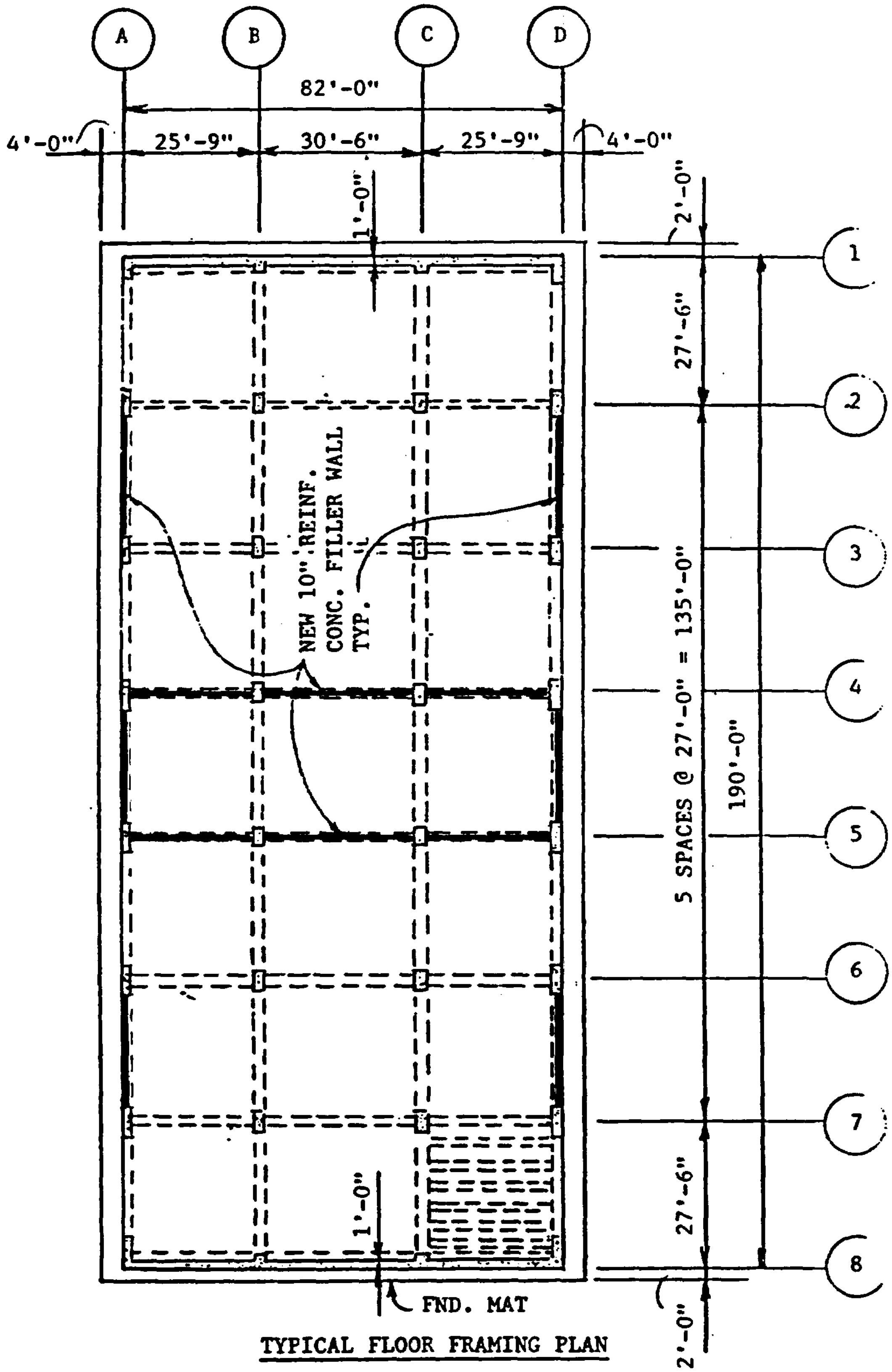
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 21 of 32)



TYPICAL FLOOR FRAMING PLAN

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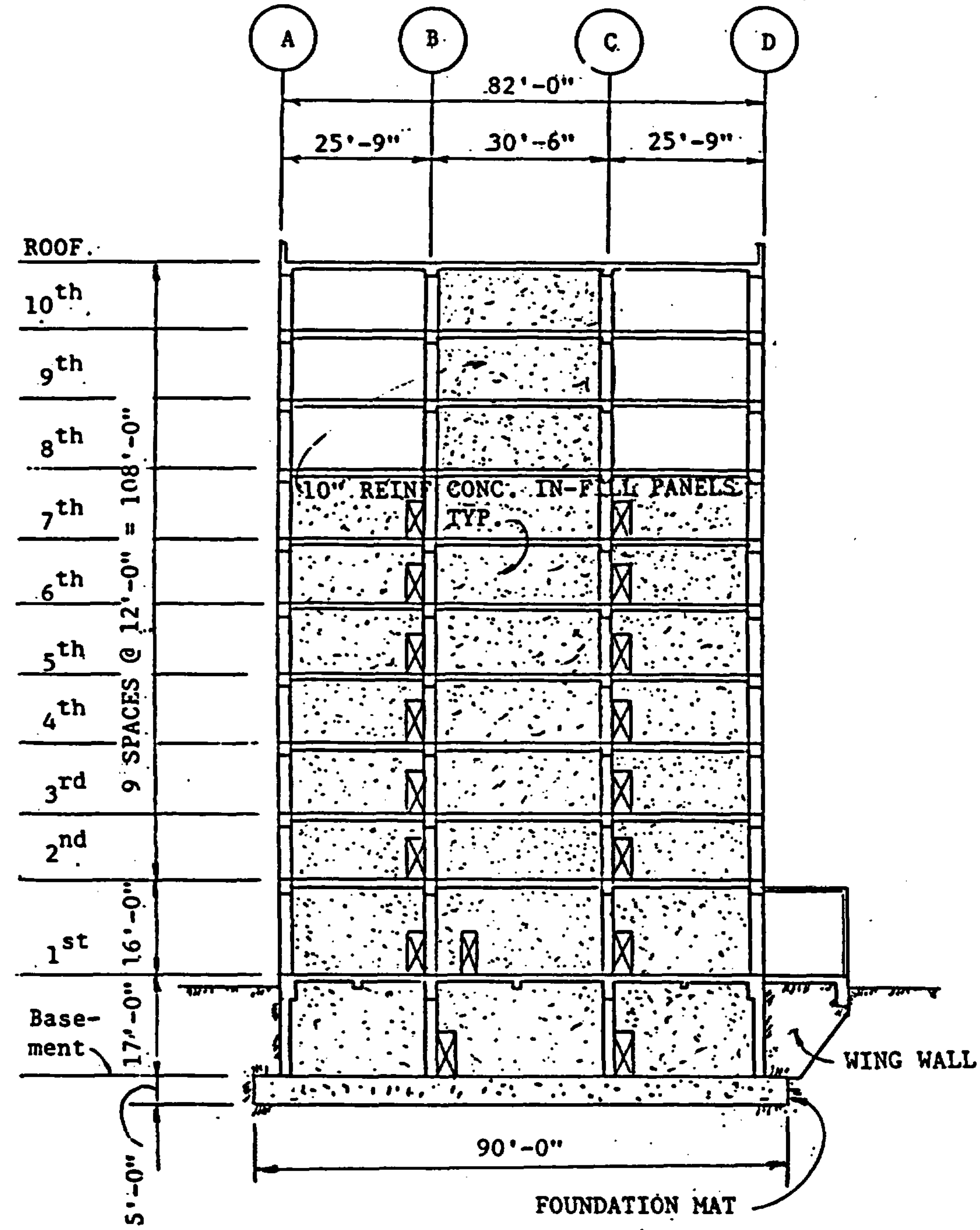
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 22 of 32)



SHEAR WALL LINES 4 & 5
(NEW)

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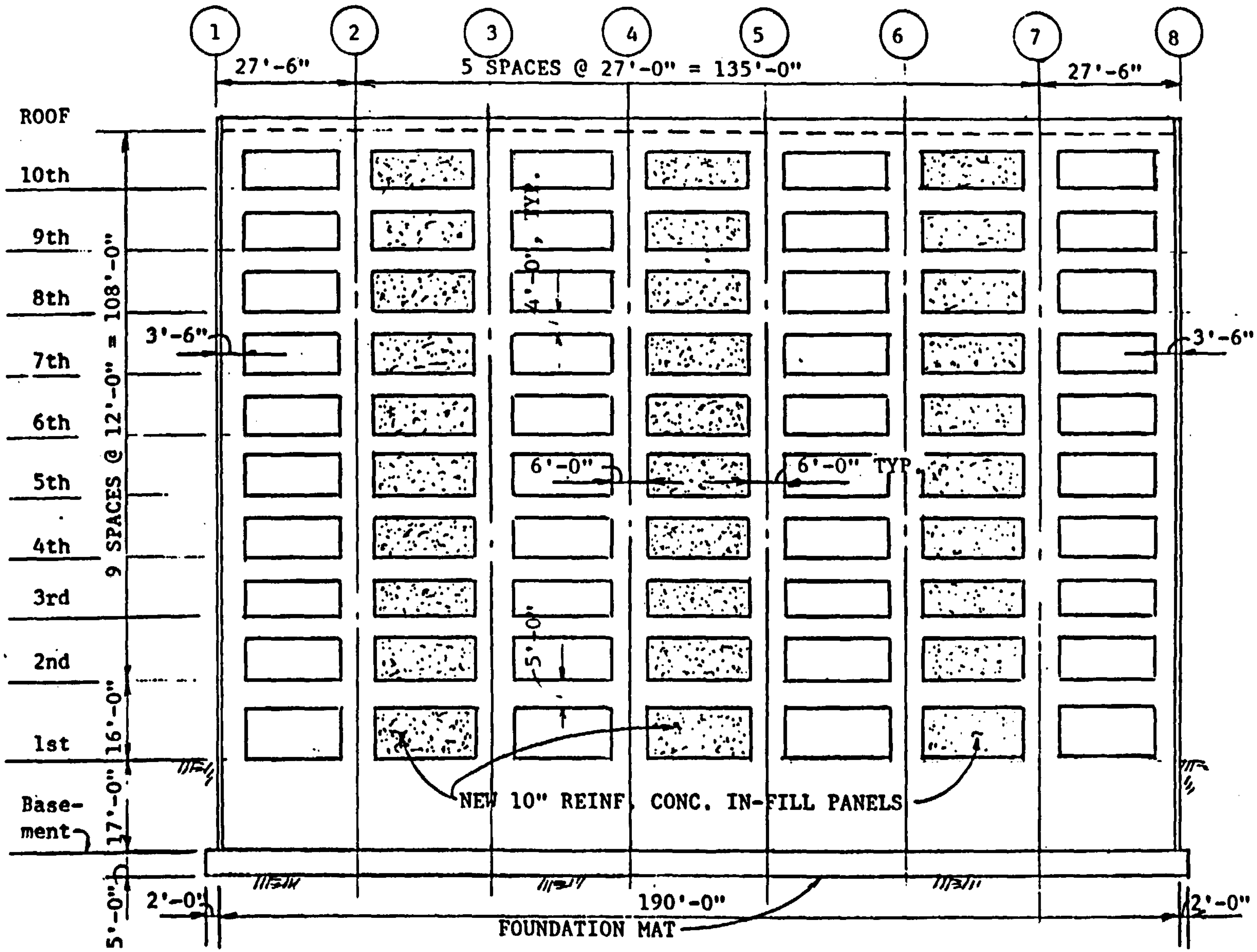
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 23 of 32)



FRAMES LINE A & D

Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 24 of 32)

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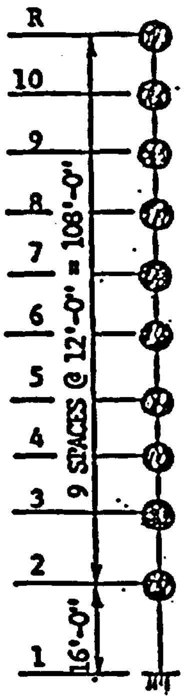
EQ II STRUCTURAL RESPONSE - SRSS

MASS (Kips-Sec/Ft.)	STORY LOAD F_x ** (kips)	STORY SHEAR V_x** (kips)	DISPLACEMENT Δ_x ** (Feet)	STORY DRIFT $\delta\Delta_x$ ** (Feet)
90.02	3033		0.202	
		3033		0.014 0.120 *
86.87	2576		0.188	
		5594		0.017
86.87	2284		0.171	
		7783		0.020
89.06	2136		0.152	
		9704		0.022
91.25	1995		0.130	
		11,394		0.024
91.25	1786		0.106	
		12,509		0.025
91.25	1577		0.082	
		13,945		0.024
91.25	1399		0.057	
		14,810		0.022
91.25	1122		0.035	
		15,406		0.019
101.48	751		0.016	
910.55		15,760***		0.016 0.160*



* MAXIMUM ALLOWABLE STORY DRIFT = 0.010H

** $F_x = \left[\Sigma F_{xm}^2\right]^{\frac{1}{2}}$ $\quad V_x = \left[\Sigma V_{xm}^2\right]^{\frac{1}{2}}$

$\Delta_x = \left[\Sigma \Delta_{xm}^2\right]^{\frac{1}{2}}$ $\quad \delta\Delta_x = \left[\Sigma \delta\Delta_{xm}^2\right]^{\frac{1}{2}}$

*** $C_b = V_b \div \Sigma W = 0.537$

MATHEMATICAL MODEL

$T_1 = 0.511$ Sec.

$T_2 = 0.144$ Sec.

$T_3 = 0.073$ Sec.

LONGITUDINAL DIRECTION

$W = 910.55 \times 32.2 = 29{,}320$ kips

NOTE: $\Sigma F_x \neq V_x$ DUE TO HIGHER MODE PARTICIPATION EFFECTS ON FORCES.

$\Sigma\delta\Delta_x \simeq \Delta_x$ BECAUSE HIGHER MODE PARTICIPATION EFFECTS ON DISPLACEMENT ARE NEGLIGIBLE FOR THIS BUILDING. (i.e. RSS DISPLACEMENTS ARE ESSENTIALLY EQUAL TO 1st MODE DISPLACEMENT.)

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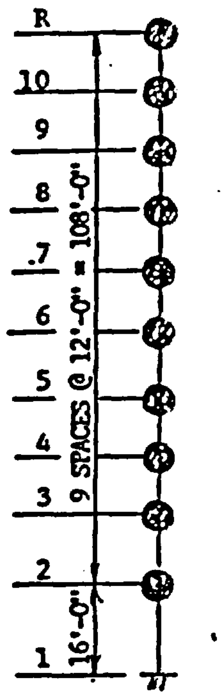
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 25 of 32)

EQ II STRUCTURAL RESPONSE - SRSS



MATHEMATICAL MODEL

T_1 = 0.352 Sec.

T_2 = 0.103 Sec.

T_3 = 0.054 Sec.

MASS (Kip-Sec./Ft.)	STORY LOAD F_x ** (kips)	STORY SHEAR V_x ** (kips)	DISPLACEMENT Δ_x ** (Feet)	STORY DRIFT $\delta\Delta_x$ ** (Feet)
90.02	3286		0.105	
		3286		0.01 0.120*
86.70	2655		0.093	
		5917		0.012
86.70	2449		0.081	
		8039		0.013
89.06	2058		0.069	
		9815		0.011
91.25	1909		0.057	
		11,363		0.011
91.25	1731		0.046	
		12,662		0.011
91.25	1571		0.035	
		13,724		0.010
91.25	1413		0.025	
		14,565		0.009
91.25	1163		0.016	
		15,182		0.008
101.48	824		0.008	
910.55		15,586 ***		0.008 0.160*

* MAXIMUM ALLOWABLE STORY DRIFT = 0.010H

** $F_x = \left[\Sigma (F_{xm})^2\right]^{\frac{1}{2}}$ $\qquad V_x = \left[\Sigma V_{xm}^2\right]^{\frac{1}{2}}$

$\Delta_x = \left[\Sigma (\Delta_{xm})^2\right]^{\frac{1}{2}}$ $\qquad \delta\Delta_x = \left[\Sigma \delta\Delta_{xm}^2\right]^{\frac{1}{2}}$

*** $C_b = V_b \div \Sigma W = 0.537$

TRANSVERSE DIRECTION

W = 910.55 x 32.2 = 29,320 kips

NOTE: $\Sigma F_x \neq V_x$ DUE TO HIGHER MODE PARTICIPATION EFFECTS ON FORCES.

$\Sigma\delta\Delta_x \simeq \Delta_x$ BECAUSE HIGHER MODE PARTICIPATION EFFECTS ON DISPLACEMENT ARE NEGLIGIBLE FOR THIS BUILDING. (i.e. RSS DISPLACEMENT ARE ESSENTIALLY EQUAL TO 1st MODE DISPLACEMENT.)

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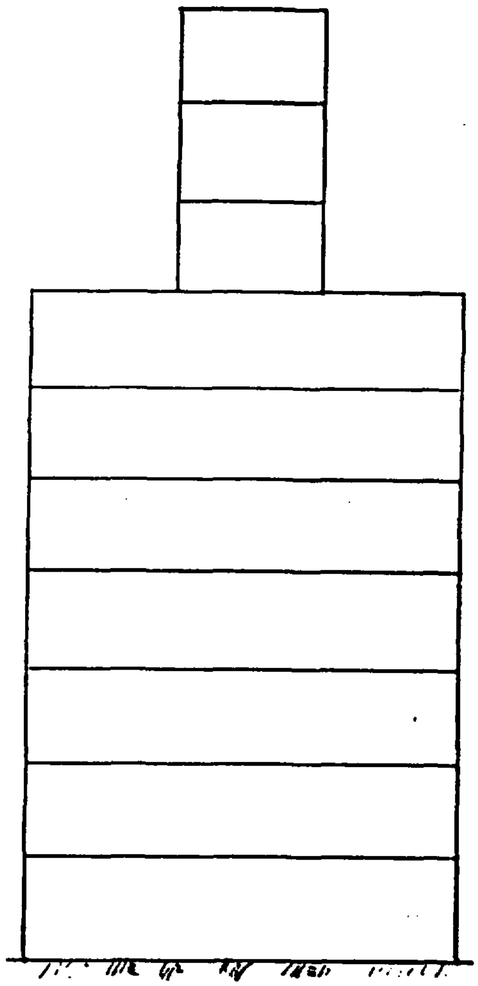
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 26 of 32)

EQ II ELEMENT FORCES: TRANSVERSE DIRECTION - WALLS 4 & 5

SEISMIC RESULTS FROM COMPUTER ANALYSIS.
UNITS ARE KIPS AND KIP-FT.

	SHEAR	MOMENT
Roof		
	329	
10th		3943
	690	
9th		12224
	1130	
8th		25759
	2778	
7th		58983
	2969	
6th		94367
	3132	
5th		131525
	3260	
4th		169990
	3335	
3rd		209139
	3361	
2nd		248405
	3269	
1st		299234

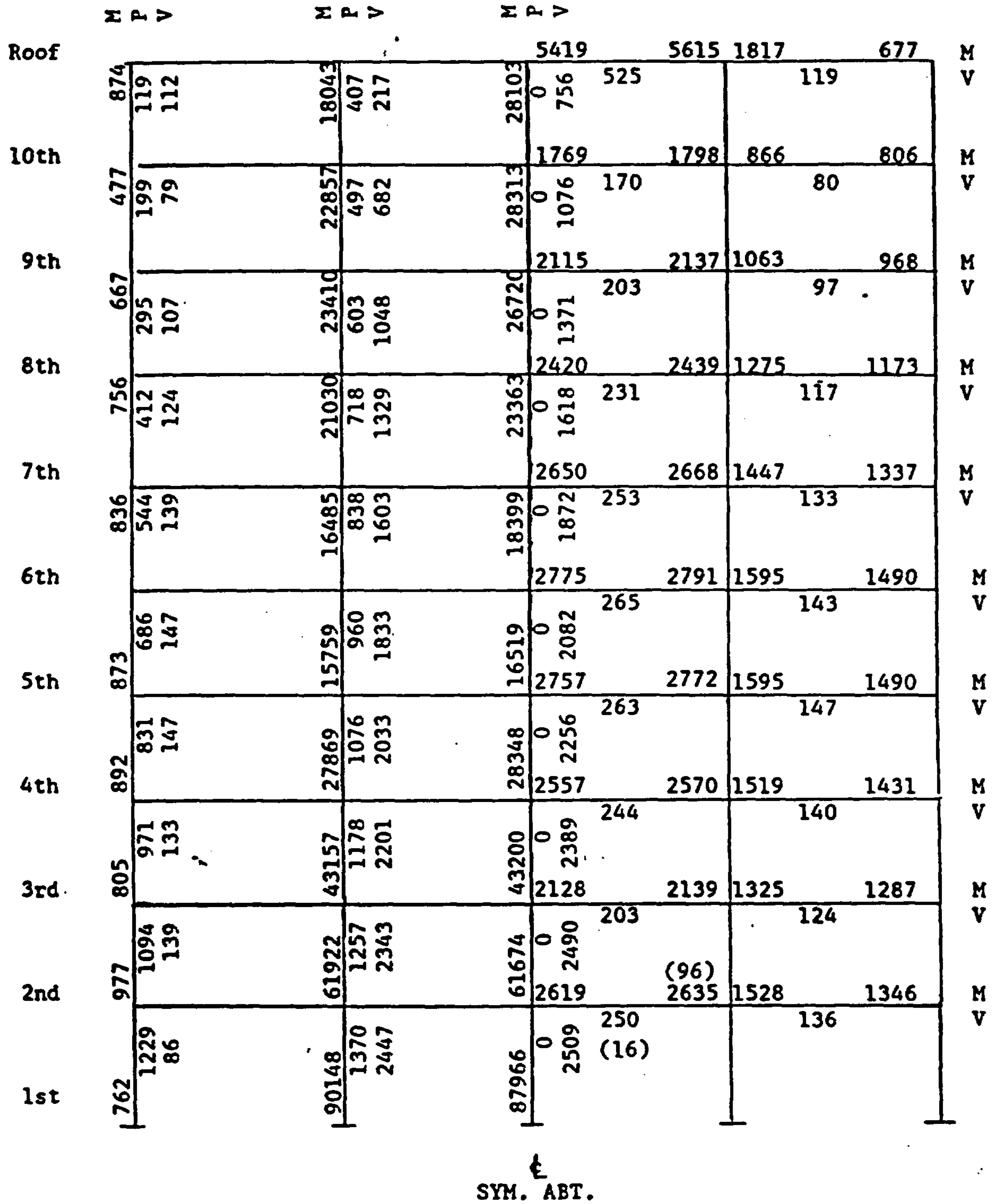


Sheet 27 of 32

Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 27 of 32)

EQ II ELEMENT FORCES: LONGITUDINAL DIRECTION - FRAME LINES A & D

SEISMIC RESULTS FROM COMPUTER ANALYSIS.
UNITS ARE KIPS AND KIP-FT.



(DL + ¼LL)

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 28 of 32)

"ACCIDENTAL" TORSION FORCES

The "accidental" torsion is the story shear, V_x, times the nominal eccentricity of 5% of the building dimension.

$$M_t = V_x \times 0.05 \times 190' = 9.5\ V_x$$

The story relative rigidity (K) of each shear element is obtained from the computer analysis.

$$\text{Torsional Shear} = \frac{Kd}{\Sigma Kd^2} \times 9.5\ V_x$$

$$\text{Direct Shear} = \frac{K}{\Sigma K} \times V_x$$

Distribution of Forces

9th Floor Level

SHEAR ELEMENT	REL K	d	Kd	Kd^2	DIRECT SHEAR	TORSIONAL SHEAR
1	19.99	94.5	1899	178516	$0.347V_T$	$0.043V_T$
2	0.35	67.5	24	1595	$0.006V_T$	$0.001V_T$
3	0.35	40.5	14	574	$0.006V_T$	$0.000V_T$
4	8.15	13.5	110	1485	$0.141V_T$	$0.002V_T$
5	8.15	13.5	110	1485	$0.141V_T$	$0.002V_T$
6	0.35	40.5	14	574	$0.006V_T$	$0.000V_T$
7	0.35	67.5	24	1595	$0.006V_T$	$0.001V_T$
8	19.99	94.5	1889	178516	$0.347V_T$	$0.043V_T$
	Σ = 57.68					
A	16.77	40.25	675	27168	$0.472V_L$	$0.015V_L$
B	1.00	15.25	15	233	$0.028V_L$	$0.000V_L$
C	1.00	15.25	15	233	$0.028V_L$	$0.000V_L$
D	16.77	40.25	675	27168	$0.472V_L$	$0.015V_L$
	Σ = 35.54		Σ =	419142		

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 29 of 32)

"ACCIDENTAL" TORSION FORCES

2nd Floor Level

SHEAR ELEMENT	REL K	d	Kd	Kd^2	DIRECT SHEAR	TORSIONAL SHEAR
1	29.66	94.5	2803	264871	$0.285V_T$	$0.043V_T$
2	0.27	67.5	18	1230	$0.003V_T$	$0.000V_T$
3	0.27	40.5	11	443	$0.003V_T$	$0.000V_T$
4	21.90	13.5	296	3991	$0.210V_T$	$0.005V_T$
5	21.90	13.5	296	3991	$0.210V_T$	$0.005V_T$
6	0.27	40.5	11	443	$0.003V_T$	$0.000V_T$
7	0.27	67.5	18	1230	$0.003V_T$	$0.000V_T$
8	29.66	94.5	2803	264871	$0.285V_T$	$0.043V_T$
	$\Sigma =$ 104.20					
A	24.84	40.25	1000	40242	$0.481V_L$	$0.015V_L$
B	1.00	15.25	15	233	$0.009V_L$	$0.000V_L$
C	1.00	15.25	15	233	$0.009V_L$	$0.000V_L$
D	24.84	40.25	1000	40242	$0.481V_L$	$0.015V_L$
	$\Sigma =$ 51.68		$\Sigma =$	622020		

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 30 of 32)

ELEMENT STRESS CHECK

Wall Lines 4 & 5

Neglect accidental torsional forces; less than 5% of the translational forces.

FLOOR LEVEL	V_D kips	V_u kips	$\frac{V_D}{V_u}$	SHEAR IDR	M_D ft-kips	M_u ft-kips	$\frac{M_D}{M_u}$	MOMENT IDR
8th	1130	1050	1.08	1.75	25729	15350	1.68	3.00
1st	3269	2710	1.20	1.75	299234	107080	2.79	3.00

Frame Lines A & D

Neglect accidental torsional forces; less than 5% of the translational forces.

At 1st Floor Level

MEMBER ELEMENT	V_D kips	V_u kips	$\frac{V_D}{V_u}$	SHEAR IDR	M_D ft-kips	M_u ft-kips	$\frac{M_D}{M_u}$	MOMENT IDR
Wall	2447	1420	1.72	1.75	90148	37740	2.39	3.00
Beam	266	281	0.95	1.75	2731	1712	1.60	1.75

Sheet 31 of 32

Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 31 of 32)

CONCLUSIONS

The structure, as modified by the upgrading concept, will conform to the acceptance criteria for EQ-II forces; however a verification of soil capacities will be required as stated on sheet 21. It should also be noted that the detailed analysis of the existing structure (without modifications) indicates that the building has good overall performance characteristics, will remain essentially elastic for EQ-I, and would satisfy acceptance criteria for an earthquake slightly smaller than EQ-II (refer to sheet 16). Because this building is not an essential or high risk facility, the need for upgrading would be set at a relatively low priority as a result of formulating a decision by means of a cost-benefit analysis.

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 32 of 32)

APPENDIX G

DESIGN EXAMPLES—NONSTRUCTURAL

G-1. Introduction

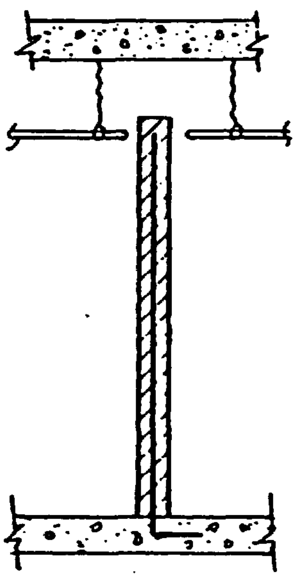
The design examples in this appendix are to illustrate principles, factors, and concepts described in this manual for the anchorage or bracing of nonstructural elements in buildings.

G-2. Design examples

The following design examples are representative of typical nonstructural elements supported on the roof or on a floor of any building. The various examples illustrate the procedures for evaluation and upgrading rigid and flexibly mounted equipment and nonstructural partitions and light fixtures. Following is a listing of the design examples.

Fig. No.	*Description of Design Examples*
G-1	Nonstructural partition: illustrates seismic evaluation and method of bracing partitions.
G-2	Unit heater—supported from structural framing: evaluates existing flexible support and upgrades support by providing rigid brace.
G-3	Light fixture: illustrates evaluation and upgrading procedure for existing light fixtures.
G-4	Tank system bolted to floor of building. Evaluates anchorage to floor system and provides bracing system.
G-5	Piping system: Provides seismic bracing for existing piping system.

GIVEN: NONSTRUCTURAL CONCRETE BLOCK PARTITIONS IN A 10-STORY REINFORCED CONCRETE FRAME BUILDING. REFER TO LONGITUDINAL DIRECTION IN FIG. F-4 FOR BUILDING PROPERTIES. PARTITIONS ARE NOT IN CONTACT WITH ANY STRUCTURAL ELEMENTS OTHER THAN FLOOR SLAB. THEY EXTEND 4" ABOVE 8-FOOT CEILING AND THE TOP OF THE BLOCKS ARE ROUGHLY 1'-8" SHORT OF THE SOFFIT OF THE FLOOR ABOVE. THE WALLS ARE 8" NOMINAL THICKNESS WITH NOMINAL REINFORCING (E.G., #4 AT 4'-0 CTRS EACH WAY)



ESTIMATE PERIOD

$$\frac{1}{T} = f = 0.56\sqrt{\frac{EI}{mL^4}} = 15.3\ Hz$$

$E = 1.5 \times 10^6$ psi
$I = 500\ in^4$ (1-FT STRIP)
$W \simeq 50^{\#}/1 \simeq 4^{\#}/in$
$L = 8'\text{-}4" = 100"$
$m = W/386 = 0.010\ \frac{\#\text{-}Sec^2}{in^2}$

$T = \frac{1}{15.3} = 0.065$ Sec (CANTILEVER WALL)

THIS IS RELATIVELY RIGID COMPARED TO BUILDING PERIOD: $T_1 = 0.80$ SEC
$T_2 \simeq 0.26$ SEC
$T_3 \simeq 0.14$ SEC

THEREFORE, ASSUME RIGID AND USE PEAK FLOOR ACCELERATION FOR PRELIMINARY CHECK.

PEAK BLDG RESPONSE: (LONG DIRECTION CONCRETE FRAME)

MODE	1	2	3	
T	0.80	0.26	0.14	
Sa	0.432	0.70	0.70	
a_q	0.54	0.25	0.08	(9TH FLR)

RSS AT 9TH FLOOR = $\sqrt{\Sigma a_q^2} = 0.60g$ (ACCELERATION)

Sheet 1 of 3

Figure G-1. Nonstructural partition. (Sheet 1 of 3)

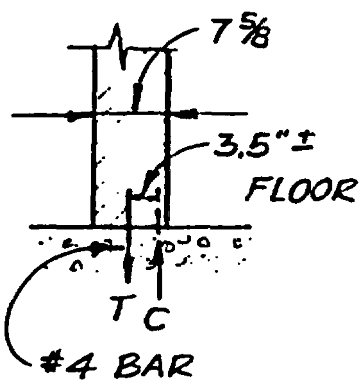
CHECK CANTILEVER WALL

FOR "RIGID" WALL CHECK FOR 0.60 g

$$m = \frac{WL^2}{2} = \frac{0.60(4 \times 50)(8.33)^2}{2} = 4160^{\#-\prime}$$

(4' WIDTH @ 0.60 g)

ESTIMATE TENSION IN #4 RE-BAR



$$T \approx \frac{m}{a} \approx \frac{4160 \times 12}{3.5} = 14,200^{\#}$$

$$\sigma = \frac{T}{AREA} = \frac{14.2^{K}}{0.20 \,\square''} = \underline{71 \text{ Ksi}}$$

N.G.

BRACE PARTITION

ESTIMATE INTER STORY DISPLACEMENTS

WORST CASE: BETWEEN 2ND & 3RD FLOORS (BY INSPECTION OF BLDG CALCS)

MODE	1	2	3	
Sd(in)	2.72"	0.45"	0.14"	
Δ_3	1.33	0.20	0.03	
Δ_2	0.84	0.15	0.03	
$\Delta_3 - \Delta_2$	0.49	0.05	0.00	RSS = 0.49"

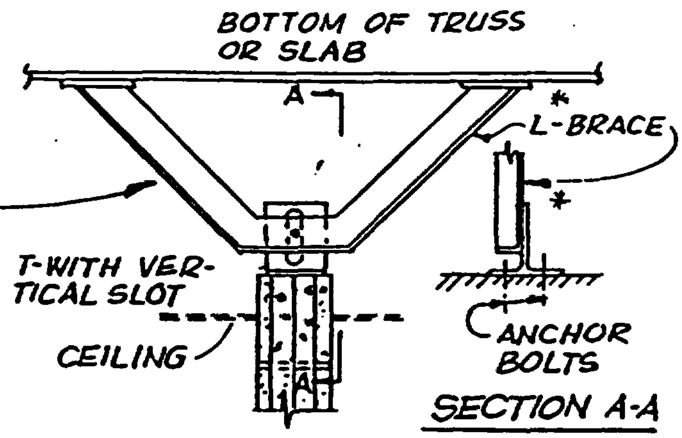
MAXIMUM INTERSTORY DRIFT

Sheet 2 of 3

Figure G-1. Nonstructural partition. (Sheet 2 of 3)

DESIGN BRACE

TRY DETAIL SHOWN IN BDM, FIG 9-1



OVERHEAD BRACING

$F_p = 0.60\ W_p$

$W_p = 4' \times 50\#/\square'$
$= 200\#/FT$
TRIB. TO TOP OF WALL

$f_p = 0.60 \times 200$
$= 120\#/FT$

SPACE BRACES AT 8-FEET CENTERS

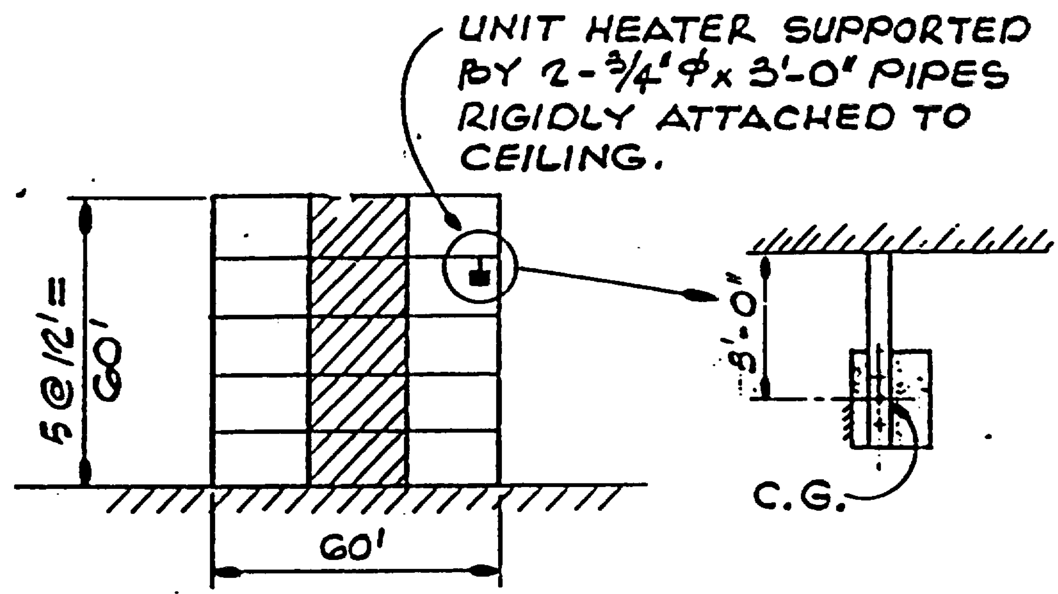
$F_p = 8 \times 120 = 960\#/BRACE$

* NOTE ON BOLTED CONNECTIONS:

USE WASHERS AS SPACERS BETWEEN BRACE AND SLAB AND BETWEEN BRACE AND SLOTTED T. THIS WILL ALLOW FOR SOME INTERSTORY DRIFT PARALLEL TO WALL.

Sheet 3 of 3

Figure G-1. Nonstructural partition. (Sheet 3 of 3)



GIVEN: NEGLECT EFFECTS OF ROTATION OF UNIT HEATER.

W_p = WT. UNIT HEATER = 350 LBS
w_x = WT. TYPICAL FLOOR = 500 KIPS
W = WT. STRUCTURE = 2300 KIPS
I (OCCUPANCY) = 1.0
ZONE 3 SEISMIC AREA
I_o (3/4"φ PIPE) = 0.037 IN^4
E (PIPE) = 30×10^3 KIPS/IN^2.

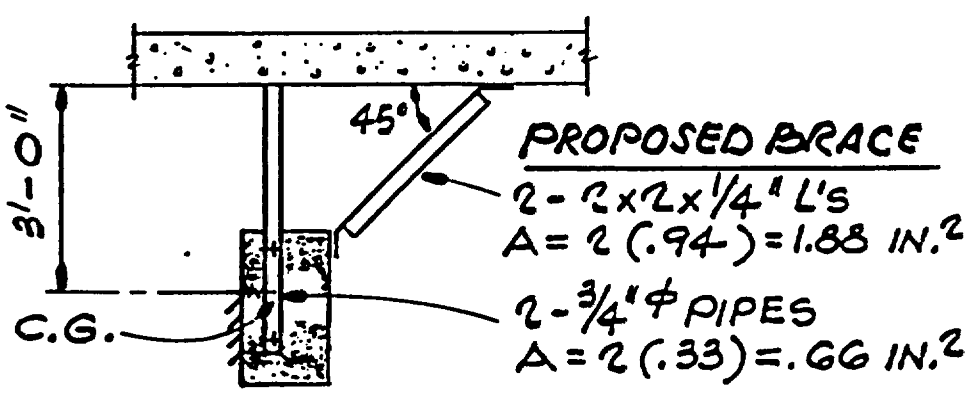


Figure G-2. Unit heater. (Sheet 1 of 2)

GIVEN: UNIT HEATERS SUPPORTED BY 2 - 3/4" PIPES, 3'-0" LONG, RIGIDLY ATTACHED TO SLAB SOFFIT.

THE UNITS ARE LOCATED IN A 7-STORY BUILDING.

THE EVALUATION OF THE RESPONSE TO EQ-I AND EQ-II ARE SHOWN IN FIGURE F-2 IN THE SDG.

EQ-II FORCES EQUAL 501 LBS.

THIS EXCEEDS THE CAPACITY OF THE PIPE SUPPORTS TO RESIST LATERAL FORCES. (SEE BDM DESIGN EXAMPLE F-5)

$$f = \frac{MC}{I} = \frac{1/2 \times (501^{\#} \times 36") \times 3/8"}{0.037\ IN^4} = 91{,}000\ \underset{NG}{PSI}$$

THEREFORE, BRACE THE SUPPORT

THE SOLUTION FOR DESIGNING THE BRACE IS SHOWN IN SDG FIG. F-3

DESIGN THE BRACE FOR A FORCE EQUAL TO 215 LBS

Sheet 2 of 2

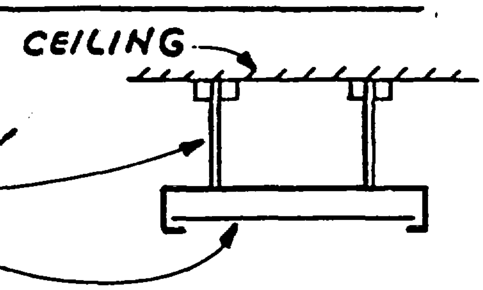
Figure G-2. Unit heater. (Sheet 2 of 2)

LIGHT FIXTURES IN EXISTING BUILDING

GIVEN:

FIXTURES SUPPORTED BY METAL RODS, RIGIDLY ATTACHED TO CEILING

LENSES ARE GLASS



EVALUATION

LATERAL FORCE DUE EQ II IN THE NEIGHBORHOOD OF 0.60 x W_F
(REFER TO FIG G-1 FOR ACCELERATIONS)
PRESENTS HAZARDS:

1. FAILURE OF METAL RODS
2. POSSIBILITY OF GLASS BREAKAGE

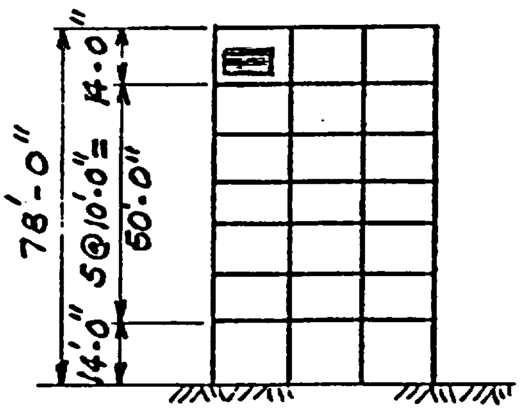
SOLUTION

ALTERNATE #1 REPLACE WITH NEW FIXTURES

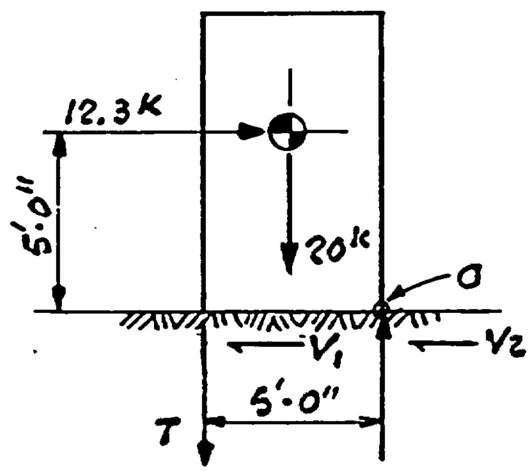
ALTERNATE #2

- REPLACE GLASS WITH PLASTIC LENSES
- ADD SUPPLEMENTARY CABLES AS BACK-UP IN CASE OF ROD FAILURE

Figure G-3. Light fixtures



BUILDING



TANK

GIVEN:

WT. OF TANK = 20 KIPS
WT. OF FLOOR = 1460 KIPS
CONSIDER TANK RIGIDLY MOUNTED TO FLOOR SLAB.
EQ. II FORCES = 12.3 K
(REFER TO SDG FIG. F-1 FOR DETERMINATION OF FORCES)
EFFECTIVE WIDTH OF BASE = 5'

REQUIRED:

FORCES ON ANCHORAGES

$$\Sigma M_o = (5 \times 12.3) - (2.5 \times 20) - 5 \times T = 0$$

$T = 2.3$ KIPS UPLIFT

$V_1 + V_2 = 12.3$ K

IF $V_1 = V_2$, $V_1 = 6.2$ K

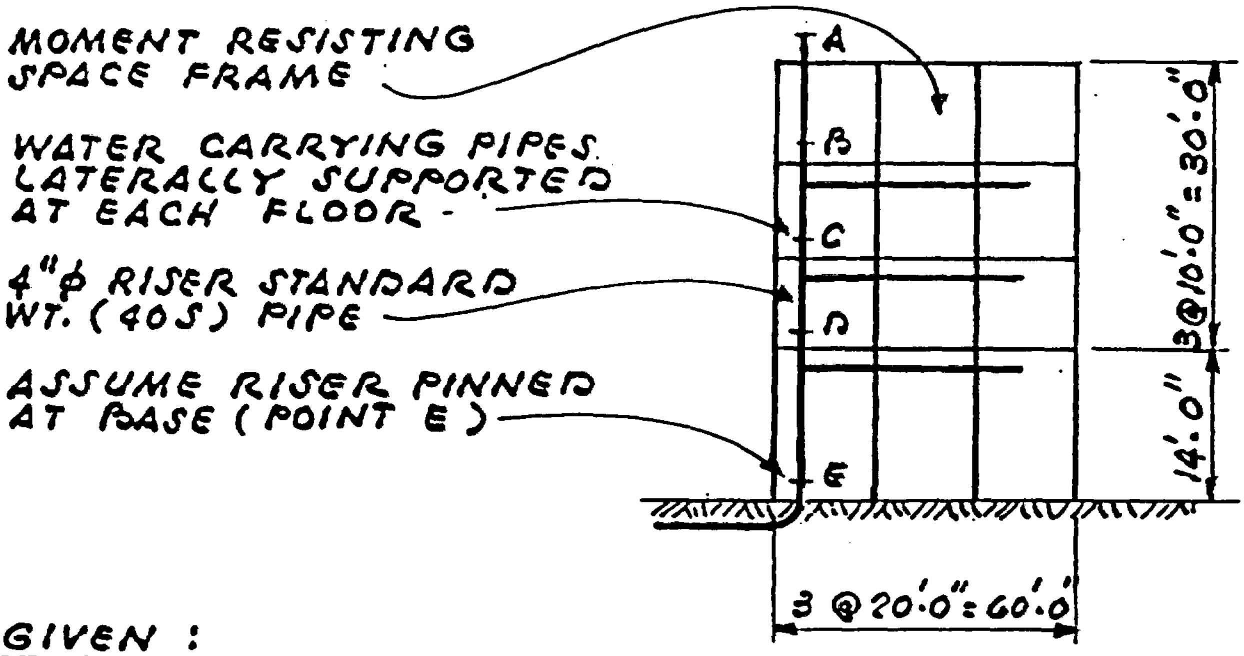
IF UPLIFT OCCURS, ASSUME TOTAL SHEAR TAKEN BY V_2

DESIGN ANCHOR BOLTS AND SUPPORT

1- 2.3 KIPS TENSION AND 6.2 KIPS SHEAR.

2. 0 - TENSION 12.3 SHEAR

Figure G-4. Tank system bolted to floor of building



GIVEN :

PIPING AS SHOWN IN 3-STORY STEEL MOMENT RESISTING SPACE FRAME.
PEAK FLOOR ACCELERATION = 0.60 g
PEAK INTERSTORY DRIFT = 1.2 IN

PIPING SYSTEM CONSIDERED RIGID
(REFER TO BDM DESIGN EXAMPLE F-9)

WT. OF PIPE AND CONTENTS = 16.3 #/FT.

MAXIMUM LATERAL FORCE AT PIPE SUPPORTS:

$$F_P = \left(\frac{14+10}{2} \times 16.3\right) 0.60 = 117 \text{ LBS.}$$

INTERSTORY DRIFTS :

1.2 IN. RELATIVE DISPLACEMENT IN 10 FT. CAN EXCEED DEFORMATION CAPACITY OF 4" PIPE ; THEREFORE ;

1. SUPPLY FLEXIBLE SUPPORTS AT B, C AND D OR
2. SUPPLY FLEXIBLE JOINTS .

Figure G-5. Piping system

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