Chapter 12

Seismic Upgrading of Existing Structures

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Key words: Structural Deficiencies, Structural Design, Seismic Rehabilitation, Reinforced Concrete, Steel, Wood, Performance Objectives, Prescriptive Requirements, Seismic Demand, Seismic Capacity, Strength, Stiffness, Flexibility, Drift, Deformation, Load Paths.

Abstract: This chapter presents important considerations for engineers upgrading the seismic resistance of existing structures including investigation of existing structural characteristics, identification of significant deficiencies, and selection of appropriate upgrade criteria and retrofit systems. In addition to all of the tasks required in design of a new structure, successful seismic upgrade of an existing structure requires development of a thorough understanding of the existing construction, research into its limiting strength and deformation characteristics, quantification of the owner's economic and performance objectives, and selection of an appropriate design criteria to meet these objectives, which is also acceptable to the building official. It also includes selection of retrofit systems and detailing which can be installed within the existing structure (which may have to remain open during the upgrade) at a practical cost and with minimum impact on building appearance, function and historic features. This chapter is organized into six sections. The differences between the seismic design philosophy for a new building and that for the upgrade for an existing building are discussed first followed by discussions on seismic deficiencies commonly found in buildings, the importance of establishing a rational seismic upgrade criteria, upgrade methods to mitigate common seismic deficiencies, and two example seismic upgrade projects. Since performance based design techniques are presented in a separate chapter of this handbook, we limit ourselves here to coverage of more traditional approaches to seismic rehabilitation.

12.1 INTRODUCTION

As compared to seismic upgrade of existing structures, design of a new structure for proper seismic performance is a relatively simple and straight-forward task. Modern building codes for new construction rigorously prescribe the design procedures to be employed based on intended building occupancy and performance and extensive research and data on seismic performance of the materials and detailing specified. The engineer designing a new structure has the opportunity to select the basic structural system and specify the materials and detailing incorporated. The engineer can participate in developing the structure's configuration and the placement of structural elements. Finally, the engineer for a new building has the opportunity to require inspection of important aspects of the construction and to confirm the quality of materials and workmanship incorporated. As a result, most structural characteristics important to seismic performance including ductility, strength, deformability, continuity, configuration and construction quality, can be controlled.

Seismic rehabilitation of existing structures presents a completely different problem. First, for most types of structures, up to very recently, there was no clear professional consensus on appropriate design criteria. That of course has changed substantially by publication of performance based design guidelines such as the FEMA 273/274[12-1, 12-2] and the ATC-40[12-3] guidelines (see Chapter 15 for application of these guidelines in seismic rehabilitation). The building codes for new construction are based on the use of modern materials and detailing, and are not directly applicable. Further, they incorporate levels of conservatism and performance objectives which may not be appropriate for use on existing structures due to economic limitations. The material strengths and ductility characteristics of an existing structure, will in general not be well defined. The configuration and materials of construction are predetermined. The details and quality of construction are frequently unknown and because the structure has been in service for some time, deterioration and damage are often a concern.

In addition to all of the tasks required in design of a new structure, successful seismic upgrade of an existing structure requires development of a thorough understanding of the existing construction, research into its limiting strength and deformation characteristics, quantification of the owner's economic and performance objectives, and selection of an appropriate design criteria to meet these objectives, which is also acceptable to the building official. It also includes selection of retrofit systems and detailing which can be installed within the existing structure (which may have to remain open during the upgrade) at a practical cost and with minimum impact on building appearance, function and historic features.

This chapter presents important considerations for engineers upgrading the seismic resistance of existing structures including investigation of existing structural characteristics, identification of significant deficiencies, and selection of appropriate upgrade criteria and retrofit systems. The chapter is organized into six sections. The differences between the seismic design philosophy for a new building and that for the upgrade for an existing building are discussed in the following section, Section 12.2. Seismic deficiencies commonly found in buildings are then discussed in Section 12.3. The importance of establishing a rational seismic upgrade criteria is presented in Section 12.4. Upgrade methods to mitigate common seismic deficiencies are then discussed in Section 12.5. The last section, Section 12.6, contains two example seismic upgrade projects. Since performance based design techniques are presented in a separate chapter of this handbook, we limit ourselves here to coverage of more traditional approaches to seismic rehabilitation.

12.2 PURPOSE OF SEISMIC STRENGTHENING

Many structural engineers believe that the purpose of seismic strengthening is to upgrade the structure, to the maximum extent practical, into conformance with the lateral force requirements of the current building code. In reality this is not the purpose of seismic strengthening, but instead a method for achieving seismic upgrade, and often an inappropriate one.

As stated by the Structural Engineers Association of California(12-1) (SEAOC), the purpose of earthquake resistance provisions incorporated into the building codes is *to maintain public safety in extreme earthquakes likely to occur at the building's site*. Such provisions are intended to *safeguard against major failures and loss of life, not to limit damage, maintain functions, or provide for easy repair*. Specifically, it is expected that buildings designed to conform with the provisions of the building code would be able to:

– Resist a minor level of earthquake ground motion without damage;
– Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some non-structural damage;
– Resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as non-structural damage.

These performance objectives were specifically formulated by SEAOC to apply to a broad range of structures and occupancies, based on trade-offs between public safety and economics. They were intended to apply to the general population of structures likely to be constructed and were specifically formulated under the influence of the seismicity of California, a region subject to frequent moderate magnitude earthquakes and occasional great earthquakes. These objectives can be reasonably attained in the design of new structures by carefully conforming to four basic sets of provisions specified by the code: *strength, materials selection, structural detailing, and construction quality*.

12.2.1 Seismic Strengthening Considerations

Since current building codes do not in general apply to existing structures, the implicit performance objectives of these codes need not be rigidly adhered to for seismic upgrades. It is therefore extremely important that the structural engineer work with the building owner to carefully define the intended purpose of seismic strengthening based on specific safety and economic performance objectives. These are likely to vary considerably from one structure to another based on several key factors. These factors include:

– Economic value of the structure and remaining years of service life.
– Occupancy of the structure including the number of persons at risk within the structure, as well as the potential for structural failure to result in release of hazardous substances and injuries outside the structure.
– Function of the structure and the economic or societal cost which would result from loss of service due to earthquake induced damage.
– Historic significance of the structure and the effects of seismic upgrades on the cultural resource.
– The site-specific seismic hazard.
– The relative cost of achieving upgrades to various criteria.

As an example, most people would agree that it is not appropriate to upgrade an unoccupied warehouse to the same level of reliability as a building with high occupancy. Similarly, a building expected to remain in service for 10 years need not have the same level of reliability as a building expected to provide service for 100 years. Reconciliation of these complex issues requires both qualitative and quantitative evaluation. Selection of appropriate design criteria cannot be made until these evaluations have been performed.

12.2.2 New Design Versus Retrofit Design Approaches

The basic design procedure for new structures consists of the selection of an appropriate level of lateral forces for design purposes, and then providing a complete, appropriately detailed, lateral force resisting system to carry these forces from the mass levels to the foundations. Deformations are checked as a secondary issue, and except for the design of flexible structures, they are not likely to control the design.

Deformation control can be relegated to a secondary consideration in the design of many new structures to code life-safety requirements because the modern materials and ductile detailing practices specified by present codes allow new structures to experience large deformations while experiencing limited damage. Older structures, however do not have the advantage of this inherent ductility. Therefore, control of deformations becomes an extremely important issue in the design of seismic retrofits. Given a ground motion criteria, and the desired performance level for that ground motion, the real task of seismic retrofit becomes one of controlling structural deformations, in response to that ground motion, to within acceptable levels. If the objective is to avoid collapse, then deformations must be controlled to an extent where stability of the vertical load carrying system is not lost. If post-earthquake functionality is the objective, then deformations must be controlled to an extent where unrecoverable cracking and bending of structural (and non-structural) elements is small enough to avoid the cosmetic appearance of an unsafe structure. This limited deformation level is necessary to ensure continued operation. Following a major earthquake, municipal building inspectors (with the assistance of local structural engineers) will perform a rapid screening assessment and make judgments as to which buildings are obviously unsafe, which are obviously safe, and which require further evaluation to ascertain whether the buildings are safe or not. Unless the building is tagged as obviously safe the local government may limit the use of the building until it can be proven safe.

There are three primary types of deformations which must be considered and controlled in a seismic retrofit design. These are: global deformations, elemental deformations and inter-structural deformations.

Global deformations are the only type explicitly controlled by the building codes and are classically considered by reviewing inter-story drift (see Chapter 7). The basic concern is that large inter-story drifts can result in P-delta instabilities. Control of inter-story drift can also be used as a means of limiting damage to non-structural elements of a structure (fascia, partitions, ceilings, utilities, etc.). It is less effective as a means of limiting damage to individual structural elements.

Elemental deformations are the amount of distortion experienced by an individual element of a structure such as a beam, column, shear wall, or diaphragm. Building codes have very few provisions to directly control these deformations. They rely on ductility to ensure that individual elements will not adversely fail at the global deformation levels predicted for the structure. In existing structures, with questionable ductility, it is critical to evaluate the deformation of each element and to ensure that expected damage to the element, at the given deformation level, is acceptable. This requirement extends to elements normally considered to participate in the lateral force resisting system as well as those that do not. For example, a common mode of collapse for older concrete structures is a punching shear failure of flat slabs at interior columns (Figure 12-1). This results from excessive rotation plus vertical accelerations (and induced punching shear concentrations) at the slab-column joint. Often, the flat system is not considered to participate in the lateral force resisting system for a retrofitted structure. However, if the

Figure 12-1. Example of slab punching shear failure-January 17,1994, Northridge earthquake

rotational deformation of these joints is not maintained below a damage threshold, the classic punching shear failure can still occur. Elemental deformations can sometimes be controlled by limiting calculated member stresses at realistic estimates of global structural deformation.

Inter-structural deformations are those that relate to the differential movement between elements of the structure. Failures which result from a lack of such control include classic failures of masonry walls which have not been anchored to diaphragms (Figure 12-2) or failures resulting from bearing connections slipping off beam seats. Building codes control these deformations by requiring interconnection of all portions of structures and the provision of continuity ties. These same "code" techniques can be effective as retrofits for an existing structure. However, in some cases provision of continuity is not practical (for example at an expansion joint of a structure). In such cases, realistic estimate of expected deformations and ensuring that stability is maintained at these deformation levels is the most effective design procedure.

12.2.3 Realistic Seismic Deformations

Determination of the realistic deformation levels expected of a structure, when subjected to the design earthquake, is the most important and also most difficult task of seismic rehabilitation design. The seismic design provisions contained in modern American building codes including the UBC-97[(12-5)] and IBC-2000[(12-6)] are all based on analysis methodologies originally presented in ATC-3-06[(12-7)].

Figure 12-2. Example of masonry wall separation – October 17,1989, Loma Prieta earthquake

The ATC-3-06 methodologies rely on elastic dynamic analysis techniques with an input ground motion that has been substantially reduced from that actually expected to be experienced by the building. This reduction factor (*R*) used to be as large as 12 but currently it is as large as 8 (see Chapter 4). The forces obtained from the elastic dynamic analysis using this substantially reduced ground motion are then used to proportion the elements of the structure. However, it is explicitly recognized that the structural deformation levels predicted by such analyses are substantially smaller than what will be experienced by the real building. All codes, therefore, specify that deformation-critical aspects of the design, such as building separations and detailing of non-structural attachments, be evaluated at amplified deformation levels (see Chapters 4 and 7). It is this amplified level of deformation rather than the deflections predicted by the code base shear forces that should be used for evaluating the adequacy of existing structural elements in a retrofitted structure.

It should be noted that even the use of amplified elastic deformations as an indication of real inelastic deformations of the structure is at best an approximation. The basis for this approach is founded in analytical research presented in a monograph by Newmark and Hall[12-8]. That research indicates that the maximum deflection (elastic plus inelastic deflection) of a structure can be predicted by the theoretical response of an elastic structure with the same initial dynamic properties.

The Newmark and Hall[12-8] basic analytical research was conducted for very simple, single degree of freedom structures only, as opposed to the complex multi-story, multi-degree of freedom structures commonly encountered in practice. Naeim and Anderson [12-9] have shown that this assumption seems to be generally, but not always, valid for regular tall building structures. The profession seems to have reached a general consensus, however, that this assumption is also valid for other structures as a method of estimating inter-story drifts, providing that several limitations are observed:

1. The deformation levels are well under the range of overall stability of the structure.
2. The structure is reasonably regular with regard to stiffness and mass distribution. Soft and/or weak stories can result in substantially different inelastic deformation distributions from those predicted by elastic analyses. Inelastic torsional instabilities can have similar effects.
3. Throughout the range of deformations experienced, the structure does not experience a net loss of lateral force resisting capacity. Ductile structures will become softer as they are pushed into the range of inelastic response. However, they will continue to retain their plastic lateral force resisting capacity, and as they strain harden, will actually become somewhat stronger. Non-ductile structures, such as many older concrete and masonry structures will experience a loss of strength resulting from spalling of compressive material and slippage in tensile elements.

When designing new structures, the building codes provide proscriptive guidance to ensure that the above assumptions are valid. Global drifts are controlled to maximum levels to satisfy the first assumption. Severe soft and weak story conditions are specifically prohibited and torsional effects are carefully evaluated to cover the second. The use of ductile detailing ensures that the third assumption is valid. In designing seismic retrofits for existing structures, it is equally important to ensure that these same assumptions are valid for the combined system of the existing structure and retrofit system.

In addition to the above, the use of elastic estimates of real earthquake deformations also has other limitations. Although the total deformation of the structure may be bounded by these techniques, it is feasible that the distribution of inelastic deformation throughout the structure is not well predicted by elastic analysis. As an example, elastic analysis of a cantilevered shear wall structure will indicate nearly uniform inter-story deformation over the height of the structure (Figure 12-3a). Direct application of the Newmark approach would lead the designer to believe that the inelastic response of the structure would also be distributed uniformly over the structure's height. In reality, however, properly designed shear wall structures become inelastic by developing a flexure hinge at the base of the wall, resulting in a concentration of inelastic behavior in the lower stories (Figure 12-3b). For such structures, the distribution of inelastic deformation is poorly predicted by this approach.

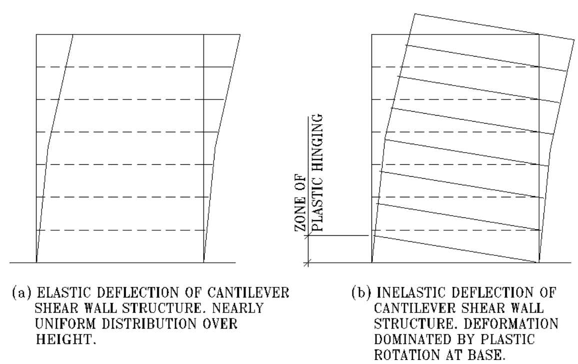


Figure 12-3. Comparison of elastic and inelastic deformations distributions

Nonlinear Analysis Techniques - As an alternative to using the code approach of amplified elastic response for estimating maximum expected deformations, direct calculation of these deformations through the use of non-linear dynamic analysis techniques is also possible and has become increasingly popular (see Chapter 15). Software systems for nonlinear static and dynamic analysis of structures are becoming increasingly available in the design office environment (see Chapter 16). Use of such techniques is required for design of certain types of seismic force resisting systems including certain classes of base isolation and energy dissipation systems and may also be appropriate for some conventional structures.

The principal advantage of nonlinear analysis techniques is that they allow direct calculation of inelastic response including the effects of any inherent hysteretic damping of the structure. To the extent that assumptions with regard to the non-linear force-deformation characteristics of the elements incorporated in the model are correct, the deformation pattern calculated by these techniques are more consistent with the real structural behavior, and can indicate the "real" distribution of inelastic deformations within the structure. However, the validity of results obtained from this approach is highly dependent on the assumptions of element properties, and in the case of time-history analysis, the appropriateness of the ground motion time histories used. Most designers using this technique attempt to conservatively estimate responses, by altering the assumptions used on element properties, and by evaluating the response to multiple time histories.

Quasi-inelastic analysis approaches are also available which permit evaluation of complex structures. The most common of these is the so-called "progressive yield" or "static pushover" analysis. A simple way to use this approach is to start with an elastic model of the structure which is analyzed for a static distribution of lateral forces. Stresses within the structure are evaluated and zones of yielding identified. The elastic model is then modified by placing "hinges" and "reduced stiffness" elements at locations of computed yielding. The revised model is then re-analyzed statically for additional static lateral forces. This process is repeated until the total structural deformation required by design criteria is attained or the structure is found to become unstable (see Chapter 15 for more information).

Regardless of the technique utilized, in order to properly understand the seismic behavior of an existing structure, it is critically important to understand the likely distribution of deformations throughout the structure under the criteria earthquake ground motion. One should recognize that deformations are likely to be substantially larger and differently distributed than is predicted by a direct elastic analysis to code specified forces.

12.3 COMMON DEFICIENCIES

This section describes typical deficiencies found in existing construction which can lead to poor earthquake performance. For the purposes of this section, poor earthquake performance is defined as endangerment of life safety through either partial or total collapse. As previously discussed, for some types of structures and occupancies it may be desirable to obtain better performance than merely protection of life safety. To obtain such performance, it is necessary to mitigate each of the deficiencies discussed in this section, as well as to ensure that expected earthquake induced deformations are kept small enough to prevent significant damage to key elements of the structure.

Until recently, there has been little consensus in the engineering profession as to appropriate methods for determining if an existing structure is seismically hazardous. Some engineers have attempted to apply the current building codes as evaluation tools for existing structures. The problem with this approach is that since the codes are revised every few years, most existing buildings will not meet the current code to some extent, a few years down the road. This would result in a finding that nearly every building is hazardous and requires upgrade. Such a finding is obviously both technically incorrect and economically not feasible to manage.

One of the most seismically hazardous class of buildings common throughout the world are structures constructed with load bearing walls of unreinforced masonry. A significant amount of research has been performed in recent years on the performance of these buildings and effective methods of improving their seismic performance. Much of this research was published as the ABK Methodology(12-10). Portions of these documents have since been adapted and placed into a code form as an appendix to the Uniform Code for Building Conservation(12-11). The procedures of these documents can be a useful guideline for the rehabilitation of masonry bearing wall structures.

A number of more general-purpose evaluation guidelines have also been recently published on the subject of seismic evaluation. These include, Rapid Visual Screening of Buildings for Potential Seismic Hazards(12-12) and the NERHP Handbook for Seismic Evaluation of Existing Buildings(12-13). The first of these is a method of rapidly determining the probability of earthquake induced failure of a building, based on identification of building type, age, configuration, condition and local site characteristics. Few calculations are performed in this method and it should be used only to obtain a preliminary indication as to whether more detailed evaluation of a structure is justified. The second publication is intended to provide detailed evaluation guidelines. It provides in-depth checklists and calculation procedures developed for different building types, which may be used to identify key seismic deficiencies present in an existing building.

Both the rapid screening and detailed evaluation methodologies are based on the observation that most earthquake induced building collapses can be attributed to several fundamental flaws. These are briefly identified in this section. The reader is referred to the references 12 and 13 for more detailed procedural guidance. The reader is also cautioned, that both references 12 and 13 are keyed to a specific ground motion criteria, (a median estimate of the strongest level of ground shaking likely to effect a site in any 500 year period). In addition, the NEHRP document is intended to identify life safety hazards only. In many cases, depending on the performance desired of a particular structure, it may be necessary to modify the evaluation criteria contained in these documents to utilize more (or less) severe ground motions and to incorporate more (or less) restrictive deformation limits.

Incomplete Lateral Force Resisting System: One of the most common causes of earthquake-induced collapse is the lack of a complete lateral force resisting system. In order to successfully resist collapse, each element of a structure must be positively connected to the whole in such a manner that inertial loads generated by the element from motion in any direction can be transmitted back to the ground in a stable manner.

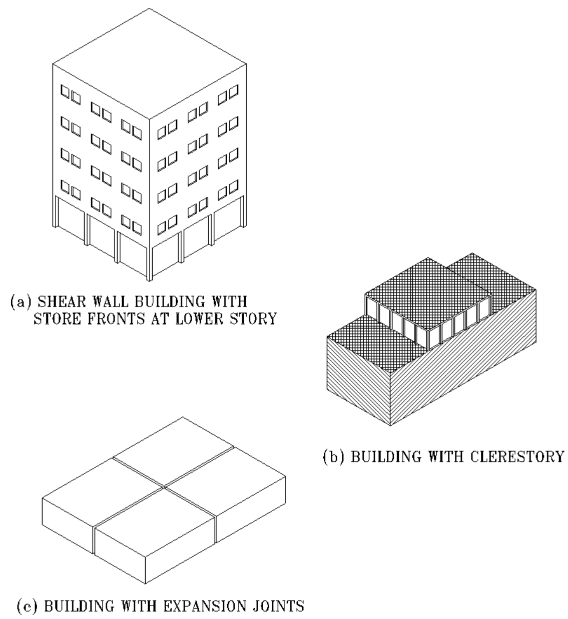


Figure 12-4. Building types in which incomplete lateral force resisting systems are common

As a minimum, a complete lateral force resisting system will include at least three non-concurrent vertical lines of lateral force resisting elements (moment frames, braced frames or shear walls) and at each level of significant mass a horizontal diaphragm to interconnect these vertical elements. Together, this assemblage of elements must provide adequate rigidity to control structural deformations to tolerable levels.

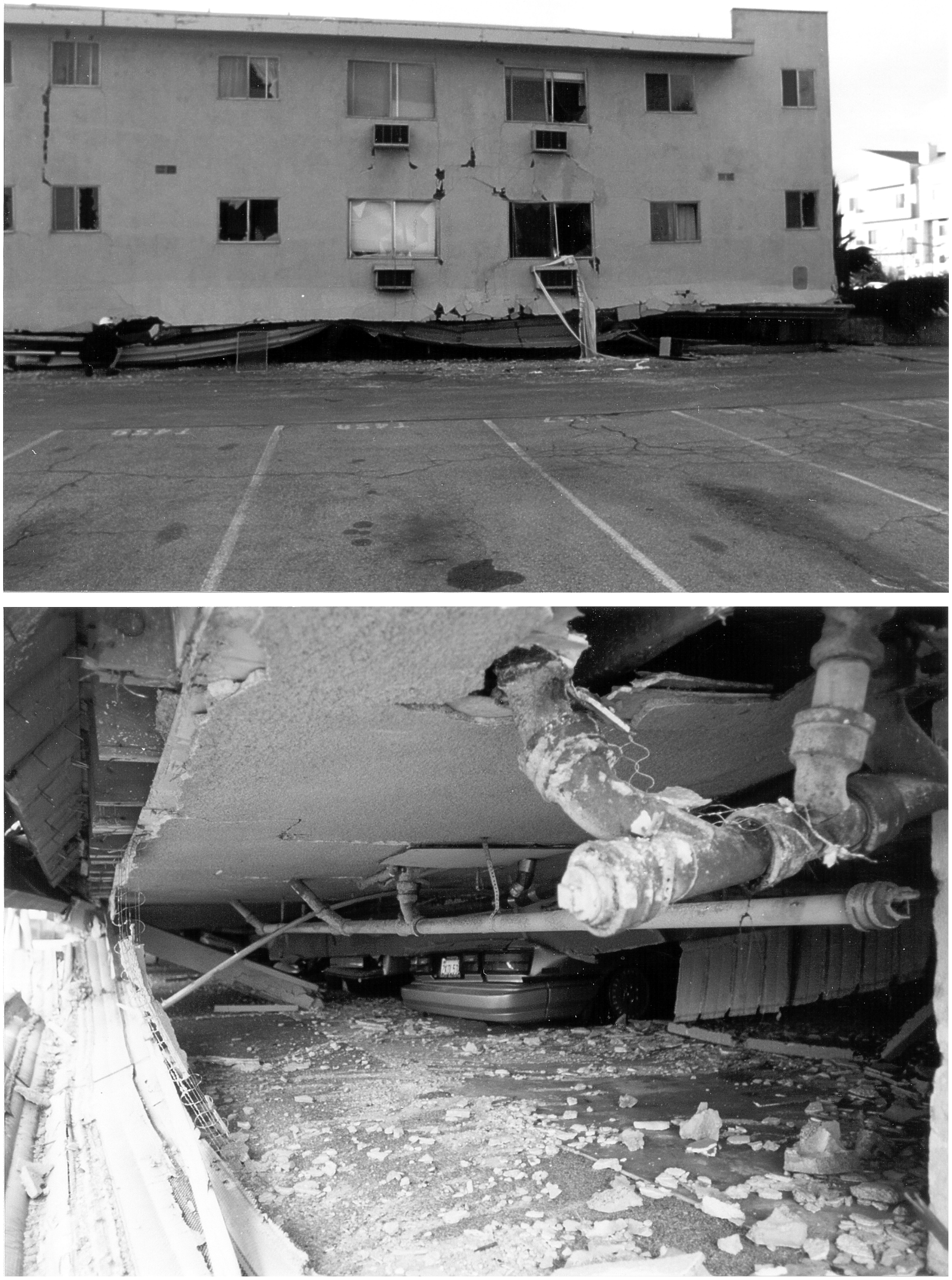


Figure 12-5. Example of building over garage collapse – January 17, 1994, Northridge earthquake

There are a number of common building configuration and design features which often result in a building without a complete lateral force resisting system. These include *open store fronts/house over garage*, *clerestory conditions*, *and expansion joint conditions*. These are schematically shown in Figure 12-4.

The *open store front or house over garage* condition, common in urban construction and for older buildings, has often lead to building collapse during strong ground motion. In older mid- and high-rise construction, the primary vertical elements of the lateral force resisting system are often the perimeter concrete or masonry walls which act as perforated shear walls. A similar condition to the open store-front is the building or house over garage. When such buildings have store-front systems or open garage fronts at the lower story, the vertical shear resistance provided by the walls of the upper stories is not present. This results in a discontinuous lateral force resisting system. Such a condition is most severe for buildings with openings on two of four sides, as the building becomes torsionally or laterally unstable at the lower story (Figure 12-5).

The *clerestory* condition is common in many low- and mid-rise buildings in either commercial or residential occupancy. The problem is that the clerestory is a major discontinuity in the horizontal roof diaphragm, which requires the structure on either side of the clerestory plus the clerestory roof to behave as independent elements. If the structure on opposite sides of the clerestory or the clerestory roof is not by itself stable, then collapse can occur. Figure 12-6 depicts damage of the column supporting a clerestory, as well as significant window damage. If the structure on both sides of the joint is stable, then differential movement of the structure on opposite sides can result in severe damage. Long narrow buildings with one end having an open store-front are also a common configuration that have a high degree of torsional instability.

Figure 12-6. Top of column and window damage due to inadequate lateral system at clerestory

Expansion joints are a common feature of many large buildings of low- and mid-rise construction, particularly in areas with significant seasonal temperature variation. They are placed in buildings to relieve stresses induced by thermal expansion of the building frame as well as to provide relief in exterior finishes (particularly roofing). The system of expansion joints placed in a building will effectively divide it into separate structural units. Some buildings with such joints have not been designed with a complete lateral force resisting system for the structural segments on each side of the joints. This can result in collapse. Another problem that can occur in buildings with expansion joints is pounding of the adjacent structures (Figure 12-7). The severity of this problem is minimized somewhat if the diaphragm levels on each side of the joint align so that the slabs of one structure do not

Figure 12-7. Example of pounding damage at a building expansion joint – October 17, 1989, Loma Prieta earthquake

act as knife edges against the columns of the adjacent structure.

Light wood framed structures are another type, which often does not have a complete lateral force resisting system. Typically, the perimeter walls, interior partitions, ceilings, floors and roofs will provide an informal but effective lateral force resisting system above the lowest floor level. However, the entire assemblage is frequently not attached to the foundations with positive connections. Failures resulting from entire residential structures sliding off their foundations have been common in past earthquakes. Even more common are failures which originate due to inadequately sheathed or braced cripple walls beneath the occupied areas of the structure.

Structural Continuity and Inter-element deformations: Structural continuity is an important factor for good seismic performance. If all of the various components of a structure are not adequately tied together, the pieces can move independently and in different directions. This can result in dislodging elements from structures and the loss of bearing support for vertical load carrying elements. Modern codes require that all elements of a structure be tied together or that sufficient accommodation be made for the real displacements such that failure does not occur. These considerations were often overlooked in older structures. Common deficiencies include: inadequate anchorage of walls to diaphragms for out-of-plane and in-plane deformations (Figure 12-8); use of sliding type beam bearing connections with undersized bearing dimensions; inadequate attachment of architectural elements including cladding, ceilings, and partitions to the structure; inadequate attachment of equipment and utilities to the structure.

Figure 12-8. Out-of-plane wall failure of tilt-up building – January 17, 1994, Northridge earthquake

Excessive Lateral Flexibility: Buildings with complete lateral force resisting systems but excessive flexibility in the elements of their lateral force resisting systems have occasionally collapsed. Such buildings can experience very large lateral displacements when subjected to ground shaking. Structures with significant gravity loading can become unstable under large lateral deformation, as a result of P-delta effects. Since flexible structures tend to have relatively long fundamental periods of vibration, such structures tend to perform adequately when located on sites with firm soils, as the energy content of ground shaking transmitted by such sites to the structures is relatively limited. However, flexible structures located on sites with deep soft soils can experience very large demands. Typically, structures with inter-story drift ratios of 1% or less at real deformation levels (as discussed in Section 12.2.3) behave acceptably.

Brittle elements: Modern design practice for buildings expected to withstand strong ground shaking requires the incorporation of ductile materials and detailing in the design of structures, such that deformations substantially larger than those expected at normal service levels can be tolerated without loss of structural capacity. Older construction rarely was provided with this ductility. As a result, elements tend to be brittle, and can rapidly loose strength when strained beyond their elastic or nominal capacities. Examples of common non-ductile construction include: unreinforced masonry walls, certain classes of concrete frames, and reinforced concrete and masonry walls, and some braced steel frame construction.

Unreinforced masonry walls can be composed of common clay brick, stone, hollow clay tile, adobe, or concrete masonry materials. Walls of these materials have limited strength, and very little ductility for in-plane demands. Slender walls, with large ratios of unsupported length to thickness have often failed due to out-of-plane demands. Inadequate anchorage of these walls to diaphragms is a common deficiency which contributes to poor out-of-plane performance.

Non-ductile Concrete Frames. If adequately designed, moment resisting frames of reinforced concrete can provide excellent behavior in strong earthquake shaking. However, many earthquake induced collapses of structures relying on non-ductile concrete frames for their lateral resistance have occurred. A number of problems can result in poor earthquake performance of concrete frames. These include deficiencies in: shear capacity, joint shear capacity, placement of reinforcement for load reversals, development of reinforcement, confinement of the concrete and lateral support for reinforcing steel.

Shear failure of reinforced concrete columns and beams is a brittle failure mode and can result in sudden loss of load carrying capacity and collapse (Figure 12-9). In frames with adequate strength to remain elastic under real deformation levels (see Section 12.2.3), the beams and columns should have greater shear capacity than required at these deformation levels. In frames which experience flexural yielding at the joints under real deformation levels, the shear strength of the elements must be greater than their flexural capacity or failure can result. The shear strength capacity of members with relatively low axial compressive stress levels should be limited to that provided

Figure 12-9. Collapse of concrete parking garage structure - October 17,1989, Loma Prieta earthquake

by the reinforcing steel as the shear strength of the concrete in such members quickly degrades under cyclic loading.

Shear failure of joints in moment resisting frames can also occur. The beam column joint of a moment resisting frame can be subjected to very large shears, resulting from the transfer of flexural stresses between the elements. Failure has occurred at such joints, particularly when the lateral confinement reinforcement in the columns does not run continuously through the joint zone. Frames with eccentric beam column joints or relatively slender beams tend to be weaker than those without such features.

Moment resisting frames subjected to strong ground shaking will typically experience large flexural load reversals at their joints. Some concrete frames designed primarily for gravity load resistance have little if any positive beam reinforcing steel (located at the bottom face of the beam) continuous through the beam column joint. As a result, the frames do not have capacity to resist load reversals. For good performance, frames must have a minimum percentage of the beam positive reinforcing developed continuously through the beam column joints.

Inadequate development of reinforcing steel is another common problem. In frames with inadequate strength to remain elastic at real deformation levels, the flexural reinforcing steel will yield. Repeated cyclic loading of the bars into the yield range results in a breakdown of the bond between the reinforcing steel and concrete, which can result in a loss of flexural strength and frame instability.

Inadequate Concrete Confinement - Normal weight concrete elements with nominal lateral reinforcement can withstand compressive strains on the order of 0.003 to 0.004. Compressive strains in excess of this amount will result in crushing and spalling of the concrete and degradation of the element's capacity to carry load. Strong ground shaking

Figure 12-10. Shear failure of concrete wall – January 17,1994, Northridge earthquake

can induce large compressive strains in concrete at flexural hinge regions of beam column joints. Large compressive strains resulting from large overturning demands can also occur in columns. Unless closely spaced lateral confinement reinforcing is provided, compressive strains at real deformation levels in excess of about 0.004% in normal weight concrete and 0.002% in lightweight concrete can result in structural failure. This is not a concern for members with low strain demands at real deformation levels.

Large tensile strains, particularly at flexural hinge regions of frames can also result in member failure, unless closely spaced lateral reinforcement is provided. When a flexural hinge forms, large tensile strains and elongation will occur in the longitudinal reinforcing steel. When structural response reverses, under cyclic motion, the elongated steel is forced into compression, and if not provided with adequate lateral support, will buckle. In addition to causing premature spalling of cover concrete, this can lead to low-cycle fatigue failure of the reinforcing and loss of structural capacity.

Reinforced concrete and masonry walls can have many of the same problems described for reinforced concrete frames, particularly if they are highly perforated by openings, or are tall and slender. Generally, walls with relatively low levels of axial load, moderate quantities of vertical reinforcing steel and shear capacities greater than their flexural capacities behave in a ductile manner, while those without these features can be quite brittle. Wall failures can occur as a result of excessive shear demands (Figure 12-10), as a result of crushing at the edges under extreme flexural strains, or as a result of failure of the reinforcement, as previously described for concrete frames. The most common wall failures occur in the spandrel beams present over door and window openings. Very large stress concentrations occur in these elements, often resulting in damage at relatively low levels of lateral load. Once the spandrels have failed, overturning demands on individual piers can increase substantially, and the stiffness and strength of the structure decrease.

Braced steel frame structures have been commonly damaged in earthquakes, but collapses have been rare. The most common damage is to the bracing itself. Light rod braces often fracture, as a result of a concentration of inelastic strain demands at the threaded portion of the rods. In heavier structures, inelastic buckling of compression braces is also common (Figure 12-11). Compression braces of intermediate slenderness, and non-compact section properties can experience brittle fracture as a result of low-cycle fatigue induced by large secondary stresses at buckled sections. Failure of bracing connections is also common, particularly when the strength of the connection is less than the strength of the brace itself. Highly eccentric brace connections tend to fail prematurely due to the large secondary stresses induced by the eccentricities.

Figure 12-11. Example of brace buckling – October 1, 1987 Whittier earthquake

Although failure of braces is one mode of common failure (Figure 12-11), other failure modes can also occur in these structures. One of the more common failure modes occurs in structures with "chevron" type bracing, where the beam at the apex of the chevrons can be severely deformed by large unbalanced force in the "tension" brace following buckling of the "compression" brace. Some structural collapses have occurred as a result of braces which were designed too strong, relative to other portions of the structure. Over-strength bracing can place very large overturning demands on columns, resulting in buckling of these critical gravity load carrying elements. Knee braced frames, in which the braces induce flexural demands on columns can also result in premature column failure.

Inadequate diaphragms - Reliance on inadequate diaphragms can be another cause of earthquake-induced collapse. Although the floors and roofs of most structures provide diaphragm capacity, unless the structures were specifically designed to resist seismic loads, these features are often grossly inadequate. Common diaphragm deficiencies in buildings include *inadequate shear capacity, inadequate flexural capacity, extreme flexibility, poor connectivity to vertical elements of the lateral force resisting system, and lack of continuity*.

Diaphragms of differing materials have widely different shear strengths. Systems consisting of cast-in-place concrete, composite systems of concrete filled metal deck, and horizontal steel braced systems tend to have very large capacities and excellent ductility. Diaphragms constructed of timber sheathing and certain metal decks have very limited capacity but intermediate ductility. Diaphragms consisting of poorly bonded precast concrete planks or of poured gypsum slabs tend to have very low shear capacity and negligible ductility.

Flexural capacity of diaphragms should also be considered. Classic engineering evaluation techniques of flexible diaphragms treat these elements as simply supported horizontal beams, spanning between the various vertical elements of the lateral force resisting systems. The diaphragm material itself (timber sheathing, metal deck, diagonal braces, etc.) are considered to act as the web of this beam while discrete continuous chord elements at the edges of the member are provided to resist flexural demands. The presence of walls around the perimeter of a diaphragm may alter the pattern of flexural demands. In such structures, the walls themselves may directly resist the shear stresses at the boundaries of the diaphragm such that the classic "simple beam" analogy is not valid. Regardless, a rational stress path must exist such that the diaphragm remains in internal as well as external equilibrium. A common deficiency in diaphragms is an absence of local flexural chords around openings. This can greatly reduce the effectiveness of otherwise competent diaphragms.

The basic functions of the diaphragm is to tie the elements of a structure together at a given level and distribute inertial loads to the various vertical elements of the lateral force resisting system. Diaphragms which are *extremely flexible* can result in very large inter-story drifts for supported elements such as walls subjected to out-of-plane loads. It is important that the diaphragm have adequate stiffness to prevent excessive inter-story drifts from developing. This problem tends to be most pronounced with diaphragms of timber construction or those of unfilled metal deck construction. The ABK methodology[12-10] provides a good procedure for estimating the deformability of timber diaphragms. Other methods for calculating diaphragm deformability are presented in the Tri-Services Manual for seismic design[12-14].

Poor connectivity of the diaphragm to the vertical lateral force resisting elements is also common, particularly in structures with relatively large diaphragms and isolated vertical shear resisting elements. It is important that collectors be provided in such diaphragms to transfer shears into the frames and walls. Another common deficiency with regard to shear transfer is a physical separation between the diaphragm web and the top of the vertical lateral force resisting elements. Examples include timber diaphragms which lack blocking of the joists at shear walls and metal deck diaphragms supported by purlins or open web joists which frame above the girders of frames. In such diaphragms the joists or purlins can roll-over at the edges under the influence of diaphragm shear demands.

Continuity is an important consideration for diaphragms constructed of materials with limited tensile capacity including plywood, gypsum and concrete. Under the influence of large concentrated inertial loads, such as generated by heavy masonry or concrete walls supported at a diaphragm edge, diaphragms with limited tensile capacity can rip apart unless directly provided with continuous elements to tie the structure together. In timber diaphragms, continuity can best be provided through the framing members. In concrete diaphragms, reinforcement must provide the required continuity.

Non-structural elements. Non-structural elements are those pieces of a structure which are not intended by the designer to act as structural load carrying elements. Common non-structural elements include non-load bearing walls, cladding, ceilings, ornamentation, and mechanical and electrical services and utilities.

Non-load bearing walls including construction of hollow clay tile, concrete masonry, concrete, and other materials are a common problem in structures. Often not directly considered by the original structural designer of the building, these elements can have substantial influence on the performance of a structure. They can alter its stiffness, deformation patterns, lateral force resisting capacity and failure modes. Common problems include partial height walls which can induce shear failures where they bear against the mid-height of columns, and irregular placement of walls in a building which can create torsional problems and soft stories. In addition to their effect on the behavior of the structure, partition walls can fail either due to in-plane deformations or out-of-plane accelerations resulting in potential personnel hazards as well as substantial architectural damage.

Buildings of recent construction often have curtain wall type *cladding systems.* A common deficiency of such systems is an inability to withstand the large lateral deformations the building experiences under strong ground motion. If the cladding has not be provided with adequate deformation capacity, panels can crush or connections can fail, creating a substantial falling hazard.

Ceilings are a frequent source of damage in earthquakes. Suspended plaster ceilings which are not adequately braced to a nearby diaphragm are a particular problem. These heavy ceiling systems can sway independently, much like a pendulum, and batter adjacent structural elements including walls. This is a common mode of failure initiation in unreinforced masonry buildings.

Exterior ornamentation on structures including parapets, statuary, balustrades, balconies and similar items can also be problem areas. Often, these decorative elements have limited capacity to resist earthquake induced lateral accelerations. Failure typically results in a falling hazard.

Mechanical and Electrical Utilities must be maintained in a serviceable condition for structures which are expected to remain functional following an earthquake. Even in less critical facilities, shaking induced damage to these elements can result in substantial consequential damage to architectural elements. For example failed mechanical and electrical systems can result in fire initiation as well as in flooding. Unfortunately, most mechanical and electrical systems in existing structures are not adequately installed to prevent earthquake induced damage. Major equipment items are not adequately anchored to the structure to prevent sliding or overturning. Piping and conduit systems typically are not adequately braced and provisions have often not been made for earthquake induced building deformation.

Poor construction quality has contributed to the earthquake induced failure of many properly designed structures. Masonry structures tend to be particularly vulnerable. A number of failures have occurred in reinforced masonry walls because grout had not been placed in reinforced cells. Poor quality mortar is also common. In concrete structures, under-strength concrete has occasionally resulted in failures. Welded reinforcing steel splices are often quite brittle and can prematurely fail if proper procedures were not followed during construction. Similar problems can occur at welded connections of steel structures. Timber buildings are also susceptible to problems arising from poor construction quality, including such basic errors as framing the structure differently than intended, or failing to provide the connectors specified.

Deteriorated condition also contributes to earthquake induced failures. Common problems include dry-rot and infestation damage to wood structures, rusting of steel and spalling of concrete on marine structures, and weather deteriorated mortar in masonry structures.

Site characteristics are also too often overlooked by structural engineers with regard to building performance. Unstable sites with propensities for liquefaction, lateral spreading, land sliding or large earthquake induced differential settlements can lead to extensive damage to structures which are otherwise adequately designed. It is critically important to assess the nature and likely stability of the local geotechnical conditions as a first step in the evaluation and retrofit of any existing structure.

12.4 UPGRADE CRITERIA

Up to very recently, there were are no consensus documents defining seismic upgrade criteria and provisions with the exception of unreinforced masonry buildings[12-11] structures. A multi-year two-phase project of the National Earthquake Hazard Reduction Program (NEHRP) which was underway for this purpose came to fruitation in 1997 by publication of the FEMA-273/274 documents (see Chapter 15).

It is very important to establish a clear statement spelling out the desired performance objectives for the upgrade, and that the design criteria to achieve these objectives be identified. The identification of the design criteria is particularly important. Even if an upgrade is required by an ordinance, it is still important that a clear understanding exists between the engineer and the owner as to what the objectives and the seismic performance of the upgraded building is likely to be.

The performance objectives, as stated earlier, are likely to vary considerably from one building to another based on several factors. These factors include: economic value of the structure, occupancy, function of the structure, historic significance, site specific seismic hazard, and the relative cost of achieving upgrades to various criteria.

A building-specific design criteria should be established that defines how the designer will accomplish the specified performance objectives. As a minimum the design criteria should address the following issues.

1. Testing program to determine existing materials properties

Existing documentation, including original drawings and specifications, material test reports, and geotechnical reports are likely to be lacking for many buildings being upgraded. Important structural elements may often be concealed, requiring destructive investigations to determine element sizes and locations.

The extent, type and location of exploration/testing for each building should be established to determine material properties of the lateral force resisting elements and other structural and non-structural elements that are to be assessed or strengthened to accomplish the performance objectives. The material testing program should provide not only material force capacity data but also deformation capacity data where practical.

2. Design force levels

A design demand level has to be established, compatible with the performance objectives to be achieved. In selecting a design demand level, one should consider the performance objectives, the importance, the size, and type of lateral force resisting system of the structure, its ability to sustain damage without collapse and the consequences of varying levels of damage, as well as the available resources. There are two common methods to establish the design demand levels (1) code based approach, in which minimum inertial lateral forces are defined: and (2) a probabilistic method, in which ground motion characteristics with a defined probability of occurring are determined, and then used to measure structure response.

The most common bases of design use the force method, with design of new elements (and check for adequacy of existing elements) to a factored percentage of the minimum lateral forces specified by the building code. Commonly, the factor is taken less than one in order to account for the reduced expected life of an existing structure as well as to control construction costs to reasonable levels.

The probabilistic approach is most commonly used for large projects, projects with restrictive performance criteria such as Emergency Operations Centers or Hazardous Materials containing facilities, and for structures in near fault regions. The probabilistic approach commonly uses a two level earthquake criteria, most commonly specifying a design level event (DBE) and a maximum credible event (MCE). The DBE is typically taken as an event in which serviceability of the structure is intended to be maintained. The MCE is an event at which collapse is to be avoided. The probability of each of these events can be adjusted depending on the importance and goals for the structure. For base isolated structures, the UBC currently specifies the DBE as an event with a 10% chance of exceedance in 50 years and an MCE as an event with a 10% chance of exceedance in 100 years. The lower the probability of exceedance of an earthquake, the more severe it is. For some structures, it may be more appropriate to take the DBE as a 10% in 100 years event and the MCE as a 10% in 500 years earthquake. Regardless, the ground motion is typically characterized as response spectra curves, which can then be utilized to determine deformations of the structure.

3. Drift limitations

As has been previously discussed drift control is much more important in the upgrade design of an existing building than in the design of a new building. Hence global and/or element drift control parameters need to be established that will provide adequate assurance that the upgraded building will meet the performance objectives.

4. Detailing criteria for existing and new elements

Detailing in existing buildings frequently does not meet the requirements of new construction and will therefore perform in a less ductile manner. Consideration for this less than desirable performance needs to be incorporated in the design criteria. This can be accomplished by not relying on existing members to participate in the lateral force resisting system or by controlling deformations in existing elements to levels where adequate participation is provided. The former is frequently not practical.

5. Compatibility of new and old construction

The stiffness and strength of existing elements may not be compatible with new upgrade elements. A steel moment frame or even a braced frame added to resist the forces of an existing unreinforced brick masonry wall with inadequate capacity is such an example. The brick wall may resist the lateral load until it's capacity is reached. The wall will then fail and the entire load would be redistributed to the steel frame. Assuming the wall participates in parallel with the frame may lead to a poor performing structure unless the capacity of the masonry wall is ignored or the steel frame is designed to control wall deformations.

Deformation and strength criteria that will provide adequate compatibility of old and new elements should therefore be specified.

6. Construction quality control

Adequate connection of new elements to existing elements is both critical and highly dependent upon existing material properties, sizes, locations and contractor accessibility. The likelihood of encountering unexpected field conditions is much greater in retrofitting existing buildings than in the construction of new buildings. It is therefor important that a quality control program involving frequent inspection, testing, and observation by the design engineer, be established and accepted by the owner.

7. Criteria for non-structural elements

Adequate performance of certain non-structural elements may be required to ensure performance objectives are achieved. Non-structural elements such as hollow clay tile partition walls around exit corridors, heavy ornamentation, light fixtures, building cladding, etc. may require supplemental anchorage reinforcement or other upgrade measures may provide for adequate life-safety. Adequate performance of essential systems, such as power and telephone service may also be required for facilities where post-earthquake functionality is required. Design force and deformation criteria for selected non-structural components therefore need to be established.

12.5 COMMON UPGRADE METHODS

Structural rehabilitation or strengthening of a building in general can be accomplished through a variety of approaches, each with its merits and limitations. The specific considerations and their relative importance in the selection of the most appropriate upgrade method are unique to each building.

The following paragraphs present methods that are commonly used to correct or improve the building deficiencies previously discussed. The structural considerations of alternate upgrade methods are presented along with their advantages and disadvantages. It should be kept in mind, however, that other factors may influence, or even dictate, the selection of a particular method for a particular building. These other factors include cost, function, and aesthetics.

Alternate upgrade approaches can generally be utilized to correct building deficiencies, each with a different impact on cost, function and aesthetics. Cost will always be a major consideration when evaluating methods to upgrade a building. Seismic upgrade costs can range greatly depending upon the deficiencies present, the performance objectives of the upgrade, the function and aesthetic constraints, and whether the building will be occupied during construction. Costs may range from as low as one dollar a square foot, to as high as one hundred dollars a square foot.

Most buildings are intended to serve one or more functional purposes (e.g. to provide housing or to enclose a commercial or industrial activity). Since the functional requirements are essential to the effective use of the building, extreme care must be exercised in the planning and design of the structural modification to an existing building to assure that the modifications will not seriously impair the functional use. For example, in a building to be utilized for leasing office space, a minimum of fixed walls or partitions is important to allow flexibility in the office layout for accommodating the space requirements of different tenants. The addition of steel braces or shear walls across the open office space may significantly decrease the flexibility and hence the value of the office space.

The preservation of existing aesthetic features may, in some cases, have a significant impact on the selection of an upgrade method. Historical buildings, for example, may require special upgrade techniques to preserve historical features. In some cases, when permissible, removal and replication of these features during the upgrade process may be more cost-effective than preservation or restoration.

12.5.1 Incomplete Lateral Force Resisting System

Three building features that commonly result in an incomplete lateral force resisting system were presented previously. These are open store-fronts, clerestory conditions, and expansion joint conditions. Lack of adequate foundation anchorage is another common example. Common methods to correct these deficiencies are presented below.

Open store-front - The deficiency in a building with an open store front is the lack of a vertical line of resistance along one or two sides of a building. This results in a lateral system that is excessively soft at one end of the building causing a significant torsional response and potential instability.

The most effective method of correcting this deficiency is to install a new stiff vertical element in the line of the open front side or sides (Figure 12-12). Should the owner desire to maintain the open front appearance braced steel frames located directly behind storefront windows are a common method utilized to provide the necessary stiffness and strength. The braces have some aesthetic impact but are commonly located to minimize functional impact. Shear walls may also be utilized to provide adequate strength. In both cases collectors are required to adequately distribute the loads into the diaphragm. Adequate anchorage of vertical elements into the foundation is also required to resist overturning forces.

Steel moment frames can also be utilized to provide adequate strength, provided that inelastic deformations of the frame under severe seismic loads are carefully considered to ensure that displacements are controlled.

Clerestory - A clerestory can result in a significant discontinuity of a horizontal diaphragm. As with all upgrades the function of the structure must be an important consideration. Clerestories are typically designed in a building to provide an open airy feeling.

A common method to address the resulting diaphragm discontinuity is the addition of a horizontal steel truss (Figure 12-13a). Light-weight steel members can be designed to transfer diaphragm shears while minimizing visual obstructions to the clerestory.

An alternate approach to correcting a clerestory deficiency is to reduce the demands on the diaphragm through the addition of new vertical lateral force resisting elements such as shear walls or braced frames (Figure 12-13b). By reducing the demands, diaphragm deformations and stresses can be controlled to within acceptable limits. Impact on space utilization must be considered in locating the vertical elements.

Expansion Joints are installed in structures for a variety of reasons including: (1) to control the effect of deformations caused by temperature changes during and after construction; (2) to control the effects of construction shrinkage or creep; or (3) merely to simplify the lateral analysis of different portions of a building, particularly when the addition to a structure is designed.

Structural members exposed to the elements (i.e. large temperature changes) prior to the installation of exterior walls, finishes, and building climate control systems, may be protected through the use of expansion joints. After the building systems are installed differential temperatures are kept to a minimum rendering the expansion joints no longer necessary.

Another common reason for the presence of an expansion joint in a building is to accommodate post-tension concrete shrinkage and creep deformations. After shrinkage and creep has stabilized (nearly all movement will have occurred within months of the construction) there is no longer any need for the expansion joint. Expansion joints are also frequently installed to control deformations of the roof membrane to prolong their life.

Once reason for the existence of the expansion joint is clearly understood intelligent decisions can be made regarding the future need of the joints. Expansion joints can present similar concerns to a building as open store fronts, i.e. lack of lateral resistance along one side of the structure. Common methods of correcting this deficiency include: (1) installing vertical lateral load resisting elements along both sides of the joint; (2) modifications to the connection such that horizontal shear can be transferred across the joint, but not axial forces; and (3) elimination the joint.

If the expansion joint needs to be maintained, installation of new vertical lateral load resisting elements on each side of the joint

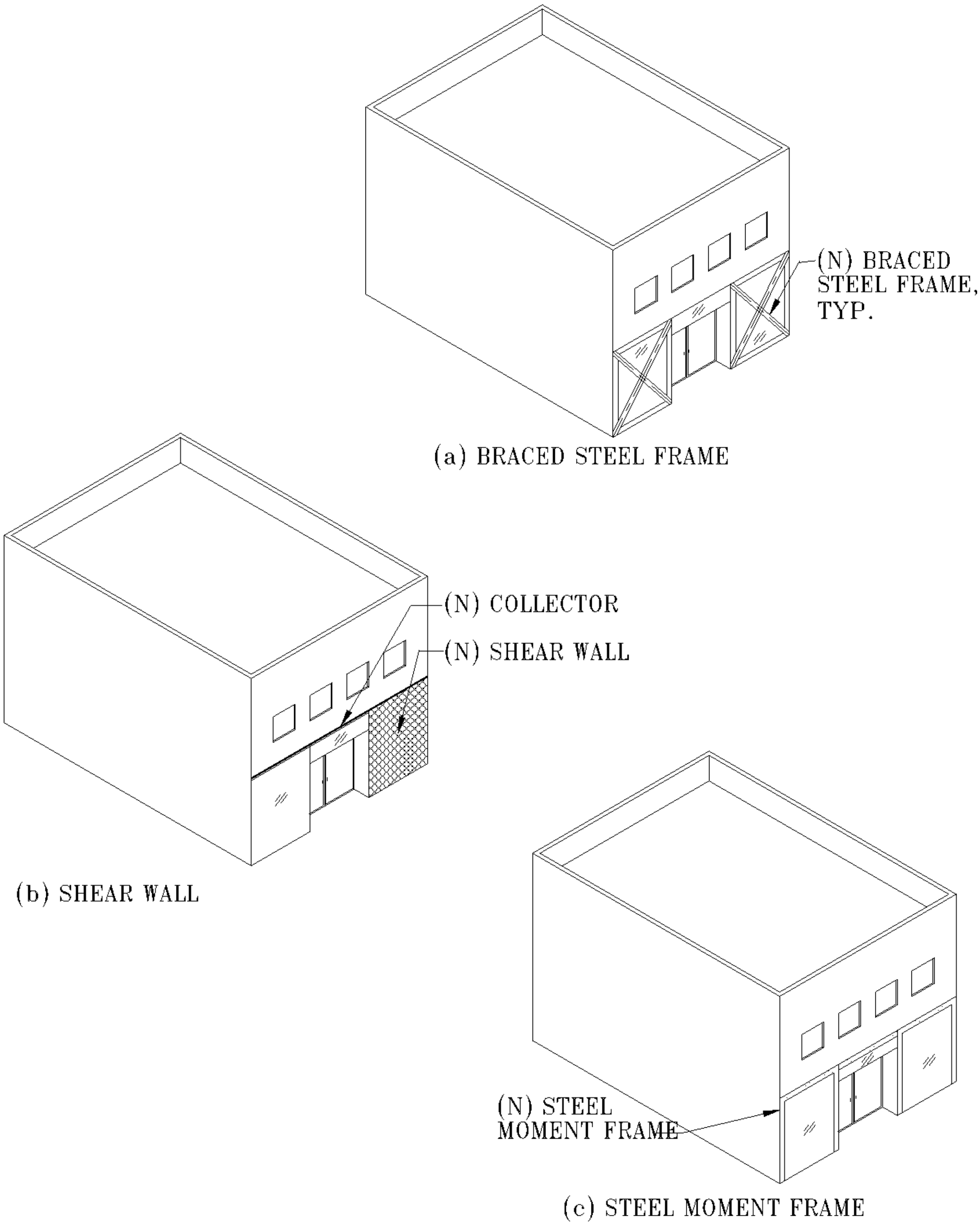


Figure 12-12. Common methods for upgrading a building with an open store front

will provide two complete lateral load resisting systems (Figure 12-14a). This method does cause a significant impact to the flexibility of the building space.

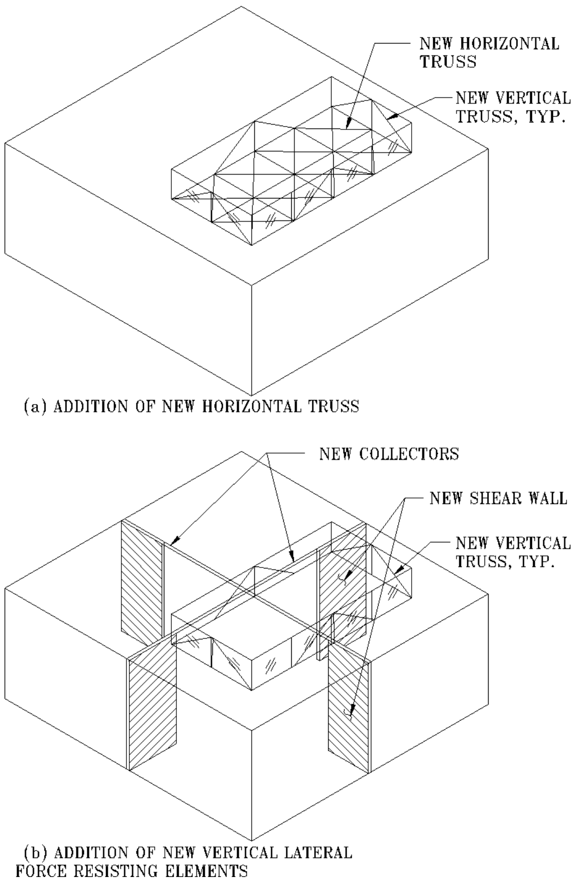
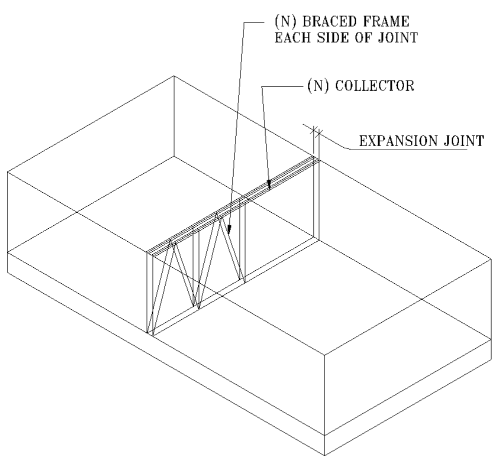


Figure 12-13. Common methods for upgrading a building with a clerestory.



a) addition of new vertical element

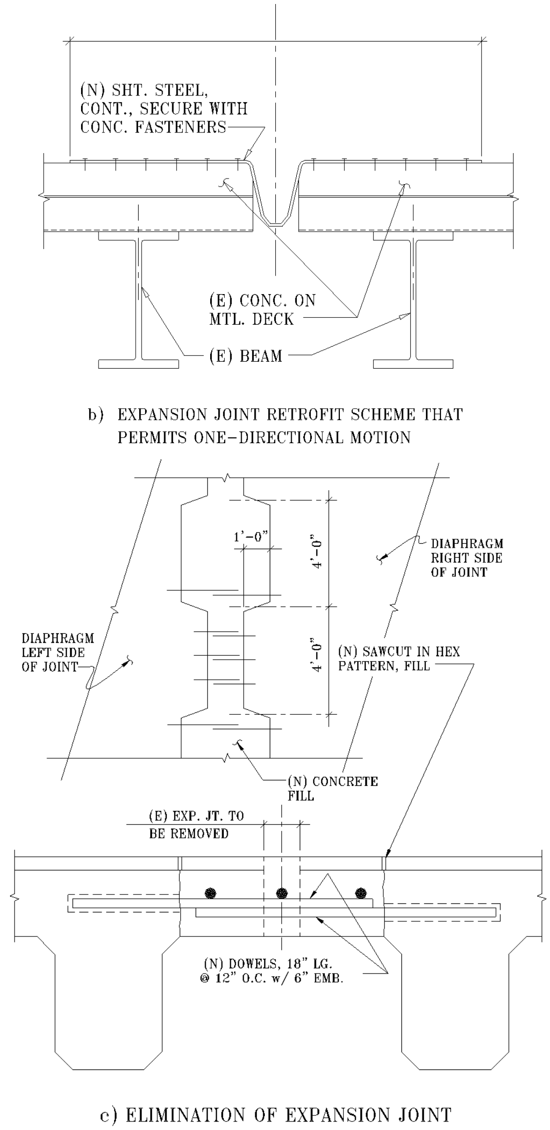


Figure 12-14. Common methods for upgrading a building with an expansion joints.

Should the vertical lateral load resisting elements on one side of the diaphragm have sufficient stiffness and strength to resist rotation, the deficiency can be corrected by modifying the connection to resist horizontal shear only. Figure 12-14b presents one option used on a metal deck with concrete fill diaphragm. The connection resists shear parallel to the joint but permits expansion in the perpendicular direction.

Elimination of the joint may be the best solution from a cost and a performance point of view if the original intent of the joint is no longer necessary. Figure 12-14c presents a common detail utilized to connect a new slab, in this case fill for an existing expansion joint, to an existing slab, thereby eliminating the joint. It is important that continuous members capable of resisting chord forces be installed at the perimeter of the diaphragm.

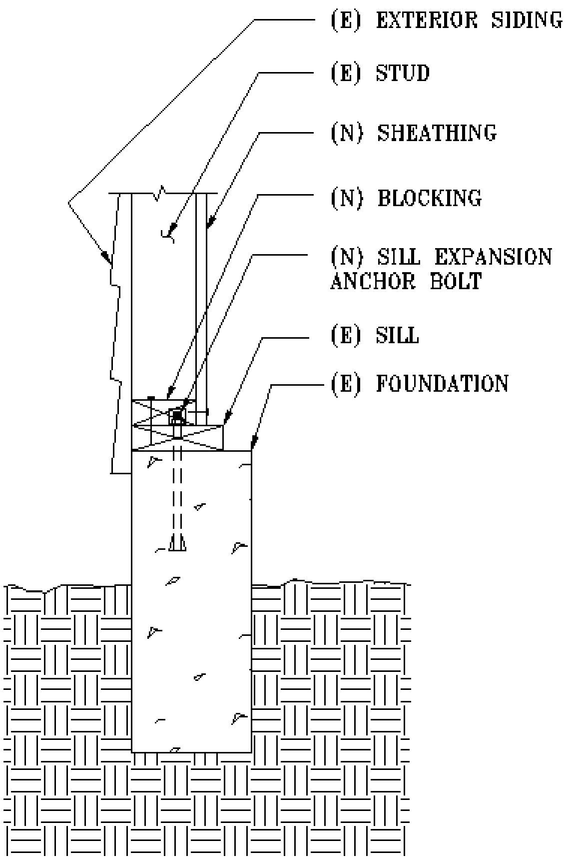


Figure 12-15. Providing wall to foundation anchors

Lack of Foundation Anchorage - Light wood-framed structures without positive connection to the foundation is another common problem where a complete load path is lacking. Providing a positive connection, (i.e. expansion anchors through the sill plate into the foundation) will correct this problem (Figure 12-15).

Light wood-framed structures also commonly have cripple stud walls above the foundation. The lack of stiffness and strength of the cripple walls can lead to failure in an earthquake. Adding plywood sheathing on the inside of the cripple wall as shown in Figure 12-16 is a common method used to correct this deficiency. Proper nailing is required to provide a continuous and adequate load path from the floor diaphragm and walls above the floor to the foundation.

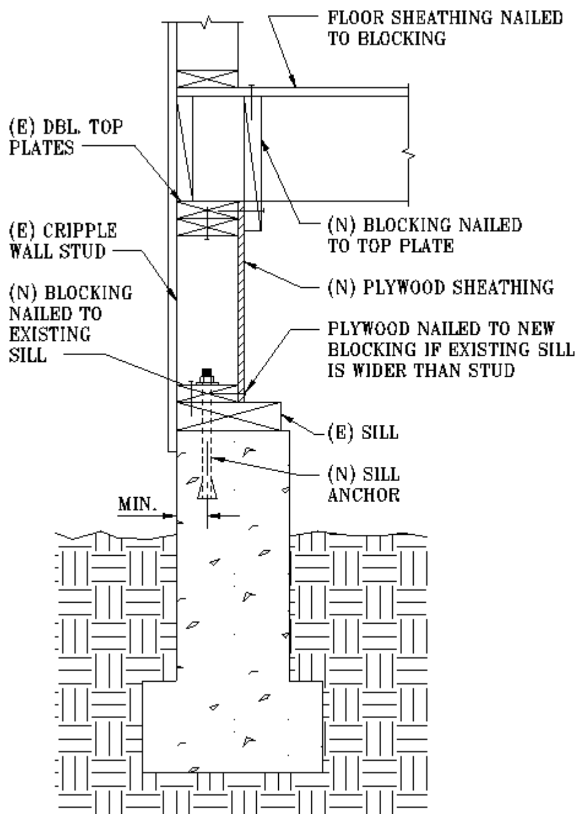


Figure 12-16. Strengthening of a cripple stud wall

12.5.2 Lack of Structural Continuity and Inter-element Deformation

Common structural continuity and inter-element deformation deficiencies were identified previously. These include: inadequate anchorage of walls to diaphragms, use of sliding type beam bearing connections with undersized bearing dimensions, and inadequate attachment of various architectural, equipment and utility elements to the structure.

Inadequate wall-to-diaphragm anchorage - In existing buildings reentrant corners are typical locations where the connection of floor and roof diaphragms to existing walls may be inadequate to accommodate real earthquake induced displacements. This problem is particularly acute with flexible diaphragm systems. Walls adjacent to the reentrant corner will keep local diaphragm deformations to a minimum, e.g. below 1/4 inch (Figure 12-17). However, global diaphragm deformations may be large, e.g. greater than 2 inches. The resulting deformation incompatibility will likely lead to a connection failure at the reentrant corner.

The common method for correcting this deficiency is to install a diaphragm collector. The collector will distribute the stresses into the diaphragm eliminating the stress concentration and deformation incompatibility at the reentrant corner. Existing roof framing members may be utilized as collectors provided the members can accommodate dead plus seismic loads. Figure 12-18 presents a common method for installing a collector in a wood diaphragm.

Structures with heavy walls and wood diaphragms may cause excessive out-of-plane stresses on the diaphragm when subjected to strong ground motions. These excessive stresses may occur at the diaphragm to wall connection or they may occur in the diaphragm if the roof or floor system is not designed for these forces. Correction of this deficiency is commonly accomplished through the installation of out-of-plane tension connections at the perimeter wall (Figure 12-17) and continuity ties across the diaphragm (Figure 12-

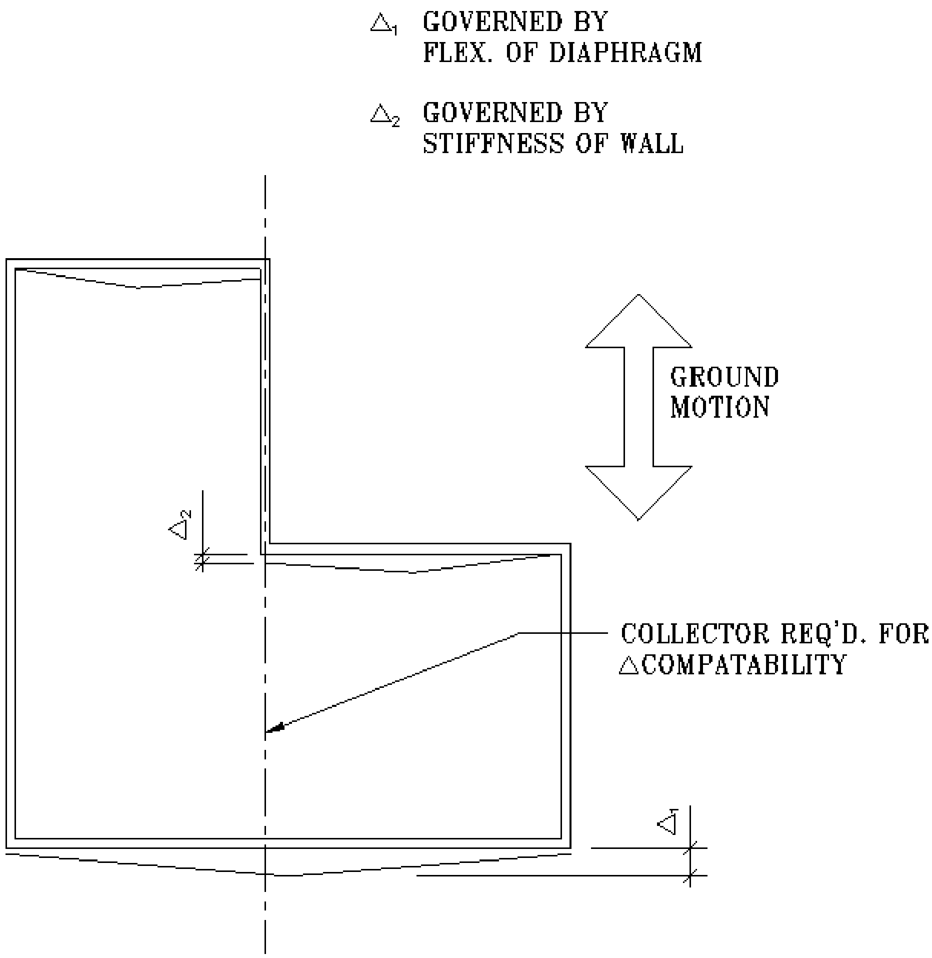


Figure 12-17. Deformation incompatibility at reentrant corner

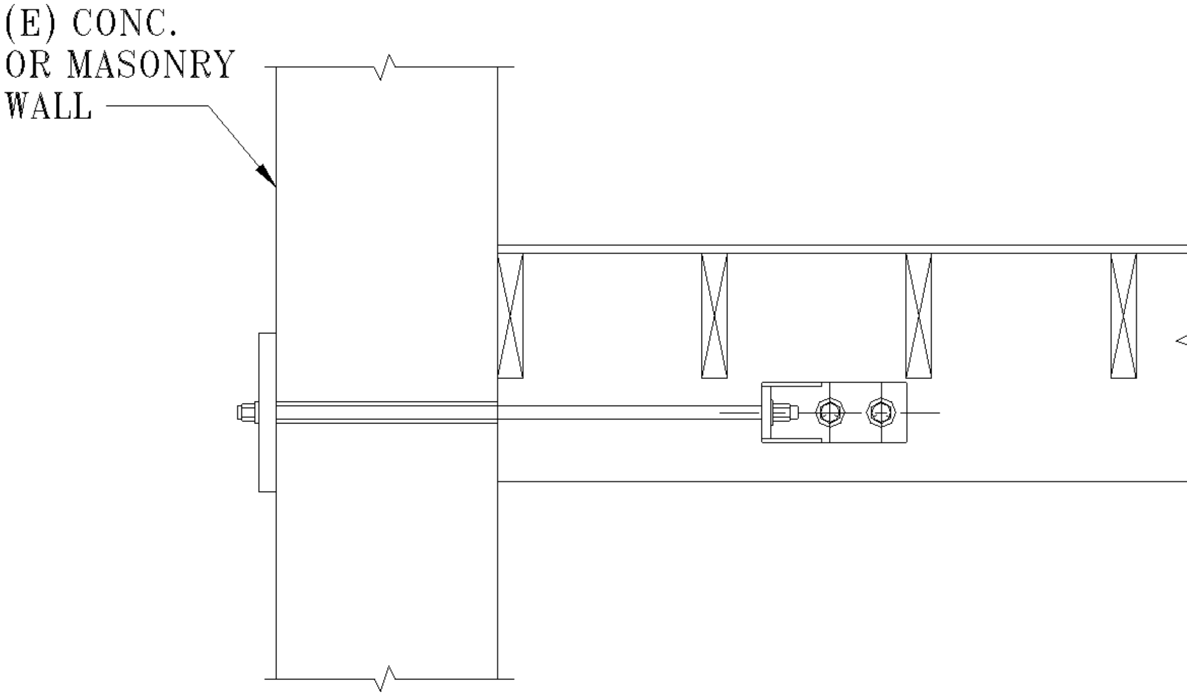


Figure 12-18. Out-of-plane wall anchor

26). With the installation of these elements the walls and diaphragms will respond as a unit, keeping inter-element deformations to a minimum.

Insufficient Bearing at Sliding Connections - Another common deficiency in existing buildings is insufficient bearing area for sliding type beam bearing connections. Floor and roof beams have slid off their bearing supports in past earthquakes and resulted in local collapse. There are four common methods for mitigating this deficiency. The first is to enlarge the beam bearing area, to accommodate the large deformations. Second, the potential for excessive differential deformations can be reduced by stiffening the lateral systems on one or both sides of the connection. Third, elimination of the sliding connection may be possible as previously discussed for expansion joints. A fourth alternative is to provide a redundant vertical support under the beam such that if the beam becomes dislodged from it's support, a local collapse will not result. The first alternative is commonly the most cost effective, however, the second and third alternatives may be less expensive if strengthening of partitions of the building are required to address other deficiencies.

12.5.3 Excessive Flexibility

Buildings with a complete lateral force resisting system but with excessive flexibility can be upgraded by introducing elements to increase stiffness and hence reduce deformations. Care needs to be taken, however, as increased stiffness is likely to result in increased amplification of seismic demands.

12.5.4 Brittle Structural Systems

The following paragraphs discuss common methods to upgrade deficiencies in buildings with brittle structural systems including unreinforced masonry (URM) buildings, non-ductile concrete frame buildings, reinforced concrete and masonry wall buildings and braced steel frame construction.

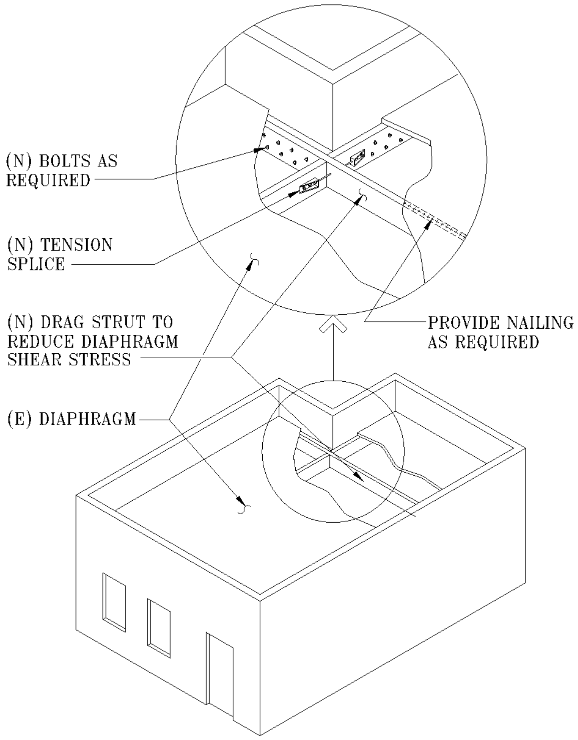


Figure 12-19. New drag strut in wood diaphragm

URM buildings - The most severe deficiency of a URM building is commonly inadequate connection of the walls to the diaphragms. URM building walls may also have limited strength and ductility, both in- and out-of-plane. Common methods for upgrading URM buildings include providing attachments between the walls and the diaphragms (Figure 12-19), and increasing the strength and ductility of the walls. In-plane deficiencies can be corrected by: (1) adding shotcrete to one face of the wall (Figure 12-20), (2) infilling existing windows, or (3) reducing the demand on existing walls through the introduction of supplemental walls.

Out-of-plane deficiencies can be corrected by: adding shotcrete, center coring the wall and installing reinforcing dowels (Figure 12-21), and adding steel "strongbacks" to stiffen and strengthen the walls (Figure 12-22). Adding strongbacks is typically the most cost effective, if out-of-plane capacity is the only consideration. Strongbacks can be installed to span either vertically or horizontally. If increased in-plane capacity is also required,

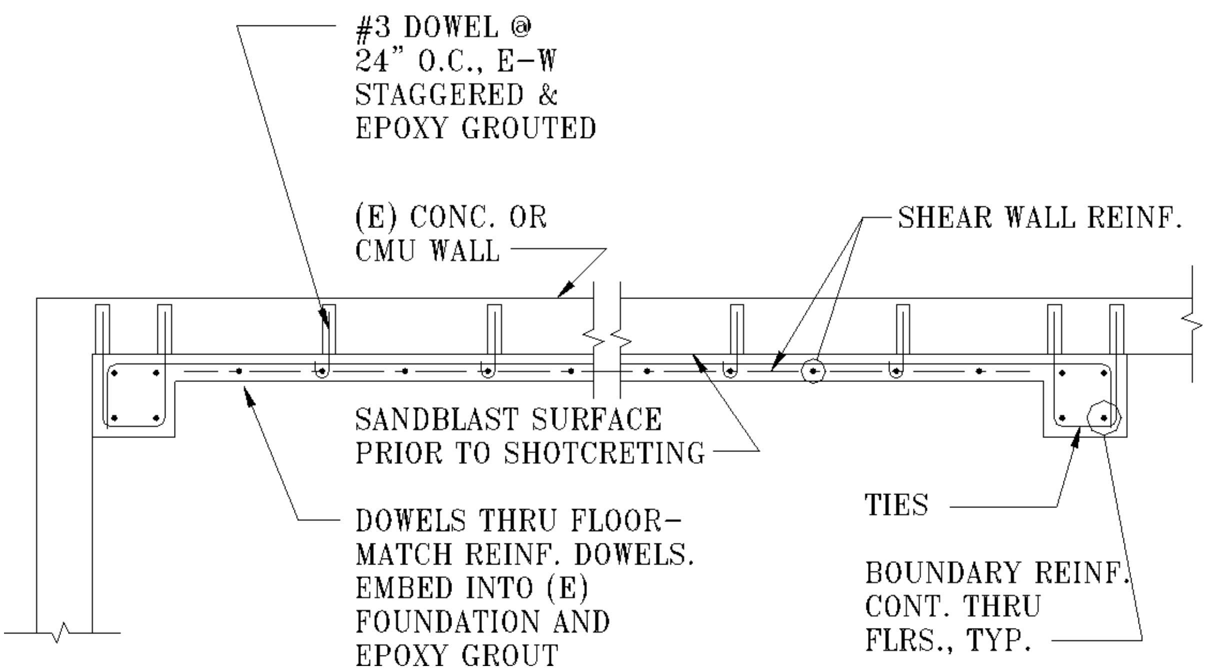


Figure 12-20. Upgrade of existing concrete or CMU wall utilizing shotcrete-Plan view

adding shotcrete may be found to be more efficient. Center coring is typically utilized when preserving the architectural appearance of both sides of the wall is desired.

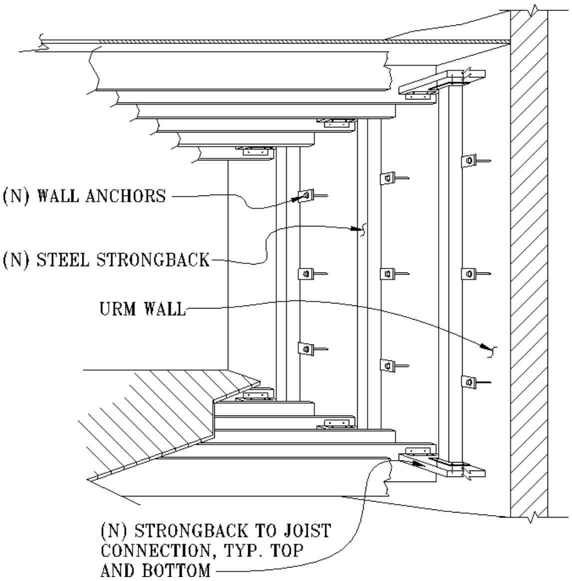


Figure 12-21. Out of plate strengthening of a URM wall using steel strongbacks.

Nonductile concrete frames - Non-ductile concrete structures have limited capability to accommodate building and element deformations. Hence, correcting the deficiencies of non-ductile concrete frame structures requires a good understanding of the behavior existing materials. This usually requires testing concrete cylinders to determine post-yield stress-strain relationships. This testing requires special equipment to monitor displacements as the load decreases. Inelastic beam and column moment-curvature relationships can then be determined using the results of the post-yield tests and estimates of available element ductility can be made.

Once available element ductilities are understood deformation limits can be defined and various upgrade methods evaluated. Common upgrade methods include: (1) reducing the drift demands by adding supplemental resisting elements, such as shear walls, braced frames or additional moment frames; (2) increasing the available ductility of

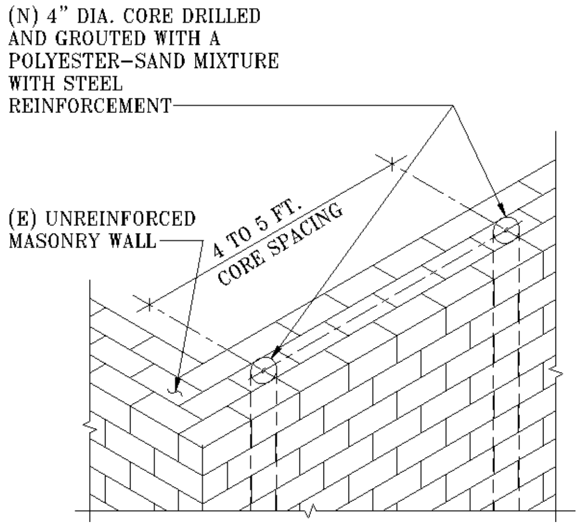


Figure 12-22. Example of center coring technique

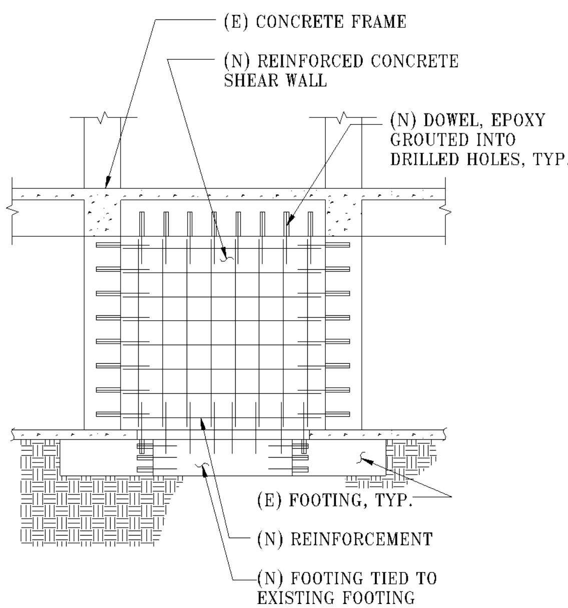


Figure 12-23. Strengthening an existing concrete frame building with a reinforced concrete shear wall.

the elements such as increasing confinement of reinforcing steel; or (3) changing the system to a shear wall system by infilling the concrete frames with reinforced concrete as indicated in Figure 12-23.

Upgrading a non-ductile concrete frame building may involve a significant amount of retrofit work. Both the first alternative, adding supplemental elements, and the third alternative, changing to a shear wall system, will likely result in the existing frames becoming ineffectual in resisting lateral loads due to the differential stiffness between new and existing elements. A significant amount of foundation work may also be required as lateral loads will be resisted at discrete locations as opposed to every foundation in an original distributed frame design. Should supplemental elements be added to control drifts, the elastic and inelastic stiffness compatibility of the new and existing members need to be evaluated.

Increasing element ductility through added confinement steel can be accomplished, however, at significant expense. New rectangular column ties added around existing members have been shown to be ineffectual in providing confinement. Concrete jackets with circular ties or round steel pipe jackets with infilled concrete provide much more effective confinement, however, this may require a significant increase in the final dimensions of the beams or columns. Details to provide adequate confinement at beam-column joints are difficult to develop and install.

Reinforced Concrete and Masonry Walls - Brittle reinforced concrete and masonry wall buildings can be upgraded by installing elements to control inelastic deformations. This can be accomplished by increasing the wall strength and stiffness through: (1) placement of reinforcing steel and shotcrete on the inside or outside of existing walls; (2) infilling window or door openings; or (3) by reducing the demands on existing walls by providing new supplemental walls.

Adding shotcrete to existing walls is the most common method to upgrade existing inadequate masonry or concrete walls. It is most cost effective to shotcrete the exterior of a building due to the ease of construction access for shotcrete and new foundation installation (if required), as well as the simplicity of providing shear and tension continuity across floor levels.

Exterior shotcrete is not always possible due to property line restrictions, access, or aesthetic reasons. Hence shotcreting of interior walls is also commonly performed. Adequate continuity of boundary elements and shear transfer across floors is required for inside applications. As shotcrete wall thickness can be as small as 3 inches, little floor space is lost. Figure 12-20 presents a typical detail of a shotcrete application to the inside of an existing concrete or CMU wall.

Infilling windows is a viable alternative if the elimination of a sufficient number of windows can be tolerated. Loss of a considerable number of windows may affect the natural air circulation in the building, will impact the amount of natural light, as well as the aesthetic appearance of the structure. When improving the capacity of shear walls by infilling windows care should be taken to ensure that adequate bond is provided between new and existing materials. The can be provided through the use of dowels. An infill material with a modulus of elasticity similar to the existing structure should utilized so that wall deformations will be uniform.

Braced steel frames structures - Common deficiencies of braced steel frame structures include: (1) weak connections, (2) non-compact members experiencing low-cycle fatigue failures; (3) beam failures in chevron braced systems; (4) column failures due to over-strength bracing; or (5) column failure of knee-braced frames.

Weak connections are common problems in existing braced frame systems as seismic codes have only recently required that braced frame connections be designed to have greater capacity than the tension capacity of the attached brace. Strengthening the capacity of the existing connection can be accomplished by the addition of new bolts or welds provided the gussets are adequate for the higher loads. Alternatively the connections can be cut out and replaced with stronger connections. If the existing brace members require strengthening or replacement with members of greater capacity, it is probable that new connections should also be designed.

Non-compact braces with intermediate slenderness can experience brittle fracture as a result of low-cycle fatigue induced by large secondary stresses at buckled sections. This deficiency can be mitigated by reducing the slenderness of the member by providing lateral bracing at intermediate locations or by increasing the capacity of the brace by increasing the area of the brace.

Beam failures may occur in chevron systems should large unbalanced forces in the "tension" brace occur following the buckling of the "compression" brace. This deficiency can be mitigated by increasing the bending capacity of the beam or by designing the braces (and their connections) to remain elastic. Increasing the beam capacity is typically the most cost effective approach. Designing the braces to remain elastic is usually not recommended as realistic design forces can not be accurately estimated.

Column failure in a braced frame system can lead to a local collapse. Where over-strength braces cause the weak link of the structure to occur in the column, design modifications are required. The existing brace could be removed and an adequately designed brace could be installed. Alternatively, the column and brace connections could be strengthened.

12.5.5 Inadequate diaphragms

Common deficiencies for diaphragms include *inadequate shear capacity, inadequate flexural capacity, extreme flexibility, poor connectivity to vertical elements of the lateral force resisting system, and lack of continuity*. The method for addressing these deficiencies is dependent upon the construction of the existing diaphragm. There are five common types of roof or floor diaphragm construction: timber, concrete, metal deck, precast, and horizontal steel bracing.

Timber Diaphragms - Timber diaphragms can be constructed of straight-laid or diagonal plank sheathing, or of plywood. *Common deficiencies include inadequate shear capacity, inadequate chord capacity, inadequate stiffness, inadequate continuity, and poor connectivity to vertical elements.*

Strengthening timber diaphragms with inadequate *shear capacity* can be accomplished by additional nailing, overlaying with plywood, or reducing the span of the diaphragm through the introduction of supplemental vertical lateral force resisting elements. Adding nails to existing plywood (with the addition of blocking) can cost effectively increase the capacity of existing plywood, however, this is not true for straight or diagonal plank sheathing. For these systems added nailing is not practical due to the large number of nails required and the propensity for existing planking to split when nailed.

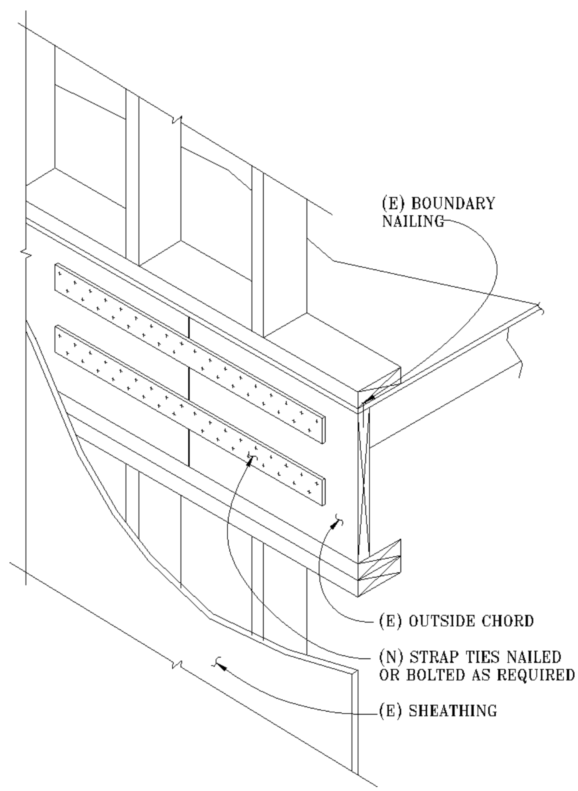


Figure 12-24. Chord splice of wood diaphragm.

The most common approach for increasing the shear capacity of plank sheathed systems is to provide a plywood overlayment. The existing planking can then be used in lieu of new blocking. Plywood should be configured such that new panel edges do not align with existing plank edges. Typically staples at close spacing on either side of the plywood joints are

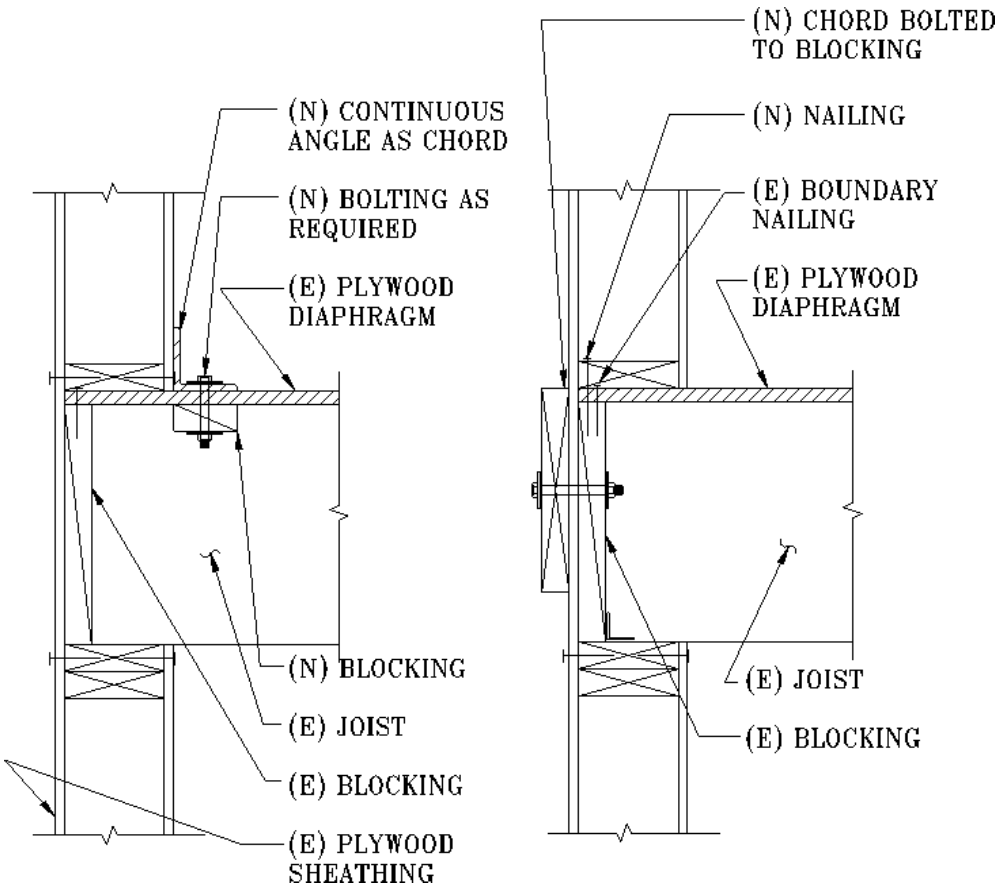


Figure 12-25. New chord member for wood diaphragm

specified as the planking provides insufficient wood depth for adequate nail penetration. The capacity of the combined plywood plus plank sheathing must be determined through a rational analysis. In addition to increasing the shear capacity of the diaphragms the plywood overlayment will also significantly increase the stiffness.

The shear capacity of existing plywood diaphragms can also be increased through the use of sheet metal strips placed over the plywood edges and securing the sheet metal to the plywood on both sides of the joints with staples. This approach is described in Reference 16.

Timber diaphragms with *inadequate chord capacity* can be upgraded by providing adequate connections to existing perimeter framing or through the addition of new continuous members. Figure 12-24 presents a detail where continuity across the connections of the existing rim joists are provided with the use of metal hardware. Figure 12-25 presents two examples where new chord members have been added to the existing diaphragm. In all cases adequate shear transfer connection capacity is required between the diaphragm and the chord member.

Drift limits frequently control member design on multi-story buildings with flexible lateral systems. *Excessive drifts* can also be expected on long span timber diaphragms, particularly when they are used with heavy walled structures. Diaphragm drifts need to be checked for these types of structures. Several alternatives can be implemented should drifts exceed acceptable levels, including: reducing the span by adding supplemental vertical lateral

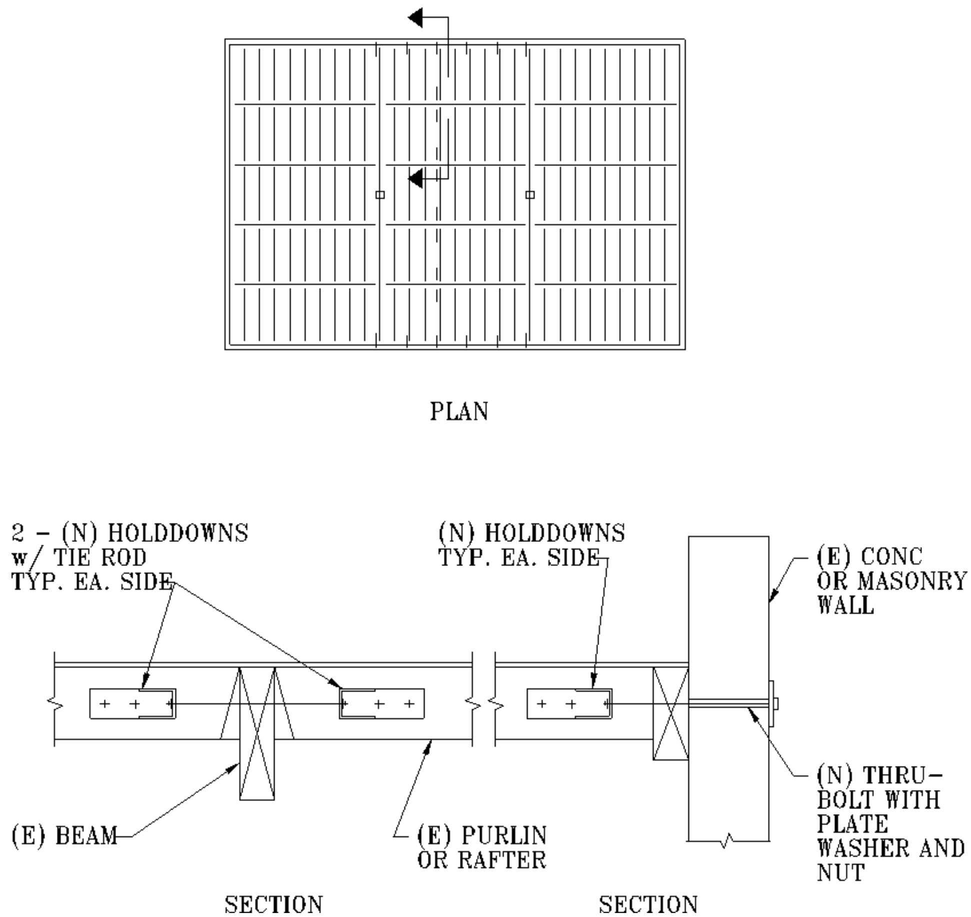


Figure 12-26. Adding continuity to an existing timber diaphragm

force-resisting members; increasing the stiffness of the diaphragm; or modifying internal structural and non-structural elements such that the excessive drifts can be tolerated.

Poor connectivity of timber diaphragms to walls is also a common problem. Timber diaphragms that lack blocking of joists at shear walls can roll-over at the edges. Adding blocking and ensuring adequate nails or metal connectors are provided to resist shears and local overturning or rolling of the blocking will address this deficiency.

Lack of continuity across diaphragms constructed of materials with limited tensile capacity, such as timber diaphragms, can lead to significant damage, particularly in structures constructed with heavy walls. Under the influence of large inertial loads at the edge of the diaphragm, diaphragms with limited tensile capacity can rip apart unless directly provided with continuous ductile elements to tie the structure together. This continuity is best provided by the timber framing members, however, timber framing connections typically have little tensile capacity. Metal hardware such as hold-downs can be installed across joints to remedy this deficiency. Figure 12-26 presents a common method for providing adequate continuity in an existing timber diaphragm. Symmetrical connectors should be utilized where possible to minimize eccentric loads on existing framing.

The number of continuity ties, their location, and capacity is dependent upon a number of factors including flexibility and tensile capacity of the diaphragm, tributary mass of walls, and dynamic response of the diaphragm. The structural community has developed a simplified method of providing for attachment of heavy walled structures to timber diaphragms and providing continuity across the diaphragm through the use of sub-diaphragms. This method can be used for new construction or retrofitting existing buildings. For more information on wood sub-diaphragms see ATC-7[12-14] or Beyers[12-15].

Concrete Diaphragms - Common deficiencies of concrete diaphragms include inadequate shear capacity, inadequate chord capacity and excessive shear stresses at diaphragm openings or plan irregularities.

Inadequate shear capacity of concrete diaphragms is commonly addressed by reducing the shear demand on the diaphragm by providing supplemental vertical lateral force resisting elements or by increasing the diaphragm capacity by adding a concrete overlayment. The addition of a concrete overlayment is usually quite expensive as this requires the complete removal of all existing partitions and floor finishes and may require the strengthening of existing beams and columns such that they can resist the added dead load demands due to the weight of the new concrete.

Adding supplemental vertical lateral force resisting elements may be more cost effective depending upon the amount of foundation work required. This approach will also reduce demands on other elements that have deficiencies.

Increasing the *chord capacity* of existing concrete diaphragms can be accomplished by adding new concrete or steel members or by improving the continuity of existing members. Figure 12-27 presents a common method for increasing the chord capacity of a concrete diaphragm with the addition of a new concrete member. This member can be placed above or below the diaphragm. Locating the chord below the diaphragm will typically have less impact on floor-space, however, details to ensure continuity of the chord as it traverses past intersecting beams can be costly. The addition of a steel strap to the outside of the building, doweled into the wall can also provide adequate chord capacity. Sufficient dowels must be provided to transfer the shears from the diaphragm to the walls.

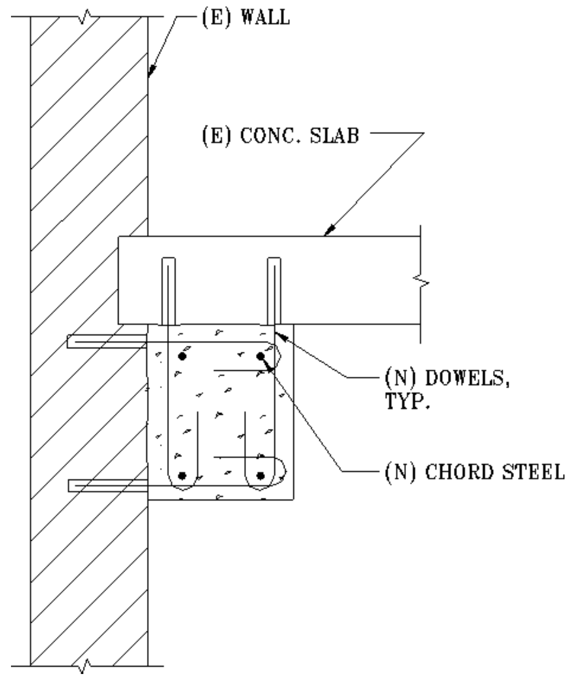


Figure 12-27. Adding a new chord member to an existing concrete diaphragm.

Existing steel frame buildings with concrete floor slabs are frequently constructed with simple or semi-rigid beam-to-column connections. The beams may have adequate capacity to resist vertical demands as well as diaphragm chord demands, however, the connections may have inadequate strength or stiffness to transmit chord forces. Figure 12-28 presents an example of a common approach used to increase the strength and stiffness of an existing steel frame connection to provide adequate chord capacity for the concrete diaphragm.

Excessive shear stresses at diaphragm openings or plan irregularities can be mitigated by distributing the forces in the diaphragm by means of reinforced concrete drag struts cast beneath the slab and made integral through the use of drilled and grouted dowels (Figure 12-29).

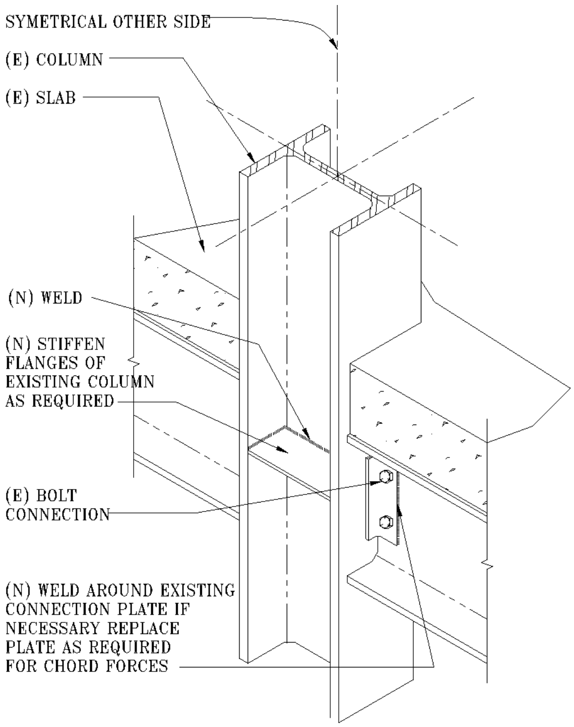


Figure 12-28. Example of details to modify simple connection to provide chord tension capacity

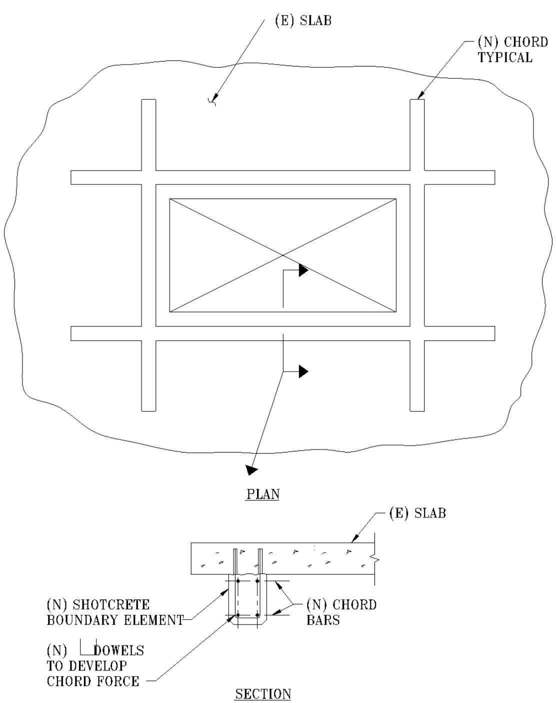


Figure 12-29. Example of diaphragm opening reinforcement.

Alternately, if the opening can be eliminated, the stress concentration can be removed by infilling the opening.

Excessive local diaphragm stresses at a reentrant corner can also be reduced through the introduction of drag struts as shown in Figure 12-30.

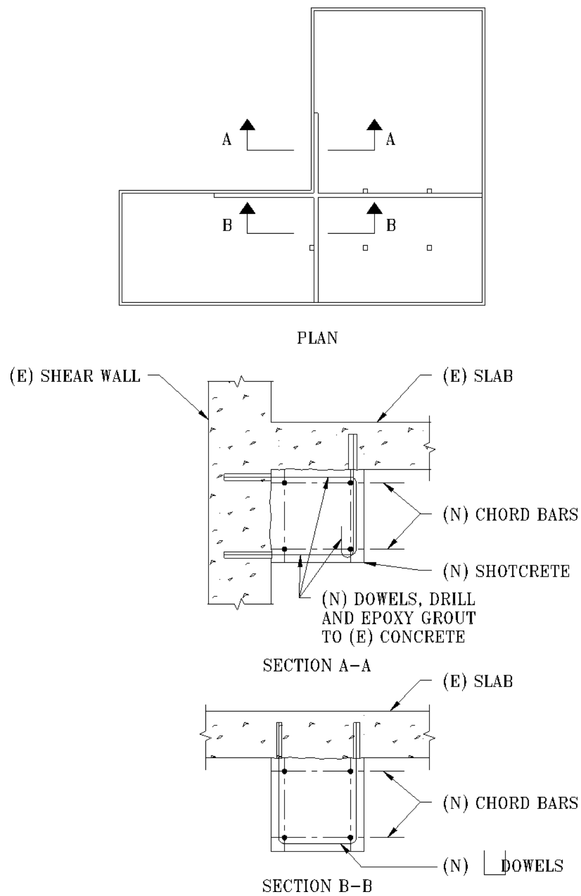


Figure 12-30. Addition of drag struts at concrete reentrant corner

Precast Concrete Diaphragms - Common deficiencies of precast concrete diaphragms include inadequate shear capacity, inadequate chord capacity and excessive shear stresses at diaphragm openings or plan irregularities.

Existing precast concrete slabs (typically constructed using precast tees or cored planks) commonly have *inadequate shear capacity*. Frequently, limited shear connectors are provided between adjacent units and a minimal topping slab with steel mesh reinforcement is placed over the planks to provide an even surface to compensate for the irregularities in precast elements. The composite diaphragm may have limited shear capacity.

Strengthening the existing diaphragm is generally not cost effective. Adding a reinforced topping slab is generally prohibitive because of the added weight. Adding mechanical connectors between units is generally not practical, because the added connectors are unlikely to have sufficient stiffness, compared to the topping slab, to resist an appreciable load. The connectors would therefore need to be designed for the entire shear load assuming the topping slab fails. The number of fasteners, combined with edge distance concerns typically makes this impractical.

The most cost effective approach is generally to reduce the diaphragm shear forces through the addition of supplemental shear walls or braced frames.

Inadequate chord capacity on a precast concrete deck can be mitigated in a similar fashion as discussed earlier for a cast-in-place concrete diaphragm. A new chord member can be added above or below the precast concrete deck as shown in Figure 12-27.

Excessive stresses at diaphragm openings or plan irregularities in precast concrete diaphragms can also be mitigated in a similar manner as described earlier for cast-in-place concrete diaphragms (as shown in Figures 12-29 and 30).

Steel Deck Diaphragms - Inadequate diaphragm shear and chord capacities, and excessive diaphragm stresses at diaphragm openings or plan irregularities are common deficiencies in steel deck diaphragms.

Steel deck diaphragm *shear capacity* is limited by the shear capacity of the corrugated sheet steel and the fastener capacity connecting adjacent deck sheets (typically through crimping of the seams or seam welding). The capacity is also controlled by the spacing of deck-to-beam connections which prevent out-of-plane buckling.

A modest amount of increased shear capacity can be achieved through additional welding at sheet seams. Removal of insulation fill on roof decks is required to provide access for the welding.

Should added welding be insufficient or impractical, reducing the demands to below the shear capacity of the diaphragm can be accomplished by adding supplemental vertical lateral force-resisting elements. New steel braced frames or shear walls can be added to cut down the diaphragm span. Drag struts connecting to the new braced frame or shear wall will be required to distribute the loads into the diaphragm.

Inadequate flexural capacity of steel deck diaphragms may occur due to incomplete or inadequate chord members. Perimeter steel beams or ledgers need to be continuous to act as chords. Beam-to-column connections at the perimeter may have inadequate stiffness or strength in the axial direction of the beams to adequately act as chords. Increasing the strength and stiffness of these connections similar to the method shown in Figure 12-31 can address this deficiency.

Excessive local diaphragm stresses at a reentrant corner in a steel deck diaphragm may be the result of an inadequate load path between girders (or beams) and the steel deck, particularly where open-web steel joist (OWSJ) construction is utilized. In this type of construction the joists span between girders with the top chord of the joist being placed on top of the top chord of the girder. The top of the joist and the girder are therefore not at the same elevation. Hence, the steel deck is not directly connected to the girder. Shear transfer between the girder and deck must therefore occur through the joist-to-girder connection. Figure 12-32 presents a common situation where this condition occurs and a typical method that is utilized to correct the deficiency.

Excessive stresses will occur in the diaphragm at the reentrant corner shown in Figure 12-32 unless adequate drag struts exist to distribute these stresses along an extended length of the diaphragm. The joist and girder at the reentrant corner may provide this drag strut function provided the joist is adequately connected to the shear wall at the reentrant corner. Frequently the framing is constructed as shown in Figure 12-32 (b), without the cap plate. The OWSJ support connection may have inadequate capacity and stiffness to transfer lateral loads from the deck to the girder, and hence the OWSJ connection may fail and/or the diaphragm may fail adjacent to the reentrant corner. The addition of a cap plate with adequate connection capacity to both the metal deck and truss will provide the necessary load path and distribute forces into the diaphragm.

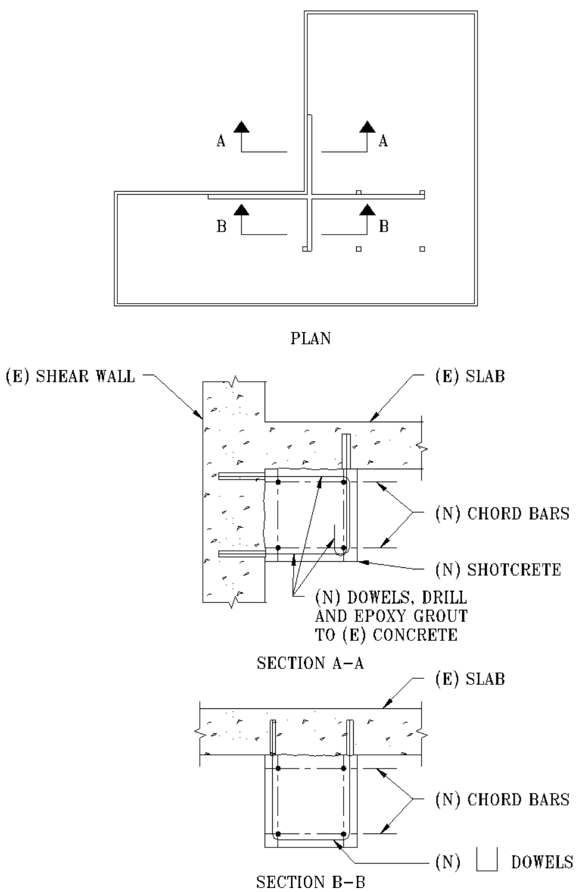


Figure 12-31. New chords at reentrant diaphragm corner.

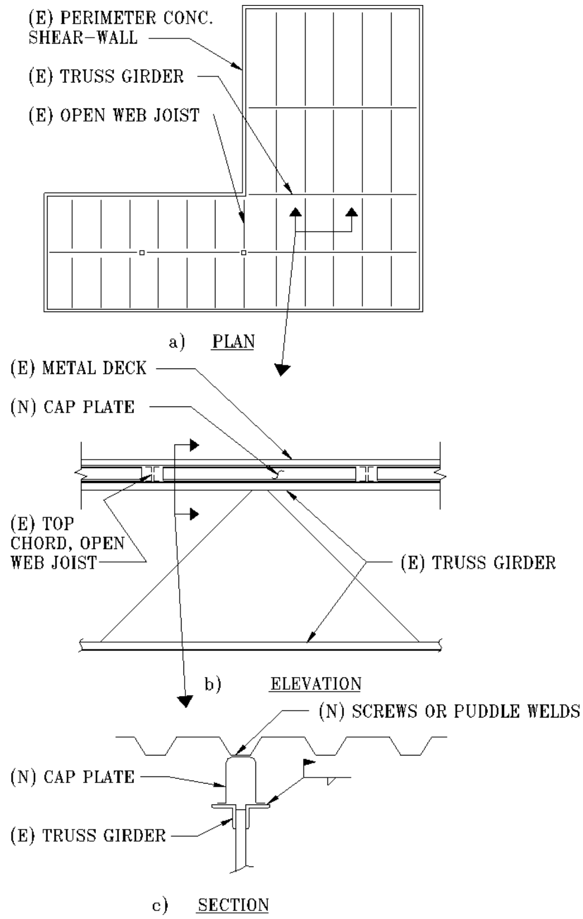


Figure 12-32. Strengthening of the steel deck-to-girder connection, (a) plan view, (b) elevation of truss girder, (c) section of metal deck and top chord of truss girder.

12.5.6 Non-structural Elements

Common non-structural elements include non-load bearing walls, cladding, ceilings, ornamentation, and mechanical and electrical services and utilities.

Non-load bearing walls - Common upgrade techniques for improving the performance of buildings with non-structural walls which adversely affect the seismic response of a building include: removing the walls; removing the walls and replacing them with walls constructed of relatively flexible materials (e.g. gypsum board sheathing); or modifying the wall connections such that they will not participate in resisting lateral loads. The first two alternatives are the most commonly utilized.

Removal and replacement of existing hollow clay tile, concrete, or brick masonry partitions is the preferred method of addressing the inadequate out-of-plane capacity of non-structural partitions. Replacement may not be practical, however, due to cost or the desire to preserve architectural finishes.

Alternatively, steel strongbacks can provide out-of-plane support. Steel members are installed at regular intervals and secured to the masonry with drilled and grouted anchors. The masonry spans between the steel members and the steel members either span vertically between floor diaphragms or horizontally between building columns. An example of a strongback installation detail is shown in Figure 12-21.

A third method for mitigating masonry walls with inadequate out-of-plane capacity is to provide a structural overlayment. The overlayment may be constructed of plaster with welded wire mesh reinforcement, or concrete with reinforcing steel or welded wire mesh. This approach is used at times merely to provide containment of the masonry. Non-structural masonry walls are frequently used as firewalls around means of egress. Egress walls with deficient out-of-plane capacity can fail or result in rubble blocking the egress. Containment of the masonry with a plaster or concrete overlayment can maintain free means of egress, although the walls may have to be replaced following a major seismic event.

Architectural Elements - Building cladding, veneers, ceilings, and partitions were frequently not designed or installed to safely accommodate seismic deformations in a building.

Precast concrete cladding panels were installed in many buildings with nearly rigid connections. The connections may not have the flexibility or ductility to accommodate large building deformations. Failure of the connection may result in heavy panels falling away from the building. Complete correction of this deficiency is likely to be costly as numerous panel connections would need to be modified to accommodate anticipated building drifts. This may require removal and reinstallation or replacement of the panels. A more economical solution is to install redundant flexible/ductile connections that will hold the panels from falling should the existing connections fail.

Improper design and installation of precast concrete cladding may also be more than just a cladding connection problem. The cladding may act as an unintended lateral load resisting element should the connections be rigid and insufficient gaps be present between panels. Correcting this deficiency can be accomplished by installing occasional seismic joints in the panels to minimize the stiffness of the cladding or by stiffening the existing lateral force resisting system.

Stone or masonry veneers on buildings may be inadequately secured. During strong ground shaking the wall to which veneers are attached may deform causing the veneer layer to separate from the wall. The veneers may become falling hazards unless their anchorages can accommodate this deformation. Remedying this situation may be expensive. Removal and replacement of the veneer with adequate anchorage is one option. A second option is to decrease the deformation of the supporting wall by adding supplemental stiffness to the structure.

Building ornamentation such as parapets, cornices, signs and other appendages are another potential falling hazard during strong ground shaking. Unreinforced masonry parapets with heights at least 1-1/2 times their width are particularly vulnerable to damage. Parapets are commonly retrofit by providing bracing back to the roof framing (Figure 12-33). Providing adequate flashing details at the roof connections is an important part of the upgrade details.

Some cornices or other stone or masonry appendages are retrofit by installing drilled and grouted anchors at regular intervals. Others are retrofit by removal and replacement in kind with adequate anchorage or replacement with a lightweight substitute material such as plastic, fiberglass, or metal.

The most common failure observed in a moderate earthquake occurs to *suspended acoustical tile ceilings*. Failure typically occurs at the perimeter of the building. Unbraced ceilings are significantly more flexible than the floors or roofs to which they are attached. The ceilings therefore will sway independent from the floor or roof, typically resulting in the runners at the walls breaking their connections. This deficiency can be reduced by stiffening the suspended ceiling system through the installation of diagonal wires at regular spacing between the ceiling grid and structural floor or roof members. Vertical compression struts are also required at the location of the diagonal wires to resist the upward component of force caused by the lateral loads. A typical installation detail is shown in Figure 12-34. Current code standards such as those contained in UBC-97 and IBC-2000 provide standards for the installation of new suspended ceiling systems that can also be utilized for the upgrade of existing ceiling systems (see Chapter 13 for more information on design of non-structural systems and components).

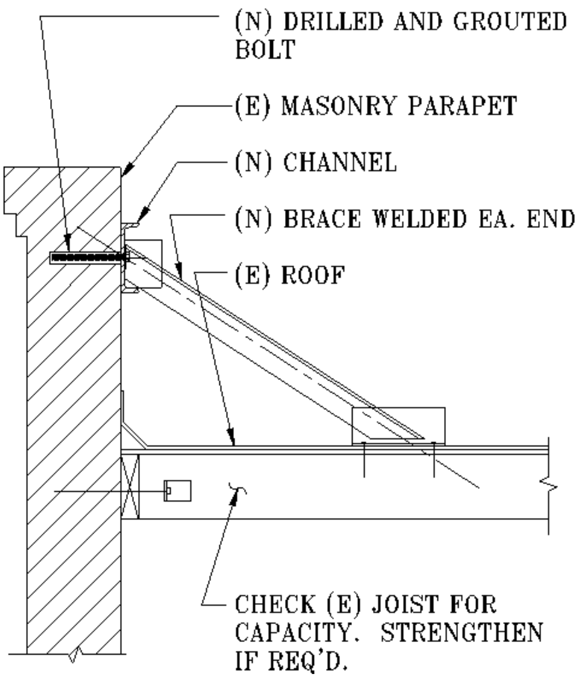


Figure 12-33. strengthening of a masonry parapet with steel braces.

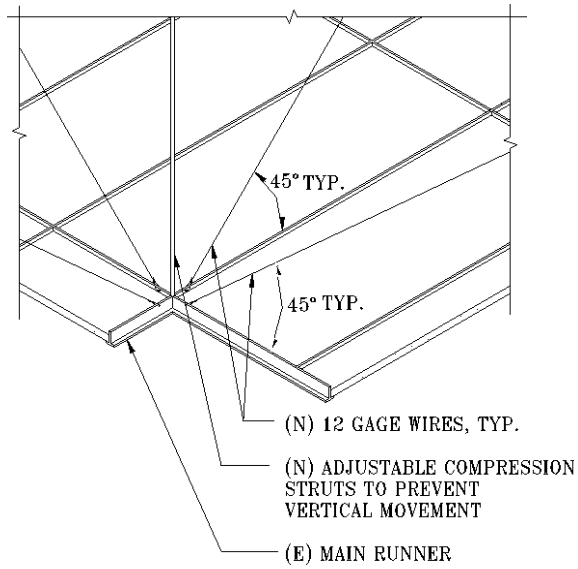


Figure 12-34. Lateral bracing of a suspended ceiling.

12.6 Examples

12.6.1 Tilt-Up Building Seismic Upgrade

A large number of precast low-rise concrete buildings with wood diaphragms were constructed in the U.S. beginning in the 1950's. This economical mode of construction was used for many office, warehouse, and light manufacturing buildings. The 1971 San Fernando earthquake, however, exposed a number of deficient conditions in typical tilt-up construction buildings. The tilt-up building shown in plan in Figure 12-35 contains many of these deficiencies. The following describes one method to upgrade the building.

The existing building has the following parameters:

- 1/2 inch C-D, Structural II roof plywood, unblocked with 8d nails at 6 inches on center.
- 3×14 wood ledgers 2×4 joists at 2 foot on center 4×14 purlins at 8-ft. on center glulam beams (GLB) at 24 feet on center, GLB are constructed with cantilever hinges.
- Total roof load including roofing and framing is 12 lb/ft^2.

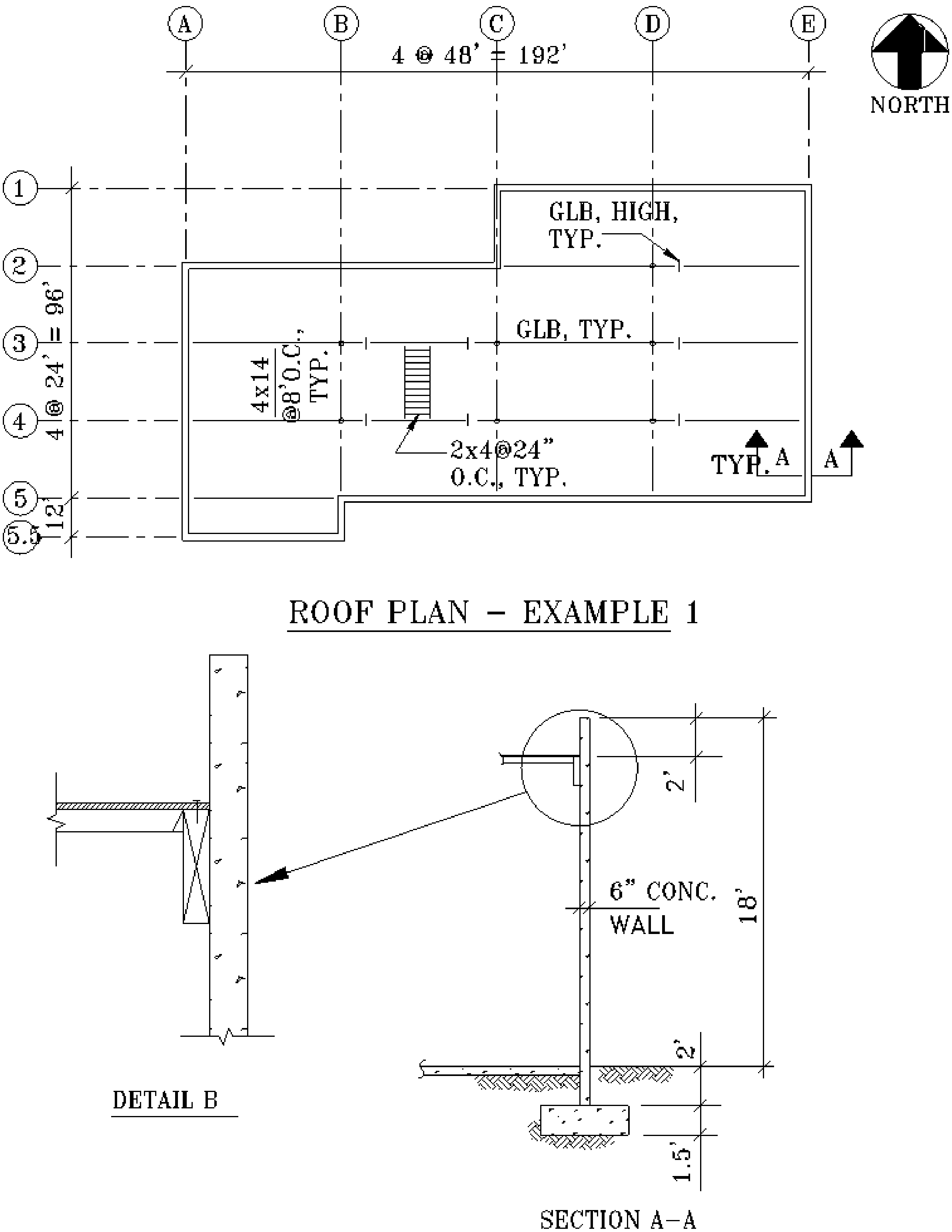


Figure 12-35. Example tilt-up building , plan and wall elevation

– Walls are 6-in. thick precast concrete panels, 18 feet high. The roof is connected 16 feet above grade.
– The wall panels are connected to the floor slab via #4 dowels at 24 inches on center.

The general upgrade objective is to bring the building up to the design provisions of the UBC-9[a] Uniform Building Code pursuant to discussions and a written understanding between the owner and the engineer. Therefore the building base shear is calculated as follows:

$V = ZICW/R_W$ where:

Z=0.4 , zone 4

I=1.0

C=2.75 (maximum)

R_W = 6 concrete shear wall, bearing

Therefore: V = 0.183W

The weight of the wall tributary to the roof diaphragm = 0.5 ft(150 pcf)(18 ft)^2/(2×16 ft) = 760 plf. The roof demands are therefore:

$W_1 = 0.183 \times (12\text{psf} \times 84\text{ ft}+760\text{ plf} \times 2\text{ walls}) = 463\text{ plf}$

Similarly:

w_2 = 436 plf

w_3 = 489 plf

w_4 = 384 plf

w_5 = 700 plf

w_6 = 489 plf

[a] Since performance based design methods are presented in Chapter 15, in the examples presented in this chapter we utilize the more traditional way of upgrading buildings. That is, to bring the structure up to one of the previous editions of the building code used to design new buildings. Rather arbitrarily, we have selected UBC-91 provisions as the objective criteria for examples of this Chapter. Obviously, other editions of this or other applicable codes may have been used.

Figure 12-36 depicts the seismic demand on the roof diaphragm in both the north-south and east-west directions and the wall reactions and diaphragm shears assuming a tributary distribution of loads (flexible diaphragm).

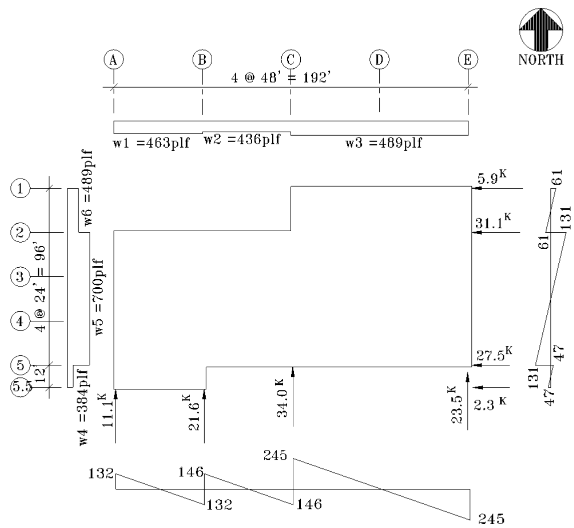


Figure 12-36. Example tilt-up building, - Seismic demands, reactions and shear diagrams.

The plywood is unblocked and configured according to UBC-91 Table No. 25-J-1, case 2 in the north-south direction and case 4 in the east-west direction. The allowable shear capacity for the diaphragm per Table 25-J-1 is 180 plf.

Deficiencies

The example building has the following obvious deficiencies:

1. The diaphragm has inadequate shear capacity at lines C and E (245 plf demand > 180 plf capacity).
2. Out-of-plane wall anchorage is provided via cross-grain bending in the ledgers, which is not permitted per UBC-91 2337(b)9D.
3. 3.No continuity ties exist per UBC-91 2337(b)9C.
4. Inadequate collector connections are provided at the reentrant corners, i.e., at lines B and C in the north-south direction and lines 2 and 5 in the east-west direction.

5. Overturning of wall panels at lines B between 5 and 5.5 and at line C between 1 and 2 are potential deficiencies based on observation of the lateral load resisting system. Therefore the wall overturning at line B between 5 and 5.5 is checked as follows:

– Weight of the wall above ground equals 16,200 lbs and the weight below ground including the footing equals 7,200 lbs. Therefore, the total gravity load for the wall is 23,400 lbs.
– The tributary lateral load from the wall equals 0.183W = 0.183×16,200 = 2,965 lbs.
– The wall overturning forces and resisting forces are shown in Figure 12-37.

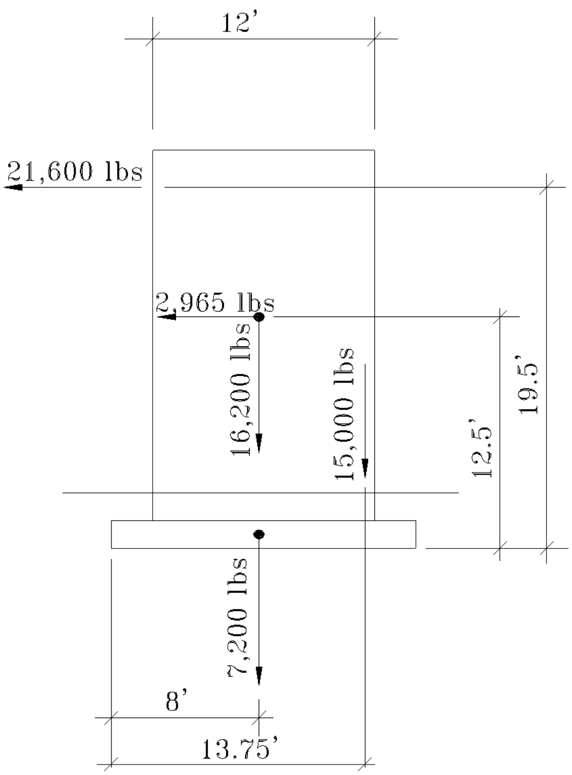


Figure 12-37. Example tilt-up - building ,Wall Reaction at line B between 5 and 5.5

The overturning moment and the resisting moment is calculated as follows (assuming rotation occurs at the toe of the footing and that a 15,000 pound dead load of the return wall will be mobilized):

$$M_{OT} = 21{,}600\text{lbs}\times 19.5\text{ft} + 2{,}965\text{lbs}\times 12.5\text{ft}$$
$$= 458{,}263 \text{ lb-ft.}$$

$$M_R = (16{,}200 \text{ lbs} + 7{,}200 \text{ lbs}) \times 0.85 \times 8\text{ft}$$
$$+ 15{,}000 \text{ lbs} \times 0.85 \times 13.75 \text{ ft}$$
$$= 334{,}432 \text{ lb-ft.}$$

Note: Dead loads are reduced by 0.85 when used to resist uplift [UBC-91 2337(a)].

$M_{OT} > M_R$, therefore not acceptable

The wall at line C between lines 1 and 2 was checked in a similar manner and the restoring moment was found to exceed the overturning moment and hence was determined to be adequate. Therefore, the fifth deficiency is that the wall at line B between 5 and 5.5 has inadequate capacity to resist overturning.

Strengthening Options

The following options are considered for addressing the above described deficiencies:

1. Correcting the inadequate roof diaphragm shear capacity can be accomplished by:
   a. Reducing the diaphragm shear by introducing a new lateral force resisting element (e.g. shear wall or braced frame) between lines C and E, or
   b. Strengthening the roof diaphragm, where demands exceed capacity by adding blocking and nailing.

   Option 1b is selected for this building. A new lateral force-resisting element (option 1a) would reduce the open space layout of the building and would require costly foundation work. Removal of roofing would be required for both options. Removal would be required for option 1a to permit nailing between the plywood and roof joist collectors required to correct deficiency number 4. Roofing removal would be required for option 1b in designated areas such that new blocking and nailing may be installed.
2. New hardware is required between the roof framing and the concrete walls to provide direct out-of-plane connection capacity.

3. New hardware is required at GLB hinge connections, and subframing intersections at main framing to provide adequate continuity.
4. New hardware is required at GLB hinge connections, and subframing intersections with main framing to provide adequate collector capacity at framing attached to reentrant corner walls.
5. Two options were considered to address the inadequate overturning capacity of the wall at line B between 5 and 5.5:
   a. Improving the foundation to resist the overturning force, or
   b. Permitting the wall to rock.

Option 5b is selected as the least costly alternative of the two. Permitting the wall to rock will result in a redistribution of diaphragm shears to the west of line C. A diaphragm shear check (shown below) demonstrates no adverse conditions result because of this redistribution.

The lateral roof load the wall is capable of resisting is determined by summing the moments about the toe of the wall (see Figure 12-37):

The sum of the moments about the toe of the wall = 0.0, therefore:

(16,200 lbs + 7,200 lbs) ×0.85×8 ft + 15,000 lbs×85×13.75 ft - 2,965 lbs×12.5 ft - P ×19.5 ft = 0.0

Solving for P, we have

P = 15,250 lbs.

Figure 12-38 depicts the new distribution of diaphragm shears in the north-south direction. The shear stress, 188 plf at the west side of Line C exceeds the capacity of 180 plf by less than 5%, hence is deemed acceptable.

Strengthening Provisions

1. Diaphragm Shear

Increasing the capacity of the roof diaphragm is accomplished by adding blocking with 8d nailing at 6 inches on center. The new capacity equals 270 plf (UBC-91 Table 25-J-1, blocked diaphragm for 15/32 C-D structural II sheathing). The blocking is to be added in the areas shown in Figure 12-39.

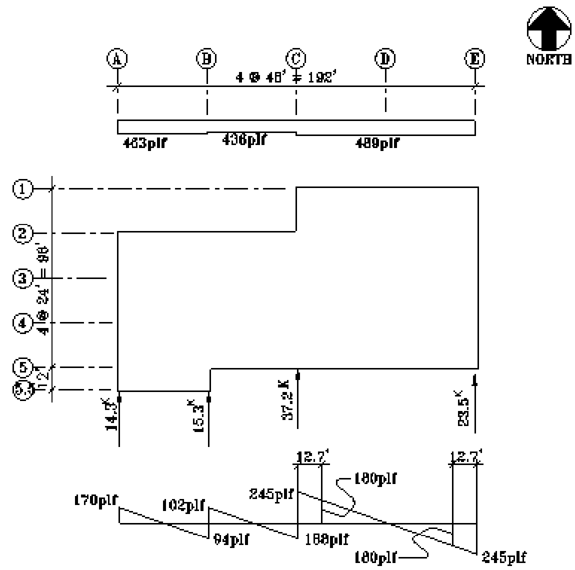


Figure 12-38. Example tilt-up building - Roof plan – Revised shear distribution assuming wall at line B resists a maximum of 15.3 kips.

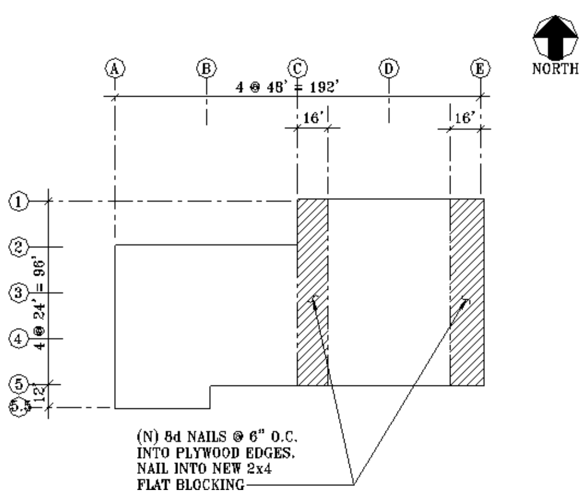


Figure 12-39. Example tilt-up building - Roof plan - Areas requiring blocking with added nailing.

2. Out-of-Plane Anchorage

Positive direct connections between the wall panels and the roof construction is required per UBC-91 2337(b)[12-19]. The demands for the out-of-plane anchorage are calculated per UBC-91 equation 36-1:

$F_p = ZIC_pW_p$

where: $C_p = 0.75$ for the outer quarters of the diaphragm, and $C_p = 1.125$ for center half of the diaphragm (UBC-91 Table 23-P, note 3)

The cost of the installation in a retrofit design is primarily labor not hardware. Therefore all out-of-plane anchorage is designed utilizing $C_p = 1.125$. The out-of-plane anchor demand is therefore:

$$F_p = ZIC_pW_p = 0.4\times1.0\times1.125\ W_p$$

$$= 0.45\ W_p$$

The tributary weight of the wall is

$$W_p = 0.5(150pcf)(18ft)^2/(2\times16ft) = 759plf$$

The wall was checked and determined to be capable of spanning 8 feet. Hence, the new out-of-plane wall anchors are to be located at 8 ft on center. The connection demand is therefore

$$F_p = 0.45 \times 759\ plf \times 8\ ft$$

$$F_p = 341\ plf \times 8\ ft = 2{,}732\ lbs/anchor$$

The joists are checked for the DL + EQ load and are found to be adequate.

3. Continuity Ties

The use of subdiaphragms is permitted to meet the continuity tie provisions of UBC-91 2337(b)9.C. The subdiaphragm configuration selected for providing continuity across the building is depicted in Figure 12-40. Alternate configurations could also be utilized.

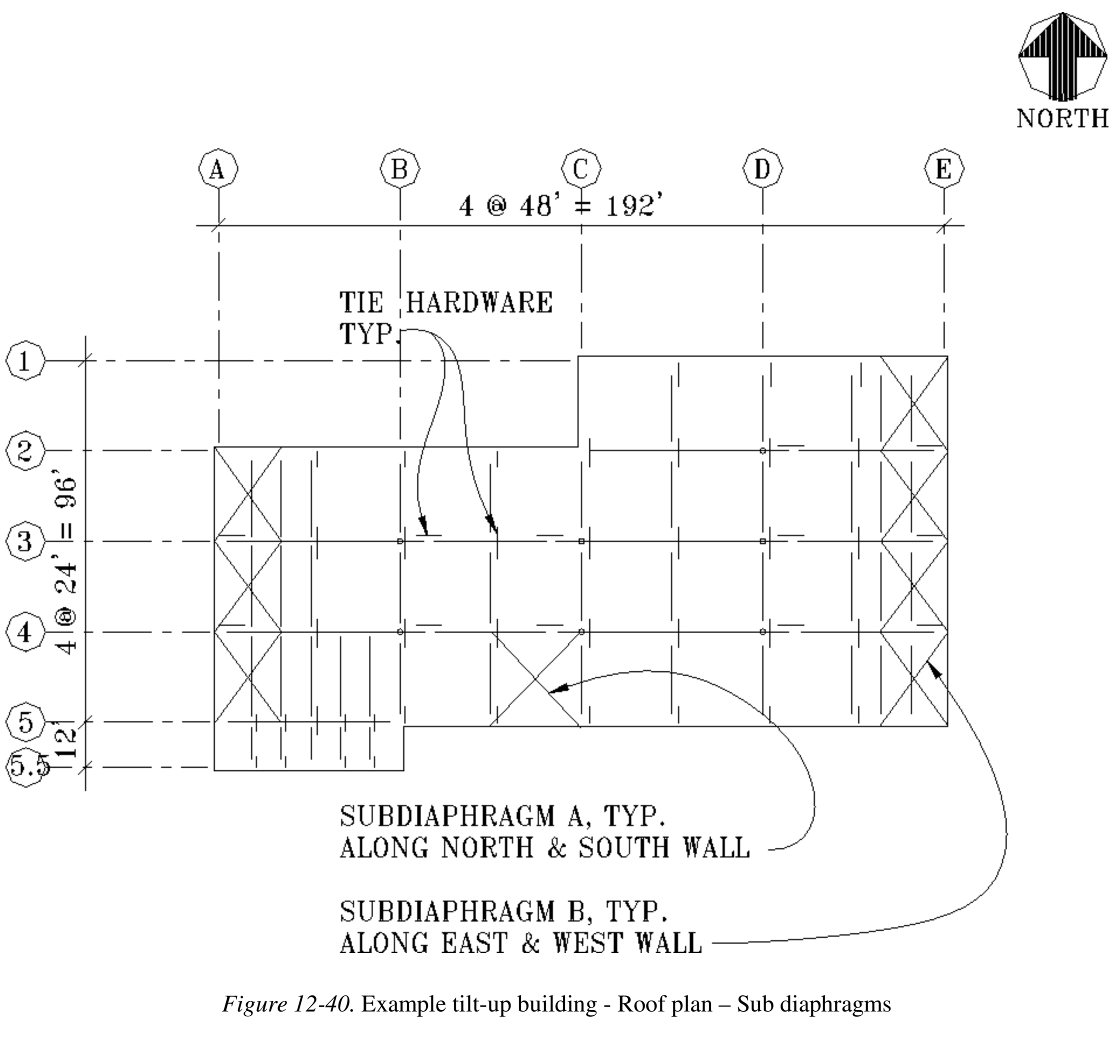


Figure 12-40. Example tilt-up building - Roof plan – Sub diaphragms

North-south direction

Continuity ties shall transfer the wall weight of

$F_p = ZICW_p = 0.4\times1.0\times0.7\times W_p = 0.3W_p$
$= 0.3\times759 = 228$ plf (> 200 plf min per UBC-91 Section 2310)

24 foot square subdiaphragms, designated "A" are provided in the north-south direction. 24 feet wide by 16 feet deep diaphragms would provide adequate shear capacity 228 plf × 12 ft/16 ft = 171 plf demand < 180 plf capacity), however connections would have to be added across the subpurlins to provide a subdiaphragm chord. The GLBs serve as the chord members when a 24 foot subdiaphragm is utilized.

Therefore, the continuity tie demand across the GLBs = 228 plf × 24 ft = 5,472 lbs

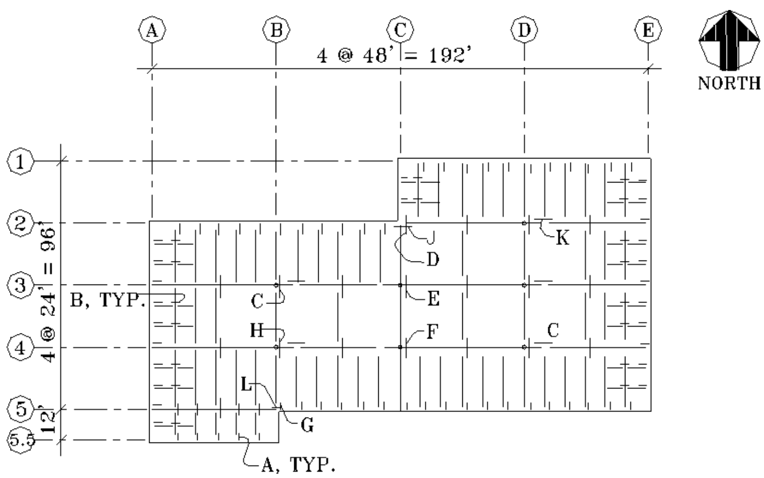
East-west direction

Subdiaphragms B are 24 feet wide and 16 feet deep in the east-west direction.

The diaphragm shear demand as calculated previously is 171 plf . The continuity demand = 228 plf×24 ft = 5472 lbs.

4. *Collectors*

The collector forces at the reentrant corners are shown in Figure 12-41. Detailing of the collector connections at the reentrant corner at line C-2 must provide for significant capacity (30,700 lbs and 18,300 lbs in the N-S and E-W directions respectively). The connection shown in Figure 12-42 is symmetrical, with respect to the wall and the roof framing in both directions, as significant additional strengthening would be required to resist moments induced by an unsymmetrical connection.



DESIGN	DESCRIPTION	DEMAND	NO. REQUIRED
A	WALL ANCHOR =	2,732 lbs	67
B	CONTINUITY TIE =	2,732 lbs	21
C	CONTINUITY TIE *=	5,472 lbs	19
D	COLLECTOR CONN=	30,700 lbs	1
E	COLLECTOR CONN=	20,500 lbs	1
F	COLLECTOR CONN=	10,200 lbs	1
G	COLLECTOR CONN=	14,500 lbs	1
H	COLLECTOR CONN=	9,700 lbs	1
J	COLLECTOR CONN=	18,300 lbs	1
K	COLLECTOR CONN=	7,600 lbs	1
L	COLLECTOR CONN=	8,600 lbs	1
			115 CONNECTION TIES REQUIRED

* CONTINUITY TIE FORCE > COLLECTOR FORCE

Figure 12-41. Example tilt-up building -Roof plan – Summary connection upgrade

Strengthening Summary

The new blocking and nailing shown in Figure 12-39 and the new connections shown in Figures 12-41 and 42 present the recommended upgrade requirements for the example tilt-up.

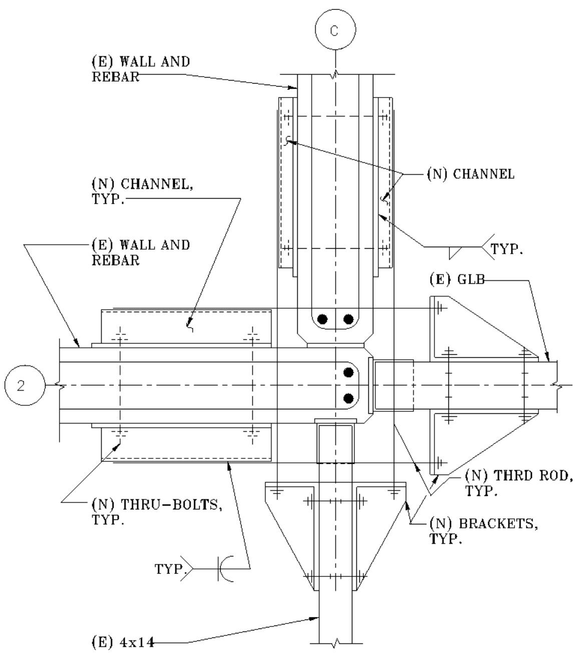


Figure 12-42. Example tilt-up building -Plan view – Collector connections at column line C-2

12.6.2 Unreinforced Brick Masonry Bearing Wall Building Upgrade

The type of building that has experienced the most severe damage in past earthquakes is the unreinforced brick masonry (URM) bearing wall building. This type of construction is prevalent throughout the United States, and is used for commercial, institutional, industrial, low-rise office, and residential occupancies. The URM bearing wall building, shown in plan in Figure 12-43 and an interior elevation in Figure 12-44, is a typical two-story URM structure. The following describes one method to upgrade the building.

The existing building has the following parameters:

Roof framing consists of:

1 by straight sheathing
2×10 joists at 2 foot on center
8×16 purlins at 20 foot on center
8×8 post columns

Total roof dead load including roofing and framing is 13 psf.

The roof is located 26 feet above the ground level first floor.

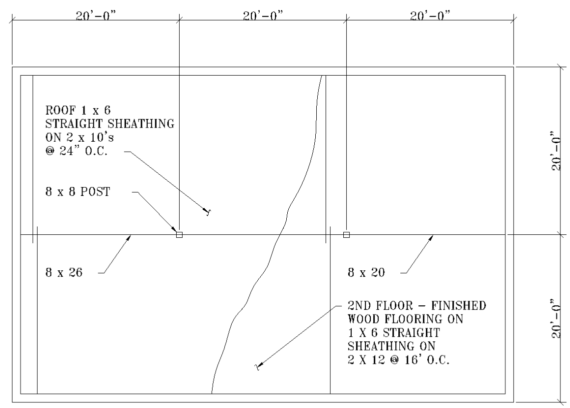


Figure 12-43. Example URM building, Plan layout – Typical URM bearing wall building.

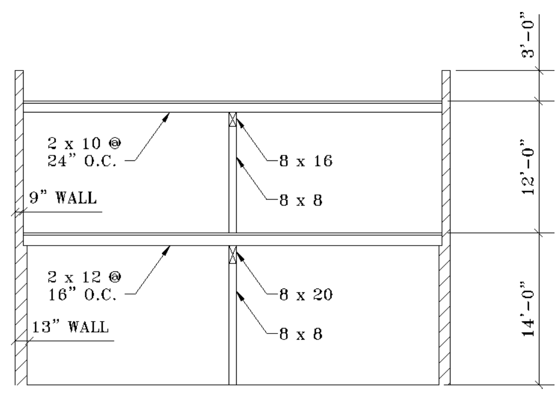


Figure 12-44. Example URM building, Interior elevation.

The second floor is located 14 feet above the ground level first floor.

A three foot tall brick parapet extends above the roof around the entire building.

Second floor framing consists of:

– 1 by straight sheathing with perpendicular finished flooring
– 2×12 joists at 16 inches on center
– 8×20 beams
– 8×8 post columns

- Total second floor dead load framing weight is 12 psf.
- Some partitions exist at the second floor and none at the first, assume 10 psf.
- The second floor walls are comprised of two wythes of brick masonry with a total thickness of 9 inches.

The first floor walls are comprised of three wythes of brick masonry with a total thickness of 13 inches.

- A 16 by 8 foot stairway opening is located in the southwest corner of the second floor diaphragm.

Criteria

The upgrade criteria selected is that specified in the 1991 Uniform Code for Building Conservation (UCBC)[b] "Special Procedure" pursuant to discussions and a written understanding between the owner and the engineer.

Dead Load Distribution

Following is the assumed distribution of roof DL's:

Roof 60'×40'×13 psf = 31,200 lbs

E&W Walls:
90 psf×(12'/2+3')×2(40')× .80[*]×= 51,800 lbs
N&S Walls:
90 psf × (12'/2+3') ×2(60') = 97,200 lbs
Partitions tributary to roof:
(10 psf/2) ×40'×60'= 12,000 lbs

Total Roof= 192,200 lbs

Following is the assumed distribution of 2nd floor DL's:

Floor 60'×40'×12 psf = 28,800 lbs

E&W Walls
[90psf×(12'/2)(2)(40')+130(14/2)(2)(40)]× ×.80[*] = 92,800 lbs
N&S Walls
[130psf×(14'/2)+90psf×(12'/2)]×2(60')= 174,000 lbs
Partitions trib to roof
(10 psf/2) ×40'×60'= 12,000 lbs

Total 2nd Floor = 307,600 lbs

Total Building = 499,800 lbs

- window area in east and west walls is assumed equivalent to 20 percent of the wall area

Seismic Demand Loads

The building base shear is calculated as follows:

V = .33ZW where: Z = 0.4, Zone 4

Therefore: V = 0.13W

The weight of the building has been calculated as 499.8 kips. Hence

V = 0.13×499.8 = 65.0 kips

Demand Vs. Capacity of the Diaphragms

Check the demand versus capacity of the diaphragms using the special procedure outline UCBC-91 Section A109(d).

Roof Diaphragm

Per UCBC-91 A109(d)4.B(i) for a diaphragm without qualifying cross-walls at levels immediately above or below:

$$DCR = 0.833ZWa/[(Sum(v_u \times D)]$$

where:

$W_{a(N\text{-}S)}$ = total tributary dead load in N-S direction = 31,200 lbs+97,200 lbs+12,000 lbs = 140,400 lbs

$W_{a(E\text{-}W)}$ = total tributary dead load in E-W direction = 31,200 lbs+51,800 lbs+12,000 lbs = 95,000 lbs

[b] Since performance based design methods are presented in Chapter 15, in the examples presented in this chapter we utilize the more traditional way of upgrading buildings. That is, to bring the structure up to one of the previous editions of the building code used to design new buildings. Rather arbitrarily, we have selected UCBC-91 provisions as the objective criteria for examples of this Chapter. Obviously, other editions of this or other applicable codes may have been used.

v_u = 100 plf (for straight sheathing per UCBC-91 Table No. A-1-C)

$D_{N\text{-}S} = 40'$
$D_{E\text{-}W} = 60'$

$\Sigma(v_u \times D_{N\text{-}S})$ = 100 plf×2×40'= 8,000 lbs

$\Sigma(v_u \times D_{E\text{-}W})$=100 plf×2×60'=12,000 lbs

Therefore:

$DCR_{N\text{-}S} = 0.833 \times .4 \times 140{,}400/8{,}000 = 5.8$

$DCR_{E\text{-}W} = 0.833 \times .4 \times 95{,}000/12{,}000 = 2.6$

From UCBC-91 Figure A-1-1 at $DCR_{N\text{-}S}$ = 5.8 a diaphragm span of 60' is unacceptable. However at $DCR_{E\text{-}W}$ =2.6 the shorter diaphragm span is acceptable.

Second Floor Diaphragm

The 2nd floor diaphragm shear demands are calculated as follows:

$W_{N\text{-}S} = (28.8^k+174.0^k+12^k) \times 0.13/60' =$

$=215^k \times 0.13/60' = 0.4$ k/ft

$W_{E\text{-}W} = (28.8^k+92.8^k+12^k) \times 0.13/40' =$

$= 134^k \times 0.13/40' = 0.4$ k/ft

$DCR = 0.833ZW_a/[(\Sigma(v_u \times D)]$

where:

$W_{a(N\text{-}S)}$ = total tributary dead load in N-S direction = 215,000 lbs

$W_{a(E\text{-}W)}$ = total tributary dead load in E-W direction = 134,000 lbs

v_u = 500 plf (for straight sheathing with perpendicular wood flooring per UCBC-91 Table No. A-1-C)

$D_{N\text{-}S} = 40' + 24' = 64'$

$D_{E\text{-}W} = 60' + 52' = 112'$

$\Sigma(v_u \times D_{N\text{-}S})$ = 500plf×(64')= 32,000 lbs

$\Sigma(v_u \times D_{E\text{-}W})$=500plf×(112')=56,000 lbs

$DCR_{N\text{-}S} = 0.833 \times .4 \times 215{,}000/32{,}000 = 2.2$

$DCR_{E\text{-}W} = 0.833 \times .4 \times 134{,}000/56{,}000 = 0.80$

Therefore the second floor diaphragm meets the UCBC-91 special procedure criteria.

Mitigate Roof Diaphragm Deficiencies

The owner of the building does not want walls in the first floor. Hence crosswalls can not be continuous from the roof to the ground. By adding a crosswall in the north-south direction between the roof and second floor diaphragms the roof diaphragm would be acceptable. Try 3/8" C-D plywood on two sides secured with 8d nails @ 6 inches on center (capacity per UBC-91 Table 25-K-1 is 264 plf). Check UCBC A109(d)4.B.(iv)

$DCR = 0.833ZW_a/[(\Sigma(v_u \times D)]$

where:

$W_{a(N\text{-}S)}$ = 140,400+215,000 lbs = 355,400 lbs

$\Sigma(v_u \times D)$= 100×2×40+500×(64')= 40,000 lbs

Therefore:

DCR = 0.833×4×355,400 lbs/40,000 lbs = 3.0

From Figure A-1-1 at $DCR_{N\text{-}S}$ = 3.0 the 60' diaphragm is acceptable. Therefore the only upgrade required to address the deficient roof diaphragm is to add a crosswall in the north-south direction. Try one 12' crosswall. Recheck A109(d)4.B.(ii):

$DCR = 0.833ZW_a/[(\Sigma(v_u \times D) + v_{cb}]$

where:

$W_{a(N\text{-}S)}$ = 140,400 lbs

$\Sigma(v_u \times D)$= 100 plf ×2×40'= 8,000 lbs

v_{cb} = 12'×2×264 plf = 6,336 lbs

Therefore:

DCR=0.833×.4×140,400lbs/(8,000+6,336 lbs)

= 3.26

and

v_{cb} = 6,336 lbs >0.3Sum(v_u×D)= .3(8,000)

= 2,400 lbs (per UCBC-91 A109(d).3.B)

∴ The diaphragm/wall assembly is acceptable

Address Cross-wall Overturning

The cross-wall will impart large vertical loads on the existing beams due to overturning moments. Therefore additional framing will be needed to address these loads.

Design the floor framing to support the capacity of the wall. For a 12' long wall 12' high, the lateral and vertical load will equal = 12 ft × 2 × 264 lbs/ft = 6,336 lbs.

The moment = 6,336 lbs×12 ft/(20 ft)×8 ft =

= 30,413 lb-ft

Two wood beams, a 4×16 in the roof framing and 4×12 beam in the second floor framing is adequate to resist this moment. The addition of these beams and the connection of the new plywood to the framing is shown in Figures 12-45. The beams on the north side of the column are utilized as collectors via the connections to the new beams shown in Figure 12-45.

Diaphragm-To-Wall Shear Connection

Check the shear transfer between the diaphragms and the wall per UCBC A109(d)5.

Roof

V = lesser of: ½ZC_pW_d or V= V_uD

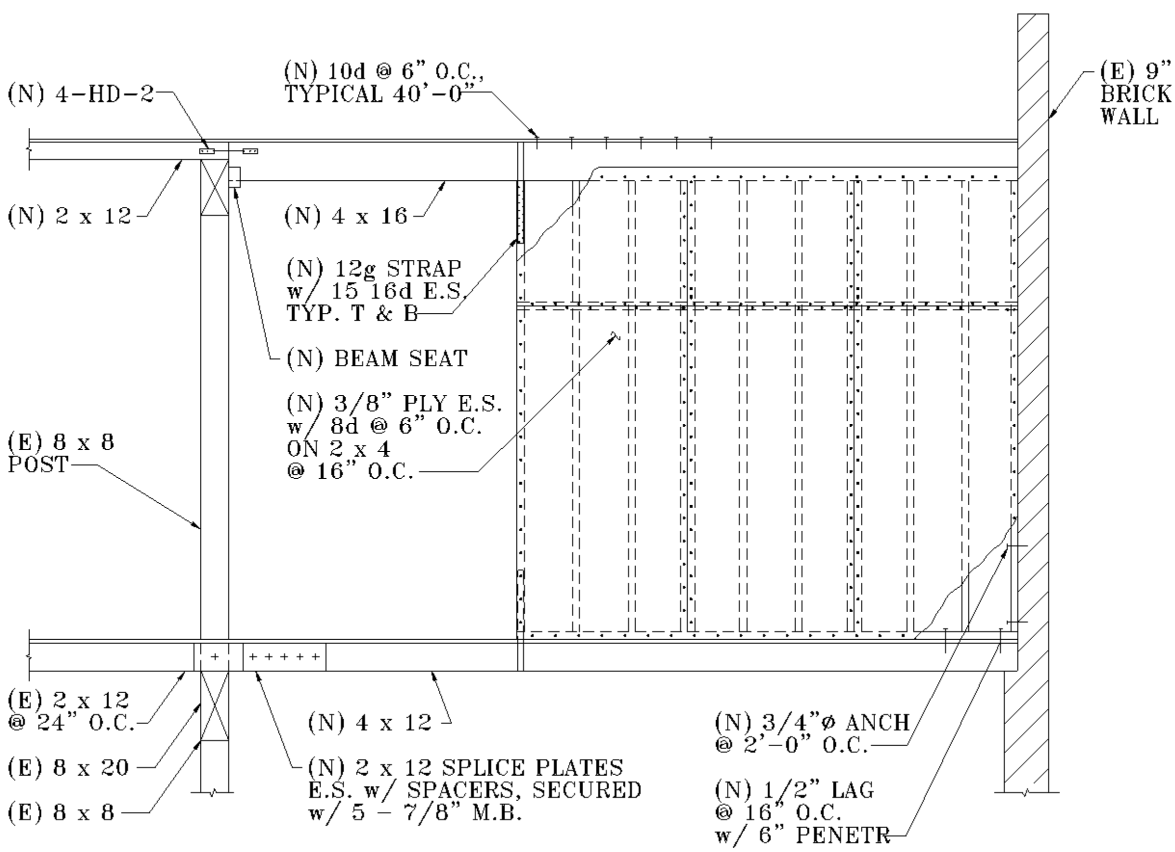


Figure 12-45. Example URM building, New cross wall

N-S

$\frac{1}{2}ZC_pW_d = 0.5\times0.4\times0.5(140.4^k) = 14.0^k$

$V=V_uD = 0.1$ klf×2×40' = 8.0^k controls
Therefore,

v = 8,000 lbs/(2×40') = 100 plf

E-W

$\frac{1}{2}ZC_pW_d = 0.5\times0.4\times0.5(95.0^k) = 9.5^k$
controls

$V=V_uD = 0.1$ klf×2×60' = 12.0^k
Therefore

v = 9,500 lbs/(2×60') = 79 plf

Per UCBC-91 Table No. A-1-D and UBC-91 Table 24M the allowable shear capacity per bolt = 1 k/bolt. Therefore required spacing = 10' o.c., use 4' minimum spacing, capacity = 250 plf.

2nd Floor

V = lesser of $\frac{1}{2}ZC_pW_d$ or $V=V_uD$

N-S

$\frac{1}{2}ZC_pW_d = 0.5\times0.4\times0.75(215^k) = 32^k$
$V=V_uD = 0.5$klf× (40'+24') = 32^k controls

Therefore

v = 32,000/(40'+24') = 500 plf

Use 3/4 bolt at 2'-0" o.c (cap=500 plf)

E-W

$\frac{1}{2}ZC_pW_d = 0.5\times0.4\times0.75(134^k) = 20.1^k$
$V=V_uD = 0.5$klf×2×60' = 60.0^k
Therefore 20.1^k controls and

v = 20,100/(2×60') = 168 plf

use 4' o.c. along E-W walls (cap = 250 plf)

Out Of Plane Anchors

H/t of 2nd story = 144"/9" = 16.0 > 14 therefore unacceptable

H/t of 1st story = 168"/13" = 12.9 < 16 therefore acceptable

Spacing per UCBC-91 A110(e)3 of wall bracing is lesser of ½ wall height or 10' therefore minimum spacing at second floor = 6'.

$F_p = ZIC_pw_p$

$= 0.4\times1.0\times0.75\times w_p = 0.3w_p$

Strength $M=wl^2/8$

where:
w = 0.3×90 psf×6' = 162 plf or 13.5 pli
therefore:
M = 162 plf× $(12')^2/8$ = 2,916 lb-ft.

$S_{req}=M/(1.33F_b)=$

$=2{,}916\times12/(1.33\times.6\times46{,}000) = 1.0$ in^3

Select

TS 3×3×3/16, S = 1.14 in^3, I = 2.60 in^4
Maximum deflection per UCBC A110(e)2.= 1/10×9" =0.9 inch

Defl=$5wl^4/(384EI)$
$= 5\times13.5(12\times12)^4/(384\times29{,}000{,}000\times2.60)$
=1.0 in > 0.9,

therefore

use TS 4×4×3/16, I = 6.59 in^4, S=3.30 in^3

Connection of Strongbacks to Roof

The strongback-to-roof connection load = 162 plf×(12'/2)= 972 lbs
Therefore:

The length of diaphragm required to transfer the load= 972 lbs/(2×100 plf) = 4.9 ft., use 6 feet.

In the direction parallel to the roof framing this can be accomplished by connecting the strongbacks to the joists and connecting the joists to the diaphragm. A typical detail showing this connection is shown in Figure 12-46.

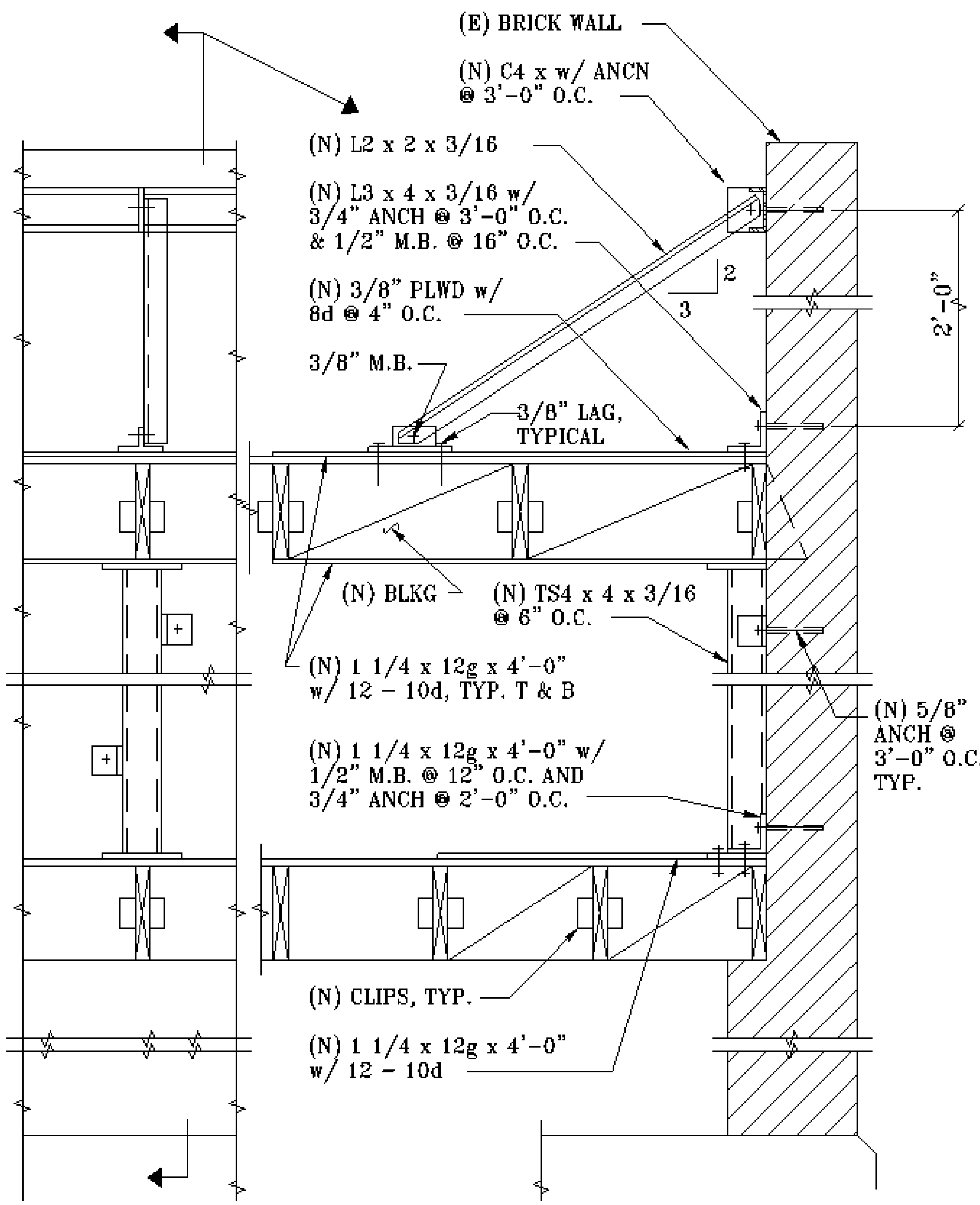


Figure 12-46. Example URM building, Strong backs on east and west walls

In the direction perpendicular to the framing blocking needs to be added such that 8 feet of diaphragm becomes engaged. Alternately plywood can be added at the end of the building reinforcing the diaphragm and permitting a reduction in the required length of the blocking. This latter option is selected as the roofing at the edge of the diaphragm needs to be removed to install new shear connections. Figure 12-47 shows a typical detail of this connection.

Connection of Strongbacks to 2nd floor

Connection load = 162 psf(12)/2 = 972 lbs

The length of diaphragm required to transfer the load therefore
= 972 lbs/(2×500plf) = 1.0 ft. use 2 ft 8 inches.

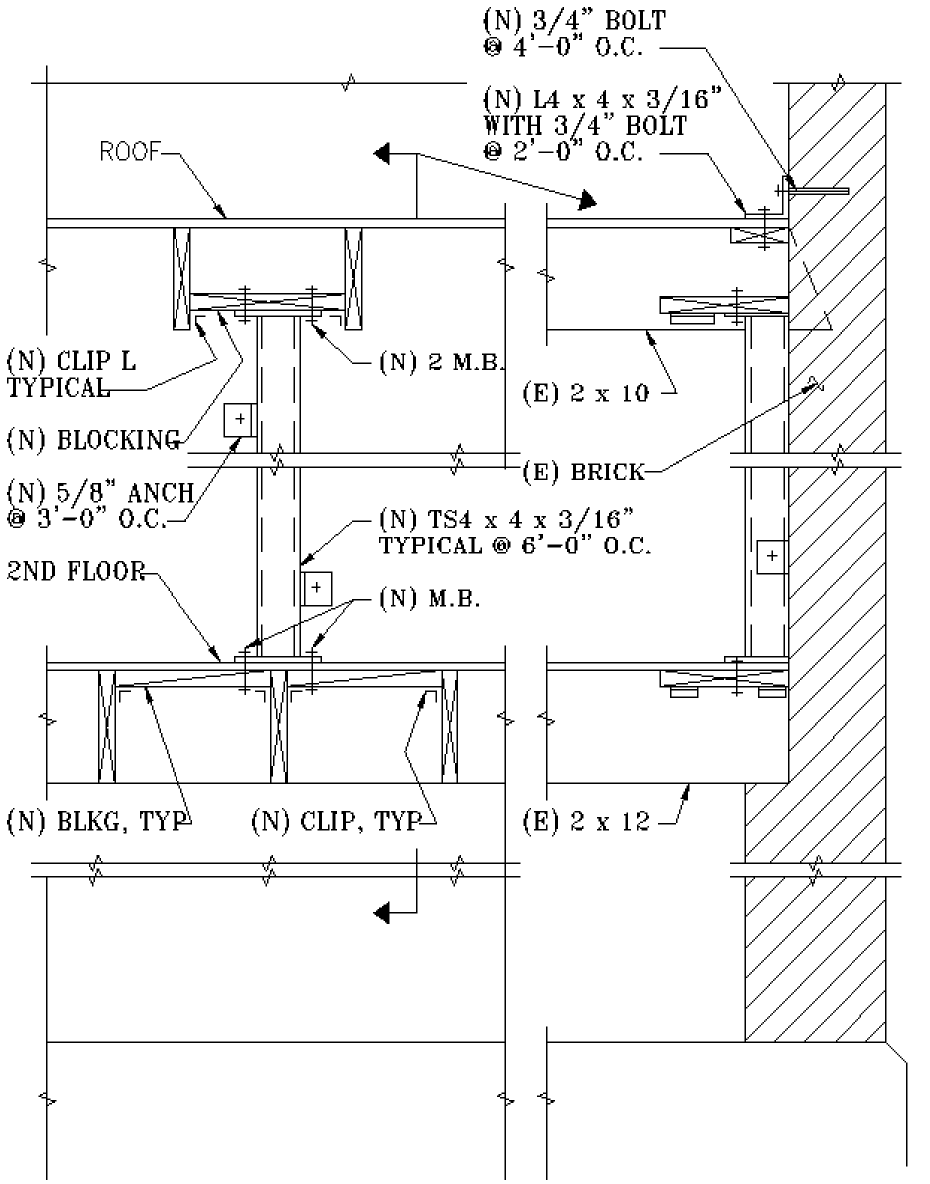


Figure 12-47. Example URM building, Strong backs on north and south walls

Parapet

Brace the parapet at 4 feet on center.

$F_p = ZIC_pw_p$,

$= 0.4\times1.0\times0.75\times w_p=0.3\times w_p$
where

$C_p = 0.75$ for braced parapet per UBC-91 Table 23-P.

$w_p = 90$ psf×3 ft×4 ft = 1080 lbs at 4' o.c.

Therefore:

$F_p = 0.3\times1080$ lbs = 324 lbs at 4' o.c.

Connect the parapet to the braces with a channel spanning 4'.
$M=wl^2/8 = .03\times90$ psf$\times2'(4')^2/8 = 108$ lb-ft.

$S_{req} = M/(1.33F_b) =$

$= 108\times12/(1.33\times.6\times36{,}000) = 0.05\ in^3$

Check deflection

Max Defl. = 0.9" = $5wl^4/(384EI)$, therefore
$I_{req} = 5wl^4/(.9\times384E)$
$=5\times(.3\times90\times2/12)(4\times12)^4/(0.9\times384\times29\times10{,}000{,}000)$
$=0.01\ in^4$

use C3×4.1, $S_y= .20\ in^3$, $I_y = .20\ in^4$
A typical detail of the parapet bracing is shown in Figure 12-48.

Wall Shear

An interior elevation of the west wall is shown in Figure 12-48. The wall piers will be checked for in-plane shear in accordance with UCBC Section A109(d)6.

Second Floor Piers

The wall story force distributed to the east and west walls is:
Smaller of:
$V_R = 0.33Z(W_{wx} + W_d/2)$ or
$0.33ZW_{wx} + v_uD$
$W_{wx} = 51{,}800\ lbs/2 = 25{,}900\ lbs$

$W_d = (31{,}200+12{,}000+97{,}200)/2 = 70{,}200\ lbs$

Therefore
$V_R = 0.33\times0.4\times(25{,}900+140{,}200/2) =12{,}700\ lbs$

and

$V_R =0.33\times0.4\times(25{,}900+100(40)) = 7{,}400\ lbs$ controls

In-place shear tests of the wall were performed in accordance with the provisions of UCBC-91 A106(c)3. The test shears, v_t, were determined to be 100 psi. Therefore, the allowable shear per UCBC-91 A103(b) is:

$v_a= 0.1v_t+0.15P_D/A$

Pier 1

$H_1=4'$, $D_1=1.67'$

$P_{D1} = 2(1.67'+2.5')(7')(90psf)= 5{,}254\ lbs$

Pier 2

$H_2=4'$, $D_2= 3.33'$

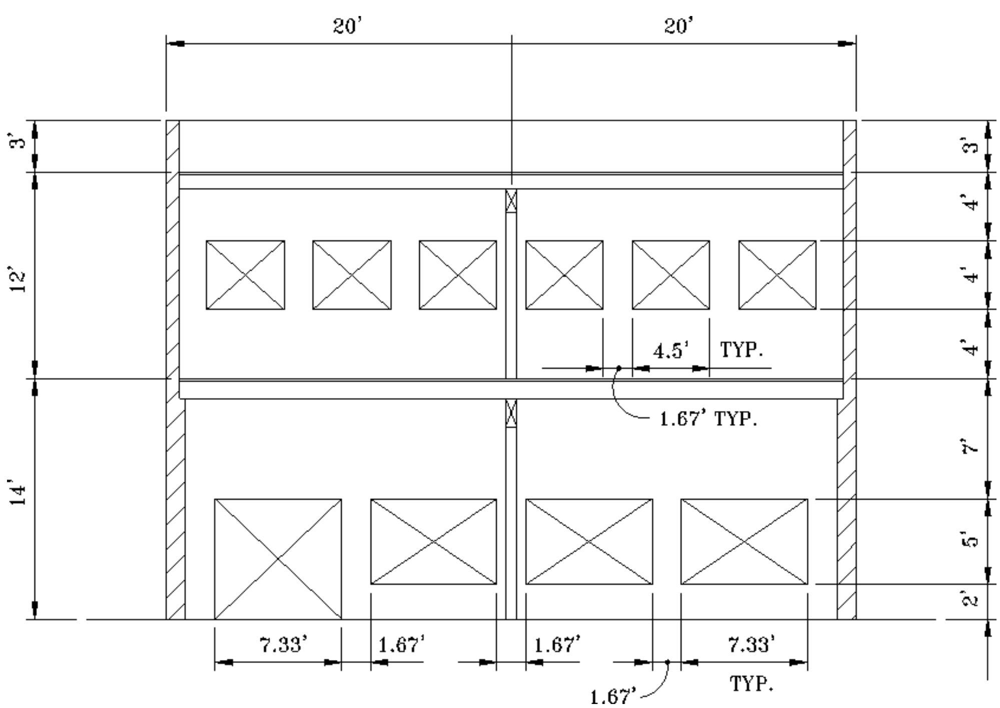


Figure 12-48. Example URM building, Interior elevation of west wall

P_{D2} = 2(5'+1.67')(7')(90psf)= 8,404 lbs

v_{a1} = 0.1(100)+(0.15)(5,254)/(1.67×12×9) =

= 10+2.2 = 14.4 psi

v_{a2} = 0.1(100)+(0.15)(8,404)/(1.67×12×9) =

=10+7.0 = 17.0 psi

The shear capacity V_a, and the rocking shear capacity V_r are calculated:

$V_{a1} = v_{a1} \times D_1 \times t$ = 14.4 psi × (1.67'×12×9) =

=2,597 lbs

$V_{a2} = v_{a2} \times D_2 \times t$ = 17.0 psi × (1.67'×12×9) =

=3,066 lbs

V_{r1}=0.5 $P_{D1} \times D_1/ H_1$=0.5×5,254lbs × (1.67'/4') =

= 1,097 lbs

V_{r2}=0.5$P_{D2} \times D_2/H_2$=0.5×8,404 lbs× (1.67/4') =

= 1,754 lbs

Therefore, rocking capacity controls for both piers and the total wall capacity =

2×V_{r1} + 5×V_{r2} = 2(1,097 lbs) + 5(1,754 lbs)

= 10,964 lbs > 7,400 lbs wall demand,

< 7,200 lbs wall demand,

∴ the wall is adequate

The first floor is checked in a similar manner and the west wall was found inadequate. The north, south and east walls at both stories were also evaluated and found to be adequate.

Add Braced Frame at West Wall

A braced steel frame is to be added behind the west wall to transfer the shear from the roof and second floor diaphragms to the foundation. A chevron braced frame and a concentric braced frame configuration were considered. The chevron braced frame required two very large 70 lbs/ft beams spanning 20 feet across the top of the chevrons (one at the roof and one at the second floor) to resist the vertical load component of the tension brace should the compression brace buckle. The concentric braced frame was therefore selected as the center post required a modest TS 5×5 center post (at 12 lbs per foot). Details of the braced frame are shown in Figure 12-49.

The dead weight of the end wall and tributary floor loads were checked and found sufficient to resist the overturning of the braced frame. Column elements were added at the south east wall and center pilaster and connected to the masonry to mobilized the dead load. The columns were designed continuous from the foundation through the second floor framing to below the roof. Steel angles were utilized as column elements at the center pilaster with the outstanding legs passing around the existing 8 by 20 in. beam to provide continuity of the column through the second floor.

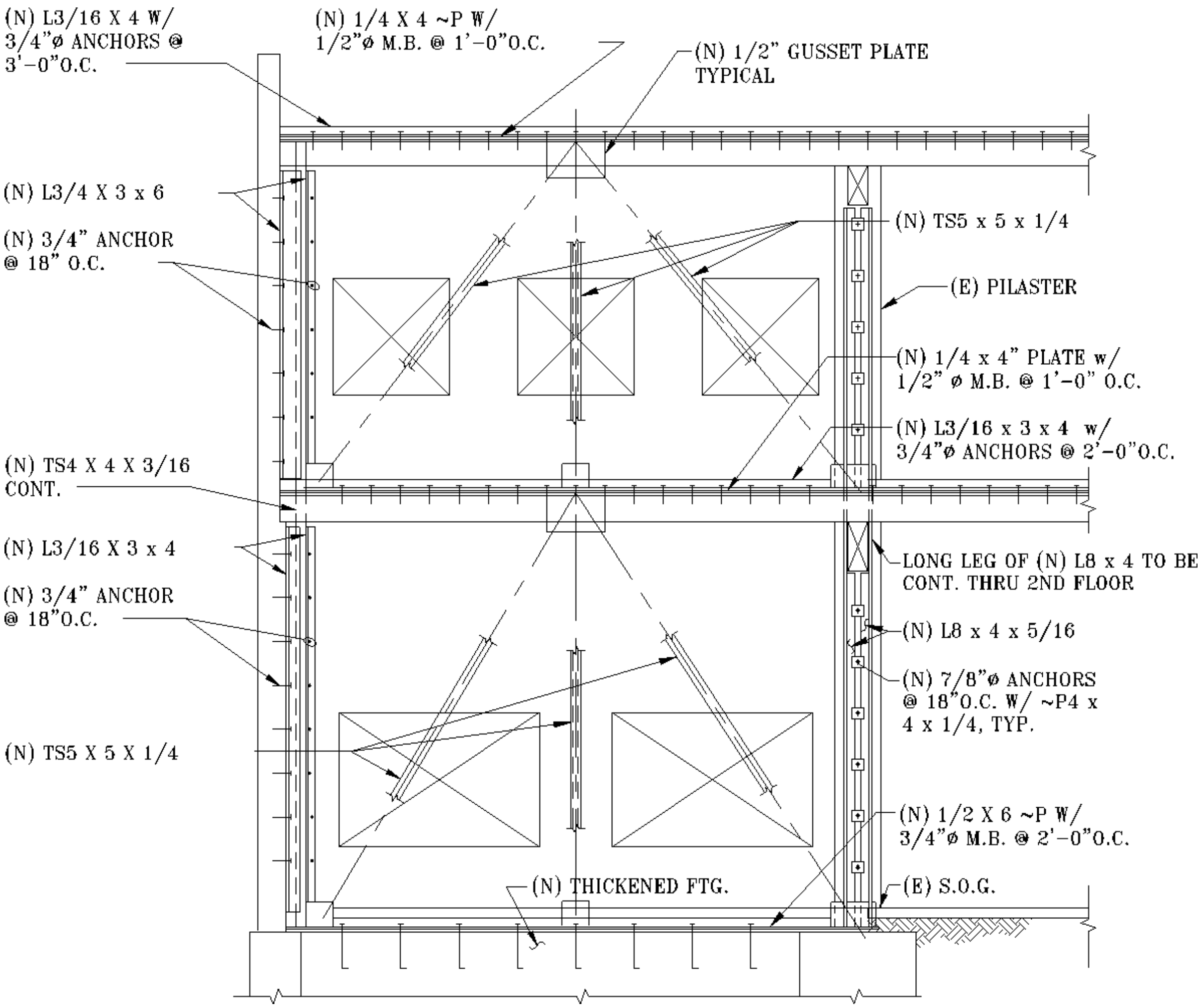


Figure 12-49. Example URM building, steel braced frame

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