COMMENTARY ON CHAPTER 2
EARTHQUAKE HAZARD REDUCTION IN EXISTING REINFORCED CONCRETE AND REINFORCED MASONRY WALL BUILDINGS WITH FLEXIBLE DIAPHRAGMS

Sec. C201 - Purpose. The provisions of this Chapter apply to buildings designed prior to adoption of *1997 Uniform Building Code (UBC)* or a code based on *1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (NEHRP)* (such as the *2003 International Building Code (IBC)*). They address deficiencies that have been found to be direct threats to life safety during an earthquake. These deficiencies first became apparent during the 1971 San Fernando, California earthquake when tilt-ups collapsed or partially collapsed. Further significant damage was observed in the 1987 Whittier Narrows, California, 1989 Loma Prieta, California and 1994 Northridge, California earthquakes. These deficiencies are as follows:

- A commonly used method of connecting wall panels to roofs and floors for out-of-plane loading was nailing the diaphragms to wood ledgers and bolting the wood ledgers to the walls. This method of connection allowed the wall panels to separate from the roof or floor by failure of the ledger in cross-grain bending, nail pullout from the ledger, or by pulling of the nails through the edge of the plywood.

- Wall anchor detailing has often included eccentric, flexible connections that are susceptible to damage. Poor installation including misalignment of anchors often created additional stresses.

- Distress occurred at the connection of the roof or floor girders to pilasters in the walls. Commonly, the anchor bolts connecting the girder to the top of the pilasters did not have adequate edge distance. Typically this resulted in spalling of the inside face of the concrete pilasters and loss of support for the girder. The containment ties required by the Building Code at the top of the pilasters did not prevent this kind of damage.

- The lack of continuity ties across the full depth of the diaphragm allowed cross-grain tension failures of the framing members at joints in the plywood sheathing. This occurred in the interior of the roof or floor diaphragms. Lack of tension ties at glulam hinge locations also led to loss of support for suspended girders.

Retrofitting of the above deficiencies was not required after the 1971 San Fernando earthquake. Voluntary upgrade provisions were developed in Los Angeles after the 1987 Whittier Narrows earthquake (Division 91, City of Los Angeles), and were then made mandatory in the City of Los Angeles after the 1994 Northridge earthquake. Where large earthquakes have previously occurred and portions of the existing wall anchorage system may have been damaged due to deficiencies in the original design, it may be appropriate to further investigate and prioritize these structures.

This Chapter adopts the concepts from the *1976 UBC* that required positive anchorage of all reinforced concrete and masonry walls to develop the out-of-plane wall loads into the floor and roof diaphragms and to prevent splitting of the ledger or tearing of the sheathing. (The concept of subdiaphragms was first introduced in the *1976 UBC*.) However, the 1994 Northridge earthquake revealed additional deficiencies in the *UBC* requirements for the wall anchorage system. This prompted changes in design forces and detailing requirements in the *UBC* for new buildings. In addition, changes to the design provisions for collector and collector connections were implemented.

Other possible deficiencies, such as inadequate diaphragm strength, inadequate diaphragm stiffness, or reinforcement at diaphragm openings, have not been reported as causing collapse, and were not addressed in this Chapter because the level of risk is generally lower. Although reinforced concrete and masonry wall buildings with metal deck diaphragms are within the scope of this Chapter, they are relatively rare in areas that have experiencedearthquakes, and consequently post-earthquakeobservationsand lessons learned are very limited.

Similarly, the analysis of the walls for either in-plane or out-of-plane forces is not part of this Chapter. Although walls are designed for smaller forces for out-of-plane bending, performance of walls in past earthquakes, provided that they have not been weakened by the introduction of large openings, has been good. There is a concern by some engineers that long walls consisting primarily of narrow piers without proper detailing may experience substantial damage in a major earthquake due to in-plane loading.

Sec. C202 - Scope.

The edition of this Chapter in the *1997 Uniform Code for Building Conservation (UCBC)* (the predecessor to the *Guidelines for the Seismic Retrofit of Existing Buildings*) applied to existing concrete tilt-ups buildings. The scope has been expanded to include all reinforced concrete and reinforced masonry buildings with flexible diaphragms. Concrete tilt-ups buildings are a subset of this type of structure.

The scope includes only buildings in higher seismic zones, and those designed under a building code in effect prior to the adoption of the seismic provisions of the *1997 UBC* or *1997 NEHRP*. (Changes based on observations in the 1994 Northridge earthquake were incorporated into both the *1997 UBC* and *1997 NEHRP*.) Note that the 1996 *Supplement to the UBC* included seismic design provisions for wall anchorage that are essentially identical to those of the *1997 UBC*, and some jurisdictions such as the City of Los Angeles adopted provisions that essentially meet the requirements of the *1997 UBC* prior to its adoption. Structures designed using such requirements may be considered exempted from the requirements of this Chapter. However, near-field effects were not considered in the interim provisions adopted by the City of Los Angeles prior to the adoption of the *1997 UBC*.

The scope of this Chapter includes only elements of the wall anchorage system and collectors and their connections. The wall anchorage system is defined in the *UBC* as including those elements within the diaphragm required to develop wall anchorage forces including: wall anchors, struts, subdiaphragms, cross ties and continuity ties. Past performance has indicated that partial and full collapse of these structures built with deficient seismic design provisions of the past is likely at ground accelerations approaching or exceeding 0.20g. (This observation is based on past earthquakes with short- to moderate- duration. Ground shaking of less severity but longer duration could also result in collapses.) For this reason, seismic zones 1 and 2A have not been included in these provisions.

Seismic design forces in building codes have changed over the years, especially design forces for out-of-plane anchorage of concrete or masonry walls. The following discussion provides a history of the pertinent changes made in the *UBC*. The design forces listed are for the highest seismic hazard zone considered in the *UBC*.

Prior to 1952, the base shear forces were used for a "part or portion of the building." The force for wall anchorage was 0.133W, with W equal to the weight of the wall tributary to the anchor. Local codes in California increased the wall anchorage force to 0.20W in 1953 for the design of anchors in concrete and masonry walls. This increase in wall design forces was adopted in the *1961 UBC*.

Changes to the provisions in the *1973 UBC* were motivated by observations made after the 1971 San Fernando earthquake. Changes included the increase the wall anchor design force, requiring continuous cross-ties across the depth of the diaphragm and prohibiting the use of wood ledgers in cross-grain bending and the use of toe nails or nails subject to withdrawal. Closely spaced ties were also required at the tops of pilasters to provide more resistance to anchor bolt shear failures. (These provisions for closely spaced ties proved to be inadequate in subsequent earthquakes.) The *1976 UBC* wall anchor forces varied between 0.20W and 0.30W due to the introduction of S (soil profile factor) in the design force equation. The *1979 UBC* modified the seismic loading on parts and portion of the building to be equal to the loading of $0.30W_p$ (regardless of soil profile), where W_p is the tributary weight of the wall component.

The format of the *UBC* changed substantially in 1988, but the design forces for wall anchorage did not. Provisions for designing structures with horizontal irregularities such as reentrant corners were introduced.

In the *1991 UBC*, a provision was introduced to increase the wall anchorage design forces for the middle half of the diaphragm by 50% to $0.45W_p$. This modification was intended to account for the amplified response in the middle portion of the diaphragms, as observed in the 1984 Morgan Hill, 1987 Whittier Narrows, and 1989 Loma Prieta earthquakes. Later nonlinear analyses of flexible roof diaphragms determined that limiting this amplification to the middle of the diaphragm was inadequate because the flexible diaphragm more closely resembles a shear yielding beam, not a flexural beam. Yielding of the diaphragm in strong ground motions tends to create a somewhat uniform pattern of maximum response along the length of the diaphragm, rather than a clear peak in the center of the span. Second and third-mode response of the diaphragm also increases the anchorage force adjacent to the supports. Therefore, the *1996 UBC Supplement* required that this amplification be applied to the full length of the diaphragm.

After the 1994 Northridge earthquake, wall anchor design forces were modified in the *1996 UBC Supplement*. In seismic zone 4, the allowable stress design forces were increased to $0.48W_p$ throughout the length of the diaphragm for buildings of standard importance, with additional material load increases required for steel (1.7) and concrete (1.7 instead of the 1.4 typically required). The intent of these different material load increases was to make the strength of the wall anchorage system sufficient to resist amplified shaking of about 1.5g at the roof diaphragm level. Using the material load increases, the capacities for all three materials (wood, concrete and steel) are more uniform. (Although the Building Code is silent regarding masonry, wall anchors in masonry can be designed for the same loads as wall anchors in concrete.)

Other detailing restrictions including minimum strut sizes (3x), concerns for symmetry in wall anchor design, and an increase in pilaster wall-anchorage design forces (due to the larger flexural stiffness of the pilaster with respect to the wall) were included in the *1996 Supplement to the UBC*. Many engineers believed that the poor detailing and construction installation contributed to the observed damage as much or more than the inadequately designed wall-anchorage system itself, and therefore, it is recommended that the engineer perform structural observations for wall anchor installation, and that testing of anchors be performed.

The requirements in the *1997 UBC* were intended to provide parity with the *1996 UBC* Supplement, but are expressed in the context of strength design. Elements of the wall anchorage system are required to be designed with $a_p = 1.5$ and $R_p = 3.0$. For elements at the roof level of a structure, this corresponds to $F_p = 2.0C_aI_pW_p$. Although the wall anchor design forces appear larger than those in the *1996 UBC Supplement* (approximately $0.8W_p$ in zone 4), it can be demonstrated that they are essentially the same by:

- Reducing the *1997 UBC* forces by a factor of 1.4 to convert to allowable stress design,
- Reducing the *1997 UBC* forces by a factor of 0.85 to account for the fact that the *1996 UBC Supplement* forces have a material factor of 1.0 for wood, whereas the *1997 UBC* forces have a material factor of 0.85 for wood (and similar changes for the other construction materials).

As mentioned above, the possible materials that comprise the wall anchorage system are concrete, masonry, wood and steel. The ratio of expected strength to design strength (i.e., overstrength) is not the same for each of these materials. The *1997 UBC* applies different material factors to the different materials in an attempt to provide ratios of expected strength versus design strength (i.e., overstrength) for each material that are approximately equal. These material factors should be used for allowable stress and strength design. The use of "material factor" may create some confusion in that it does not have a clear meaning for allowable stress design in *UBC* jurisdictions. "Material load increase" is was used in describing the factors used in the *1996 UBC Supplement* in this Chapter, but as some of the material factors in the *1997 UBC* are less than one (i.e., the factor for wood is 0.85), the use of "increase" can also be confusing.

The *1997 UBC* design forces include near fault effects. Near fault effects may increase wall anchor forces by as much as 50% for structures located within 2 km of a major fault. Modifications due to changes in soil type classification may increase or decrease wall anchor forces a relatively small amount. The wall anchorage forces in the *1997 UBC* are derived using a formula that is dependent on the height of the diaphragm in the structure. Thus in 2-story structures, wall anchorage forces at the second floor are lower than corresponding forces in the *1996 UBC Supplement*.

After the Northridge earthquake this Chapter was first included in the *1996 UCBC Supplement* as Appendix Chapter 5. The source of Appendix Chapter 5 was Division 91 of the Los Angeles Building Code, which was written prior to the Northridge earthquake, and consequently used smaller design forces for wall anchorage. In order to increase the forces to an acceptable level, materials factors larger than those required for new buildings were required. This created some confusion that has been eliminated in this edition of the Chapter by adopting the *1997 UBC* material factors for the different materials.

The authors of the *1997 NEHRP*, and later the authors of the *2000 IBC* took a different approach than the *1997 UBC* with respect to material factors and the dependence of the wall anchor forces on the relative height of the diaphragm in the structures. Rather than provide different material factors for the different structural materials, a strength design force of $1.2W_p$ was used for geographic areas equivalent to *UBC* seismic zone 4 for all of the materials. This force is slightly larger than the design force for steel in the *1997 UBC*. This change was made partly due to the fact that the use of different material factors can become cumbersome, and because it was believed more appropriate to deal with inconsistencies in different material safety factors in the different material chapters. However, as a result, a significant inconsistency in design forces between the *1997 UBC* and in the *2000 IBC* exist. Trial designs in high seismic zones using the *IBC* provisions have resulted in large subdiaphragm shears and the need for substantially deeper subdiaphragms.

The *2003 IBC* reduced the design force in high seismic zones to $0.8W_p$, but employs a 1.4 material factor for steel. Whereas the *1997 UBC* wall anchor forces vary linearly with the height within the structure due to the h_x/h_r term in Formula 32-2 for wall anchor forces, the *2000 IBC* and *2003 IBC* has one equation for wall anchor forces, regardless of the height. Thus the *2003 IBC* and the *1997 UBC* are reasonably consistent for wall anchorage design at the roof with the *IBC* being slightly more conservative for wood components. At floor levels the *IBC* is significantly more conservative.

Prior to the 1994 Northridge earthquake the *UBC* required that buildings be required to resist out-of-plane loading of 200 pounds/foot regardless of the seismic zone, regardless of seismic zone. (This requirement was introduced in the *1958 UBC*, apparently for wind loading.) This load was increased to 300 pounds/foot (allowable stress loads) in the *1997 UBC* for seismic zones 3 and 4. Using the *1997 UBC*, this minimum load is checked against loading that is directly proportional to seismic zone coefficients. In the *2003 IBC*, in Seismic Design Category B (SDC-B) the wall anchor force for buildings of standard importance is larger of 10% of the tributary wall weight, or 40% of the tributary wall weight times S_{DS}, the spectral response at short periods. For higher performance categories (SDC-C and above), the 40% value is increased 80% per the discussion above.

In summary, prior to the *1973 UBC*, wall anchorage design was minimal (and cross-grain tension and bending were not prohibited). The *1976 UBC* included *many* of the requirements currently used for wall anchorage design today, but at much lower design force levels. The design forces increased by 50% in the *1979 UBC*, and an additional increase of 50% was applied for the wall anchors in the middle of the diaphragm in the *1991 UBC*. Design forces were increased again in 1994, and detailing restrictions were introduced. Therefore structures constructed prior to adoption of the *1973 UBC* represent the highest risk, with significant improvements occurring in 1976, 1979, 1991 and finally in 1996 (1997). In regions that previously adopted the *UBC* but now adopt the *IBC*, the *2000 IBC* represented a significant increase in design loads for wood and concrete components and a slight increase in the design loads for steel components of the wall anchor system. The significant increases in design loads for concrete and wood were eliminated in the *2003 IBC*. It may be appropriate for risk reduction programs to take the design codes used above into account when identifying or prioritizing when and which buildings should be retrofitted. Nevertheless, only those buildings designed using building codes based on the *1997 UBC* or *1997 NEHRP* provisions are exempt from having to comply with the provisions of this Chapter.

Sec. C203 - Definitions. No commentary.

Sec. C204 - Symbols and Notations. No commentary.

Sec. C205 - General Requirements. The provisions of this Chapter are intended to address deficiencies that represent direct threats to life safety. The primary concern is providing an adequate wall anchorage system. Lack of adequate collectors or collector connections at reentrant corners or internal walls can also be a significant deficiency that can lead to localized collapse. If the engineer finds that a complete load path is absent (e.g., some shear walls have no blocking for force transfer from the roof to the shear walls), the requirements of the Building Code for a complete load path (with reduced force levels) should be applied.

Sec. C206.1 - Analysis and Design. Concrete Walls and Masonry. Currently it is a standard practice to evaluate existing buildings to a reduced force level of 75% of the design force for new buildings. That is, buildings with more than 75% of the capacity required by the Building Code need not be retrofitted. Some documents recommend retrofit to this same force level, while others like FEMA 356 recommend retrofitting to 100% of current Building Code forces. Given the poor performance of wall anchorage systems for buildings with flexible diaphragms in past earthquakes, and the relatively small cost associated with upgrading wall anchorage to higher force levels once some retrofit is required, it is recommended that where retrofits are performed, that the engineer consider providing capacity in excess of 75% required (i.e., use the design forces in the Building Code).

This Chapter requires that wall anchorage systems in existing buildings be evaluated using a force level of 75% of the design force required for new buildings. The wall anchorage system is defined in the *1997 UBC* as including the elements within the diaphragm required to develop wall anchorage forces including: wall anchors, struts, subdiaphragms, cross ties and continuity ties.

Sec. C206.2 - Special Requirements for Wall Anchors and Continuity Ties. The wall anchorage forces are dependent on the span and stiffness of the diaphragm and the weight, thickness, height and flexibility of the walls. The probable dynamic forces will exceed the design force. Recent studies of strong motion records in buildings with flexible diaphragms show amplification at the roof diaphragm level of two to four times the ground acceleration. (The higher amplifications typically occur at lower ground accelerations, where lower damping and no inelastic behavior occurs.) Such amplifications of peak ground acceleration are expected for structures that typically have diaphragm periods corresponding with the peak of the response spectrum. Yielding of the anchor system contributes negligible energy absorption. Therefore the capacity of the wall anchorage system should be at the expected level of earthquake loading caused by the dynamic response of the diaphragm and the walls, i.e. three times the effective ground acceleration (1.2g the roof diaphragm level for 0.4g ground acceleration before the near source effect and soil condition are considered).

As discussed previously, the *1997 NEHRP* and the *2000 IBC* do not use load factors for different materials. In the *2003 IBC* and *ASCE 7-02* a material factor of 1.4 is used for steel. Although it is not recommended that wall anchors yield when subjected to major earthquakes (yielding would result in large deformations that could cause damage to diaphragm nailing at the ledgers) it is still recommended that the designer provide wall anchors that will yield rather than fail in a brittle manner at reduced sections (e.g. twisted straps).

Structural steel, which has a known ductile behavior and a small coefficient of variation in material strength has the lowest ratio of yield stress to allowable stress. The material factor of 1.4 given in the *1997 UBC* for strength design adjusts the strength of the steel to meet the expected anchorage loading and thus maintains consistency in the capacity of all of the materials used in the wall anchorage system. Typically only the anchor hardware is designed using this load factor. The embedment of the anchor bolt is evaluated using the concrete material factor. If the 1.33 increase for allowable stress design is used the resulting steel member size is approximately 25% smaller than required by strength design. In order to provide an allowable stress design that is consistent with the strength design, no 1.33 increase should be used.

The *1997 UBC* permits use of a 0.85 material factor for wood including nailing of the subdiaphragm and attachment of the wall anchors to the wood member. This same factor may be applied to allowable stress design even though it is not explicitly stated in the Chapter. It was believed necessary to state clearly that the 1.4 factor for steel applied to working stress, as it would be unconservative to ignore it. This is not the case with the 0.85 factor for wood.

It is also important to consider the stiffness of the steel and other materials in the anchorage. For example, the wall anchors should be substantially stiffer than the ledger nails in order to prevent them from resisting substantial load and failing in cross-grain bending. Several mechanisms for ensuring stiff wall anchors were considered in developing the provisions in the *UBC*, and such considerations were a motivation for increasing the minimum out-of-plane forces from 200 to 300 pounds per foot (allowable stress design). Clearly stiff wall anchors and minimal bolt slip (e.g., no oversized holes) are goals in a good wall anchorage system.

Existing ledger bolts embedded in the concrete or masonry wall may be used as a part of the wall anchorage system if their tension values can be established by testing or through analysis. The size and spacing of these anchor bolts must be shown on the approved plans and verified by site investigation. A strength design analysis must show that these embedded anchor bolts are capable of resisting the factored tension load in combination with the factored combination of earthquake- and gravity-induced shear loads in both the horizontal and vertical directions.

Expansion anchors are only allowed if they are installed with special inspection and if they meet the appropriate acceptance criteria (e.g., AC 193 "Acceptance Criteria for Mechanical Expansion Anchors in Concrete Elements").

Wall anchorages that rely on the tear-out strength (nails pulling through the edge) of the plywood for attachment to a steel ledger are not considered as in compliance with the provisions of this Chapter. This is because such connectors at the edge of the plywood sheets would be loaded with a combination of shear and tension (in-plane shear and tear-out). This means that the local demand can be much higher than the original design capacity. In addition, tear-out values of connectors adjacent to the edge of the plywood sheet are not specified in the Building Code.

Welding steel decking to a steel ledger angle is not considered positive anchorage, as required by this Chapter, when flutes are parallel to the wall. The reasons are the same as for plywood attachment, and also because the flutes have substantial flexibility. Testing and analysis to show that the decking with flutes perpendicular to the wall could provide the positive anchorage without significant displacement normal to the wall surface, but the welds are also loaded in shear due to orthogonal loading of the diaphragm. The local demand on the welds can be much higher than the original design capacity.

Sec. C206.3 - Development of Anchor Loads into the Diaphragm. This section presents the requirements of Chapter 16 of the Building Code for development of wall anchorage loading. Continuous ties or struts between diaphragm chords are required to distribute wall anchorage forces into the body of the diaphragm. Subdiaphragms created with subdiaphragm chords at their boundaries may be used to transmit the anchorage forces to the main cross ties.

The span-depth ratio of subdiaphragmsshall not exceed 2.5. This allowable span-depth ratio is substantially less than the 4.0 value permitted in previous editions prior to the 1997 Building Code. This is an indirect attempt to limit subdiaphragm shear. (Some engineers argue that some relaxation of this requirement is appropriate for evaluating existing buildings as design forces have been reduced by 25%, and this requirement is related to the strength of the subdiaphragm. They argue that retrofitting a building that meets all of the other requirements of the Chapter simply because the subdiaphragm ratio is 3 instead of 2.5 may not be necessary. However, this is a minority opinion and the requirement a retrofitting subdiaphragms with a length-to-depthratio of more than 4.0 is a requirement of this Chapter. The engineer should keep in mind that the limit on the subdiaphragm ratio of 2.5 originated because of the belief of some engineers that subdiaphragm shear demand should be limited to 300 pounds per foot. If higher subdiaphragm shears are calculated it is recommended that the engineer increase the depth of the subdiaphragm.) Girders, trusses and beams used as cross ties shall have a positive connection to the roof sheathing. This configuration of continuous orthogonal ties confines sections of the diaphragm. This confinement is needed to resist tensile forces caused by shearing stresses in the diaphragm.

The use of toe nails and nails subject to withdrawal as any part of wall anchorage system is prohibited. Ledgers or plates on the top of walls are not allowed as a part of the wall anchorage system when the loading will result in cross-grain bending or cross-grain tension.

The engineer should consider the plywood layout when selecting locations for wall anchors. Only members with edge nailing should be used as part of the wall anchorage system unless the reduced capacity for field nailing is considered. (This includes locations where the plywood is staggered and the subpurlin in the first bay has edge nailing, but the subpurlin in the second bay does not.) If the diaphragm nailing is unknown (no design drawings exist), the roofing should be removed locally in a number of locations to determine edge nailing. As nailing will probably vary over the length of the diaphragm, the nailing should be verified in a number of regions. If there are indications of poor construction quality, the engineer should consider removal of all roofing to verify nailing at all wall anchor locations.

At re-entrant corners, the concrete or masonry return walls may have been ignored in the diaphragm analysis performed in the original design, but due to their in-plane stiffness, they function as a diaphragm support. (The same is true for short interior walls or fin walls.) Deflection of the diaphragm spanning between end walls of the building may not be compatible with the deformation of the top of the short interior return walls. This deformation incompatibility results in diaphragm damage and separation of the diaphragm from the return wall. Rather than attempting to calculate a deflection compatibility approach, a simple comparison of strengths or capacities is recommended. The objective of the comparison is to find the minimum load needed to adequately tie the return wall to the diaphragm. The capacities of the diaphragm system or the return wall could be estimated as follows for a single-story structure:

- Rocking capacity of a wall:The actual rocking capacity V_R of the wall shall be calculated from an equilibrium equation, using all the dead loads on the walls, including the weight of the wall, taken about an extreme edge of the footing. Other factors such as soil weight and contributions from continuous footings should be included, as underestimating the overturning resistance in this context is unconservative. Load factors specified in the Building Code that provide the maximum resistance shall be applied.

- Shear capacity of the wall: $V_n$shall be the nominal shear strengthof the wall provided by the concrete and shear reinforcement without any strength reduction factor.
- Maximum force that can be delivered by the diaphragm:

$$V_d = (v)(L_1 + L_2)$$

Where:

L_1, L_2 = Depth of the diaphragm measured on each side of the return wall.

v = Allowable diaphragm shear value from the Wood Chapter of the Building Code times a factor of 3. The *1997 NEHRP* may be used for strength design values for diagonal sheathing.

No drag strut from the return wall into the diaphragm need be installed if the minimum lateral load that must be developed in the return wall does not exceed the diaphragm capacity V_d along the length of the return wall. It is assumed in such cases that the shear will transfer through the diaphragm even if a non-uniform shear throughout the depth of diaphragm is required to do so.

A redundant vertical support is required beneath major framing members, such as girders or trusses supported by ends of return walls, if these return walls are limited in capacity (when either $V>V_n$ or $V>V_R$) and $V > V_d$, where V is the demand on the wall (strength design). The failure mode could be shear in the reinforced concrete or reinforced masonry wall, or a dynamic response that causes rocking in the return wall. A wall that fails in in-plane shear may lose its capacity to support vertical loads. A wall that rocks on its foundation may cause damage to the bearing surface providing support for the member framing onto the return wall.

No redundant support is required for smaller members such as purlins or subpurlins. It is likely that loss of support for such minor roof-framing members will not result in significant collapse or threat to life safety.

Sec.C206.4 - Anchorage at pilasters. It has been past practice to anchor walls to the floor and the roof diaphragms for tributary wall loading, assuming the wall panel spans between adjacent floors or between the floor and roof. However, when pilasters are present in the walls, the stiffening effect of the pilasters on the deformed shape of the wall-pilaster system must be taken into account. Since the relative stiffness of the walls and pilasters may vary, a rational method is required to calculate the wall panel load carried by the pilaster. The reaction of a pilaster at the diaphragm level, resulting from the analysis of the wall-pilaster system, is the load applied directly to the girder through its existing anchorage to the top of the pilaster.

Two concerns must be addressed: (1) the existing connection to the girder must have adequate strength for the pilaster reaction (2) the shear strength of the existing anchors, with consideration of the edge distance of the embedded bolts in the top of the pilaster, must be adequate. If the shear strength of the existing bolts is not adequate due to their edge distance, additional exterior confinement at the top of the pilaster must be provided, or the existing anchors can be bypassed with new, stiff wall anchors designed to take the entire load. The stiffness of such a bypass system must be checked to verify that it has significantly less deformation under loading than the existing system. If not, the existing system should be disconnected (e.g., remove bolts from girder).

A single-story wall panel with pilasters at each end is supported at four edges by the pilasters, the roof diaphragm and the slab-on-grade. Typically, the support at the slab-on-grade should be considered as a support without moment restraint. (The panel could also be modeled with lateral restraint at both the slab-on-grade and the footing in cases where the elevations of the two are substantially different.) The support condition of the panel at the pilaster can be a continuous span, having a moment restraint, or be a simple support. The moment restraint depends on whether continuity of the panel through the pilaster is shown on the drawings or whether an open joint exists at one side of the pilaster. The support at the roof level is typically considered as a support without moment restraint. Any textbook or standard for the design of slabs supported on four edges may be used to determine reactions at each edge of the panel. These references consider the aspect ratio of the panel and the rotational restraint at each panel edge. Analyses can be complicated by the presence of openings and parapets. In such cases, approximate, conservative,simplifying assumptions should be used. A logical approach regarding tributary area for the design of pilaster wall anchorage is included in the commentary of the 1999 Bluebook (Figure C108.1).

Sec. C206.5 - Symmetry. Symmetry of wall anchorage connectors that are placed on the vertical sides (i.e., would generate bending about minor axis) of the framing members is required unless the eccentricity is accounted for in the design. In the past, engineers often used allowable values in hardware manufacturer's catalogs without consideration of member capacity, and specifically the stresses caused by eccentric connections. The values in the hardware catalog typically were developed by testing the hardware on a steel jig, and thus did not consider the stresses in the wood members. The flexural stresses about the minor axis caused by unsymmetrical connectors must be combined with stresses caused by gravity loading. Eccentricity between the line of loading on the connector and the centerline of the resisting member must be resisted by supports normal to the line of loading or by a resisting couple. The reactions at these supports must be determined by calculations or testing. The engineer should be aware of the fact that eccentricity can arise either because the anchor design includes eccentricity, or because of bolt or strap misalignment (i.e., poor installation).

Sec. C206.6 - Minimum Member Size. The minimum member size of three inches nominal is only required for a new member added to the existing construction or used as replacement of an existing member. All members are to be designed for combined earthquake and gravity loading. An existing 2-inch nominal member may be supplemented with another 2-inch nominal member if the additional member is internailed to the existing subpurlin or joist for load sharing.

Sec. C206.7 - Combination of Anchor Types. The sharing of the load on a single framing member between anchors of different types requires an analysis of their relative stiffness. If no data exists for calculation of relative stiffness of existing anchors, load sharing between different types of anchors is not permitted.

Sec. C206.8 - Miscellaneous. Existing buildings commonly have mezzanines that are used for office or light storage. Mezzanines may be dependent on the wall and pilasters, for lateral support. The seismic load of mezzanines is additive to the seismic load of the walls. This combined loading should be used for calculation of wall anchorage forces at the roof diaphragm level above and below (if applicable) the mezzanine. An alternative would be to provide a lateral-load resisting system for the mezzanine and use the mezzanine for support of the adjacent walls. In either of these cases, the mezzanine must be anchored to the wall. If the mezzanine is isolated from the wall in accordance with the building separation requirement of the Building Code and has independent lateral and vertical support, no consideration of the anchorage of the mezzanine is required.

This Chapter does not require that the flexural strength of the walls and/or pilasters be analyzed for lateral loads from the mezzanine. However, it is recommended that consideration be given to the existing strength of these elements before making a decision about the alternatives of providing lateral bracing below the existing mezzanine, or using the wall to transmit the seismic loading of the mezzanine to the roof and slab-on-grade levels.

This section requires that each wall that extends to the diaphragm level is provided lateral support by a wall anchorage system; and if the wall is not isolated in the plane of the wall, it must be analyzed as required for a return wall in Section A206.3.

Sec. C207 - Materials of Construction. No commentary.