

Anchorage of Concrete Masonry Elements

Introduction

The damage that occurred to out-of-plane connections between panelized plywood roofs and perimeter concrete or masonry walls during the 1971 San Fernando and 1994 Northridge earthquakes demonstrated that earlier codes may not have provided buildings with sufficient protection against out-of-plane forces during strong ground shaking. The most significant failures occurred in large box-type structures with high walls and flexible diaphragms. Recordings from instrumented buildings showed that in such buildings, the acceleration at the center of the diaphragm is often over three times the acceleration at the ground level. This led to changes in the code requirements for out-of-plane anchorage of concrete and masonry walls, particularly when they are supported by flexible diaphragms.

This edition of masonry chronicles will discuss the procedures for designing to provide anchorage for concrete masonry walls subjected to out-of-plane earthquake loads. Examples that highlight some specific code requirements will also be presented.

Anchorage Forces on Masonry Walls

As shown in Figure 1, masonry walls that are subjected to lateral loads from earthquake and wind demands typically span between floors and/or roof levels. This means that adequate reactions, or anchorages, must be developed at floor and roof supports to resist the tendency of the wall to pull away and collapse during earthquakes and high winds.

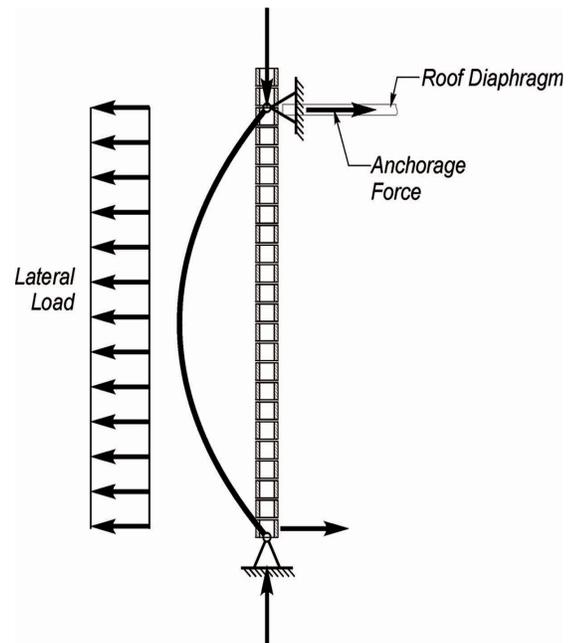


Figure 1 Development of Anchorage Forces Due to Lateral Load

In the past, out-of-plane failure of wall to roof connections was one of the more common types of failure of that occurring in masonry buildings subjected to severe ground shaking.

Code requirements for wall anchorage design have improved significantly in recent years, and most well designed concrete masonry buildings have performed excellently during recent earthquakes. The earlier failures however, highlight the importance of correctly designed anchorages and the fact that engineers must pay special attention to this part of the design process to ensure satisfactory performance.

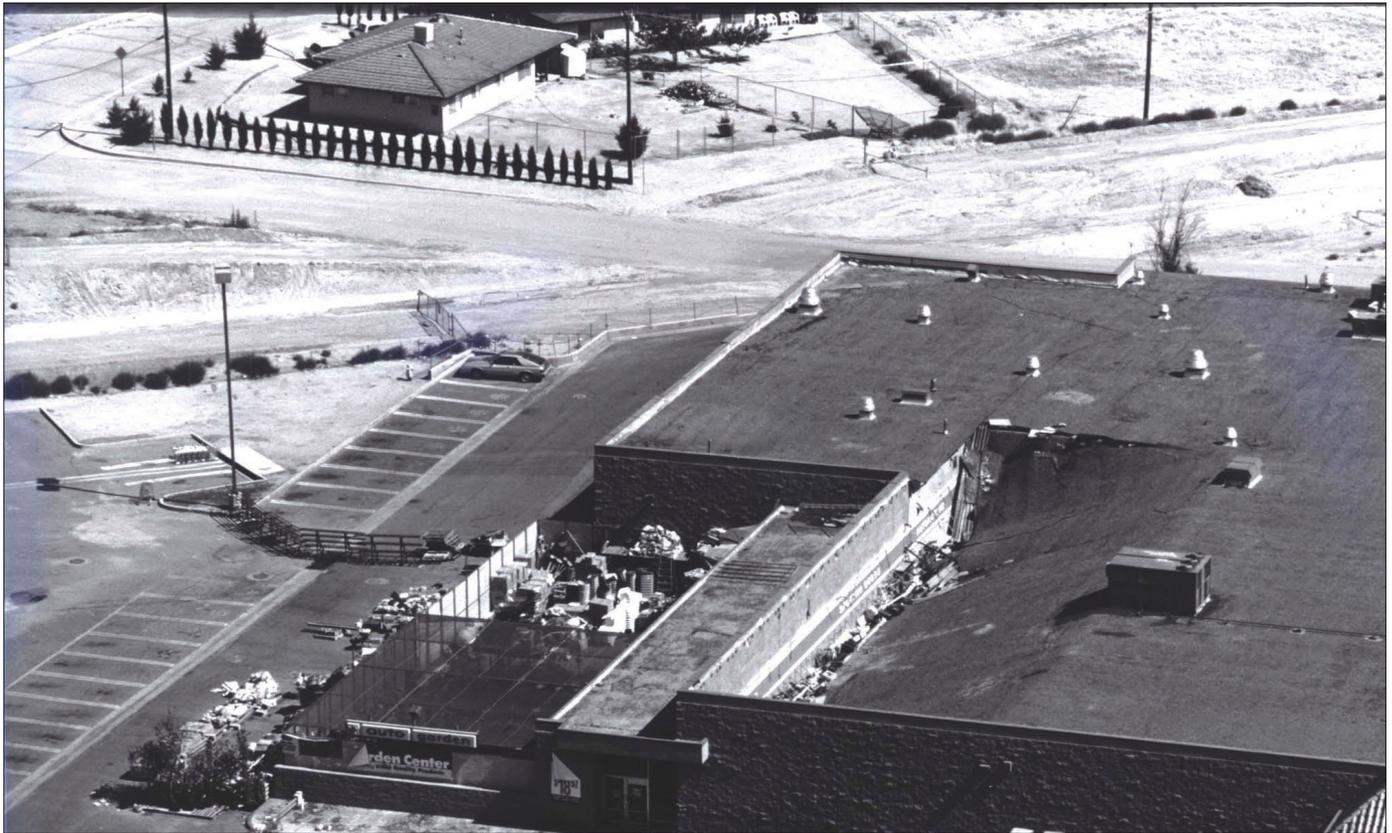


Figure 2 Failure of Roof Anchorage Due to Inadequate Anchorage and Collector Design (Courtesy Gregg Brandow, Brandow and Johnston)

When designing out-of-plane anchorages for concrete masonry walls, loads are determined by considering the walls as elements of the structure – as stipulated in Section 1632 and 1622 of the 1997 Uniform Building Code (UBC)[1] and 2003 International Building Code (IBC) [2], respectively. This is because walls perpendicular to the direction of loading do not participate as part of the lateral load resisting system. Instead, the walls respond by transferring the inertial forces generated by their self weight to the diaphragms, which in turn transfer loads to the lateral load resisting elements. Figure 3 shows a plot of accelerations recorded at various elevations in buildings during recent California Earthquakes [3]. The figure shows that a trapezoidal distribution of floor acceleration within a building can be assumed, with a roof acceleration of about two and a half times the acceleration at the ground surface.

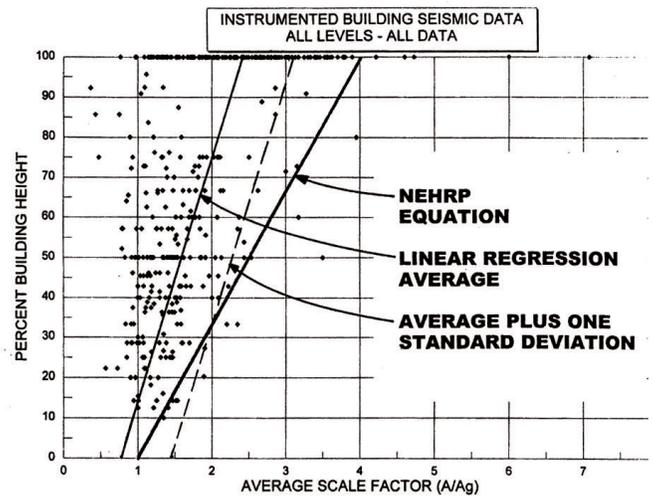


Figure 3 In-structure Accelerations Recorded Over Building Height [3]

Since earthquake forces are directly proportional to earthquake-induced accelerations, the 1997 UBC provides the following upper bound equation for calculating the out-of-plane anchorage forces:

$$F_p = \frac{a_p C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_r} \right) W_p \quad (1)$$

$$; 0.7 C_a I_p W_p \leq F_p \leq 4 C_a I_p W_p$$

where

- a_p = In-structure Component Amplification Factor, which is equal to 1.0 (1.5 for flexible diaphragms).
- C_a = Seismic Coefficient for ground motion.
- I_p = Importance Factor.
- R_p = Component Response Modification Factor, which is equal to 3.0 (1.5 for shallow anchors with a with an embedment length-to-diameter ratio of less than 8).
- h_x = height of point of wall attachment with respect to the base.
- h_r = average roof height of structure with respect to the base.
- W_p = Weight of wall tributary to diaphragm providing lateral support.

Figure 4 illustrates how the anchorage forces are calculated in a multi-story building taking into account the wall weight tributary to each floor.

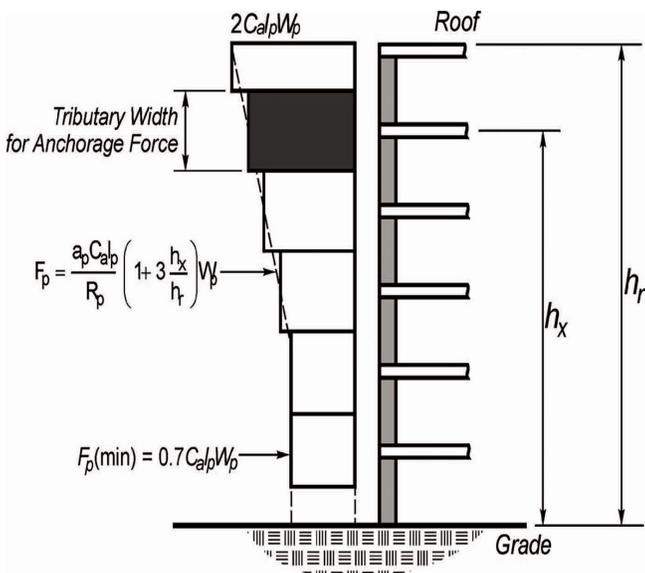


Figure 4 Calculation of Anchorage Forces in a Multi-story Building

The Component Amplification Factor, a_p accounts for the dynamic amplification of an element relative to the building's fundamental period of vibration. In flexible diaphragms, a_p is increased by 50% to 1.5 because of the amplification of the building acceleration by the diaphragm, as shown in Figure 5. Earlier codes required that the flexible diaphragm effect be included only at the diaphragm mid-span. However, recent research indicates that since a flexible diaphragm is a shear yielding beam, and because of secondary mode effects, the amplification should be applied uniformly across the length of the diaphragm.

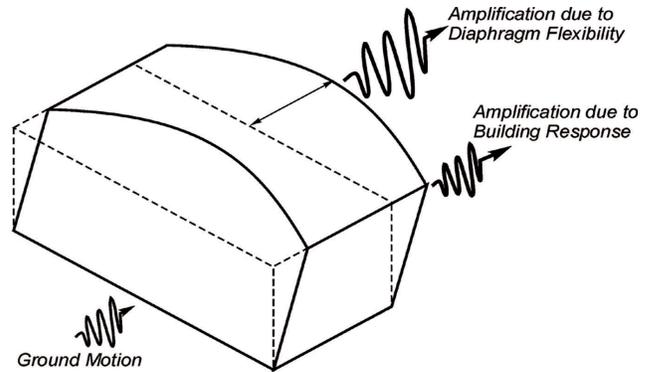


Figure 5 Amplification of Anchorage Forces Due to Flexible Diaphragm Response

The 2003 IBC provides a similar procedure for calculating anchorage forces that are required to support masonry walls. The following equation calculates the seismic design force F_p on the wall, which is to be supported by the diaphragm:

$$F_p = \frac{0.4 a_p S_{DS} W_p}{R_p / I_p} \left(1 + 2 \frac{z}{h} \right) \quad (2)$$

where

- S_{DS} = short period 5% damped spectral response acceleration at the building location. The value $0.4 S_{DS}$ represents the effective ground acceleration at the site.
- a_p = amplification factor that represents the dynamic amplification of the wall relative to the fundamental period of the structure. For most masonry walls, $a_p = 1.0$, except for parapets and unbraced walls for which $a_p = 2.5$. For fasteners used to design the anchorage system $a_p = 1.25$.
- I_p = importance factor that varies from 1.0 to 1.5.
- W_p = wall weight tributary to the diaphragm.

R_p = response modification factor that represents the wall overstrength and ductility or energy absorbing capability. For reinforced masonry walls, $R_p = 2.5$, while for unreinforced masonry walls, $R_p = 1.5$.

z = height of point of wall attachment with respect to the base.

h = average roof height of structure with respect to the base.

The seismic force need not exceed

$$F_{p,max} = 1.65S_{DS}I_pW_P$$

and should not be less than

$$F_{p,min} = 0.3S_{DS}I_pW_P$$

Design of Out-of-Plane Anchorage of Concrete Masonry Walls

After the anchorage forces are determined as described in the previous section, the elements of the anchorage system must be designed to resist the calculated demands. Some specific requirements stipulated by the 1997 UBC include the following:

- In Seismic Zone 4, elements must be designed to resist an anchorage force of at least 420 pounds per lineal foot.
- The strength design forces for steel elements of the wall anchorage system shall be increased by 1.4 times.
- The strength design forces for wood elements of the anchorage system shall be multiplied by 0.85.

A primary aspect of the design process is the design of anchor bolts to resist the out-of-plane anchorage forces. When anchor bolts embedded in concrete masonry are loaded in tension, the capacity is determined by the following equations when using strength design:

$$B_{tn} = \phi A_p \sqrt{f'_m} \quad (3)$$

$$B_{tn} = \phi 0.4 A_b f_y \quad (4)$$

where

f'_m = Masonry compressive strength.

f_y = Tensile yield stress of reinforcement.

ϕ = Strength reduction factor = 0.8.

A_b = Cross-sectional area of anchor bolt.

A'_p = Area of tension pullout cone of embedded anchor projected onto the masonry surface. $A_p = \pi l_b$ where l_b is the anchor bolt embedment. When the projected areas of adjacent bolts overlap, the value of A_p for each bolt shall be reduced by half the overlapping area. The edge distance l_{be} should be used in place of l_b if the edge distance is less than the embedment.

Equation (3) represents the allowable capacity related to concrete masonry pullout, while Equation (4) is the allowable load associated with masonry failure of the bolt in tension. The smaller of the values obtained from the two equations should be used. For allowable stress design, the following equations should be used for masonry pullout and anchor bolt yield, respectively:

$$B_t = 1.33(0.5)A'_p \sqrt{f'_m} \quad (5)$$

$$B_t = 0.2A_b f_y \quad (6)$$

Ties and Subdiaphragms

When designing buildings to resist earthquakes, it is important that all parts of the structure are tied together with struts, collectors or ties that are capable of transmitting the earthquake-induced forces to the primary lateral-load resisting system. Bearing in mind that this is particularly important when flexible diaphragms provide out-of-plane support for relatively heavy concrete or masonry walls, section 1633.2.9 of the 1997 UBC states that:

Diaphragms supporting concrete or masonry walls shall have continuous ties or struts between diaphragm chords to distribute the anchorage forces specified in Section 633.2.8.

One way of interpreting this requirement is this would mean that each joist in the north-south direction in Figure 6 would have to transfer the load from one end of the building to the other. Each joist-to-girder connection would have to be capable of transmitting the anchorage force, and require splice with the capacity of resisting the horizontal loads. Such a design would be quite expensive

and is clearly not the most effective method of providing continuity. Fortunately, the UBC also states that:

Added chords of subdiaphragms may be used to form subdiaphragms to transmit the anchorage forces to the main continuous cross-ties. The maximum length to width ratio of the wood structural subdiaphragm shall be 2 1/2 :1

This concept of subdiaphragms is extremely useful in providing continuous ties, as illustrated in Figure 7. While each subdiaphragm chord must be designed to resist axial chord forces, the number of splice connections required to transmit the anchorage load is reduced significantly.

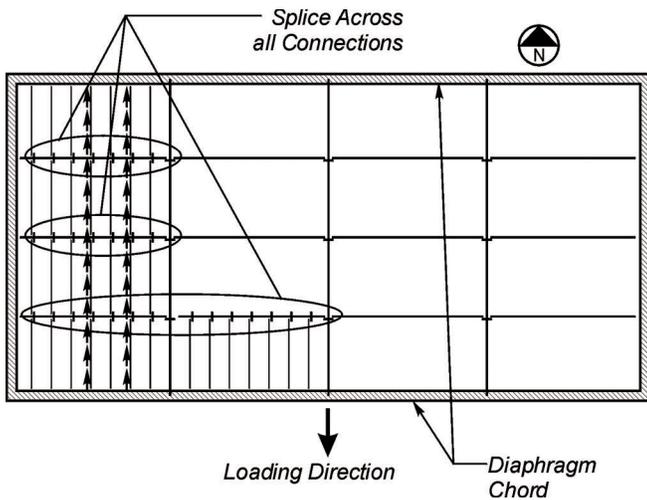


Figure 6 Continuous Ties Without the Use of Subdiaphragms

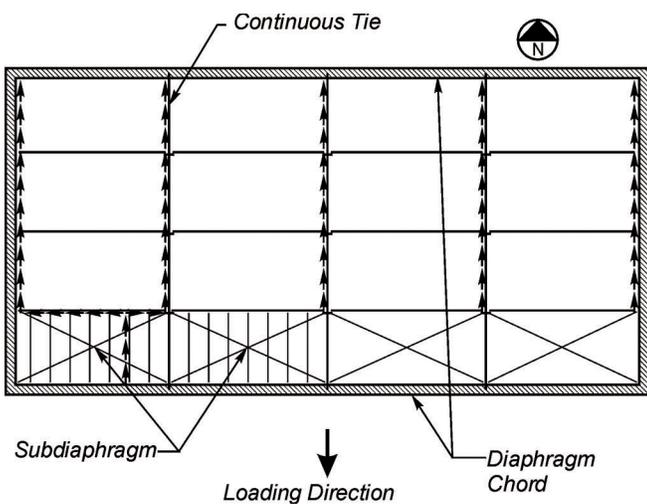


Figure 7 Continuous Ties With Subdiaphragms

Out-of-Plane Anchorage Details

Figures 8 and 9 show typical details for connecting concrete masonry walls to wood-framed diaphragms. The 1997 UBC requires that wall anchorage utilizing embedded straps must have the straps attached or hooked around reinforcing steel. In addition, wood framing or ledgers should not be used in cross-grain bending or cross-grain tension and the use of toe nails or nails subject to withdrawal is not permitted. All wood elements of the anchorage system must have a minimum actual net thickness of 2 1/2 inches.

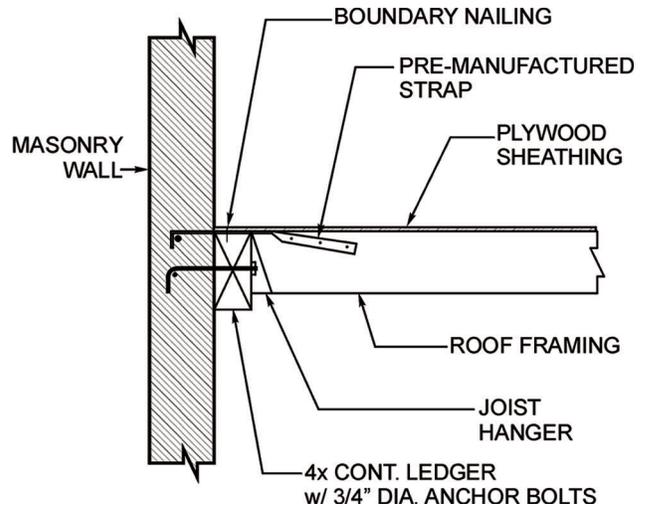


Figure 8 Typical Connection for Anchorage of Concrete Masonry Wall to Wood Diaphragm

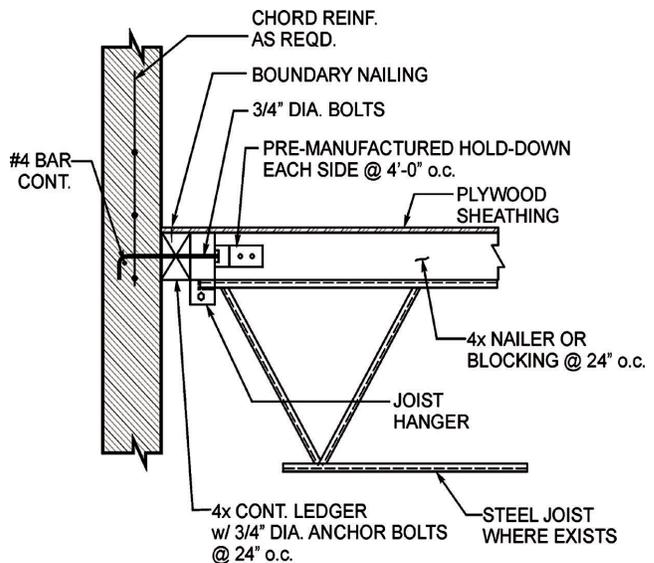


Figure 9 Typical Connection for Anchorage of Concrete Masonry Wall to Wood Diaphragm

Example

Design the anchorage connection for the wall shown in Figure 10. The building is constructed with 12-inch concrete masonry (weight = 124 plf, $f'_m = 1500$ psi) and is located on a site less than 2 km from a Type A seismic source ($C_a = 0.48$).

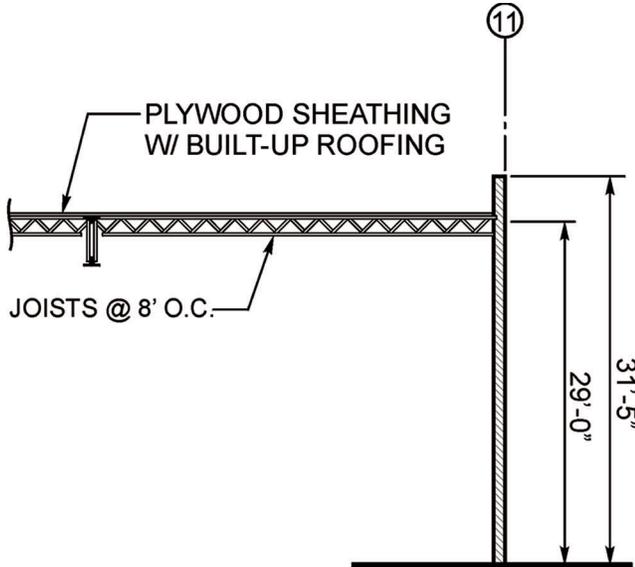


Figure 10 Example Problem

Solution

Tributary wall weight:

$$W_p = 124 \left[\frac{29}{2} + 2.5 \right] = 2108 \text{ lbs/ft}$$

Earthquake load:

$$\begin{aligned} F_p &= \frac{a_p C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_r} \right) W_p \\ &= \frac{1.5 C_a I_p}{3.0} \left(1 + 3 \frac{29}{29} \right) W_p = 2 C_a I_p W_p \\ &= 2(0.48)(1.0) W_p = 0.96 W_p \\ &= 0.96(2108) = 2024 \text{ lbs/ft} > 420 \text{ lbs/ft} \end{aligned}$$

Anchor Bolts:

- Pre-manufactured Hold-Down with 2-3/4-inch Anchor Bolts spaced 7 inches apart
- Embedment Length, $l_b = 6$ in
- Separate Bolts for Gravity Loads

$$A_p = \pi l_b^2 = \pi(6)^2 = 113.1 \text{ in}^2$$

Reduction due to overlapping area:

$$\begin{aligned} A'_p &= A_p - \left[l_b^2 \left\{ \cos^{-1} \left(\frac{s}{2l_b} \right) \right\} - \left(\frac{s}{2} \right) \sqrt{l_b^2 - \left(\frac{s}{2} \right)^2} \right] \\ &= 113.1 - \left[(6)^2 \left\{ \cos^{-1} \left(\frac{7}{2(6)} \right) \right\} - \left(\frac{7}{2} \right) \sqrt{(6)^2 - \left(\frac{7}{2} \right)^2} \right] \\ &= 96 \text{ in}^2 \end{aligned}$$

Strength Design:

$$b_t = E_n = 2024 \text{ lbs/ft}$$

Anchor bolts pullout capacity:

$$\begin{aligned} \phi B_{tn} &= \phi 1.0 A_p \sqrt{f'_m} \\ &= 0.8(1.0)(96) \sqrt{1500} \\ &= 2975 \text{ lbs} \leftarrow \text{Governs} \end{aligned}$$

Anchor Bolts Yield Strength:

$$\begin{aligned} \phi B_t &= \phi(0.4 A_b f_y) \\ &= 0.8(0.4)(0.44)36000 = 5068 \text{ lbs} \end{aligned}$$

$$s_{reqd} = \frac{2B_t}{b_t} = \frac{2(2975)}{2024} = 2.94 \text{ ft}$$

Working stress design:

$$b_t = \frac{E_h}{1.4} = \frac{2024}{1.4} = 1446 \text{ lbs/ft}$$

Anchor bolts pullout capacity:

$$\begin{aligned} B_t &= 1.33(0.5) A'_p \sqrt{f'_m} \\ &= 1.33(0.5)(96) \sqrt{1500} \\ &= 2473 \text{ lbs} \leftarrow \text{Governs} \end{aligned}$$

Anchor Bolts Yield Strength:

$$\begin{aligned} B_t &= 1.33(0.2 A_b f_y) \\ &= 1.33(0.2)(0.44)36000 \\ &= 4213 \text{ lbs} \end{aligned}$$

$$s_{reqd} = \frac{2B_t}{b_t} = \frac{2(2473)}{1446} = 3.42 \text{ ft}$$

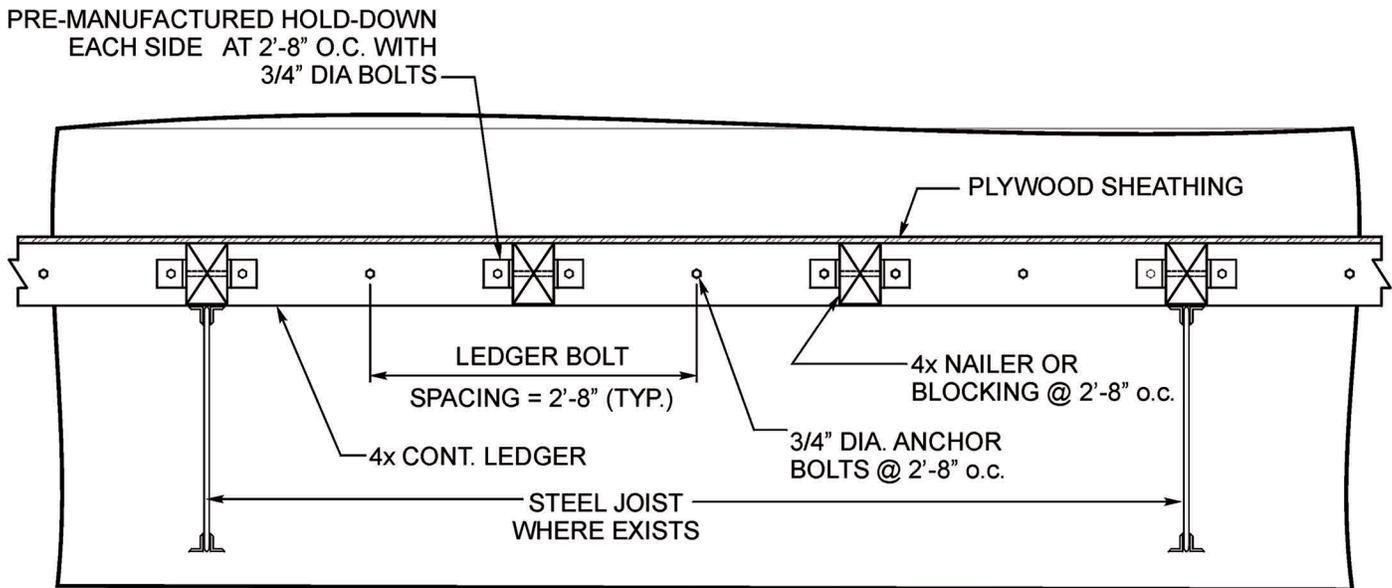


Figure 11 Solution to Wall Anchorage Design Example

References

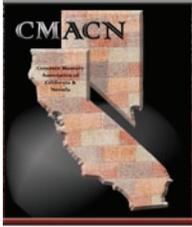
- [1] International Conference of Building Officials (ICBO), 1997 Uniform Building Code, International Conference of Building Officials, Whittier, California, 1997.
- [2] International Code Council (ICC), 2003 International Building Code, International Code Council, Inc., Falls Church, Virginia, 2003.
- [3] Seismology Committee, Structural Engineers Association of California, Recommended Lateral Force Requirements and Commentary, Structural Engineers Association of California (SEAOC), Sacramento, CA 1999.

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Dr. Chukwuma Ekwueme received his BSCE from the University of Nigeria, Nsukka, Nigeria 1987, in Civil Engineering, his MSSE, at the University of California, Los Angeles, CA, 1990, in Earthquake and Structural Engineering, a DEng, (Degree of Engineer) from the University of California, Los Angeles, CA, 1992, and his PhD, from the University of California, Los Angeles, CA, 1994, in Structural Engineering. He is a registered Structural Engineer in California. Dr. Ekwueme has worked for Hart-Weidlinger in many capacities since 1992 and since 2004 continues to work as a Senior Associate.

Dr. Ekwueme is a member and leader of many organizations and committees such as ASCE, SEI, EERI, SEAOC, ACI and TMS. He has written several publications and co-authored CMACN's Seismic Design of Masonry Using the 1997 UBC and our soon to be released 2006 edition of Design of Reinforced Masonry Structures. He has also received awards for his outstanding work as a structural engineer.



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