Chapter 11

Seismic Design of Wood and Masonry Buildings

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Abstract: The purpose of this chapter is to present criteria and example problems of the current state of practice of seismic design of wood and reinforced masonry buildings. It is assumed that the reader is familiar with the provisions of either the Uniform Building Code (UBC), Building Officials and Code Administrators (BOCA), or Southern Building Code Congress International (SBCCI), or international code council, international building code (IBC). For consistency of presentation the primary reference, including notations and definitions, will be to the UBC 97. Included within the presentation on diaphragms are criteria and example problems for both rigid and flexible diaphragms. Also included is the UBC 97 criteria for the analytical definition of rigid versus flexible diaphragms. Wood shear walls and the distribution of lateral forces to a series of wood shear walls is presented using Allowable Stress Design (ASD). Masonry slender walls (out-of-plane loads) and masonry shear walls (in-plane loads) are presented using Load and Resistance Factor Design (LRFD).

11.1 INTRODUCTION

The design process can be separated into two basic efforts; the design for vertical loads and the design for lateral forces. The design for vertical loads for both wood and masonry is currently in transition from Allowable Stress Design (ASD) to Load and Resistance Factor Design (LRFD). The draft LRFD criteria for wood[11-52, 11-53] is currently being reviewed by various industry committees prior to being submitted to the IBC codes for adoption.[11-28, 11-36] The LRFD criteria for masonry walls for both in-plane and out-of-plane loads is currently in the Uniform Building Code - 1997.[11-38]

The current state of practice is to design wood members for vertical loads using ASD including all the unique Wood Design Modification Factors, see Table 11-1.[11-35, 11-51] Masonry members are designed for vertical loads using Working Stress Design (WSD) with the standard linear stress - strain distribution assumptions. Wood members, both horizontal diaphragms and vertical diaphragms (shear walls), are designed for lateral forces using ASD; while masonry shear walls are designed for lateral forces using LRFD.

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Wood shear walls and the distribution of lateral forces to a series of wood shear walls is presented using Allowable Stress Design (ASD). Masonry slender walls (out-of-plane loads) and masonry shear walls (in-plane loads) are presented using Load and Resistance Factor Design (LRFD).

11.2 LRFD/ Limit-State Design for Wood Construction

A United States and Canadian wood industry-sponsored effort to develop a reliability-based, load and resistance factor design (LRFD) Specification for engineered wood construction in the U.S. has been underway since 1988[11-49]. Far-reaching changes in design and material property assessment methodology have resulted. Not only has an LRFD Specification been developed using accepted principles of reliability-based design but many other up-to-the-minute applications of recent design and materials research have been incorporated. Now undergoing a Joint American Society of Civil Engineers (ASCE)/Industry Standards Committee review, the LRFD Specification for Wood Construction is expected to be presented in the international building code in the near future.

11.2.1 Design Methodology

Important advances in design methodology and in procedures for assessing the strength of components and connections have been made for the new LRFD Specification.[11-42, 11-43, 11-46, 11-47, 11-50]

Load and resistance factor design (LRFD) methodology has become the standard procedure for practical application of the principles of reliability-based design. For the U.S. LRFD Specification, a simple format was chosen:

$$\lambda \phi R \geq \sum \gamma_i Q_i$$

where:

λ = time effect (duration of load) factor

Table 11-1. Wood Matrix of Design Modification Coefficients, Ref NDS[11-51]

	NDS	Allowable Stresses									Mod	Bolts		
Factor	Section	F_b	F_c	F_{cp}	F_n	F_r	F_{rc}	F_{rb}	F_t	F_v	E	p	q	Comment
C_c	5.3.4	X												Curvature (Gluelams Only)
C_F	4.3.2	X	X						X					Size Factor for Sawn Members Only
C_f	2.3.8	X												Form
C_R	2.3.6	X	X	X	X	X	X	X	X	X		X	X	Fire Retardant Treatment
C_b	2.3.10			X									X	Compression Perpendicular to Grain
C_D	2.3.2	X	X		X	X	X	X	X	X		X	X	Load Duration
C_M	2.3.3	X	X	X					X	X	X	X	X	Wet Service
C_p	2.3.9/3.7		X									X		Column Stability
C_L	2.3.7/3.3.3	X												Slenderness/Stability – Do not use with C_V
C_t	2.3.4	X	X	X	X	X	X	X	X	X	X	X	X	Temperature
C_T	4.4.3										X			Deflection Critical – Buckling Stiffness for 2x4 Truss
C_G	7.3.6											X	X	Group Action
C_{fu}	4.3.3/5.3.3	X												Flat Use (2” to 4” thick and Glulam only)
C_H	4.4.2									X				Horizontal Shear
C_V	5.3.2	X												Volume Factor GluLam Member Only
C_r	4.3.4	X												Repetitive Member
C_i	2.3.11	X	X	X					X	X	X			Incising to Increase Penetration of Preservatives

F_b = Bending
F_c = Compression
F_{cp} = Compression Perpendicular to Grain
F_n = Hankinson Formula (3.10)
F_r = Radial Stress
F_{rc} = Radial Stress Compression (5.4.1)
F_{rb} = Radial Stress Tension (5.4.1)
F_t = Tension
F_v = Horizontal Shear

E = Modules of Elasticity
p = Parallel to Grain
q = Perpendicular to Grain

Examples:

$F_x' = F_x \times \text{sum}\ (C_i \ldots C_n)$

$F_b' = F_b(C_c)C_{v,}\ C_F \text{ or } C_L\ (C_f)(C_R)(C_D)(C_M)(C_t)$

$$\text{Defl}' = \frac{\text{Deflection x E}}{EC_tC_mC_R}$$

ϕ = resistance factor
R = reference resistance
γ_i = load factors
Q_i = effects of prescribed nominal loads

The reference resistance, R, includes all the necessary corrections for the effects of moisture and/or other end-use conditions. The load factors have been chosen to conform with U.S. practice for most engineered construction using values from ASCE 7-98[11-53]. Time effect factors, λ, have been completely reassessed. Using the latest stochastic load models and applying damage, the accumulation models of Gerhards and Link [11-45], new time effect factors have been developed by Ellingwood and Rosowsky [11-43]. These time effect factors apply to the short term (5 minute) test strength of the wood member. The values resulting from these studies are summarized in Table 11-2.

Table 11-2. Time Effect Factors (λ)

Load Combination		Time Effect Factor
1.4 D		0.6
1.2 D+1.6L + 0.5 (L_l or S or R)	$L_{storage}$	0.7
	$L_{occupancy}$	0.8
	l_{impact}	1.25*
1.2D+1.6(L_l or S or R) + 0.5L		0.8
1.2D+1.6(L_l or S or R) + 0.8W		1.0
1.2D+1.3W+0.5L+0.5(L_l or S or R)		1.0
1.2D+1.5E+(0.5L or 0.2S)		1.0
0.9D-(1.3W or 1.5E)		1.0

*For connections, = 1.0 for L from impact.

Resistance factors, ϕ , have been assigned for each limit state, i.e., tension, compression, shear, etc. The following factors have been assigned for the current draft of the LRFD Specification:

ϕ_b (flexure) = 0.85
ϕ_c (compression) = 0.90
ϕ_s (stability) = 0.85
ϕ_t (tension) = 0.80
ϕ_v (shear) = 0.75
ϕ_z (connections) = 0.65

The use of simple factors for each limit state requires that the strength of components and connections include adjustment from a basic fifth percentile value (or average yield limit value for connections) to a level which will maintain prescribed levels of reliability. This method achieves designer simplicity and enables accurate strength assessment for each component, member and connection[11-47].

As an example, the basic equation for moment design of bending members is

$$\lambda\phi_b M' = \lambda\phi_b F_b' S > M_U$$

Where
λ, = The Effect Factor
ϕ_b = 0.85
F_b’ = $F_b\ C_L\ C_f\ C_R\ C_D\ C_M\ C_t$
S = Section Modulus
M’ = Adjusted Moment Resistance
M_U =Factored Moment (i.e. 1.2D+1.6L)

11.2.2 Serviceability / Drift

Serviceability issues have long been recognized as an important consideration in the design of wood structures. Current specifications include limitations on deflection such as span/360 aimed at preventing cracking and providing protection from excessive deflection. While such restriction have proved to be adequate in many cases, they do not uniformly address problems of vibration and other serviceability issues [11-50].

The U.S. LRFD Specification has taken a different approach which more nearly reflects practice regarding serviceability issues with other construction materials. The Specification requires structural engineers to address serviceability in design to ensure that "deflections of structural members and systems due to service loads shall not impair the serviceability of the structure." To assist the structural engineer in checking for serviceability, a comprehensive commentary is provided. Serviceability is defined broadly to include:

- Excessive deflections or rotation that may affect the appearance, functional use or drainage of the structure, or may cause damaging transfer of load to non-load supporting elements and attachments.

- Excessive vibrations produced by the activities of building occupants or the wind, which may cause occupant discomfort or malfunction of building service equipment.

- Deterioration, including weathering, rotting, and discoloration."

It should be noted that checks on deflection and vibration should be made under service loads. The Specification defines service loads as follows:

> "Service loads that may require consideration include static loads from the occupants and their possessions, snow on roofs, temperature fluctuations, and dynamic loads for human activities, wind-induced effects, or the operation of building service equipment. The service loads are those loads that act on the structures at an arbitrary point in time. In contrast, the nominal loads are loads with a small probability (in the range of 0.01 to 0.10) of being exceeded in 50 years (ASCE 7-98). Thus, appropriate service loads for checking serviceability limit states may be only a fraction of the nominal loads."

Detailed guidance is provided in the Specification Commentary for serviceability design for vertical deflections, drift of walls and frames, deflection compatibility, vibration prevention and for long-term deflection (creep). While this approach is not as prescriptive as in past codes, it is felt that by providing detailed guidance on methods for preventing serviceability problems, structural engineers will deal more realistically with these issues. In the past, structural engineers have often been misled into believing that by simply meeting a prescriptive requirement, SPAN/360 for example, that serviceability requirements would automatically be satisfied. Of course, this has not always been the case.

11.3 LRFD/ LIMIT-STATE DESIGN FOR MASONRY CONSTRUCTION

The seismic design of masonry structures has made significant advances in the last decade. Initially the lead was provided by New Zealand and Canadian structural engineers and their contributions can be noted in the proceedings of the first three North American Masonry Conferences[11-1,11-2,11-3] plus the third and forth Canadian Masonry Symposia[11-4,11-5]. In the United States the work of the Masonry Society in the development of the 1985 Uniform Building Code[11-6] provided a point which marks a change in attitude and direction of seismic masonry design. While notable earlier masonry research efforts by Hegemier[11-7] and Mayes[11-8] were directed at seismic design considerations, it was the development of the 1985 UBC code, the Structural Engineers Association of California (SEAOC) review of the proposed code, and finally the adaptation in the 1985 by International Conference of building Officials that started the new direction for seismic design of masonry structures.

The development of this new seismic design approach from the design implementation perspective is documented by approval by the International Conference of Building Officials (ICBO) of three design standards. They are:

1. The Strength Design Criteria for slender walls in section 2411 of the 1985/1991 UBC.
2. The Strength Design Criteria for one to four story buildings in ICBO Evaluation Services Inc., Evaluation Report Number 4115, first published in 1983[11-9]
3. The Strength Design Criteria for shear walls in Section 2412 of the 1988/1991 UBC[11-10].

11.3.1 Behavior and Limit States

The behavior of a masonry component or system when subjected to loads can be described in terms of behavior and limit states. For illustrative purposes, we will use the slender wall shown in Figure 11-1.

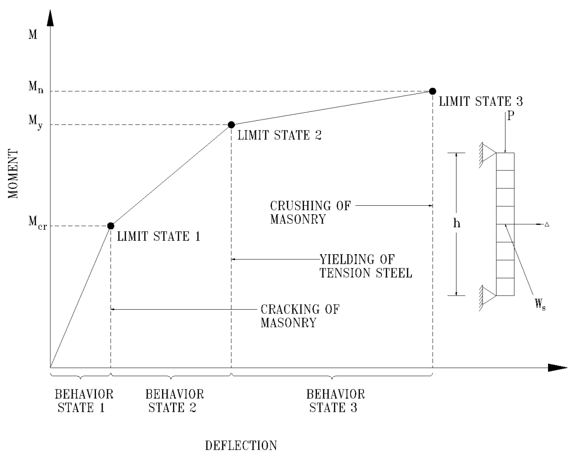


Figure 11-1. Moment-deflection curve for a typical slender wall

Table 11-3. Behavior and Limit States for a Ductile Slender Wall.

State	Description
Behavior state 1	Uncracked cross-section and $M < M_{cr}$
Limit state 1	$M = M_{cr}$ and stress in the masonry equal to the modulus of rupture.
Behavior state 2	Cracked cross-section with strain in the steel less than its yield strain and $M_{cr} < M < M_y$.
Limit state 2	$M = M_y$ and strain in the steel equal to its yield strain.
Behavior state 3	Cracked cross-section with strain in the steel greater than its yield strain but the maximum strain in the masonry less than its maximum usable strain and $M_y < M < M_u$
Limit state 3	$M = M_u$ and strain in masonry equal to maximum usable strain.

As indicated in this figure the slender wall can be idealized for structural design as evolving through several identifiable states of behavior prior to reaching its final deformed position. We can define this evolution in terms of "Behavior States". Table 11-3 defines the behavior states for the slender wall. For example, the first behavior state corresponds to the stress condition where the load-induced tensile stress is less than the modulus of rupture. In this behavior state, the wall cross section is uncracked and the load-induced moment is less than the cracking moment capacity of the wall cross section.

A "Limit State" exists at the end of each behavior state (see Table 11-3). For example, at the end of the first behavior state, we have the first limit state and it exists when the lateral load on the wall produces a tensile stress equal to the modulus of rupture.

The slender wall, goes through several behavior states prior to reaching its final or "Ultimate Limit State". For example, if we consider the load-induced moment as a measurable variable, it can be used to define the existence of the first limit state. In this case, the load-induced moment M will be equal to the cracking moment of the cross section (M_{cr}). The second limit state exists when the moment M is equal to the yield moment (M_y) and the third limit state exists when M is equal to the moment capacity of the wall (M_n). Therefore, we have identified three limit states whose existence can be numerically quantified as follows:

Limit State	Moment	Condition/Comment
1	M_{cr}	Serviceability/Cracking of Cross Section
2	M_y	Damage Control/ Permanent Steel Deformation
3	M_n	Ultimate/Nominal Moment Strength

Each of these limit states can be the focus of concern for the structural engineer according to different client or design criteria requirements. For example, the first limit state relates to the cracking of the cross section, and thus, possible water penetration. It can be viewed as a "Serviceability Limit State". The second limit state defines the start of permanent steel deformation or significant structural damage. It can be viewed as either a "Serviceability" or a

"Structural Damage Limit State". Finally, the third limit state defines the limit of our acceptable wall performance from a life safety perspective. Therefore, it is an "Ultimate" or "Strength" Limit State. Typically, it is this limit state that we are concerned with when we use the design approach called strength design. Limit state design can be thought of as a generalization of strength design where we leave open the possibility of addressing limit states other than the strength limit state.

The structural engineer must review the limit states that can exist for the structure he or she is designing. Then, a design criteria must be established that ensures, with an acceptable level of reliability, that the limit states that the structural engineer has identified as undesirable do not exist. For example, current slender wall design criteria adopted by the International Congress of Building Officials (ICBO) in the 1994 and 1997 Uniform Building Codes (UBC) identify an ultimate or strength design limit state that corresponds to limit state 3 in Table 11-3[11-6, 11-10]. For this example, the "Limit State Equation" is:

$$M_u \leq \phi M_n \tag{11-1}$$

where

M_u = Factored Moment or Load induced moment obtained from factored design loads.

M_n = nominal moment strength of the wall.

ϕ = capacity reduction factor that is intended to ensure that an acceptable level of reliability exists in the final design.

The design criteria must address both sides of Equation 11-1. The load-induced moment is obtained from a structural analysis using factored deterministic design loads. We calculate the nominal moment capacity of the wall using the nominal design values of the structural parameters, e.g., specified compressive strength, modulus of elasticity, etc., and the equations of structural engineering.

11.3.2 Limit States and Structural Reliability

One task in the United States-Japan coordinated research program under the direction of the Technical Coordinating Council for Masonry Research (TCCMAR) focused on the evaluation of available approaches whereby masonry design could incorporate the analytical method of structural reliability into "Limit State Design"[11]. These reliability methods ranged from the very direct to the extremely sophisticated. It is the conclusion of the TCCMAR Category 8, Task 8.1 research that it is possible to significantly extend the rigor of today's masonry code to incorporate structural reliability. The new Steel Design Criteria accepted for the 1988 Uniform Building Code is Load and Resistance Factor Design (LRFD) and is based on structural reliability[11-12,11-13,11-14,11-15]. LRFD will, in all probability, be the basis of modern reinforced masonry design. The remainder of this section presents the basics of the LRFD approach and indicates why the identification and quantification of behavior and limit states is so important.

A limit state occurs when a load, Q, on a structural component equals the resistance, R, of the component. The occurrence of the limit state exists when F=0, where

$$F = R - Q \tag{11-2}$$

Consider our slender wall example and the third (or strength) limit state. We can consider R to be the moment capacity of the wall and Q to be the dead plus live plus seismic moment demand. If we denote the factored moment or "Moment Demand" as M_u, and the nominal moment strength or "Moment Capacity" as M_n, then Equation 11-2 can be written as

$$F = M_n - M_u \tag{11-3}$$

This equation is called the limit state design equation. The strength limit state exists when $M_u = M_n$ or, alternatively, $F = 0$. Stated differently, if F is greater than zero we know that one of the first three behavior states exists and that the third limit state does not exist.

The economics of building design and construction requires us to have a balance between the safety that a limit state will not exist or be violated and construction costs. This, historically, has been attained by using a term called the factor of safety. In structural reliability, the parallel term is referred to as the "Reliability Index" associated with the limit state under consideration.

Because M_n and M_u are not known with certainty they are called random variables. F is a function of M_n and M_u. Hence, it is also a random variable with a mean F and standard deviation σ_F. The reliability index is defined in terms of the statistical moments of F. The reliability index β can be defined as

$$\beta = F/\sigma_F \tag{11-4}$$

Structural reliability theory and the associated mathematics is typically too complex for most design applications. Therefore, for design purposes, we must develop a more direct design criteria. Ideally, it is based on structural reliability concepts. This can be accomplished using the "First Order Second Moment" structural reliability theory. This theory first performs a Taylor's series expansion of F in terms of the random variables, for example R and Q. This expansion is done about the mean value of the random variables and only the first order partial derivatives are retained in the Taylor's series expansion, i.e., the name first order. Next, the mean and standard deviation of F in its Taylor's Series expanded form are calculated in terms of the mean and standard deviation (or, alternatively, coefficient of variation) of R and Q. Thus, the second term in the name "first order second moment" refers to second order statistical moments. With the mean and standard deviation of F so calculated, the reliability index can be expressed in terms of a constant α, the means (R and Q) and coefficient of variations (V_R and V_Q) of the random variables. So doing, we can write:

$$Q\,e^{\mu\beta V_Q} = R\,e^{-\mu\beta V_R} \tag{11-5}$$

Note that the right side of the equation relates to the resistance and the left side to the load effect. If we again consider the slender wall example, we can express this equation as:

$$M_u\,e^{\mu\beta V_{M_u}} = M_n\,e^{-\mu\beta V_{M_n}} \tag{11-6}$$

where

M_u and M_n = mean of M_u and M_n.
V_{Mu} and V_{Mn} = coefficient of variation of M_u and M_n.

The left hand side of Equation 11-6 is the factored moment or "Design Moment Demand" and ideally is equal to the left hand side of Equation 11-1. The ASCE 7-88 [(11-41)] load factors or similar reliability based factored loads define this design moment demand.

The right hand side of Equation 11-6 is the nominal moment strength or "Design Moment Capacity" that will have a level of structural reliability or safety β. This can be written as:

$$M_n = M_u\,e^{-\mu\beta V_{M_u}} \tag{11-7}$$

If we recall the right hand side of the limit state design equation for moment capacity given in Equation 11-1, it follows that:

$$M_u = \phi M_n = \phi M_u\;e^{-\mu\beta V_{M_u}} \tag{11-8}$$

Therefore, the capacity reduction factor ϕ, for this limit state is:

$$\phi = \frac{M_u}{M_n}\,e^{-\mu\beta V_{Mu}} \tag{11-9}$$

Equation 11-9 shows the dependence of the capacity reduction factor ϕ on: (i) the ratio of the factored moment to nominal design moment, (ii) the uncertainty or quality of construction and analytical modeling as manifested in the value of V_{Mu}, and (iii) the level of reliability, β value, that the design criteria seeks to attain. These three items can and must be the focus of discussion among those involved in future masonry design criteria development.

11.4 SEISMIC LATERAL FORCES AND HORIZONTAL DIAPHRAGMS

11.4.1 Seismic Lateral Forces

Most wood and masonry buildings are one to three stories in height and qualify to be designed using a static lateral force procedure (SLFP). Thus the total design base shear in a given direction (V) is determined from the following Formula:

$$V = \frac{C_V I}{RT} W \qquad (11\text{-}10A)$$

The total design base shear need not exceed the following:

$$V = \frac{2.5 C_a I}{R} W \qquad (11\text{-}10B)$$

The total design base shear shall not be less than the following:

$$V = 0.11 C_a I W \qquad (11\text{-}10C)$$

In addition, for seismic zone 4, the total base shear shall also not be less than the following:

$$V = \frac{0.8 Z N_V I}{R} W \qquad (11\text{-}10D)$$

Where:

C_V = Seismic coefficient dependent upon soil profile type, as set forth in table 16-R of UBC 97(11-38). C_V is a function of Z, seismic zone factor (effective peak ground acceleration) and N_V, near-source factor in seismic zone 4.

I = Importance factor.

R = Numerical coefficient represen-tative of the inherent over strength and global ductility capacity of lateral – force – Resisting systems, as set forth in table 16-N or 16-P of UBC 97 (11-38).

T = Elastic fundamental period of vibration, in seconds, of the structure in the direction under consideration. The fundamental period T may be approximated from this following formula:

$$T = C_t \left(h_n\right)^{3/4} \qquad (11\text{-}10E)$$

Where:

C_t = 0.035 for steel moment-resisting frames
= 0.030 for reinforced concrete moment resisting frames
= 0.020 for all other buildings

W = The total seismic dead load including partition loads, snow loads, weight of permanent equipment and a minimum of 25 percent of storage live load (Note: Sotrage live load is defined as a uniform load of 125 PSf or greater).

C_a = Seismic coefficient dependent upon soil profile type, as set forth in table 16-Q of UBC97(11-38). C_a is a function of Z, seismic zone factor (effective peak ground acceleration) and N_a, near-source factor in seismic zone 4.

N_a = Near-source factor used in the determination of C_a in seismic zone 4 related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in tables 16-S and 16-U of UBC97(11-38). Note the magnitude of N_a (and thus the increase in base shear V) varies from 1.0 to 1.5.

N_V = Near-source factor used in the determination of C_V in seismic zone 4 related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in tables 16-T and 16-U of UBC97([11-38]. Note the magnitude of N_V (and thus the increase in base shear V) varies from 1.0 to 2.0.

A comparison of design base shear values for a 3-story wood building and a 3-story masonry building are presented in tables 11-10 and 11-11 (last chapter page). Note that for these types of buildings (relatively stiff/Rigid structural system with short period) The total design base shear is governed by Eq. 11-10B. Also note special provisions for near field effects in seismic zone 4 (i.e. N_V and N_a) and special minimum base shear equation 11-10D.

The vertical distribution of the design base shear over height of the structure is determined by the following formula:

$$F_x = \frac{(V - F_t)W_x h_x}{\sum_{i=1}^{n} W_i h_i} \qquad \text{(Eq.11-11)}$$

Where:

F_x = force applied at level n
w_x = that portion of W located at level x
h_i = height above base to level x
F_t = 0.07TV
= 0 for T of 0.7 seconds or less
= 0 for most wood or masonary buildings

The story force F_x at each level is applied to the diaphragm, then distributed through the diaphragm, collected by the drag or collector members, and delivered to the vertical lateral force resisting elements, such as shear walls, frames, braces, etc. The walls, frames or braces which resist these forces at each level, shall be analyzed and designed to meet stress and drift requirements.

Horizontal diaphragms (floor and roof diaphragms) shall be designed to resist forces determined in accordance with the following formula:

$$F_{px} = \frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} W_i} Wpx \qquad \text{(Eq. 11-12)}$$

Where:

F_{px} need not exceed $0.5C_aIWpx$ but shall not be less than $1.0C_aIWpx$.

The forces in both formulas are inertia forces at each level which represents the acceleration of the weight at each level. Formula (Eq. 11-11) produces the triangular distribution of forces for the overall analysis of the building which should fairly represent the distribution of forces from a dynamic analysis where the modes are combined. Formula (Eq. 11-12) represents a diaphragm design force which should represent the acceleration determined from the dynamic analysis for each diaphragm times the weight of the diaphragm. It is preferable to use the term "seismic coefficient" rather than acceleration/g since both formulas do not represent true earthquake acceleration but rather scaled design forces. Both formulas yield the same seismic coefficient for a one story building or at the roof of a multi-story building. The diaphragm design seismic coefficients are always larger than those for the story forces for the other levels.

The weight terms in Formula (Eq. 11-11) and (Eq. 11-12) are different. The term W_x in Formula (Eq. 11-11) is the total weight of each level of the building including all seismic resisting elements (walls, etc.) in both directions. The term W_{px} is the weight of the diaphragm and the seismic resisting elements which are being accelerated with the diaphragm and typically does not include the weight of the seismic resisting elements parallel to the direction of the forces (perpendicular to the span of the diaphragm)

Concrete or masonry walls shall be anchored to all floors and roofs which provide lateral support for the wall. The anchorage shall provide positive direct connections between the wall and floor or roof construction capable of resisting the forces specified or a minimum force of 280 plf, whichever is greater. Walls shall be designed to resist bending between anchors when the anchor spacing exceeds 4 feet. Diaphragm deformations shall be considered in the design of the supported walls.

Diaphragms supporting concrete or masonry walls shall have continuous ties or struts between diaphragm chords to distribute the anchor forces. Added chords may be used to form sub-diaphragms to transmit the anchor forces to the main cross ties. A sub-diaphragm is a portion of a larger diaphragm designed to anchor and transfer local forces to primary diaphragm struts and the main diaphragm.

In Seismic Zones Nos. 2,3 and 4 anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension, and the continuous ties required shall be in addition to the diaphragm sheathing.

11.4.2 Horizontal Diaphragms

The total shear at any level will be distributed to the various vertical lateral force resisting elements (VLFR) of the lateral force resisting system (shear walls or moment-resisting frames) in proportion to their rigidities considering the rigidity of the diaphragm. The effect of diaphragm stiffness on the distribution of lateral forces is discussed below. For this purpose, diaphragms are classified into two groups rigid or flexible.

A rigid diaphragm (concrete) is assumed to distribute horizontal forces to the VLFR elements in proportion to their relative rigidities.[11-29, 11-30, 11-31, 11-32] In other words, under symmetrical loading a rigid diaphragm will cause each VLFR element to deflect an equal amount with the result that a VLFR element with a high relative rigidity or stiffness will resist a greater proportion of the lateral force than an element with a lower rigidity factor.

A flexible diaphragm (maybe plywood) is analogous to a shear deflecting continuous beam or series of simply supported beams spanning between supports. The supports are considered non-yielding, as the relative stiffness of the vertical lateral force resisting elements compare to that of the diaphragm is great. Thus, a flexible diaphragm will be considered to distribute the lateral forces to the VLFR elements on a tributary area basis. A flexible diaphragm will not be considered capable of distributing torsional stresses, see Figure 11-2A & 11-2B.

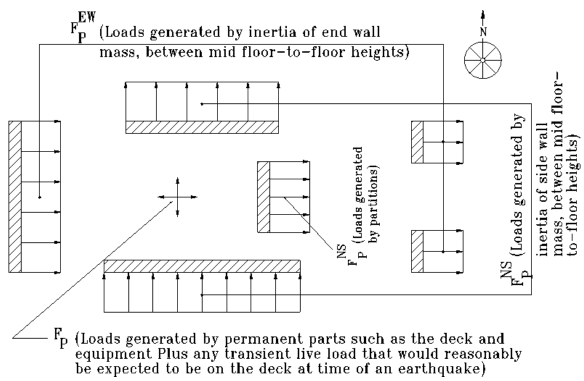


Figure 11-2A. Flexible/Plywood Diaphragm

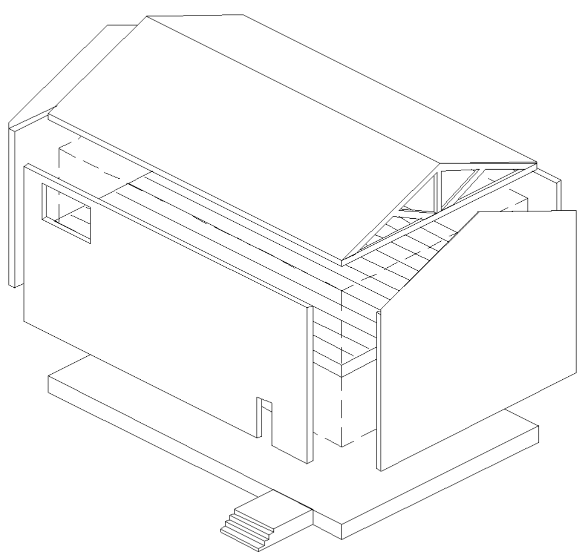


Figure 11-2B. Lateral Force Resisting System in all wood Building

Generally, it is assumed that the in-plane mass of a shear wall does not contribute to the diaphragm loading unless the shear wall is interrupted at the specific level. In case a shear wall does not extend below the floor level, both its horizontal and vertical loads must be distributed to the remaining walls. Of course, major difference in rigidities may be cause for redistribution.

A torsional moment is generated whenever the center of gravity (CG) of the lateral forces fails to coincide with the center of rigidity (CR) of the VLFR elements, providing the diaphragm is sufficiently rigid to transfer torsion. The magnitude of the torsional moment that is required to be distributed to the VLFR elements by a diaphragm is determined by the sum of the moments created by the physical eccentricity of the translational forces at the level of the diaphragm from the center of rigidity of the resisting elements ($M_T = F_p e$, where e = distance between CG and CR) plus the "accidental" torsion of 5%. The "accidental" torsion is an arbitrary code requirement intended to account for the uncertainties in the location of loads and stiffness of resisting elements. The accidental torsion is equivalent to the story shear acting with an eccentricity of not less than 5% of the building dimension at that level perpendicular to the direction of the force under consideration. The torsional distribution by rigid diaphragms to the resisting elements will be assigned to be in proportion to the stiffness of the elements and its distance from the center of rigidity.

The torsional design moment at a given story shall be the moment resulting from the eccentricities between applied design lateral forces at levels above that story and the VLFR elements in the story plus an accidental torsion. Negative torsional shear shall be neglected. Flexible diaphragms shall not be used for torsional distribution. Cantilever diaphragms on the other hand will distribute translational forces to VLFR elements, even if the diaphragm is flexible. In this case, the diaphragm and its chord act as a flexural beam on supports (VLFR elements) whose resistance is in the same direction as the forces.

Diaphragms shall be considered flexible for the purposes of distributions of story shear and torsional moments when the maximum lateral deformation is more than two times the average story drift of the associated story. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm itself under lateral force with the story drift of adjoining vertical lateral force resisting elements under equivalent tributary lateral force.

The critical aspect of this new definition is that it may require that a given diaphragm be designed as rigid in one direction and flexible in the other orthogonal direction. For example, the plywood roof of a large and narrow masonry building with minimal shear walls in the long direction could qualify as a rigid diaphragm in the long direction and flexible in the narrow or short direction; which is probably closer to the actual behavior and observed performance of this type of building during an earthquake. See Tables 11-4 and 11-5 for equations for deflections of walls and diaphragms.

The general characteristics of motion of a flexible diaphragm is that the walls, being relatively rigid, respond to the accelerations of the ground, but a flexible (wood or metal deck) roof diaphragm, responds with an amplified motion. In seismic zones 3 and 4 with flexible diaphragms as defined above provide lateral support for walls, the values of F_p for anchorage shall be increased 50 percent.

11.5 FLEXIBLE HORIZONTAL DIAPHRAGM (PLYWOOD)

A horizontal plywood diaphragm acts in a manner analogous to a deep beam, where the plywood skin acts as a "web", resisting shear, while the diaphragm edge members perform the function of "flanges", resisting tension and compression induced by bending. These edge members are commonly called chords in diaphragm design.

Table 11-4. Concrete/CMU/Brick Wall Displacements

Fixed – Fixed $\beta = 1.2 \ \& \ G = E/2.2$	Fixed – Hinged $\beta = 1.2 \ \& \ G = E/2.2$	Comments
$\begin{aligned}\Delta &= \frac{Ph^3}{12EI} + \frac{BPh}{GA} \\ &= \frac{P}{E}\left[\frac{h^3}{12I} + \frac{1.2h(2.2)}{A}\right] \\ &= \frac{P}{E}\left[\frac{h^3(12)}{12td^3} + \frac{2.64h}{td}\right] \\ &= \frac{P}{Et}\left[\left(\frac{h}{d}\right)^3 + 2.64\frac{h}{d}\right]\end{aligned}$	$\begin{aligned}\Delta &= \frac{Ph^3}{3EI} + \frac{BPh}{GA} \\ &= \frac{P}{E}\left[\frac{h^3}{3I} + \frac{1.2h(2.2)}{A}\right] \\ &= \frac{P}{E}\left[\frac{h^3(12)}{3td^3} + \frac{2.64h}{td}\right] \\ &= \frac{P}{Et}\left[4\left(\frac{h}{d}\right)^3 + 2.64\frac{h}{d}\right]\end{aligned}$	The Value for G as given in the literature varies from E/2.2 to E/2.5 $I = td^3/12$ A=td Where: t = Wall Thickness d = Wall Depth h = Wall Height p = Load applied at top of Wall (lbs)
Fixed – Fixed $\beta = 1.2 \ \& \ G = E/2.5$	Fixed – Fixed $\beta = 1.2 \ \& \ G = E/2.5$	
$\begin{aligned}\Delta &= \frac{P}{E}\left[\frac{h^3}{12I} + \frac{1.2h(2.5)}{A}\right] \\ &= \frac{P}{Et}\left[\left(\frac{h}{d}\right)^3 + 3.0\frac{h}{d}\right]\end{aligned}$	$\begin{aligned}\Delta &= \frac{P}{E}\left[\frac{h^3}{3I} + \frac{1.2h(2.5)}{A}\right] \\ &= \frac{P}{Et}\left[4\left(\frac{h}{d}\right)^3 + 3.0\frac{h}{d}\right]\end{aligned}$	

Table 11-5. Concrete Diaphragm Displacements

Hinged – Hinged $\beta = 1.2 \ \& \ G = E/2.2$	Hinged – Hinged $\beta = 1.2 \ \& \ G = E/2.2$	Comments
$\begin{aligned}\Delta &= \frac{5Wl^4}{384EI} + \frac{\beta l^2 W}{8AG} \\ &= \frac{5WL^4(12)}{384Etb^3} + \frac{1.2l^2W(2.2)}{8btE} \\ &= \frac{Wl^4}{6.4Etb^3} + \frac{0.33l^2W}{btE} \\ &= \frac{Wl}{Et}\left[\frac{1}{6.4}\left(\frac{l}{b}\right)^3 + 0.33\left(\frac{l}{b}\right)\right] \\ &= \frac{Wl}{6.4Et}\left[\left(\frac{l}{b}\right)^3 + 2.13\left(\frac{l}{b}\right)\right]\end{aligned}$	$\begin{aligned}\Delta &= \frac{5Wl^4}{384EI} + \frac{\beta l^2 W}{8AG} \\ &= \frac{5WL^4(12)}{384Etb^3} + \frac{1.2(l^2)W(2.5)}{8btE} \\ &= \frac{Wl^4}{6.4Etd} + \frac{0.375l^2W}{btE} \\ &= \frac{Wl}{Et}\left[\frac{1}{6.4}\left(\frac{l}{b}\right)^3 + 0.375\left(\frac{l}{b}\right)\right] \\ &= \frac{Wl}{6.4Et}\left[\left(\frac{l}{b}\right)^3 + 2.4\left(\frac{l}{b}\right)\right]\end{aligned}$	The Value for G as given in the literature varies from E/2.2 to E/2.5 $I = tb^3/12$ A = tb Where: t = Diaphragm thickness b = Diaphragm Depth l = Diaphragm Length/Width w = Load applied along length of diaphragm (Plf)

Due to the great depth of most diaphragms in the direction parallel to application of force, and to their means of assembly, their behavior differs from that of the usual, relatively shallow, beam. Shear stresses have been proven to be essentially uniform across the depth of the diaphragm, rather than showing significant parabolic variation as in web of a beam. Similarly, chords, in a diaphragm are designed to carry all "flange" stresses, acting in simple tension or compression, rather than sharing these stresses significantly with the web. As in a beam, consideration must be given to bearing stiffeners, continuity of webs and chords, and web buckling.

Plywood diaphragms vary considerably in force-carrying capacity, depending on whether they are "blocked" or "unblocked". Blocking consist of lightweight nailers, usually 2 X 4's, framed between the joist, or other primary structural supports, for the specific purpose of connecting the edges of the plywood sheets. The reason for blocking the diaphragms is to allow nailing of the plywood sheets at all edges for better shear transfer. Design of unblocked diaphragms is controlled by buckling of unsupported plywood panel edges, with the result that such units reach a maximum load above which increased nailing will not increase capacity. For the same nail spacing, allowable design forces on blocked diaphragm are from 1½ to 2 times allowable design forces on its unblocked counter part. In addition, the maximum forces for which a blocked diaphragm can be designed are many times greater than those without blocking.

In a uniformly loaded floor or roof plywood diaphragm the shear normally decreases from a maximum at the exterior wall or boundary to zero at the centerline of a simple single diaphragm building. The four regions of diaphragm nailing are as follows: (1) Boundary - exterior perimeter of the diaphragm;(2) Continuous panel edges - based on the lay of the plywood, the continuous panel edges consist of multiple panel edges in a straight line parallel to the direction of diaphragm shear; (3) Other panel edges - including staggered (or discontinuous) panel edges; and (4) field - interior of plywood panels. See UBC97 Table 23-11-H for diaphragm values and figures.

A common method of plywood diaphragm design is to vary the nail spacing of the boundary/continuous panel edges and the other panel edges based on the shear diagram. Using this procedure the engineer assigns regions of nail spacing. The transition areas between shear capacity regions are not considered boundary conditions. Boundary nailing only occurs at the perimeter of the plywood diaphragm (i.e. exterior wall). More complicated buildings may be comprised of two or more diaphragms which will require boundary nailing along interior walls and drag struts/collector elements.

The three major parts of a diaphragm are the web, the chords, and the connections. Since the individual pieces of the web must be connected to form a unit, and since the chord members in all probability are not single pieces; connections are critical to good diaphragm action. Their choice actually becomes a major part of the design procedure. Diaphragms are most commonly used for roofs and floors. They function usually as simple beams, and sometimes as cantilever beams. Shear walls or vertical diaphragms function as cantilevered beams. Each diaphragm serves, like a beam, only to transfer force. It must, therefore, be properly connected to resisting elements which can accommodate the force.

Horizontal and vertical diaphragms sheathed with plywood may be used to resist horizontal forces not exceeding those set forth in the code, or may be calculated by principles of mechanics without limitation by using values of nail strength and plywood shear values.[11-39] Plywood for horizontal diaphragms should be at least ½ inch thick with joist spaced a maximum of 24 inches on center. It is not uncommon to specify 5/8 inch thick plywood with joist spaced a maximum of 24 inches on center for roof construction and 3/4 inch plywood with joist spaced a maximum of 16 inches on center for floor construction to minimize vertical load deflection and vibration concerns.

All boundary members shall be proportioned and spliced where necessary to transmit direction stresses. Framing members shall be at least 2-inch nominal in the dimension to which the plywood is attached. In general, panel edges shall bear on the framing members and butt along their center lines. Nails shall be placed not less than 3/8 inches from the panel edge, and spaced not more than 6 inches on center along panel edge bearings. Nails shall be firmly driven into the framing members. No unblocked panels less than 12 inches wide shall be used.

Lumber and plywood diaphragms may be used to resist horizontal forces in horizontal and vertical distributing or resisting elements, provided the deflection in the plane of the diaphragm as determined by calculations, test, or analogies drawn there from, does not exceed the permissible deflection of attached distributing or resisting elements.

Permissible deflection shall be that deflection up to which the diaphragm and any attached distributing or resisting element will maintain its structural integrity under assumed force/load conditions (i.e. continue to support design loads without danger to occupants of the structure).

Connections and anchorages capable of resisting the design forces shall be provided between the diaphragms and resisting elements. Openings in diaphragms which materially affect their strength shall be fully detailed on the plans, and shall have their edges adequately reinforced to transfer all shearing stresses. Flanges shall be provided at all boundaries of diaphragms and shear walls.

Additional restrictions are sometimes imposed by local jurisdictions. For example same cities limit the maximum distance between resisting elements of horizontal diaphragms to 200 feet for plywood with blocking, 150 feet for special double diagonal sheathing, 75 feet for plywood without blocking, and 75 feet for diagonal sheathing, unless evidence is submitted and approved by the Superintendent of Building illustrating that no hazard would result from deflections.

11.5.1 Deflections and Deflection Compatibility

Codes do not usually require deflection calculations if diaphragm length-width ratios are Restricted. The Uniform Building Code(11-38) limits these ratios to 4:1 for horizontal diaphragms, and 2:1 for vertical diaphragms.

The deflection formula, taken from Douglas Fir Plywood Association Laboratory Report No. 55 by David Countryman - March 28, 1951, and Published in Uniform Building Code Standards 97,(11-38) is

$$d = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\sum(\Delta_c X)}{2b} + EWD \tag{11-13}$$

Where:

d = mid-span deflection, inches

v = maximum shear, due to design loads in the direction under consideration, lb/per ft.

L = length of diaphragm, feet

E = modulus of elasticity of chords, (Approximately 1,800,000 psi)

A = cross-sectional area of chords, inches^2

b = width of diaphragm, feet

G = Shear modulus, psi (Approximately 90,000 psi)

t = effective thickness of plywood panels for shear,in

e_n = nail deformation/slip inches, see Table 11-6

$\Sigma(\Delta_c X)$ = Sum of Individual chord-splice slip values each multiplied by its distance to nearest support

EWD = End wall deflection

Δ = L/480 = Guideline allowable deflection

The first term represents deflection due to bending, the second term represents deflection due to panel shear, the third term represents the deflection from panel rotation caused by nail deformation/ slippage, the fourth represents

Table 11-*6*. "e_n" values (inches) for use in calculating diaphragm deflection due to nail deformation/slip (structural 1 plywood)[1,2,3,4]

Loads Per Nail (Pounds)	Nail Designation/Size		
	6d	8b	10d
60	0.012	0.008	0.006
80	0.020	0.012	0.010
100	0.030	0.018	0.013
120	0.045	0.023	0.018
140	0.068	0.031	0.023
160	0.102	0.041	0.029
180	----	0.056	0.037
200	----	0.074	0.047
220	----	0.096	0.060
240	----	----	0.077

1 Increase "e_n" values 20 Percent for plywood grades other than STRUCTURAL I.
2 Values apply to common wire nails.
3 Load per nail = maximum shear per foot divided by the number of nails per foot at interior panel edges.
4 Decrease values 50 percent for seasoned lumber.

deflections due to slip in chord splices, and the fifth accounts for end wall deflections.

Example: Calculate the deflection at the center of the long wall of a 200 foot by 400 foot building caused by a seismic force of 800 Plf, assuming all panel edges are blocked.

Thus:

v = 800 PLF(400ft)/2 (200ft) = 800 Plf
L = 400 ft
A = 25 in^2 equivalent area of wood
E = 1,800,000 Psi
b = 200 ft
G = 90,000 Psi
t = 15/32 = 0.4653 in.
e_n = 0.047 For 10d nails @ 3 inch on center (i.e. 200lb/nail)
Δ_c = 1/16 = 0.0625 at each splice (40 ft on center)

Now:

$$d = \frac{5(800PLF)(400ft)^3}{8(1{,}800{,}000PSI)25in^2(200ft)} = 3.56in$$

$$+\frac{800PLF(400ft)}{4(90{,}000PSI)(0.4683in)} = 1.90in$$

$$-0.188(400ft)(0.047) = 3.53in$$

$$+\frac{0.0625(200)+2(0.0625)[40+80+120+160]}{2(200ft)} = 0.16in$$

$$d = 9.15\,inch$$

Recall guideline allowable deflection (Δ)

$$\Delta = L/480 = \frac{400ft(12in/ft)}{480} = 10\,inch$$

Note calculated deflection (d) is less than guideline deflection (Δ).

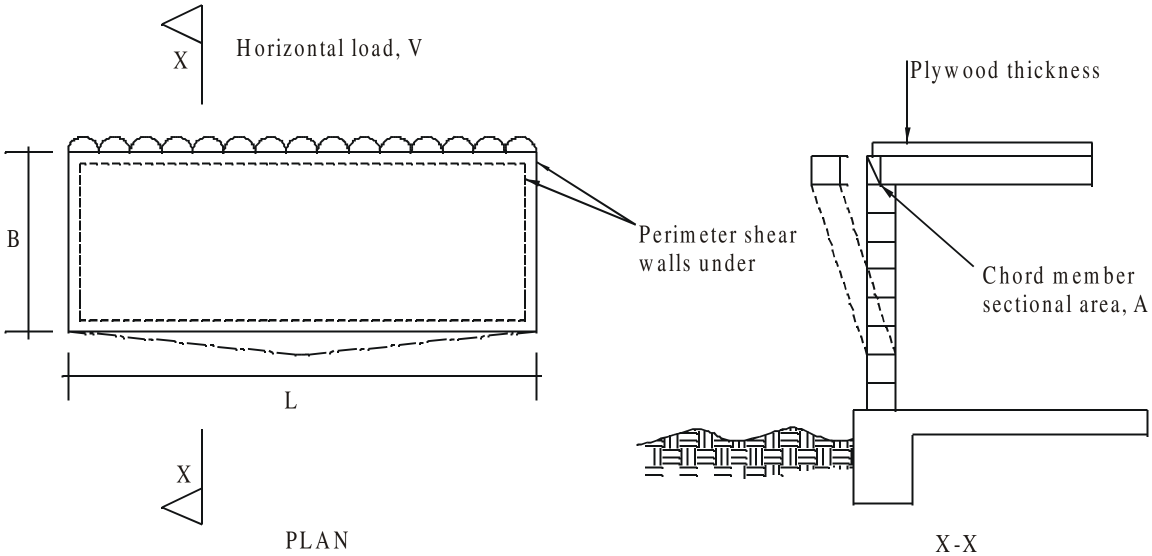
The calculated deflections obtained by the formula conservatively correspond with the results obtained from the full scale 20' x 40' blocked plywood diaphragm tests. The validity of this formula when applied to a span that is 10 times that of the test span is not known. However, the formula does represent the best available means for determining deflections of large spans. It is not applicable to unblocked plywood or diagonal sheathed diaphragm.

The formula for allowable deflection of concrete of masonry walls was developed by the "Horizontal Bracing Systems in Buildings having Masonry or Concrete Walls", Committee of the Structural Engineers Association of Southern California and was published in their Technical Bulletin No. 1, February, 1951. The formula is:

$$d = \frac{75H^2 f_b}{Eb} \tag{11-14}$$

Where:

d = maximum allowable deflection, inches



A typical horizontal timber diaphragm showing the effect on supporting walls of deflections under horizontal loading

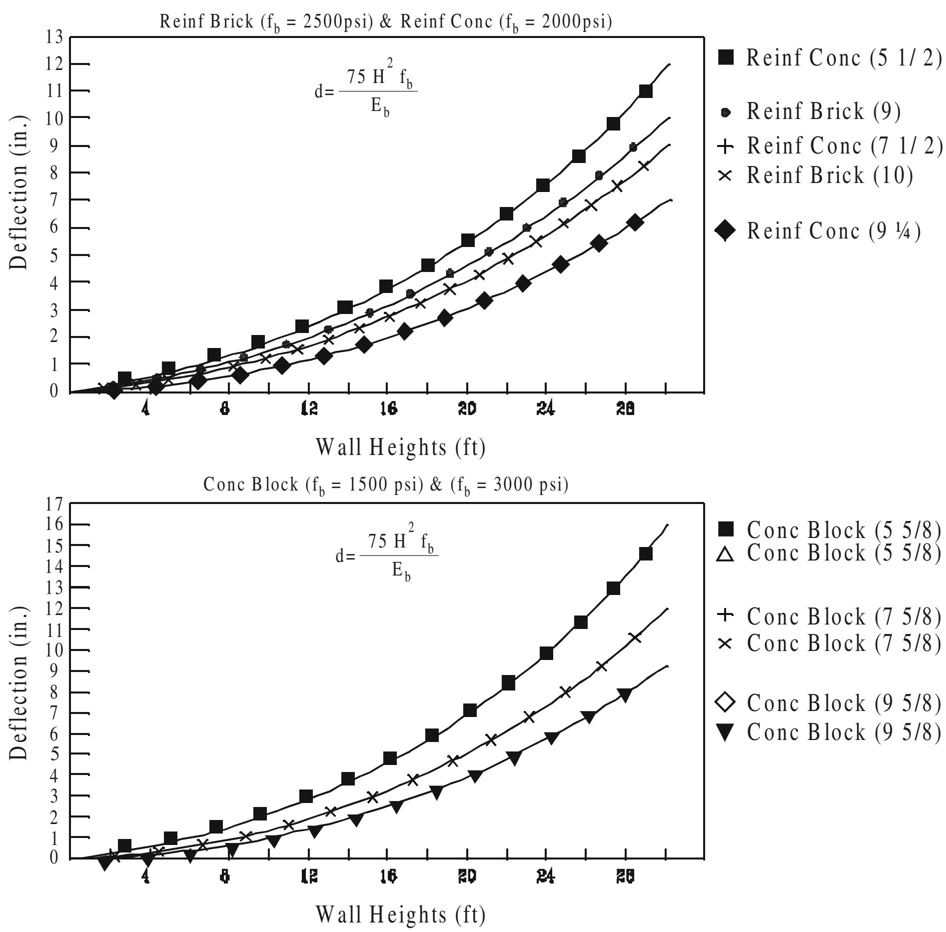


Figure 11-3. Permissive/Allowable Deflection of Concrete and Masonry Walls

H = wall height between horizontal support, feet
f_b = allowable flexural compressive stress in psi
E = modulus of elasticity in psi
b = wall thickness, inches

See Figure 11-3 for plot of above formula and sketch of building and wall deflected shape.

11.5.2 Subdiaphragms

A subdiaphragm is unique to flexible diaphragms. Experience encountered in the San Fernando earthquake of February 1971, revealed that there was a basic weakness present in many of the modern industrial type buildings.

Over the years the practice of installing strap anchors between the walls and wood framing had been for the most part eliminated. The prevalent assumption was that as long as some of the ledger bolts were installed within 3½ to 4 inches of the top of the ledger, the cross grain bending of the ledger would be of a low enough magnitude that it would not result in a failure. This assumption was proven to be incorrect, also a split or crack at the upper ledger bolt might occur simply as a result of shrinkage of an unseasoned member. Especially where two rows of ledger bolts occurred, this split or crack would leave virtually no capacity of cross grain bending. Failures of predominantly tilt-up type buildings occurred at the roof to wall connections in this earthquake.

Much has been said about cross grain bending of wood ledgers which prior to 1972, were utilized for anchoring walls to roof or floor diaphragms. Many of the failures were attributed to cross grain bending, however, many of the failures occurred where the plywood connected to the ledger or in some cases at a point 4 to 8 feet and in some cases 20 feet away from the wall to roof joint. In other words, the wall fell over with a section of the roof still attached, or with the ledger completely attached.

This experience, like previous earthquakes, taught the engineering community an expensive but important lesson in the behavior of structures. It is vital that we look at not just the building design as a whole, but that we must closely examine all the connections in the load path and make sure that they have the capacity to not only support the calculated load safely, but that they also have the reserve capacity to withstand the short term dynamic forces which may be several times the magnitude of the calculated force and where possible exhibit a yielding type failure rather than a brittle type failure.

The design methodology can be described simply as first calculating and designing the vertical load carrying system of the structure, followed by the lateral design for the structure as a whole establishing the diaphragm shears, nailing patterns and zones in the traditional manner. After this is complete, the members are selected for the required continuity ties across the building. For some framing systems the selection is quite obvious, however, for others it requires some judgment or possible investigation of alternate schemes.

The anchorage force shall be determined using the formula:

$$F_p = 4.0\,C_a\,I_p\,W_p \tag{11-15A}$$

Alternatively, F_p may be calculated using the following formula:

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W \tag{11-15B}$$

Except that:
F_p shall not be less than $0.7C_aI_pW_p$ and need not be more than $4C_aI_pW_p$.

Where:
h_x = Element or component attachment elevation with respect to grade. hx shall not be taken less than 0.0.
h_r = Structure roof elevation with respect to grade.
a_p = In structure component amplification factor that varies from 1.0 to 2.5, as set forth in table 16-O

of UBC97[11-38]; except a_p = 1.5 vs 1.0 for anchorage of walls to flexible diaphragms in seismic zones 3 and 4

R_p = Component response modification factor as set forth in table 16-O of UBC97[11-38]; except that:

- R_p = 1.5 for shallow expansion anchor bolts, shallow chemical anchors or shallow cast-in-place anchors. Note shallow anchows are those with an embedment length-to-diameter ratio of less than 8.
- R_p = 3.0 for most other connection with anchor embedment length to diameter ratio equal to or greater than 8.

If the anchors are spaced greater than 4 feet apart, the wall must be designed to span between the anchors. This is generally not a problem for spacing up to 10 feet.

Next, if the members to which the walls are anchored are not continuously tied across the building, the subdiaphragms which carry and distribute these loads to the members and tie across the building, must be selected and analyzed both for shears, and chord forces. Note, the subdiaphragm length to width ratios must meet the 4 to 1 code requirements for plywood diaphragms regardless of the load levels. It is also possible and in some cases desirable to incorporate subdiaphragms into another larger subdiaphragm.

The methodology is probably best understood by the use of design examples.[11-35] The following example problem will present the seismic design for lateral forces including the design of subdiaphragms for a one-story masonry building with a flexible plywood diaphragm.

11.6 EXAMPLE PROBLEM 1 - L-SHAPED BUILDING WITH CMU WALLS

A framing plan for a one story structure is shown on Figure 11-4. The structure is located in Seismic Zone 4. The importance factor is 1.0. Design for seismic forces only, neglect wind forces. Note walls along lines A,E and G contains 50% openings for truck doors which weighs 10 psf.

Required

A) Design the roof diaphragm for N-S lateral forces so as to minimize nailing.

B) Determine the chord forces at grid lines A and E.

C) Design for the critical lateral forces along line E (3 locations). Indicate by detail how to nail, bolt, etc.

D) Design the typical ledger bolting to wall along line A between 7 and 8.

E) Analyze the subdiaphragms so as to minimize the number of cross ties based on the nailing determined in A.

F)Check for flexible versus rigid diaphragm E-W direction only.

11.6.1 Part A

Lateral loads Seismic - Follow UBC 1997

$$V = \frac{C_V I}{RT} W = 0.763W \qquad (11\text{-}10A)$$

$$V = \frac{2.5 C_a I}{R} W = 0.256W\,* \qquad (11\text{-}10B)$$

$$V = 0.11 C_a I W = 0.051W \qquad (11\text{-}10C)$$

$$V = \frac{0.80 Z N_V I}{R} W = 0.90W \qquad (11\text{-}10D)$$

* Governs

Given:

Soil profile type S_D

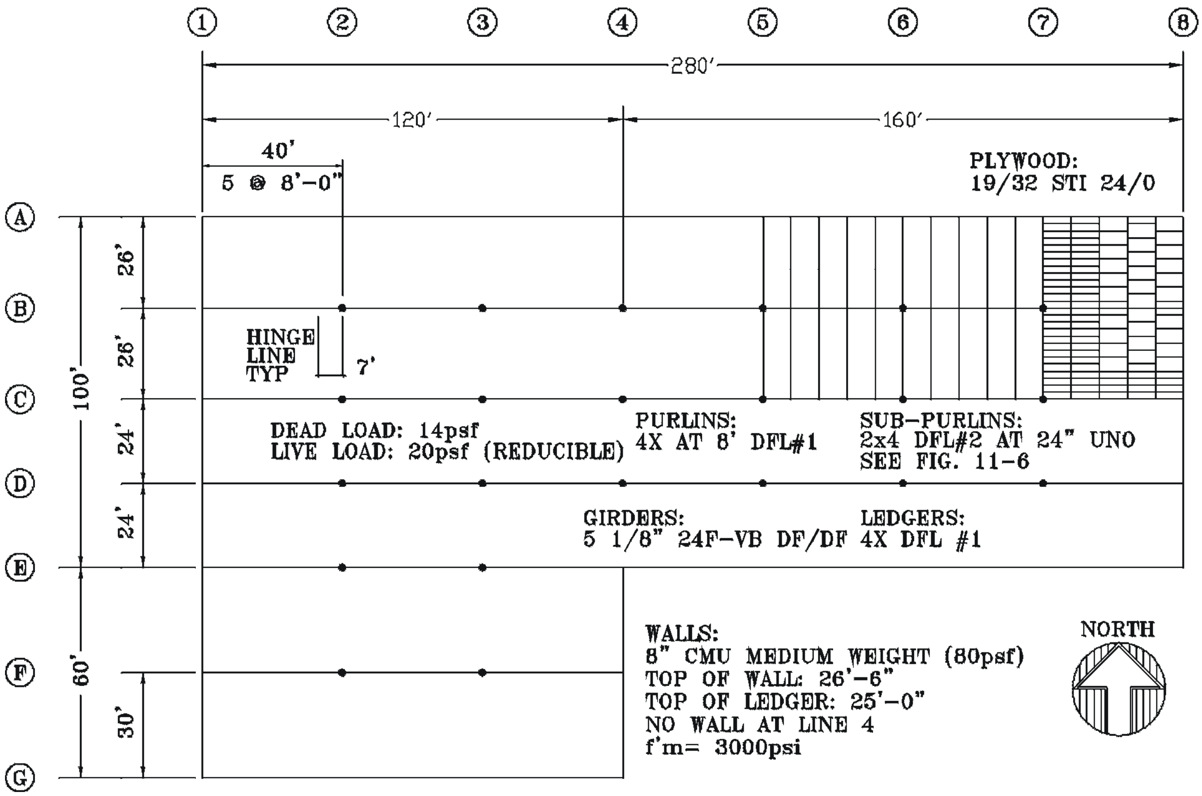


Figure 11-4. Roof Framing Plan

Closest distance to known seismic source = 4.5 km

N_a = 1.05

N_V = 1.27

Seismic Zone 4, Z = 0.40

C_a = 0.44(1.05) = 0.462

C_V = 0.64(1.27) = 0.81

T = $0.020(265)^{3/4}$ = 0.233 SEC

R = 4.5

I = 1.0

Recall that UBC97 is a strength design code, thus to design wood elements using allowable stress design the seismic forces computed from strength design shall be divided by 1.4.

Therefore: for allowable stress design

$$V = \frac{0.256W}{1.4} = 0.183W$$

<u>N-S Loads:</u>

Roof 14 PSF X 100 FT = 1,400 lb/ft
14 PSF X 160 FT = 2,240 lb/ft

$$8 \text{ inch CMU wall} = 80 \text{ psf}\left[\frac{(26.5)^2}{2\times 25}\right]$$

$$=1{,}123.6 \text{ lb/ft}$$

Recall 50% openings for truck doors at walls A,E and G:

Revised wall weight = 1123.6 x 0.50
= 561.8 plf

$$\text{Weight of doors} = 1123.6\left[\frac{10\,psf}{80psf}\right]$$

= <u>140.5 plf</u>

Total effective wall weight = 561.8+140.5=702.3 plf

Therefore:

W_1 = 0.183 [1,400 + 702.3 x 2] = 513 lb/ft

W_2 = 0.183 [2,240 + 702.3 x 2] = 667 lb/ft

ΣW = 513 x 160ft + 667 x 120 ft = 162,120 lb

$\Sigma M_H = 0$

Therefore : (See Figure 11-5)

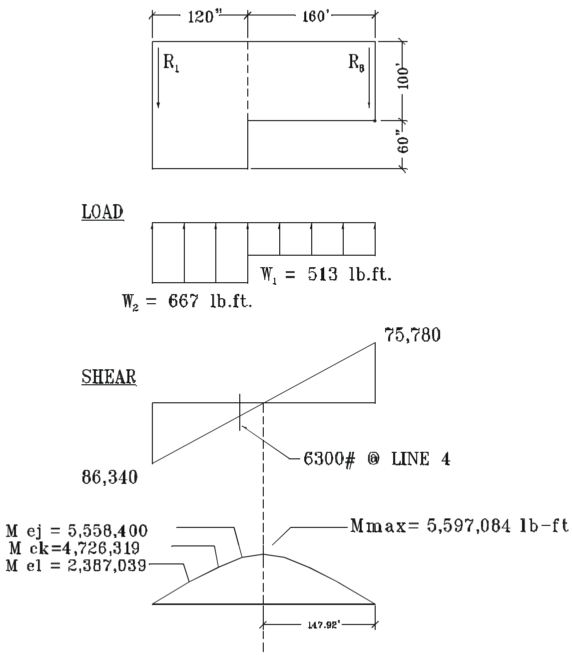


Figure 11-5. Diaphragm Loading

$$R_1 = \frac{1}{280\,ft}\,[(513 \text{ x } 160ft \text{ x } 80ft) + 667 \text{ x } 120 \text{ x } (60ft + 160ft)]$$

= 86,340 lb

R_8 = 162,120 lb - 86,340 lb = 75,780 lb

<u>N - S Roof diaphragm shear : (</u>See Figure 11-6 and Table 11-7)

$$V_{r1} = \frac{86{,}340 lb}{160\,ft} = 539.6\,lb/ft$$ panel type B

$$V_{r8} = \frac{75{,}780\ lb}{100\,ft} = 757.8\,lb/ft$$ panel type A

$$V_{r7} = \frac{75{,}780 - 513\ x\ 40\,ft}{100\,ft} = 552.6\,lb/ft$$ panel type B

$$V_{r3} = \frac{86{,}340 - 667\,x\,80\,ft}{160\,ft} = 206.1\,lb/ft$$ panel type C

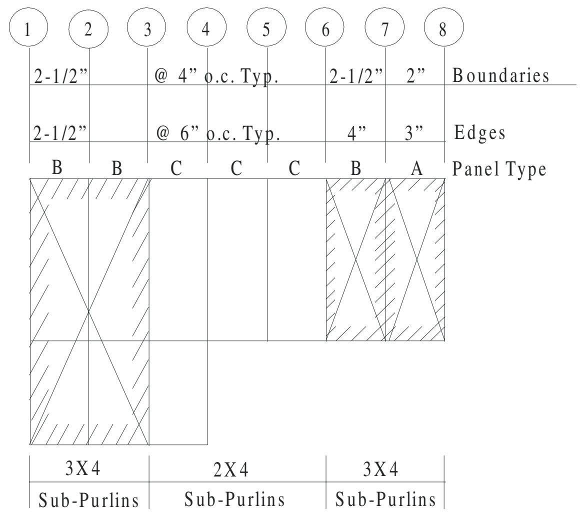


Figure 11-6. NS Loading - Diaphragm Boundaries

Table 11-7. NS Loading - Diaphragm Capacities

Diaphragm Capacity Table				
Type	Bound Nailing	Edge of Nailing	Width of Framing	Capacity Plf
A	2"	3"	3"	820
B	2 ½"	4"	3"	720
C	4"	6"	2"	425
D[1]	2 ½"	4"	2"	640

Ref. UBC 91 table 25-J-1

1. Framing at adjoining panel edge shall be 3-inch <u>nominal in wich with staggered nail spacing.</u>

$$V_{r6} = \frac{75{,}780 - 513\ x80\,ft}{100\,ft} = 347.4\,lb/ft$$ panel type C

Use 19/32 in. plywood str. I All edges blocked

Nailing schedule: Boundary: 10d (see Figure 11-6)

Edges: 10d (see Figure 11-6)

Field: 10d @ 12 ft o.c.

Minimum allowable diaphragm shear = 425 lb/ft (See Table 11-7)

Note: Alternate use of panel type D instead of panel type B would require 3x4 sub purlins at adjoining panel edge versus all 3x4 members as shown.

11.6.2 Part B

Maximum moment in N-S direction:

$$x = \frac{75{,}780}{513} = 147.72\,ft \quad from\, H$$

Therefore,

M_{max} = 75,780 (147.72) – 513 (147.72) (147.72/2) = 5,597,084 lb-ft

$$F = \frac{M}{D} = \frac{5{,}597{,}084}{100\,ft} = 55{,}971\,lb$$

cord stress @ lines A & E

11.6.3 Part C

Consider 3 locations at joints J,K & L on line E, see details on Figure 11-7

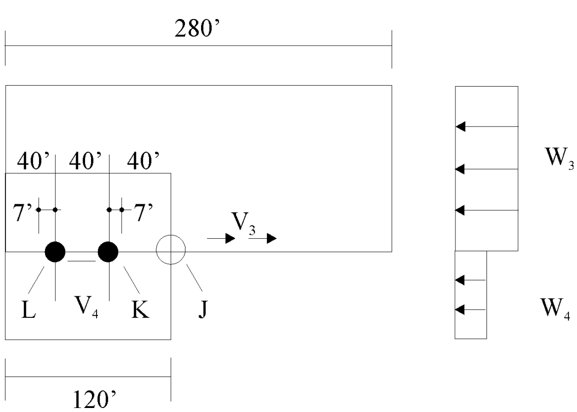


Figure 11-7. Diaphragm Splice Locations

Seismic force in E-W direction: Note to complete the design of joint "J" a similar drag strut connection is required for NS tension, reentrant forces along line 4.

Roof 14 psf x 280 ft = 3920 lb/ft

Roof 14 psf x 120 ft = 1620 lb/ft

$$8\text{ inch CMU wall} = 80\,\text{psf}\left[\frac{(26.5)^2}{2x25}\right] = 1123.6\,lb/ft$$

Therefore

W_3 = 0.183[3920+1123.6 x 2] = 1128.6 lb /ft

W_4 = 0.183[1620 + 1123.6] = 502.1 lb/ ft

$$R_{E1} = R_A = 1128.6\,lb/ft\,\frac{100\,ft}{2} = 56{,}430\,lbs$$

$$R_{E2} = R_G = 502.1\,lb/ft = \frac{60\,ft}{2} = 15{,}063\,lbs$$

$$R_E = R_{E1} + R_{E2} = 71{,}493\,lbs$$

$$V_3 = 1128.6\frac{100\,ft}{2x280\,ft} = 201.5\,lb/ft$$

$$V_4 = 502.1\left[\frac{60\,ft}{2x120\,ft}\right] = 125.5\,lb/ft$$

11.6.3.1 @ joint J(see detail C on Figure 11-8)

$$\text{Chord stress} = [86{,}340 \times 120\text{ ft} - 667 \times \frac{120^2}{2}][\frac{1}{100\,ft}] = 55{,}584\,lb$$

Drag force = (201.5 plf + 125.5 plf)(120 ft) = 39,240 lb < chord stress

Connections

a. To GLB Girder - Design using 1 in. diameter bolts in double shear with 2 bolts in a row (1.25 increase for metal side plates plus 1/3 for seismic). Allowable load parallel to grain for a 1" diameter bolt in a 5 1/8 member:

p = 5070 lbs/bolt.

$$\text{Therefore No. of bolts} = \frac{55{,}584 lbs}{5{,}070 lbs/bolt \, x 1.25 x 1.33} = 6.6$$

Use eight 1 inch diameter A307 bolts 1/4 in. x 18 in. A36 steel plate @ bolt side of beam

$$A_{plate} = \frac{55{,}584 lbs}{22{,}000 psi \times 1.33}$$

$$= 1.90 in.^2 \; required$$

$$A_{provide} = 0.25 in. [18 - 4.2(2)] \times 2$$

$$= 5.00 in^2 > 1.90 in^2 \quad OK$$

<u>b. To concrete wall</u> - design using #8 A706 reinforcing steel ($A_s = 0.79$)

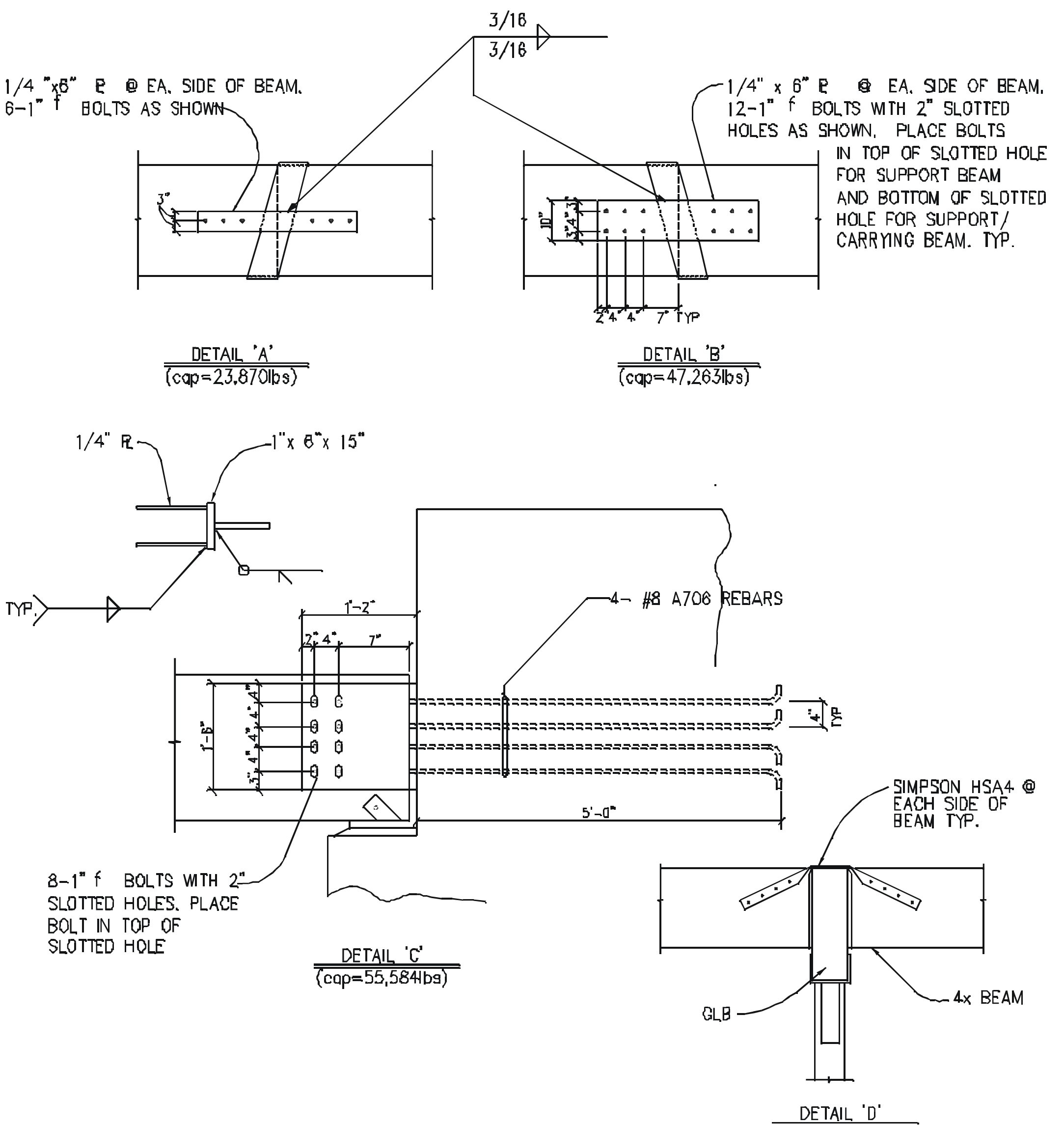


Figure 11-8. Details

Therefore No. of bars

$$= \frac{55{,}584 lbs}{24{,}000 psi \times 1.33 \times 0.79 in.^2} = 2.20$$

Development length $l_d = 0.002\ d_b\ f_s$
$= 0.002(1.0 \text{ in.})(24{,}000 \text{ psi})$
$= 48$ in.

<u>Use:</u> 4 - #8 A706 60 ksi
l = 5'- 0

<u>c. Back Plate</u> (See Figure 11-9)

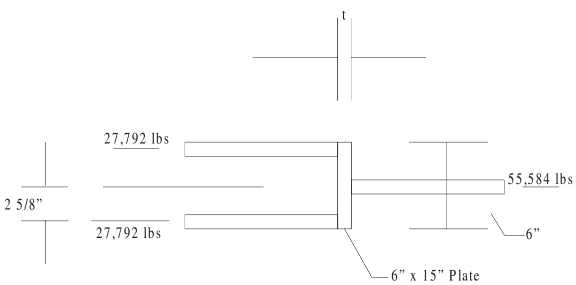


Figure 11-9. Loads on Back Plate - Detail C

Maximum moment on plate

M_e = 27,792 lb x 2.63 in. = 73,093 in.-lb

$$\text{Therefore } t = \left[\frac{6M}{bF}\right]^{\frac{1}{2}}$$

$$= \left[\frac{73{,}093 in\text{-}lb \times 6}{15 in. \times 27{,}000 \times 1.33}\right]^{1/2}$$

$$= 0.902 in.$$

Use 1 in. x 6 in. x 15 in. A36 steel back plate & 1/4 in. x 14 in. x 18 in. A36 steel side plates with eight 1 in. diameter A307 bolts to GLB and four #8 A706 reinforcing steel in CMU wall.

<u>11.6.3.2@ Joint K</u>(see detail B in Figure 11-8)

Chord Stress =

$$= \left[83{,}340 x 87 ft - 667 x \frac{87^2}{2}\right]\left[\frac{1}{100 ft}\right]$$

$$= 47{,}263 lb$$

Drag force = (201.5 + 125.5)(87 ft) = 28,449 lb < Chord Stress

<u>Try</u> 1/4 in. plate @ each side of beam with 1 in. diameter bolts

$$\text{No. of bolts } \frac{47{,}263 lb}{5{,}070 lb/bolt\ x 1.25 \times 1.33} = 5.61$$

$$A_{plate} = \frac{47{,}263 lb}{22{,}000 psi\ x 1.33}$$

$$= 1.62 in^2 \qquad required$$

$A_{provided}$ = 0.25 in [10-2(2)]x 2

= 3.0 in^2 > 1.62 in^2 OK

Use: Six 1 inch diameter A307 bolts 1/4 in. x 10 in. A36 steel plate @ each side of beam

<u>11.6.3.3@Joint L</u>(see detail A in Figure 11-8)

$$\text{Chord Stress} = \left[83{,}349 \times 33 ft - 667 \times \frac{33^2}{2}\right]$$

$$\times \left[\frac{1}{100 ft}\right] = 23{,}870 lb$$

Drag force = (201.5 + 125.5)(33 ft) = 10,791 lb < Chord Stress

Try 1/4 in. plate @ each side of beam with 1 in. diameter bolts

Therefore No. of bolts

$$= \frac{23{,}870 lb}{5{,}070 lb/bolt\ x 1.25\ x 1.33} = 2.83$$

$$A_{pl} = \frac{23{,}870 lb}{22{,}000 psi\ x 1.33} = 0.82 in^2$$

$A_{provide}$ = 0.25 in. [6-2] x 2 = 2.0 in.^{2} > 0.82 in.^{2} OK

Use: Three 1 in. diameter bolts 1/4 in. x 6 in. steel plate @ each side of beam

Notes: 1. Capacity governed by bolts = 3(5070)(1.25)(1.33) = 25,287 lbs

2. Revise to use detail B as required by section 11.6.5, subdiaphragm “Y” below.

11.6.4 Part D

Loads along line A, between 7 and 8
Vertical loads:
w = 14 psf DL+20psf LL(26 ft/2)= 442 lb/ft

Allowable single shear load perpendicular to grain for a 3/4" diameter bolt in a 3 1/2 " member: q = 630 lbs/bolt

Therefore bolt spacing

$$S = \frac{630 lbs/bolt\, x 1.25}{442 lb/ft} x 12 in. = 21.4 inch$$

Use: 3/4 inch diameter A307 anchor bolt with 4 x ledger with spacing of 18 in. o.c.

Therefore: load on bolt = 442 x 1.5 ft. = 663 lb/bolt

Recall: Lateral shear under seismic force:
V_3 = 201.5 lb/ft (see item 11.6.3 above)
Load on bolt = 201.5 x 1.5 ft = 302.3 lb/bolt
Allowable single shear load parallel to rain for a 3/4" diameter bolt in a 3 1/2" member: p = 1400 lb/bolt

Check stress in ledger with Hankinson formula

$$F_n = \frac{F_c F_{c\perp}}{F_c \sin^2\theta + F_{c\perp}\cos^2\theta}$$

Where:

$$\tan\theta = \frac{302.3}{663} = 0.456$$

Therefore:

θ = 24.51 sin θ = 0.415
cos θ = 0.910

Therefore:

$$F_n = \frac{1{,}400\, x\, 630\, x 1.33}{1{,}400\, x (0.415)^2 + 630\, x (0.910)^2}$$
$$= 1537.8 lb/bolt$$

Actual force: (Seismic + Dead Load)

DL = 663 (14/34) = 273.

P = [$(273)^2 + (302.3)^2$]$^{1/2}$
= 407.3 lb/bolt < 1537.8 OK

11.6.5 Part E

Subdiaphragms, see Figures 11-10 and 11-6 for panel types

1. Subdiaphragm "X": (Critical case between line E & G)
(Span: depth) = (30 ft: 8 ft) = (3.75: 1) < 4:1 OK
Lateral force:
Note for center 1/2 of diaphragm F_p = 0.30(1.5) = 0.45 W_p

Recall $F_p = ZIC_pW_p$ = 0.40(1.0)(0.75)W_p = 0.30 W_p(1.5) = 0.45 W_p

Wall: $80 \left[\frac{(26.5)^2}{2x25 ft}\right] 0.45$ = 505.62 lb/ft > 200 lb/ft

Design wall anchors @ 4 ft o.c. (check for one Bay only)

$$V_x = \frac{505.62\, x\, 30 ft}{2\, x\, 8 ft}$$
= 948 lb/ft>(720 lb/ft panel type B) NG

Expand subdiaphragm to 2 bays use continuity ties at each 2 x 4 at 2'-0 o/c similar to detail D, Figure 11-8.

$$V_x = \frac{505.62(30\,ft)}{2(16)} = 474\,lb/ft$$
$$< (720\,lb/ft \; panel\,type\,B) \quad OK$$

Chord load = $\dfrac{505.62\,plf\;x30^2\;ft^2}{8\,x16\,ft}$ = 3555

Required As = $\dfrac{3500\,lbs}{24000\,psi}$ = 0.146

Two #4 in CMU wall (As = 0.40) .....OK

Note: Purlins at first line from lines 1 and 8 require investigation for combined flexural and axial stresses due to dead loads plus chord forces.

Subdiaphragm "Y" Boundaries (5, 6, D & E)

(Span: depth) = (40 ft: 24 ft) = (1.67: 1) < 4:1 OK

Wall line E: = 505.62 lb/ft

$V_y = \dfrac{505.62\,x\,40\,ft}{2\,x\,24\,ft}$ = 421.4 lb/ft < 425 lb/ft

(panel type C) OK

Chord = $\dfrac{505.62\,x\,40^2}{8(40)}$ = 2528.1 lb at midspan

for girder on line - D and wall line - E

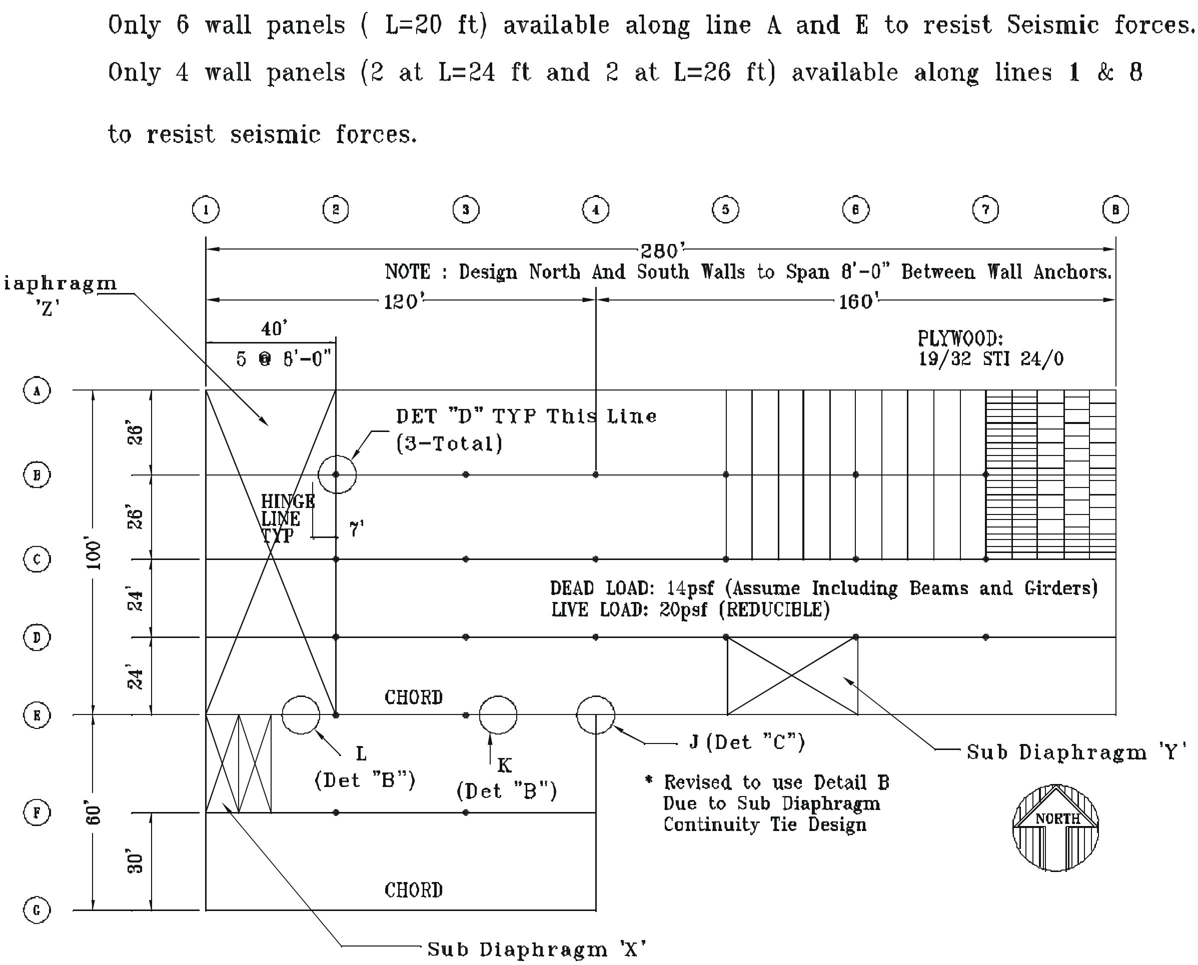


Figure 11-10. Subdiaphragms

<u>Chord load @ girder support joint</u> (7 ft from column)

$V_y = 505.62 \text{ x } \dfrac{40\,ft}{2} = 10{,}112$ lb

Therefore:

M= 10,112 lb x 7 ft - 505.62 plf ($7^2/2$)
= 58,399 ft-lb

Therefore:

Chord load = $\dfrac{58{,}399\,ft-lb}{24\,ft}$ = 2433 lbs

Use: Simpson hinge connector HC3T,similar to Detail A Figure 11-8. Typ. @ all GLB to GLB connections

<u>Girder tie across line D:</u>

505.62 plf x 40 ft = 20,225 lb

Use: Simpson strap connectors HSA68 @ each side of beam, similar to Detail D, Figure 11-8

Capacity = 2 x 11,000 lb= 22,000 lb

Typ. over all columns
See detail "D", Figure 11-8

<u>Subdiaphragm "Z":</u> @ boundaries (1, 2, A & E or 7, 8, A & E)

Span: depth = 100 ft = (2.5:1) < 4:1 OK

Wall load @ line 1 = 505.62 lb/ft

$V_y = \dfrac{505.62\,plf\ x100\,ft}{2(40\,ft)}$ = 632 lb/ft < 720 lb/ft panel type B OK

Chord = $505.62 \text{ x } \dfrac{100^2\ ft^2}{8\,x\,40\,ft}$
= 15,801 lb < 22,000 lb

(See girder tie across line D, above)

<u>Drag force at line E for subdiaphragm:</u>

From Z: 505.62plf x $\dfrac{100\,ft}{2}$ = 25,281 lb

From X: 505.62plf x $\dfrac{30\,ft}{2}$ = 7,584 lb

Total = 25,281 lb+ 7,584 lb = 32,865 lb

Recall: Capacity @ L = 25,287 < 32,865 NG
Use: Detail B @ Joint L (Revise from section 11.6.3.3 above)

11.6.6 Part F

<u>Check for flexible versus rigid diaphragms EW dir. only</u>

Recall: d = mid-span deflections of diaphragm

$= \dfrac{5VL^3}{8EAb} + \dfrac{VL}{4Gt} + 0.188Le_n$ + chord splice slip (css)

In the E-W direction between grid lines A + E:

V_3 = 201.5 lb/ft
E = 29,000,000 psi for chord steel
A = 0.40 in.^{2} (2 - #4 bars for chord steel)
L = 100 ft
b = 280 ft
G = 90,000 psi for plywood
t = 19/32 = 0.593
e_n = 0.029 (based on 160 lb/ft and 10d nails)
css = Zero . Bar elongation at splices is negligible for these loads
Δ = Guideline allowable deflection
= L/480 = 100(ft)x12(in./ft)/480=2.5 in.

Now:

$$d = \frac{5(201.5)(100)^3}{8(29{,}000{,}000)(0.40)(280)} + \frac{201.5(100)}{4(90{,}000)(0.593)} + 0.188(100)(.029)$$
$$= 0.0387 + 0.094 + 0.545 = 0.678\,in.$$

Δ_A = Deflection of wall on line - A (see Table 11-4)

$$= \frac{P}{Et}\left[4\left(\frac{h}{d}\right)^3 + 3\left(\frac{h}{d}\right)\right]$$

Where:

P = R_A = 56,430 lbs total (Ref. Section 11.6.3)
P = P per panel = P/6 = 56,430/6 = 9405 lbs
f'_m = 3000 psi
E = 750 f_m' = 2,250,000 psi
t = 8 in
h = 25 ft (top of ledger)
d = 20 ft

Now:

$$\Delta_A = \frac{9405\,\text{lb}}{2{,}250{,}000\,\text{psi} \times 7.625\,\text{in}}\left[4\left(\frac{25}{20}\right)^3 + 3\left(\frac{25}{20}\right)\right]$$
$$= 0.0064$$

For wall on line E we have:

P= R_E = 71,493 lb (Ref. Section 11.6.3)
P= P per panel = 71,493/6 = 11,915 lb

$$\Delta_E = \frac{11915\,\text{lb}}{2{,}250{,}000\,\text{psi} \times 7.625\,\text{in}}\left[4\left(\frac{25}{20}\right)^3 + 3\left(\frac{25}{20}\right)\right]$$
$$= 0.008$$

Thus the average story drift = (0.0064 + 0.0080)/2 = 0.0072 in

Recall for flexible diaphragm behavior deflection of the diaphragm must be more than 2 times the average story drift:

$$0.678 > 2(0.0072)$$

Thus the E-W diaphragm is a flexible diaphragm and will behave consistent with the analysis presented herein.

From the above analysis and similar calculations it can be shown that most one story industrial/warehouse buildings with wood diaphragm and concrete or CMU walls will qualify as flexible diaphragms. It can also be shown that one to three story apartment or office buildings with light weight concrete topping slab over a wood diaphragm and wood shear walls may very well qualify as a rigid diaphragm in one or both directions.

PLYWOOD SHEAR WALLS

Vertical diaphragms sheathed with plywood (plywood shear wall) may be used to resist horizontal forces not exceeding the values set forth in the code. Plywood shear walls are designed as a dual system; the overturning forces (compression/tension) are resisted by the boundary members while the shear forces are resisted by the web or plywood. As part of the consideration given to the design for uplift caused by seismic loads, the dead load shall be multiplied by 0.90 when used to reduce uplift. This criteria is required for materials which use working stress procedures and is intended to account for variations in dead load and the vertical component of an earthquake.

The deflection (d) of a blocked plywood shear wall uniformly nailed throughout may be calculated by use of the following formula:[(11-38)]

$$d = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + \frac{h}{b}d_a \qquad (11\text{-}16)$$

Where:
d= the calculated defection, in inches.
v = maximum shear due to design loads at the top of the wall, in pounds per lineal foot.
A = area of boundary element cross section in square inches (vertical member at shear wall boundary)
h = wall height, in feet.
b = wall width, in feet.
d_a = deflection due to vertical displacement at anchorage details including slip in holddown, bolt elongation and crushing of sill plate.
E = elastic modulus of boundary element (vertical lateral force resisting member at

shear wall boundary), in pounds per square inch (approximately 1,800,000 psi).

G = modulus of rigidity of plywood, in pounds per square inch (approximately 90 x 10^3 ksi)

t = effective thickness of plywood for shear,in inches

e_n = nail deformation/slip, in inches (see Table 11-6).

Δ = Allowable story drift = 0.005h for allowable stress loads.

For a typical plywood shear wall constructed of structural I plywood on 2 x 4 studs spaced at 16 inches on center with 4 x 4 boundary elements:

V = 500 plf
A = 12.25 in^2
h = 8'-0
b = 10'-0
d_a = 1/8 inch = 0.125 inch
E = 1.8 x 10^6 psi
G = 90 x 10^3 psi
t = 15/32 in
e_n = 0.036

where:

10d nails at 4 inch on center load/nail
= 500 plf (4/12) ft/nail
= 167 lb/nail.

Thus:

$$d = \frac{8(500)(8)^3}{1.8x10^6(12.25)(10)} + \frac{500(8)}{90x10^3\left(15/32\right)} + 0.75(8)(0.036) + 0$$

$$= 0.009 + 0.089 + 0.216 + 0.125\left(\frac{8}{10}\right)$$

$$= 0.0414$$

$$\Delta = 0.005(8ft)(12in/ft) = 0.48in. > 0.414 \quad OK.$$

More important than the magnitude of the displacement is the contributions of the components. The flexural component is negligible while the shear and nail deformation/ slip components are the dominate contributions. An evaluation of the deflection is that loads can be distributed to a series of wood shear walls based upon only the length of each wall when using the same plywood and nailing for walls of equal height.

Two example problems are presented. The first example problem presents a design procedure for an isolated plywood shear wall. The second example problem presents a design procedure for distribution of lateral seismic forces to a series of plywood shear walls.

EXAMPLE PROBLEM 5 - ISOLATED PLYWOOD SHEAR WALL

Isolated plywood shear wall is shown in Figure 11-11. Determine if the plywood shear wall is adequate.

Note: All shear in plywood web; all overturning moment loads in columns (boundary elements)

$$\text{Shear} = \frac{2400lb}{4ft} = 600 \text{ lbs/ft}$$

Use 15/32" plywood Structure I

Perimeter nails = 10d @ 3" with 1 5/8" penetration o/c for each panel edge
Field nails = 10d @ 12 in. o/c
4 x 4 post = boundary elements
Allowable shear = 665 lbs/ft > 600 lbs/ ft OK

Check Bolts

Use 3/4 in. diameter at sill plate bolts (P = 1420 lbs for single shear in wood).

$$\text{No. required} = \frac{2400lbs}{1420lb/bolt\,x1.33} = 1.27$$

Use 2 bolts

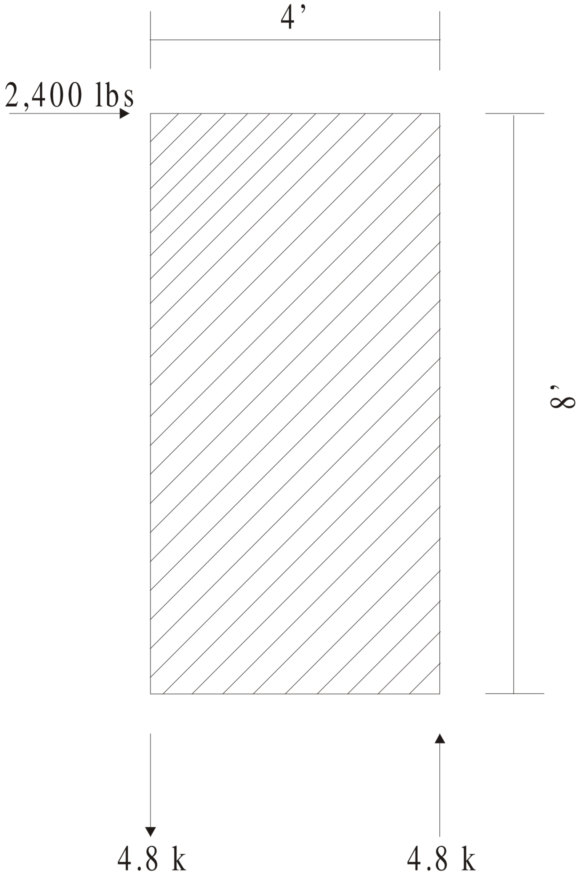


Figure 11-11. Isolated Plywood Shear Wall

$$\text{Overturning} = \frac{2400 lbs \, x \, 8 ft}{4 ft} = 4800 \text{ lbs}$$

Compression perpendicular to grain in sill plate: = 4800 lb/(3.5 in.)^2 = 392 psi < 625 psi OK

3/4" anchor bolt OK for 0.3 x 20 = 6 kips
Connection must resist 4.8 kips pull out
OK

Note:Net area of 3/4" dia. anchor bolt is 0.30 in^2. with an allowable tension of 20 ksi.

Pull out of concrete for 3/4" ϕ:

F = 2.25(2)(1.33) = 6 kips; with special inspection and 1/3 seismic increase.

F = 6.0 > 4.8 OK

Check End Stud
Check End Post for Compression
Recall: $F_c' = F_c * C_p$
Now:

$$Cp = \frac{1 + F_{CE}/F_C *}{2c} - \left[\left[\frac{1 + F_{CE}/F_C *}{2c}\right]^2 - \frac{F_{CE}/F_C *}{c}\right]^{1/2}$$

Where:

$$F_{CE} = \frac{K_{CE} E'}{(le/d)^2} = 638$$

K_{CE} = .30 for visually graded lumber
l_e = 96 in
d = 3.5 in
E = 1.6 x 10^6 PSI
C = 0.80 For Sawn Lumber
F_C* = $F_C \, C_F \, C_R \, C_D \, C_N \, C_t$ = 1596 PSI
Where: F_C = 1200 PSI
C_F = 1.0
C_R = 1.0
C_D = 1.33
C_M = 1.0
C_t = 1.0
Thus:

$$Cp = \frac{1 + 638/1596}{2(0.8)} - \left[\left[\frac{1 + 638/1596}{2(0.8)}\right]^2 - \frac{638/1596}{0.8}\right]^{1/2}$$

$= 0.262$

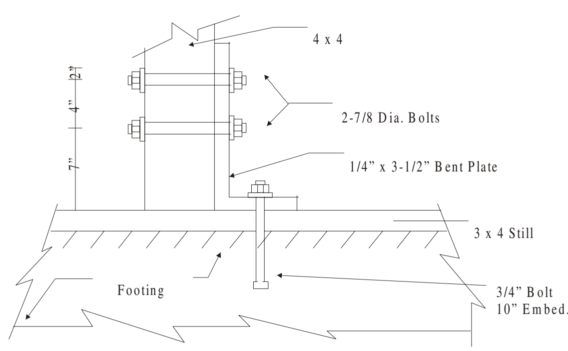


Figure 11-12. Shear Wall Post Connections

Therefore:

$F_c' = 1596\ (0.262) = 418$ psi

Now:

$F_h = P/A = 4800$lbs/12.25 inc
$= 392$ psi < 418 OK.

Bolts to 4 x 4 post: (See Figure 11-12)

$V_{allow} = 2(1790$ lbs/bolt$)(1.25)(1.33)$
$= 5.95$ kips > 4.8 kips...OK

where

1.25 = increase for metal side plates
1.33 = increase for seismic (short term) force

Check Deflection:

V = 600 plf
A = 12.25 in^2
h = 8.0 ft
b = 4.0 ft
da = 0.1 in.
E = 1.6×10^6
G = 90×10^3
t = 15/42 in.
$e_n = 0.029$

$$d = \frac{8(600)(8)^3}{1.6{\times}10^6\,(12.25)(4)} + \frac{600(8)}{90{\times}10^3\,(15/32)} +$$
$$+0.75(8)(0.029)+0.1$$
$$=0.0313+0.107+0.174+0.10$$
$$=0.412\,inch.$$

$$\Delta = 0.005h = 0.005\,(8\,ft)(12\,in\,/\,ft)$$
$$=0.48\,in. > 0.412\,in. \qquad OK.$$

Note that deflection/stiffness criteria will govern on short plywood walls with high shear load.

EXAMPLE PROBLEM 6 - DISTRIBUTION OF LATERAL SEISMIC FORCES TO A SERIES OF PLYWOOD SHEAR WALLS

Determine the distribution of lateral seismic force to series of plywood shear walls shown in Figure 11-13.

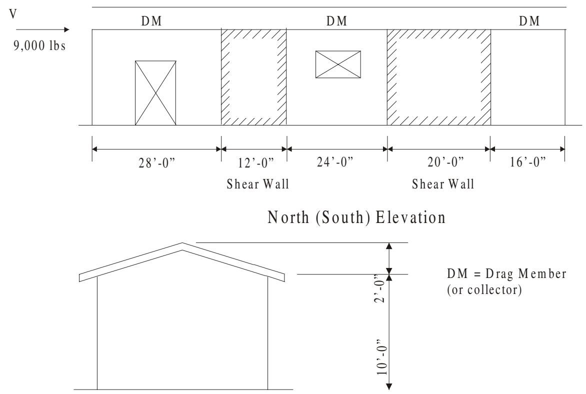


Figure 11-13. Building Elevations

Load to walls

Total length of walls = 12 + 20 = 32 ft
Load per foot of wall = 9000 lbs/32 ft=281.25 plf
Load to 12 ft wall = 281.25 plf (12) = 3375 lbs
Load to 20 ft wall = 281.25 plf (20) = <u>5625 lbs</u>
Total = 9000 lbs

Load to drag struts/collectors

q = load per foot at collector
= 9000 lbs/ft
= 90 plf

Force diaphragm of collector/shear wall load

Thus:(See Figure 11-14)

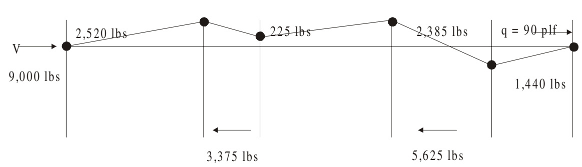


Figure 11-14. Collector/Drag Force Diagram

Drag strut at b: F = 2520 lbs compression
Drag strut at c: F = 225 lbs compression
Drag strut at d: F = 2385 lbs compression
Drag strut at e: F = 1440 lbs tension

11.7 CMU SLENDER WALL (OUT-OF-PLANE FORCES)

The design of masonary walls can be divided into two separate procedures. The first procedure is the design of the wall for out-of-plane forces (forces perpendicular to the face of the wall). Walls designed using WSD are limited to an h'/b ratio of 30; where h' is the effective wall height and b is the effective wall thickness. Walls designed using LRFD are really slender walls and are not limited to an h'/ b of 30 but must comply with srict reinforcement criteria and have special inspection. Walls designed as slender walls are becoming more prevalent and will be discussed in detail in the following chapter.

The second procedure is the design of the wall for in-plane forces (forces parallel to the length of the wall). Walls designed using WSD usually require a concentration of bars at the extreme ends of the wall to resist flexure stresses and overturning forces; and shear forces are carried either by the masonry or the steel. Walls designed using LRFD are called limit state or strength design shear walls and are allowed to account for the distributed vertical wall steel to resist flexure stresses and overturning forces; shear strength is proportioned to both the masonry and the steel. Strength design shear walls are a relatively new concept and will be discussed in detail following the section on slender walls.

Manual calculations are presented to demonstrate the procedure, but as the reader will quickly realize that for production design a computer software program is mandatory. A computer software program has been developed for both the slender wall computations and the shear wall computation and is available from the concrete masonry association of California and Nevada.[11-33]

11.7.1 Interaction Diagram

The appropriate method to model the capacity of a member subjected to both bending and axial loads is an interaction approach which accounts for the relationship between the stresses caused by bending and axial loads. An "Interaction Diagram", such as that shown in Figure 11-15, may be constructed by establishing the capacity of the member under various combinations of axial and flexural loads. Although an infinite number of points may be calculated, the critical points identified by numbers 1 through 6 on Figure 11-15 should be more than sufficient to construct an accurate interaction diagram. Each point is described by the axial capacity P_n and moment capacity M_n. Thus, M_n can be computed for a given P_n, or vice versa.

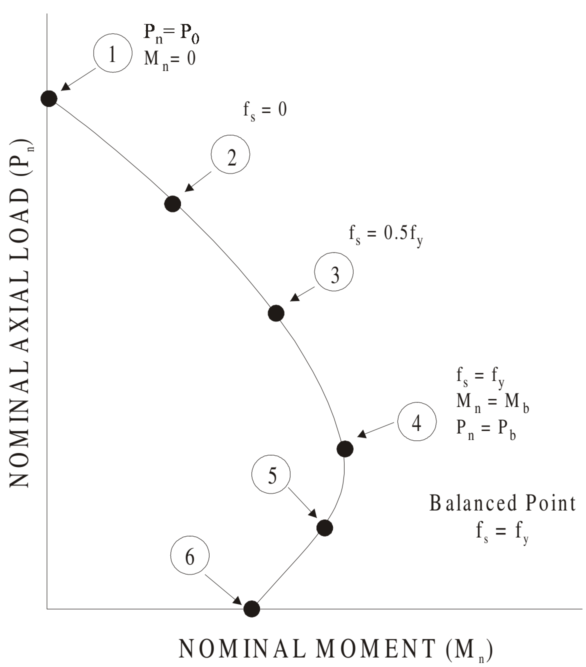


Figure 11-15. Interaction diagram for an eccentrically loaded member

For example, at one extreme, point 1, where no externally applied moment is imposed on the wall the nominal axial capacity of the wall, is:[11-20]

$$P_n = 0.85f_m'(A_n - A_s) + A_sF_y \qquad (11\text{-}17)$$

The other extreme, point 6, is where the capacity of the member is the pure bending nominal flexural capacity of the wall, or:

$$M_n = 0.85f_m'ab[d-(a/2)] \qquad (11\text{-}18)$$

The intermediate points may be established by choosing several condition of strain and, using the force-equilibrium and stress-strain relationships developed in Reference 11-16 for calculating P_n and M_n.

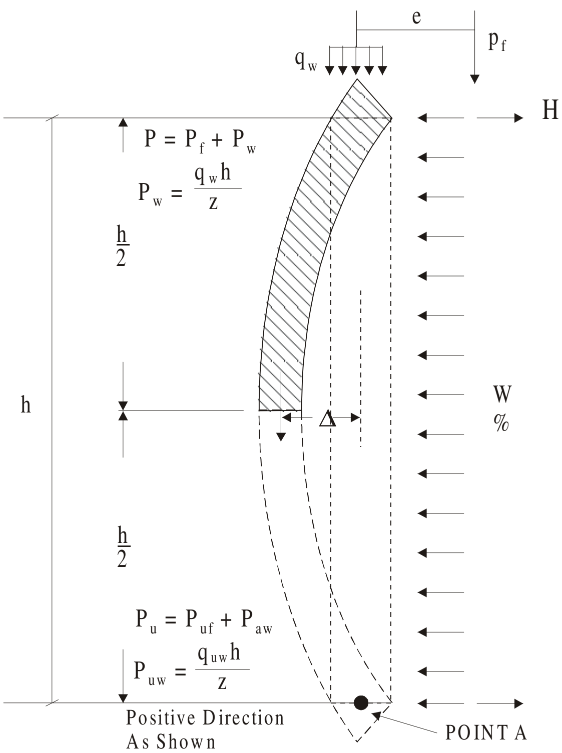


Figure 11-16. Loading geometry of slender wall

11.7.2 Structural Mechanics

The load-induced moment on a wall is a function of lateral wall deflection. If the wall is slender, usually a wall with height to thickness of 25 or more, herein referred to as a "Slender Wall", the lateral deflection can produce moments that are significant relative to the moment obtained using small deflection theory.

Figure 11-16 shows the forces acting on a slender wall with a pin connection at each end. The summation of moments about the bottom of the wall, point A, gives the equation for the horizontal force at the upper wall support. That is:

$$P_f e + Hh - w(h^2/2) - q_w h(\Delta_a) = 0 \qquad (11\text{-}19)$$

where

P = Design axial load = $P_f + P_w$
P_f = vertical load on wall per linear foot
e = eccentricity of vertical load
w = uniform lateral load on wall per linear foot
$P_w = q_w h/2$
q_w = weight of wall per linear foot
Δ_a = "effective" lateral deflection used to estimate dead load moment

If we assume that

$$\Delta_a = 2\Delta/3 \qquad (11\text{-}20)$$

where Δ is the wall's mid-height lateral deflection, then

$$H = wh/2 - P_f e/h + 2q_w\Delta/3 \qquad (11\text{-}21)$$

The first term corresponds to the classical small deflection reaction, the second term represents the change in the magnitude of the force due to an eccentric wall loading, and the third term incorporates the lateral wall deflection.

If we take the moment about the mid-height of the wall, the moment induced on the cross section from the external loads is

$$M = H(h/2) + P_f(\Delta + e) + (q_w h/2)\Delta_b - (wh/2)\,h/4 \qquad (11\text{-}22)$$

where Δ_b is the "effective" lateral deflection used to estimate dead load moment. If we assume that

$$\Delta_b = \Delta/3 \qquad (11\text{-}23)$$

which is consistent with Δ_a above and substitute H into the moment equation, it follows that

$$M = wh^2/8 + P_f e/2 + (P_f + q_w h/2)\Delta \qquad (11\text{-}24)$$

The first term corresponds to the moment due to the classical small deflection moment from the uniform lateral load, the second term corresponds to the moment due to the eccentric vertical load on the wall, and the third term represents the moment due to large lateral deflections. This last term can be referred to as the P-Delta load.

The moment M and lateral force H are a function of Δ, which in turn is a function of the wall's cross-sectional properties and steel reinforcement as well as the moment M and the lateral load H. Therefore, the problem of calculating the moment M is iterative.

The ultimate axial load computed using the factored axial forces must be less than the evaluated nominal capacity:

$$\phi P_n \geq P_u \qquad (11\text{-}25)$$

The slender wall must have a capacity equal to the sum of the superimposed factored axial dead and live loads, P_{uf}, factored wall dead load for the upper one-half, $q_u wH/2$, along with the factored lateral load from the wall and/or loading above (see Figure 11-16). The moment capacity of a wall section is calculated, assuming that axial strength does not govern the design, and it is checked against the moment generated under the applied lateral load and by the P-Delta effect.

Although most walls are loaded at a level which is considerably less than their axial load strength, a check can be made to determine if flexure controls the design, that is,

$$\phi P_b \geq P_u \qquad (11\text{-}26)$$

in which

$$P_b = 0.85f'_m b a_b - \Sigma A_s f_y$$

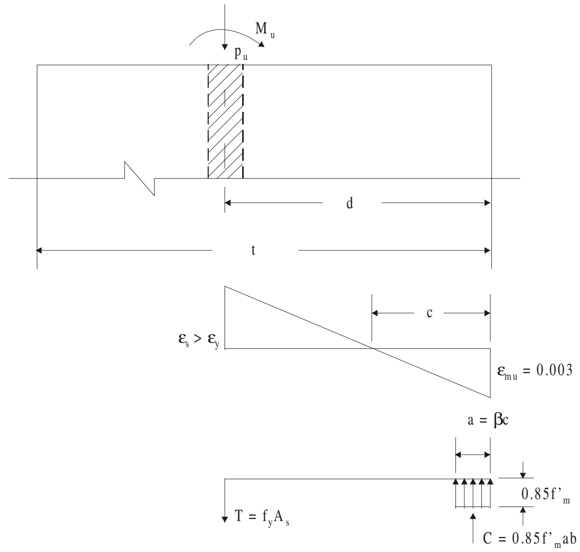


Figure 11-17. Stress and strain diagrams for steel at center of wall

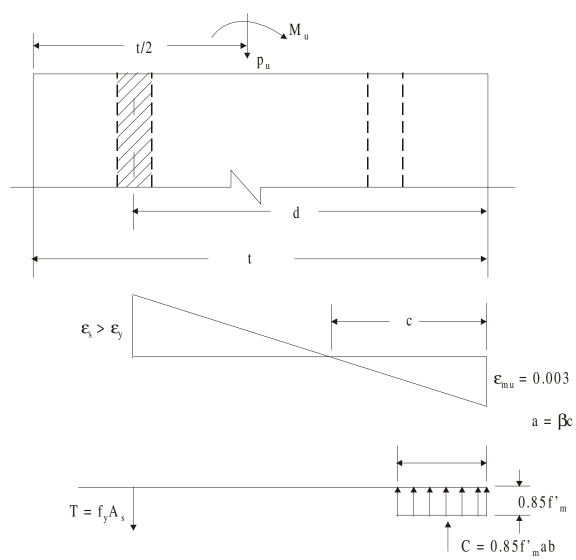


Figure 11-18. Stress and strain diagramss for steel at two faces (ignoring compression steel)

where

$$a_b = \left(\frac{87{,}000}{87{,}000 + f_y}\right)\beta d$$

The nominal moment capacity of the wall section loaded with a concentrically applied load may be determined from force and moment equilibrium (see Figures 11-17 and 11-18). The axial load is

$P_u = C - T$

Thus:

$C = P_u + T$

$$0.85f'_m ba = P_u + A_s f_y \qquad (11\text{-}27)$$

and solving for "a" yields

$$a = (P_u + A_s f_y)/(0.85f'_m b) \qquad (11\text{-}28)$$

Summing the internal and external moments about the tension steel yields

$M_u + P_u(d - t/2) - C(d - a/2) = 0$

Substituting Equation 11-27 for C, and assuming $M_n = M_u$, the nominal moment capacity of a member with steel at two faces (Figure 11-18) is

$$M_n = (P_n + A_s f_y)(d - a/2) - P_n(d - t/2) \qquad (11\text{-}29)$$

In the more typical case with steel in one layer of reinforcement at the centerline of the wall (Figure 11-17), the nominal moment capacity is

$$M_n = (P_n + A_s f_y)(d - a/2) \qquad (11\text{-}30)$$

If the imposed moment, M_u, is less than the reduced moment capacity, ϕM_n, the wall section

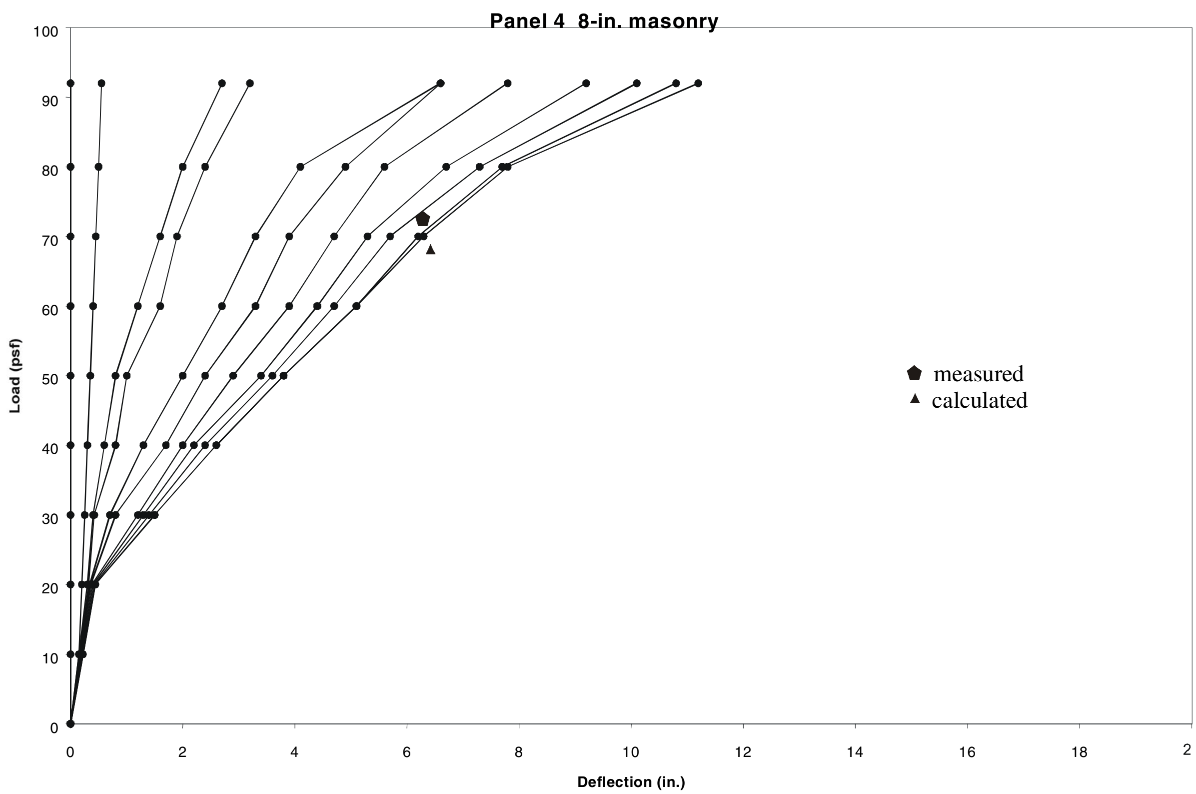


Figure 11-19. Load deflection curves (slender walls)

is acceptable.

$$\phi M_n \geq M_u \qquad (11\text{-}31)$$

This may be determined by comparing Equation 11-24 with Equation 11-29 or 11-30, multiplied by the appropriate ϕ factor.

In 1981, the Structural Engineers Association of Southern California (SEAOSC) tested 32 slender concrete, brick, and concrete masonry panels subjected to a constant axial and increasing lateral load[11-17]. Panel capacities were predicted using the strength method developed by SEAOSC. The procedure for calculating ultimate moments and deflections is presented in Equations 11-30 and 11-31. Load deflection results of these tests for eight inches thick concrete masonry walls are presented in Figure 11-19. A close correlation was obtained between calculations and test data.

11.7.3 LRFD/Limit-State Design Criteria

The Limit State design procedure concerns reinforced hollow unit concrete masonry slender walls subjected to vertical and horizontal forces causing out-of-plane flexure.

<u>A. Conditions for the design procedure:</u>

1. The minimum nominal thickness of the masonry wall shall be six inches. Note : eight inch minimum wall is recommended.
2. The ratio of unsupported height to nominal wall thickness may not exceed 30 unless the axial stress at the location of maximum moment is equal to or less than 0.04 f'_m.(Same as concrete)
3. Minimum reinforcement ratio shall be 0.0007 in either direction and 0.002 total.(Title #4 requires a minimum of 0.003)
4. Maximum reinforcement shall not exceed 50 percent of the balanced steel ratio, ρ_b. Maximum steel in each cell shall not exceed 0.03 times the cell area unless the reinforcing steel is lap spliced and then it is 0.06 times the cell area. (see Table 11-8). Note: $\rho < 0.6\, \rho_b$ for concrete
5. The principal wall reinforcement in the direction of span shall not be spliced within the middle third of the span.
6. All units shall be laid in running bond unless the wall is grouted solid. Note that running bond and solid grouting are recommended.
7. Masonry walls at corners and intersecting cross walls shall be effectively anchored to each other or separated to prevent seismic batter.
8. All grouts shall have a minimum compressive strength, f_C', not less than of 2000 psi nor greater than 4,000 psi. f_c' shall be determined by prism tests. f_c' shall be greater than f_m'
9. All grouts shall be consolidated by mechanically vibrating over the height of pour (vibration shall be performed after the initial loss of water and before initial set). Grout space shall be not less than the minimum necessary for mechanical vibration.
10. The specified compressive strength, f_m', shall not be less than 1,500 psi nor greater than 3,000 psi. f_m' shall be determined by prism tests.
11. An inspector shall provide continuous inspection during all key phases of wall construction as identified on the structural plans.

<u>Design Procedures:</u>

Design of hollow unit reinforced concrete masonry shall be based on forces and moments determined from analysis. The analysis that considers slenderness of walls by representing effects of axial load and deflection in the calculation of required moments must be used. This design procedure must satisfy both strength and deflection limit states. The slender wall design procedures given herein shall be used when the ratio of unsupported height to nominal wall thickness is equal to or greater than 30 and when the vertical load stress at the location of the maximum moment does not exceed 0.04 f_m'.

$(P_w + P_f)/A_g \le 0.04\ f_m'$ (11-32)

where

P_f = Unfactored axial load from tributary floor and/or roof area, pounds.

P_w = Unfactored weight of the wall tributary to section under consideration, pounds.

f_m' = Specified compressive strength psi.

A_g = Gross area of wall, square inches.

Recall for working stress designs of CMU walls:

$f_a = 0.20\ f'_m\ [1 - (h'/42b)^3]$
$= 0.20\ f'_m\ [1 - (30/42)^3]$
$= 0.127\ f'_m$ @ $(h'/b)max = 30$
$= 0.04\ f_m'$ @ $(h'/b) = 39$ aside

Versus:

$f_a = 0.040\ f'_m$ without (h'/b) limit
For LRFD/limit-state design

Design Load Factors:

1. General: Strength required by a masonry wall shall be based on factored loads
2. Basic Load Combinations: Loading combinations shall be based on the selected loading criteria shown below:

Required strength, U, to resist factored loads and forces shall be as follows:

$U = 1.4D$ (11-33a)

$U = 1.2D + 1.6L + 0.5(L_r \text{ or } S)$ (11-33b)

$U = 0.9D \pm (1.0E \text{ or } 1.3W)$ (11-33c)

$U = 1.2D + 1.0E + (0.5L + 0.2S)$ (11-33d)

Where:

D = Dead loads or related internal moments and forces.

L = Live loads or related internal moments and forces.

E = Load effects of earthquake or related internal moments and forces.

W = Wind loads or related internal moments and forces.

U = Required strength to resist factored loads or related internal moments and forces.

Design Assumptions for Nominal Strength:

1. Nominal strength of singly reinforced concrete masonry wall cross-sections subject to combined flexural and axial loads shall be based on applicable conditions of equilibrium

Table 11-8. Maximum Reinforcement for Masonry Slender Walls

Nominal Thickness inch	Actual Thickness inch	f_m' = 1500 psi w/ $(\rho_u)max$ =0.00535		f_m' = 3000 psi w/ $(\rho_u)max$ = 0.0107	
		Reinforcement $(\rho_u)_{max}$ bd As in^2/ft	Reinforcement As/b # in^2/ft	Reinforcement $(\rho_u)_{max}$ bd As in^2/ft	Reinforcement As/b # in^2/ft
6	5.625	0.1805	# 4 @ 16 (0.15)	0.361	# 6 @ 16 (0.33)
8	7.625	0.2445	# 5 @ 16 (0.23)	0.489	# 7 @ 16 (0.45)
10	9.625	0.309	#5 / #6 @ 16 (0.28)	0.618	# 8 @ 16 (0.59)
12	11.625	0.373	# 6 @ 16 (0.33)	0.746	# 9 @ 16 (0.75)

$(\rho_b)_{max} = 0.00535$
$(\rho_b)_{max} = 0.5\ (\rho_b)$ masonry
$(\rho_b)_{max} = 0.6\ (\rho_b)$ concrete

$$\rho_b = \frac{0.85\,\beta\, f'_m}{f_y} \times \frac{87{,}000}{87{,}000 + f_y}$$

and compatibility of strains. Strain in reinforcement and masonry shall be assumed directly proportional to the distance from the neutral axis.

2. Maximum usable strain at extreme masonry compression fiber shall be assumed equal to 0.003 i.e at 0.85 f_m'.

3. Maximum usable strain at extreme masonry compression for confinement limits e to 0.001 at 0.40 f'_m

4. For steel strains less than the steel yield strain, the stress in reinforcement shall be taken as E_S times the steel strain. For steel strains greater than the steel yield strain the stress in the reinforcement shall be considered independent of strains and equal to f_y, where:

f_y = Specified yield strength of the reinforcement, psi

E_s = Modulus of Elasticity of reinforcement, = 29,000,000 psi

5. The tensile strength of masonry shall be neglected in flexural calculations of strength, except when computing the nominal cracking moment strength.

6. In the calculation of nominal moment strength the relationship between masonry compressive stress and masonry strain may be assumed to be rectangular. Masonry stress of 0.85 f'_m shall be assumed uniformly distributed over an equivalent compression zone bounded by the edges of the cross-section and a straight line located parallel to the neutral axis at a distance "a" from the fiber of maximum compressive strain.

Design Strength:

Required moment strength, M_u, shall be equal to or less than the nominal moment strength multiplied by a strength reduction factor.

$$M_u \leq \phi M_n \qquad (11\text{-}34)$$

where:

M_n= Nominal moment strength.

ϕ = Strength reduction factor for nominal strength

= 0.80 for nominal wall thickness of 8 inches or greater

= 0.65 for nominal wall thickness of 6 inches or smaller

Modulus of Elasticity:

The nominal value of the modulus of Elasticity of the masonry, E_m shall be assumed as follows:

$$E_m = 750\, f_m' \qquad (11\text{-}35)$$

Modulus of Rupture:

The nominal value of the modulus of rupture (f_r) of the partially grouted or solid grouted hollow unit masonry wall system shall be assumed as follows:

$f_r = 4.0\sqrt{f_m'}$, 235 maximum ... Fully grouted wall

$f_r = 2.5\ \sqrt{f_m'}$, 125 maximum ... Partially grouted wall

Deflection Limitations:

The maximum wall deflection relative to the support, Δ_s, under unfactored lateral and vertical loads shall be 0.007h where h is the height of wall between supports. Note that 0.007h is approximately *l/142* and may not be compatible with some non-structural elements such as doors and windows systems. One may want to use *l*/240 or 0.004 criteria to avoid possible conflicts.

Design Equations:

1. Deflections: The mid-height deflection for simple wall support conditions top and bottom due to the unfactored loads, Δ_S, shall be computed using either of the following equations:

$$M_s \leq M_{cr} \qquad \Delta_s = \frac{5 M_s h^2}{48 E_m I_g} \qquad (11\text{-}36)$$

$M_{cr}<M_s<M_n$ $$\Delta_s = \frac{5M_{cr}h^2}{48E_mI_g} + \frac{5(M_s - M_{cr})h^2}{48E_mI_{cr}} \quad (11\text{-}37)$$

where:

I_g = Moment of inertia of the uncracked wall cross-section, in^4.

I_{cr} = Moment of inertia of the cracked wall cross-section, in^4.

M_{cr} = Cracking moment strength.

$$M_{cr} = S\, f_r \quad (11\text{-}38)$$

S = Section modulus of the uncracked wall cross-section,in^3

M_s = Moment due to unfactored loads for a simple wall support condition top and bottom.

$$M_s = wh^2/8 + P_f(e/2) + (P_w+P_f)\Delta_S \quad (11\text{-}39)$$

where:

w = Distributed lateral load.

e = Eccentricity of the vertical load, P_f.

For other wall support conditions the maximum wall deflection shall be calculated using the equations of structural mechanics.

2. Required Moment Strength: The required moment strength or factored moment, M_u, for a simple wall support conditions top and bottom is the moment given by:

$$M_u = w_uh^2/8 + P_{uf}(e/2) + (P_u)\Delta_u \quad (11\text{-}40)$$

where:

w_u = Factored distributed lateral load.

Δ_u = Horizontal deflection at mid-height of wall calculated using Equation 11-40 for factored loads and M_s=M_u.

P_{uw} = Factored weight of the wall tributary to the section under consideration.

P_{uf} = Factored axial load on the wall from tributary floor and/or roof loads.

e = Eccentricity of the factored axial load, P_{uf}.

$P_u = P_{uw} + P_{uf}$

= Factored axial load at mid height of wall, including tributary wall weight.

3. Nominal Moment Strength: The nominal moment strength, M_n, of the wall is as follows:

$$M_n = A_{se}f_y\,[d-(a/2)] \quad (11\text{-}41)$$

where:

$$a = \frac{(P + A_sf_y)}{0.85\,f_m{}'b} \quad (11\text{-}42)$$

$$A_{se} = \frac{(P + A_sf_y)}{f_y} \quad (11\text{-}43)$$

b = Tributary width

d = Distance from extreme compression fiber to centroid of tension reinforcement.

11.7.4 Comments on the State of the Art Limit State Design Criteria

Reinforced hollow unit masonry that is constructed with good quality control and has its grout vibrated has been shown through experimental measurements to perform in flexure in a very similar fashion to reinforced concrete. The slender wall test conducted by the Structural Engineers Association of California and presented in Section 2411 of the 1985/1991 UBC is developed recognizing this similarity of basic engineering mechanics performance.

One basic assumption of the existing working stress design approach for axial load and flexure is that plane cross-sections remain plain during axial load and bending moment deformations. Alternatively stated, this means that the variation of strain is a linear function of the distance from the neutral axis. The proposed strength design approach for masonry shear walls makes the same assumption. This assumption is consistent with the assumption used in the strength design of reinforced concrete and is supported by experiments on masonry shear walls such as those presented for a six meter tall wall in Figure 11-20[11-18].

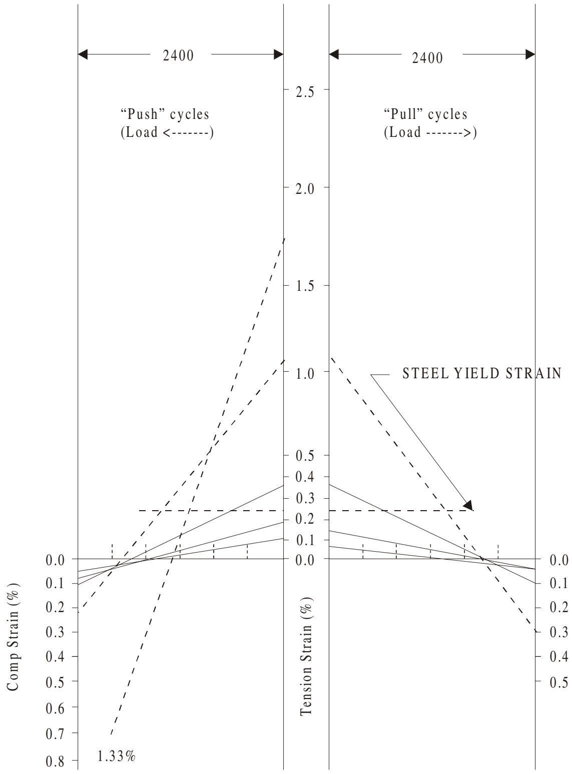


Figure 11-20. Strain profiles at 200 mm above base of a 6 m wall for different deformations

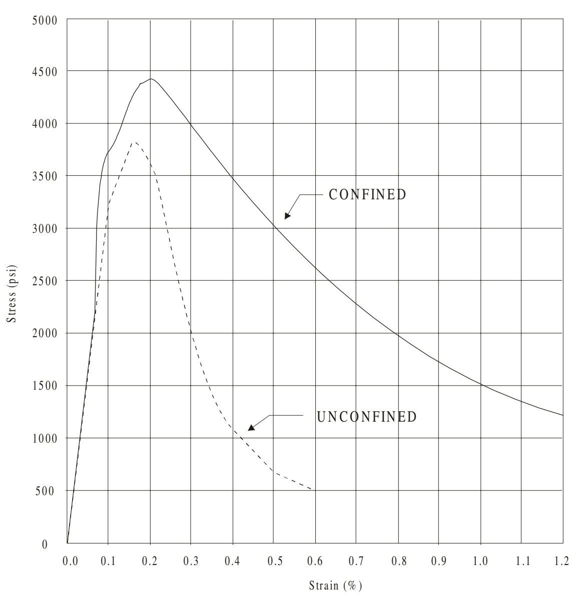


Figure 11-21. Priestly's stress-strain curves

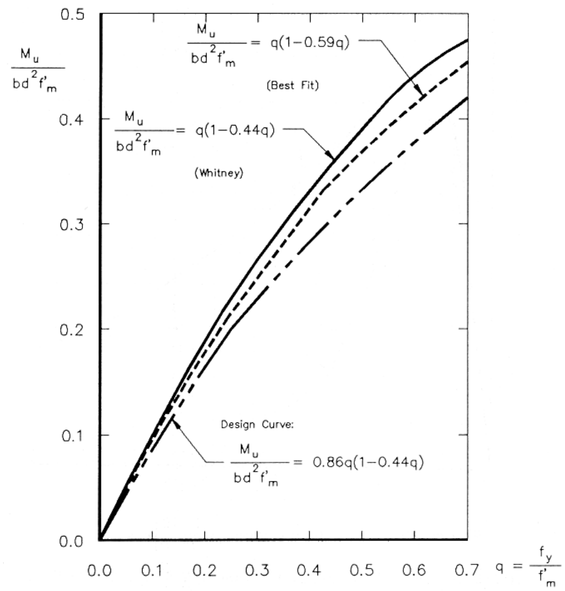


Figure 11-22. Tension controlled flexural test results

The assumption is made in the proposed design criteria that a rectangular stress block can be used to calculate the flexural capacity of shear walls. Stress-strain curves such as those presented in Figure 11-21 indicate that the stress-strain curve for masonry is not rectangular in shape but follows more closely a parabolic form. The reason for the selection of the rectangular stress block is one of convenience, and also, the recognition that the moment capacity of a section with a rectangular stress block closely approximates the moment capacity obtained using the more accurate representation of the stress strain curve.

Figure 11-22 shows the results of tests conducted in Canada for beams in flexure (11-21). The test results are compared with the estimated nominal moment capacity using a rectangular stress block and the design value using a strength reduction factor of 0.86.

Figure 11-23 shows an idealized stress strain curve with the parameters defined in Table 11-9 identified on the curve. Based on the TCCMAR data, the value of 0.003 for the maximum usable strain is slightly less the average value obtained from the test results.

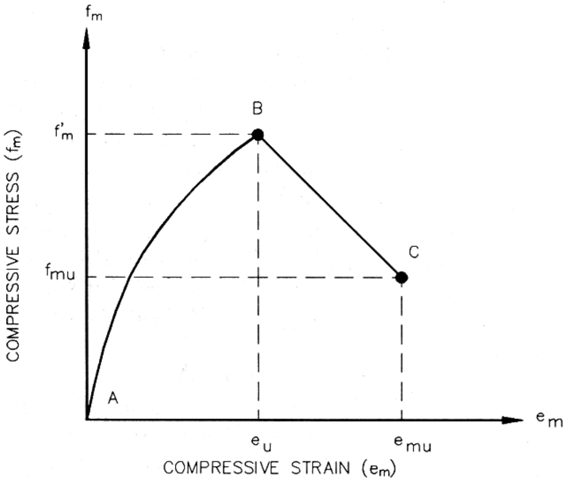


Figure 11-23. Unconfined concrete masonry stress-strain curve

The value of the maximum usable strain selected as part of this criteria is equal to the value most often cited for the design of reinforced concrete members. One might be inclined to be concerned with our selection of 0.003 because it is the same value as used for reinforced concrete. However, as indicated in Figure 11-24, the maximum usable strain value for concrete with maximum compressive value comparable to those values specified in the criteria for masonry far exceeds the 0.003 value. In particular, as reinforced concrete can obtain significantly higher maximum compressive values, it is only at these maximum compressive values where the 0.003 limitation is reasonable.

Table 11-9. Design Parameters for the Unconfined Concrete Masonry Stress-Strain Curve

Parameter	Comment
f'_m	Ultimate compressive stress. Nominal design value is specified by design engineer.
ε_u	Strain corresponding to f'_m. We recommend a nominal design value of 0.0020 to 0.0025.
f_{mu}	The minimum usable compressive stress in the strain region defined by strain values greater than the strain at ultimate compressive stress, ie., ε_{mu}. We recommend a nomial design value of 0.5 f'_m.
ε_{mu}	Maximum usable unconfined strain. Alternately stated, it is the strain corresponding to the minimum usable compressive stress. We recommend a nominal design value of 0.0030.

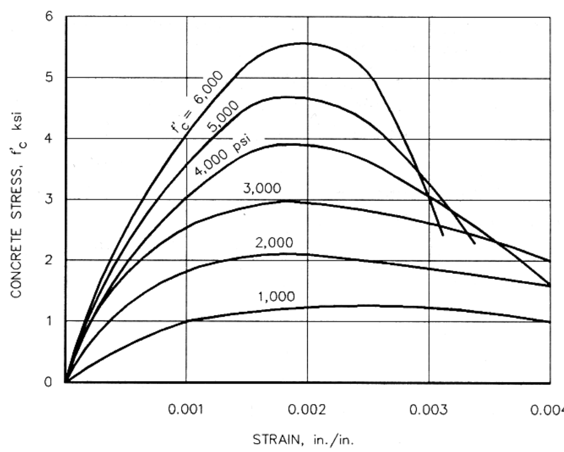


Figure 11-24. Typical stress-strain curves for concrete under short-time loading

The maximum strain can be increased where confinement is provided (see Figure 11-21). Experimental evidence indicates that confinement increases the maximum usable strain, and therefore, the component curvature ductility.[11-19, 11-22]

11.7.5 Example Problem - Out of Plane loads on Reinforced Masonry Wall (Strength Design)

Determine if the fully grouted medium weight concrete masonry unit (CMU) slender wall (out-of-plane loads) shown in Figure 11-25 is adequate. Seismic Zone 4 (Ca=0.44), with special inspection.

Wall Properties:

Wall is fully grouted (medium wt.)	= 80 psf
Nominal block thickness	= 8 inch
Actual block thickness (b)	= 7.6 inch
Tributary width of roof	= 26 ft/2
Specified compressive stress (f_m')	= 3000 psi

Modulus of Rupture (f_r) = $4.0(f_m')^{1/2}$ = 219 psi
Modulus of elasticity of CMU(E_m) = 750 f_m'
Specified yield str. of steel (F_y) = 60 ksi
Modulus of elasticity of steel (E_s) = $29\text{x}10^6$ psi
Area of vertical steel (A_s) = 0.33 in^2/ft
Eccentricity(e) (3.5/2 + 7.625/2) = 5.56 in
Depth to steel (d) = 3.81 in

Strength Reduction Factor for Flexure:
$\phi = 0.80$

Unfactored Loads:
Self Weight of Wall (P_w) at mid-wall height
P_W = [(25/2) + 1.5] 80 psf = 1120 plf

Roof Tributary Load (P_f)
P_f = (D+L_r)(26ft/2) = (14psf+20psf)(13 ft)
= 442 plf

Seismic Lateral Load (w)

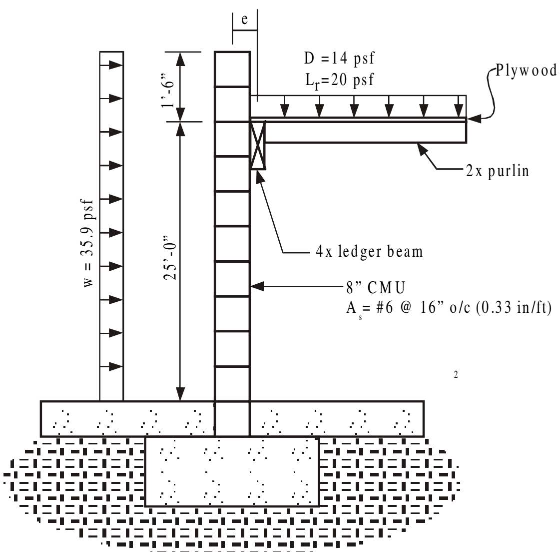


Figure 11-25. Cross-Section of Slender Wall

The wall is laterally supported at the base and roof. At the roof level, $h_x = h_r$, and so the lateral force is equal to:

$$F_p = \frac{(1.0)C_a I_p}{3.0}\left(1 + 3\frac{h_x}{h_r}\right)W_p$$
$$= 1.33 C_a I_p W_p < 4 C_a I_p W_p$$

At the base of the wall, $h_x = 0$, and so the lateral force coefficient is equal to:

$$F_p = \frac{(1.0)C_a I_p}{3.0}\left(1 + 3\frac{h_x}{h_r}\right)W_p$$
$$= 0.33 C_a I_p W_p < 0.7 C_a I_p W_p$$

Thus, use $0.7C_aI_pW_p$ at the base. The design lateral forces are to be distributed in proportion to the mass distribution of the element. Therefore, the average force, which is uniformly distributed over the wall height, is given by:

$$F_p = \frac{(1.33 + 07)}{2} C_a I_p W_p = 1.02 C_a I_p W_p$$
$$= 1.02(0.44)(1.0)W_p$$
$$= (0.45)(80\,psf) = 35.9\,psf$$

SOLUTION OUTLINE:

A. Vertical load stress check
B. Maximum Reinforcement Check
C. Cracking moment
D. Moment of inertia (gross/cracked)
E. Nominal moment strength (M_n)
F. Unfactored service moments and displacements
G. Factored moments and displacements.
H. Design moment capacity

Vertical Load Stress Check

$(P_w + P_f)/A_g \le 0.04\, f_m'$

Where:

P_w = Weight of wall = 1120 plf
P_f = Tributary load = 442 plf
A_g = Gross area of wall = tb
= 7.625 in (12 in/ft) = 91.5 in^2/ft
$0.04f_m'$= 0.04(3000psi) = 120 psi
Now

(1120+442)/91.5 =17.07psi< $0.04f_m'$…OK

Maximum Reinforcement Check

$(\rho_b)max = 0.0107$
$(A_s)max = 0.489$...Ref. Table 10-8
$(A_s)actual = 0.33 < 0.489$...OK

Cracking Moment (M_{CR}): (w/o dead load)

$M_{cr} = Sf_r$

Where:

$f_r = 4.0\,(f_m')^{1/2}$...235 psi, max
$= 4.0\,(3000)^{1/2}$
$= 219$ psi
$S = Lb^2/6$
$= 12\text{ in }(7.625)^2/6$
$= 116.3\text{ in}^3/\text{ft}$

Now:

$M_{CR} = 116.3\text{ in}^3\,(219\text{ psi})(1/1000\text{ k/lb})$
$= 25.5$ k-in/ft
$= 2.12$ k-ft/ft

Moment of Inertia (Gross/Cracked)

A. Gross Moment of Inertia (I_g)
$I_g = Lb^3/12$
$= 12\text{ in }(7.6)^3/12$
$= 443.3\text{ in}^3$

B. Cracked Moment of Inertia (I_{cr})
$I_{cr} = nA_{se}(d - c)^2 + (bc^3)/3$

where:

$A_{se} = (A_sf_y + P_u)/f_y$ = effective area of steel

$A_s = 0.33\text{ in}^2/\text{ft}$
$f_y = 60$ ksi
$P_u = 1.2D + 0.5L_r$
$P_u = 1.2(1120+14(26/2)) + 0.5(20(26/2))$
$= 1.69$ kips

Now:

$A_{se} = [0.33\text{ in}^2(60\text{ ksi}) + 1.69]/60\text{ ksi}$
$= 0.36\text{ in}^2/\text{ft}$

Next:

$a = (P_u + A_sf_y)/0.85\,f'_mb$
$= [1.69 + 0.33(60)]/0.85(3.0)(12.0)$
$= 0.71$

Now:

$c = a/0.85$
$= 0.71/0.85 = 0.84$ in.

Next:

$b = 12.0$ in.
$d = 3.8$ in.
$n = E_s/E_m = 29 \times 10^3\text{ ksi}/750(3.0\text{ ksi})$
$= 12.9$
$p = A_s/bd$
$= 0.33\text{ in}^2/(12\text{ in} \times 3.8\text{ in}) = 0.0072$
$np = 12.9\,(0.0072) = 0.093$

Thus:

$I_{cr} = 12.9(0.36)[3.8\text{-}0.84]^2 + 12\text{ in}(0.84\text{ in})^3/3$
$= 40.9 + 2.34$
$= 43.2\text{ in}^4$

Note : ratio of I_g to $I_{cr} = 443.3/43.2 \approx 10:1$

Nominal moment strength (ϕM_n)

$\phi M_n = \phi\, A_{se}f_y[d-(a/2)]$
$= 0.80(0.36\text{ in}^2)(60\text{ ksi})[3.81 - (0.71/2)]$
$= 59.7$ k-in.
$= 4.98$ k-ft.

Unfactored Service Moments and Displacements (Design for Deflection)

$M_s = (wh^2/8) + P_f(e/2) + (P_w + P_f)\Delta_s$

Where:

Δ_s = Midheight deflection under service lateral and vertical loads (without load factors [w=F_p/1.4]). Maximum Δ_s = 0.007h = 0.007(25)(12) = 2.1 in. Note that a deflection criteria used by some window systems is *l*/240 = 0.004h; thus 0.007 = *l*/143 may be liberal for attached glazing.

$$\Delta s = \begin{cases} \dfrac{5M_s h^2}{48E_m I_g} \\ (For\, M_s \le M_{cr}) \\ \dfrac{5M_{cr}h^2}{48E_m I_g} + \dfrac{5(M_s - M_{cr})h^2}{48E_m I_{cr}} \\ (For\, M_{cr} < M_s < M_n) \end{cases}$$

Recall:

M_{cr} = 25.5 k-in = 2.12 k-ft
ϕM_n = 59.7 k-in = 4.98 k-ft
e = 5.56 in.

<u>Start:</u> Try: $\Delta_1 = 0$

Recall: $P_w + P_f$ = 1120 + 442 = 1562

M_1= [(35.9/1.4) x $(25)^2$/8]
+ 442 x 5.56/(2 x 12) + 1562(0)
= 2003 + 102
= 2105
= 2.11 k-ft < M_{cr} = 2.12 k-ft

Thus:

$$\Delta_2 = \frac{5(2.11)(25)^2(1728)}{48(750)(3)(433)} = 0.24\text{in}$$

Try: Δ_2 = 0.26 in.

M_2 = 2110 lb-ft + 1562(0.26/12)
= 2144 lb-ft
= 2.14 k-ft

$$\phi M_n > 2.14 > M_{CR}$$

$$\Delta_3 = \frac{5(2.12)(25)^2(1728)}{48(750)(3)(433)} + \frac{5(2.14-2.12)(25)^2(1728)}{48(750)(3)(44.18)} = 0.26\text{in}$$

Use M_s = 2.14 k-ft
= 25.72 k-in
Δ_S = 0.26 in. < (.007h = 2.1 in)...OK

Factored (Ultimate) Moments and Displacements.

<u>Load Case 1</u>: U = 0.9D + 1.0E

Thus:

w_u	= 1.0(39.5 plf)	= 39.5 plf
P_{ufd}	= 0.9(14 psf x 26 ft/2)	= 164 plf
P_{ufl}	= 0	
P_{uw}	= 1.2(1120 plf)	= 1344 plf
P_{uf}	= $P_{ufd} + P_{ufl}$	
	= 164 plf + 0 plf	= 164 plf
P_u	= $P_{uf} + P_{uw}$	
	= 164 plf + 1344 plf	= 1508 plf

Now:
$M_u = (w_u h^2/8) + P_{uf}(e/2) + P_u\Delta_u$

Where:
M_u = Factored moment at midheight of wall
Δ_u = Midheight deflection under factored lateral and factored service loads

Try $\Delta_1 = 0$

$M_1 = 35.9(25)^2/8$
+ 164 [5.56/(2 x 12)] + 1508(0)
= 2805 + 38.0 + 0
= 2843 lb-ft = 2.84 k-ft

$M_{cr} < 2.84 < \phi M_n$

$$\Delta_2 = \frac{5(2.12)(25)^2(1728)}{48(750)(3)(433)} + \frac{5(2.84-2.12)(25)^2(1728)}{48(750)(3)(43.2)}$$

$= 0.245 + 1.157(2.84\text{-}2.12) = 1.08$ in.

Try $\Delta_2 = 1.2$ in.

$M_2 = 2843$ lb-ft + 1508(1.2/12)
= 2994 lb-ft
= 2.99 k-ft

$M_{cr} < 2.99$ k-ft $< M_n$

Δ_3 = 0.245 + 1.157(2.99-2.12)
= 0.245 + 1.01
= 1.25 in

Try $\Delta_3 = 1.27$

$M_3 = 2843$ lb-ft +1508 (1.27/12) = 3.00 k-ft

$\Delta_4 =$ 0.245 + 1.157 (3.00 - 2.12)
= 1.27 in

Use $M_u = 3.00$ k-ft
= 36.0 k-in
Δ_u = 1.27 inch

Load Case 2: U = 1. 2D + 1.0E

The earthquake load on an element is given by:

$$E = \rho E_h + E_v$$

where E_h is the horizontal component and E_v is the vertical component of the earthquake load. The variable, ρ is the redundancy/ reliability factor and is equal to 1.0 for elements of structures. For strength design, the vertical component is given by:

$$E_v = 0.5 C_a I D$$
$$= 0.5(0.44)(1.0)D = 0.22D$$

Thus, the load combination U=1.2D + 1.0E becomes:

U = 1.2D + 1.0E
= 1.2D + 0.22D +1.0E_h
=1.42D + 1.0E_h

w_u = 1.0(35.9 plf) = 35.9 plf
P_{ufd} = 1.42(14 psf x 26 ft/2) = 258.4 plf
P_{ufl} = 0
P_{uw} = 1.42(1120 plf) = 1590 plf
P_{uf} = $P_{ufd} + P_{ufl}$
= 258.4 plf + 0 plf =258.4plf
P_u = $P_{uf} + P_{uw}$
= 258.4 plf + 1590 plf =1848.4 plf

Now:
$M_u = (w_u h^2/8) + P_{uf}(e/2) + P_u\Delta_u$

Where:
M_u = Factored moment at midheight of wall
Δ_u = Midheight deflection under factored lateral and factored service loads

Try $\Delta_1 = 0$

$M_1 = 35.9(25)^2/8$
+ 258.4 x 5.56/(2 x 12) + 1848.8(0)
= 2805 + 59.8
= 2864.8 lb-ft = 2.86 k-ft

$M_{cr} < 2.86 < \phi M_n$

$$\Delta_2 = \frac{5(2.12)(25)^2(1728)}{48(750)(3)(433)} + \frac{5(2.86-2.12)(25)^2(1728)}{48(750)(3)(44.18)}$$

$= 0.245 + 1.157(2.86\text{-}2.12) = 1.1$ in.

Try $\Delta_2 = 1.2$ in.

$M_2 = 2864.8$ lb-ft + 1848.8 (1.2/12)
= 3050 lb-ft
= 3.05 k-ft

$M_{cr} < 3.05$ k-ft $< M_n$

Δ_3 = 0.245 + 1.157(3.05-2.12)
= 0.245 + 1.076
= 1.32 in

Try $\Delta_3 = 1.34$

M_3 =2864.8 lb-ft+1848.8(1.34/12)=3.07k-ft

$\Delta_4 =$ 0.245 + 1.157 (3.07 - 2.12)
= 1.34 in

Use M_u = 3.07 k-ft
= 36.8 k-in…Controls
Δ_u = 1.34 inch

Design moment capacity $M_u < \phi M_n$

ϕM_n = 59.7 k-in

Where:
Strength Reduction Factor for Flexure
$\phi = 0.80$
M_u = 36.8 k-in < 59.7 k-in …OK

Conclusion:
Slender wall is OK as shown

11.8 Shear Wall Design

11.8.1 General

Over 100 masonry shear walls with different steel ratios, axial load levels and sizes have been tested in the last decade. Therefore, it is possible to develop design criteria that are based on good quality, typically cyclic load reversal, test data. The design criteria for reinforced hollow unit concrete masonry shear walls in many respects follow the design criteria for reinforced concrete shear walls. However, as we shall later discuss, a major area of disagreement exists between many engineers who design concrete shear walls and many masonry designers over the use of highly reinforced boundary members. With that issue put aside it is possible, as this section will illustrate, to design ductile masonry shear walls that will perform well during seismic loading.

11.8.2 Structural Mechanics

The reader is referred to Volume two of the books entitled "Earthquake Design of Concrete Masonry Buildings" by Englekirk and Hart(11-16) and Design of Reinforced Masonry by Schneider and Dickey(11-23) for excellent discussions of the structural mechanics of reinforced masonry design. In most respects it parallels the standard development of structural engineering design we are familiar with. For example, plane cross-sections are assumed to remain plane and a rectangular (Whitney) stress block replaces a more complex stress strain curve. The reader may wish to refer to these two references prior to reading the next subsection.

11.8.3 State-of-the-art Limit State Design Criteria

The following design criteria is very similar to the UBC design criteria. The reader is referred to Reference 11-24 for a history of that development.

A. Notations

A_e = effective area of masonary

A_n = net cross sectional area perpendicular to axial load square inches.

A_{mv} = net area of masonry section bounded by wall thickness and length of section in the direction of shear force considered, square inches.

A_s = area of tension reinforcement, square inches.

a_b = length of compressive stress block. inches.

b = effective width of wall, inches.

C_d = masonry shear strength coefficient as obtained from Figure 11-26.

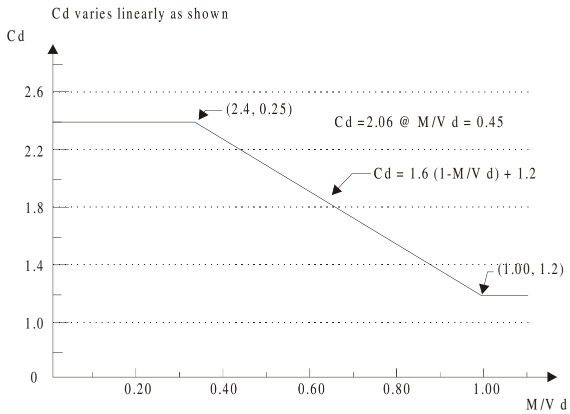


Figure 11-26. Nominal Shear Strength Coefficient (Cd)

d = distance from extreme compression fiber to centroid of tension reinforcement, inches.

D = dead loads, or related internal moments and forces.

E = load effects of earthquake, or related internal moments and forces.

E_s = modulus of elasticity of steel, 29,000,000 psi.

e_{mu} = maximum usable compressive strain of masonry.

F_s = allowable stress in reinforcement. psi.

f_s = computed stress in reinforcement, psi.

f_m' = specified compressive strength of masonry at the age of 28 days, psi

f_y = specified yield strength of reinforcement, psi.

L = live loads, or related internal moments and forces.

L_w = length of wall.

P_b = nominal balanced design axial strength.

P_o = nominal axial strength without bending loads.

P_u = required axial strength.

U = required strength to resist factored loads, or related internal moments and forces.

V_n = nominal shear strength.

V_m = nominal shear strength provided by masonry.

V_s = nominal shear strength provided by shear reinforcement.

ρ_n = ratio of distributed shear reinforcement on a plane perpendicular to plane of A_{mv}.

ϕ = strength reduction factor.

B. Quality Control Provision.

1. Special, inspection during construction of the shear wall is required, especially after placement of the steel and prior to the pouring of the grout.
2. f_m' shall not be less than 1,500 psi nor greater than 4,000 psi. However, in concrete masonry a limit of 3,000 psi is recommended unless special quality control measures are taken or specified by the engineer.
3. f_m' shall be verified with prism testing.

C. Design Procedure

1. Required strength:

• For earthquake loading, the load factors shall be

$$U = 1.2D + 1.0E \tag{11-44}$$

$$U = 0.90D \pm 1.0E \tag{11-45}$$

• Required strength U to resist dead load D and live load L shall be at least equal to

$$U = 1.2D + 1.6L + 0.5(L_r + S) \tag{11-46}$$

2. Design Strength: Design strength provided by the shear wall cross section in terms of axial force, shear, and moment shall be computed as the nominal strength multiplied by the strength reduction factor, ϕ.

Shear walls shall be proportioned such that the design strength exceeds the required strength.

Strength reduction factor ϕ shall be as follows:

• Axial load and axial load with flexure: $\phi = 0.65$

For members in which f_y does not exceed 60,000 psi, with symmetrical reinforcement, ϕ may be increased linearly to 0.85 as ϕP_n decreases from 0.10 $f_m'A_e$ or 0.25 P_b to zero.

For solid grouted wall P_b may be calculated by Equation 11-47:

$$P_b = 0.85\, f_m' b a_b \qquad (11\text{-}47)$$

where

$$a_b = 0.85\left[\frac{e_{mu}}{e_{mu} + \dfrac{f_y}{E_s}}\right] d \qquad (11\text{-}48)$$

• Shear: $\phi = 0.60$

The shear-strength reduction factor may be 0.80 for any shear wall when its nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength for the factored-load combination

3. Design Assumptions for Nominal Strength: Nominal strength of shear wall cross sections shall be based on assumptions prescribed in Section 11.8.

The maximum usable strain, e_{mu}, at the extreme masonry compression fiber shall not exceed 0.003 unless compression tests on prisms indicate higher values are justified.

4. Reinforcement:

• Minimum reinforcement shall be 0.0007 in either direction and 0.002 total. (0.003 for California Hospitals and schools)

• When the shear wall failure mode is in flexure, the nominal flexural strength of the shear wall shall be at least three times the cracking moment strength of the wall from Equation 11-38.

• All continuous reinforcement shall be anchored or spliced in accordance with 1997 UBC Section.

• The minimum amount of vertical reinforcement shall not be less than one half the horizontal reinforcement.

• Maximum spacing of horizontal reinforcement within the region defined in Section 6C(i) below shall not exceed three times nominal wall thickness or 24 inches, whichever is less.

5. Axial strength: The nominal axial strength of the shear wall supporting axial loads only shall be calculated by Equation 11-49.

$$P_o = 0.85\, f_m'(A_n - A_s) + f_y A_s \qquad (11\text{-}49)$$

Axial design strength provided by the shear wall cross section shall satisfy the equation:

$$P_u \le \phi(0.80) P_o \qquad (11\text{-}50)$$

6. Shear strength:

a. The nominal shear strength shall be determined using either Section 6b or 6c. Figure 11-26 gives the values for C_d.

b. The nominal shear strength of the shear wall shall be determined from Equation 11-51, except as provided in Section 6c.

$$V_n = V_m + V_s \qquad (11\text{-}51)$$

where

$$V_m = C_d\, A_{mv} \sqrt{f_m'} \qquad (11\text{-}52)$$

and

$$V_s = A_{mv}\,\rho_n\,f_y \qquad (11\text{-}53)$$

c. For a shear wall whose nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength two shear regions exist.

(i) For all cross sections within the region defined by the base of the shear wall and a plane at a distance L_w above the base of the shear wall the nominal shear strength shall be determined from:

$$V_n = A_{mv}\rho_n\,f_y \qquad (11\text{-}54)$$

The required shear strength for this region shall be calculated at a distance $L_w/2$ above the base of the shear wall but not to exceed one-half story height.

(ii) For the other region the nominal shear strength of the shear wall shall be determined from Eq 11-51.

7. Confinement of Vertical Steel: All vertical reinforcement whose corresponding masonry compressive stress, corresponding to factored forces, exceeds $0.75f_m'$ shall be confined when the failure mode is flexure. Vertical steel when it needs to be confined shall be done with a minimum of No.3 bars at a maximum of 8-inch spacing or equivalent within the grouted core and within the region defined as the base of the shear wall. When confinement is needed the vertical steel confined shall be at least from the end of the wall to a lateral distance three times the thickness of the wall.

11.8.4 Comments on State of the Art Design Criteria for Shear Walls

The design strength is obtained by multiplying the nominal strength by a strength reduction factor. The nominal strength is ideally the best professional estimate of the true strength of the member. The strength reduction factor is selected to account for the uncertainty of the value of the parameters in the nominal strength equation, the workmanship in the field, and the general confidence in the equation's ability to predict the actual performance of the member.

For walls subjected to flexure and axial load the variation in the numerical value of the strength reduction factor is a function of the axial load on the shear wall. The primary reason for this is to insure that the walls performance is that of an under-reinforced flexural member. Therefore, we have divided the interaction diagram for the shear wall into two zone for the purpose of setting a value for the strength reduction factor. Zone 1 corresponds to sufficiently low axial loads to insure a very ductile shear wall performance. We have provided an axial load limit of less than 65% of an approximate calculation of the balance design axial load, P_b. This alternative approach, by being a function of the balance design axial load, places a stronger emphasis on the importance of quantifying the intensity of the axial load as a function of the balance design axial load in order to promote ductility. The value of 65% P_b is reasonable based on a reliability analysis which incorporated uncertainty in material properties and the design equation[11-25]. To provide a straightforward calculation of the balance design axial load, we have provided an equation which is a good approximation of the balance design axial load for purposes of the use here (i.e.,typically less than 10% error). This approximation assumes that the forces from the positive tension steel and the negative compression steel balance each other in the equilibrium equation.

Zone 2 is for value of axial load greater than 65% of the balance design axial load. The numerical value of the strength reduction factor in Zone 2 is equal to 0.65. To ensure that the quality of the masonry is consistent with the engineering design assumptions, the minimum value of f_m' is set at 1500 psi. The maximum recommended value for f_m' is 3,000 psi unless a special level of quality control is used for concrete masonry. Unless the engineer has performed a check with his local block supplier it is reasonable to assume that 3,000 psi is a practical limit.

The strength reduction factor for shear walls where the mode of failure is shear is equal to 0.60. This typically represents shear walls that are long compared to their height.

For walls where flexure is a possible failure mode, the shear resistance that is provided is checked to ensure that the shear corresponding to the development of the full nominal flexural strength of the wall is provided. This approach is consistent with the approach taken for reinforced concrete in the 1997 UBC. In this situation, the strength reduction factor for shear is equal to 0.80.

The equation used to calculate the axial strength of the wall is equal to the specified compressive strength times the net area of the wall times an effective stress parameter value of 85% plus the yield stress of the steel times the area of the steel. This equation is directly consistent with the equation used in reinforced concrete design.

For pure axial load design, the strength reduction factor is equal to 0.65 and was discussed in Section 11-8. A further reduction is made to reduce the axial load by multiplying the nominal strength by 0.8 in order to account for accidental eccentricities.

The shear strength of shear walls can be determined using either of two alternative approaches. The first approach is used for shear walls where the failure mode is shear. In this situation, the strength reduction factor is equal to 0.60 and the nominal shear strength is obtained by adding two terms. The first term is the shear strength assumed to be provided by the masonry in a reinforced masonry wall. The second term is the shear strength provided by the shear reinforcement.

The second approach used to calculate the nominal shear strength of a wall is appropriate for shear walls where a flexural mode of failure is possible. The intent of this approach is to require that sufficient shear reinforcement is placed in the wall to insure a ductile flexural failure. In this situation, the strength reduction factor for shear is equal to 0.80. The flexural failure mode will result in a shear wall where the region near the base will be called upon to undergo an inelastic moment curvature response. Therefore, we have identified two shear regions for such a shear wall. Shear region number one is a region defined from the base of the wall up to a distance equal to the length of the wall and is a plastic hinge region. In this region because of the inelastic cyclic response, only the shear resistance provided by the steel is considered in the design. In this region, the region above the plastic hinge, the masonry and the steel are both used to calculate the shear strength of the wall.

The use of boundary members in shear walls is a highly controversial topic in masonry design. The New Zealand Building code does not allow boundary members to be used in masonry shear walls[11-26]. The New Zealand approach is to encourage the structural engineer to uniformly distribute the vertical steel along the length of the wall. This, they argue, provides a more consistent distribution of shear stress between the wall and the foundation. The counter to this argument is the current approach taken by reinforced concrete design criteria. In essence, the current approach for reinforced concrete walls is to design the shear wall as if it were essentially a second class ductile frame and discount the concrete between the boundary members. The net result of this design is high axial loads at the ends of the wall.

The approach defined in UBC 97 for steel confinement determination specifies that the factored loads are applied to the shear wall and, using the principles of mechanics, the compressive stress in the masonry immediately adjacent to the vertical reinforcing bars is calculated. If this stress exceeds 75% of the maximum specified compressive stress, the vertical reinforcement must be confined. The 75% number is based on an approximate unconfined masonry prism strain value of 0.0015 for a stress strain curve that is parabolic between zero stress and maximum compressive stress (see Figure 11-21). If the strains are below 0.0015 then based on observations of prism tests we can expect no significant loss of strength or stiffness due to cyclic loading and small internal masonry cracking.

11.8.5 Example Problem – Reinforced Masonry Shear Wall (Strength Design)

This example problem is a variation of example 3K on page 95 of the book entitled "Reinforced Masonry Engineering Handbook - Clay and Concrete Masonry" by James Amrhein[11-27]. Determine if the CMU shear wall shown in Figure 11-27 is adequate for the following vertical and seismic loads. Use strength design UBC 97.

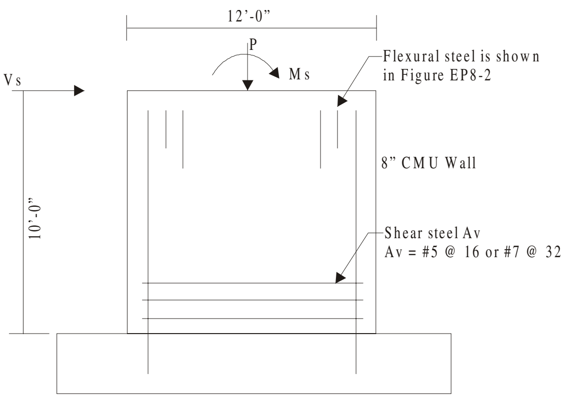


Figure 11-27. Elevation of Shear Wall

Loads: Dead Load= 30 kips
Live Load = 0 kips
Lateral Shear Force (V_E) = 75 kips
Seismic Moment (M_E) = 400 kip-ft

Load Factors: U = 1.2D + 1.6 L
U = 1.2D + 0.5L + 1.0E
U = 0.9D ± 1.0E

Reduction Factors: ϕ = 0.65 Axial
ϕ = 0.65 Axial plus flexure
ϕ = 0.80 Flexure only
ϕ = 0.60 Shear

Wall Properties:

Wall is fully grouted ($M_n \geq 1.8\, M_{cr}$)
Normal block thickness = 8 inch
Actual block thickness (b) = 7.625 inch
Length of wall (L) = 12 ft
Specified compressive strength(f_m') = 1500 psi
Modulus of rupture (f_r) = $4.0\sqrt{f_m'}$
Maximum usable masonry strain (e_{mu}) = 0.003
Modulus of elasticity of CMU (E_m) =$750f_m'$
Shear modulus of masonry (G) =$0.4E_m$
Specified yield strength of steel(f_y) =60 ksi
Modulus of elasticity of steel (E_s) = 29×10^6psi

<u>SOLUTION OUTLINE:</u>

A. Interaction diagram (generate/draw)
B. Cracking moment strength (M_{cr})
C. Load cases (axial plus flexure)
D. Boundary members
E. Shear

<u>A. Interaction Diagram</u>

1. Nominal axial load strength (P_o)

$P_o = 0.85\, f_m'(A_e - A_s) + f_yA_s$
= 0.85(1.5ksi)[12 ft (12 in/ft)(7.625 in) - 10 bars0.31 in^2/bar] + 60 ksi (10 bars)(0.31 in^2/bar)
= 1581.99 kips

2. Design axial load strength (P_u)

$P_u = \phi(0.80)(P_o)$
= 0.65(0.80)(1581.99 kips)
= 822.64 kips

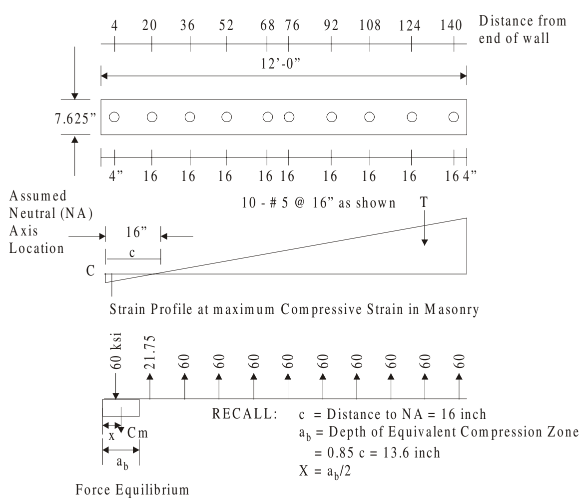


Figure 11-28. Steel locations, strain profile and force equilibrium diagrams

3. Nominal bending moment strength (M_o): See Figure 11-28.

Must solve for location of neutral axis (NA) such that sum of axial forces on cross section is zero.

_ Assume location for NA; c = 16 inch.

_Use maximum allowable CMU strain of 0.003.

_ Iterative solution.

_ Take sum of moments about extreme compression fiber (end of wall).

$T = A_s f_s = [21.75 \text{ ksi} + 8(60 \text{ ksi})](0.31 \text{ in}^2)$
$= 155$ kips

$C = A_s f_s + \phi f_m' b a_b$
$= 0.31 \text{in}^2 \ (60 \text{ ksi}) + 0.85 \ (1.5 \text{ ksi})(7.625 \text{ in})(13.6 \text{ in})$
$= 150.82$ kips

T-C = 4 kips close enough <u>use</u> c = 16".

$M_o = A_s f_y - 0.85 f_m' b a_b$

$M_o = 0.31 \text{ in}^2[21.75(20) + 60(36 + 52 + 68 + 76 + 92 + 108 + 124 + 140] - 0.31 \text{ in}^2 \times(60)(4) - 0.85(1.5)(13.6)^2(1/2)(7.625)$
$= 13080.4 - 74.4 - 899$
$= 12{,}107$ k-in
$= 1009$ k-ft

4. Design bending moment strength (M_u)

$M_u = 0.80 M_o$
$= 0.80(1009 \text{ k-ft})$
$= 807.2$ k-ft

5. Nominal balanced design axial strength (P_b): See Figure 11-29.

$C_m = 0.85 f_m' b a_b$

Where:

$$a_b = \left[\frac{e_{mu}}{e_{mu} + \dfrac{f_y}{E_s}}\right] d$$

$$= 0.85\left[\frac{0.003}{0.003 + 60/29000}\right] d$$

$$= 0.85(0.5918)d$$

$$= 0.503(140\text{inch})$$

$$= 70.43\text{inch}$$

Recall:

c = Distance to NA = $a_b/0.85$
$= 70.428/0.85$
$= 82.86$ in

$T = \Sigma A_s f_y$
$= 0.31 \text{ in}^2 \ (9.6 + 26.4 + 43.2 + 60)\text{ksi}$
$= 43.2$ kips

Now:

$C = \Sigma A_s f_y + 0.85 f_m' b a_b$
$= 0.31 \text{ in}^2(7.2 + 15.6 + 32.4 + 49.2 + 60 + 60)\text{ksi} + 0.85(1.5)(7.625)(70.428)$
$= 69.56 + 684.69$
$= 754.25$

Thus:

$P_b = C - T$
$= 754.25 - 43.2$
$= 711$ kips

6.Design balanced design axial strength (P_{bu})

$P_{bu} = \phi P_b$
$= 0.65 \ (711 \text{ kips})$
$= 462$ kips

7.Nominal balanced design moment strength (M_b): See Figure 11-29. Take sum of moments about plastic centroid (center of wall):

$M_b = A_s f_y - 0.85 f_m' a_b X_b b$
$= 0.31[60(68) + 43.2(52) + 26.4(36) + 9.6(20) - 7.2(4) + 15.6(4) + 32.4(20) + 49.2(36) + 52(60) + 68(60)] + 0.85(1.5)(70.428)(36.76)(7.625)$
$= 5308 + 25169$
$= 30477$ k-in
$= 2540$ k-ft

8. Design balanced design moment strength (M_{bu})

$M_{bu} = \phi M_b$
$= 0.65(2540 \text{ k-ft})$
$= 1651$ k-ft

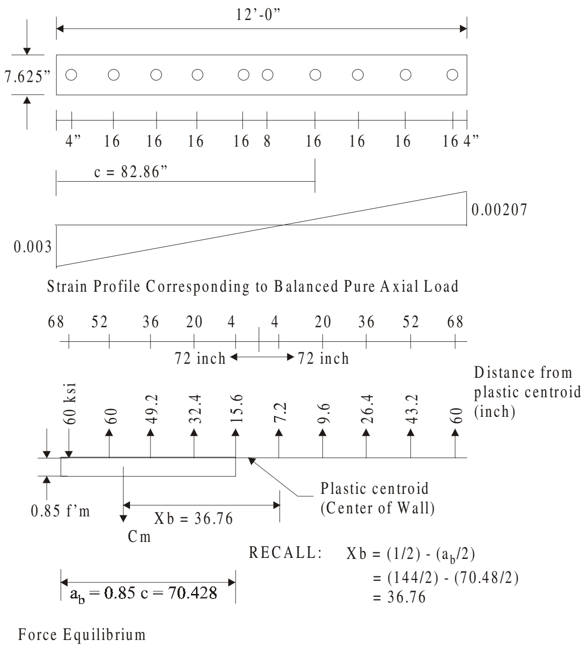


Figure 11-29. Balanced design load condition

B. Cracking moment strength

-Linearly elastic model
-Gross section properties

$(P/A) + M_{cr}/S = f_r$

Thus:

$M_{cr} = S[(P/A) + f_r]$

Where:

$A = bl = 7.625(144) = 1098 \text{ in}^2$
$s = bl^2/6 = 7.625 (144)^2/6 = 26{,}352 \text{ in}^3$
$f_r = 4.0\sqrt{f_m'} = 4.0(1500)^{1/2} = 155$ psi
P = Dead Load = 30,000 lbs

Thus:

Mcr = 26352[(30000/1098) + 155][(1/1000)(k/1b)]
= 4804.6 k-in
= 400 k-ft

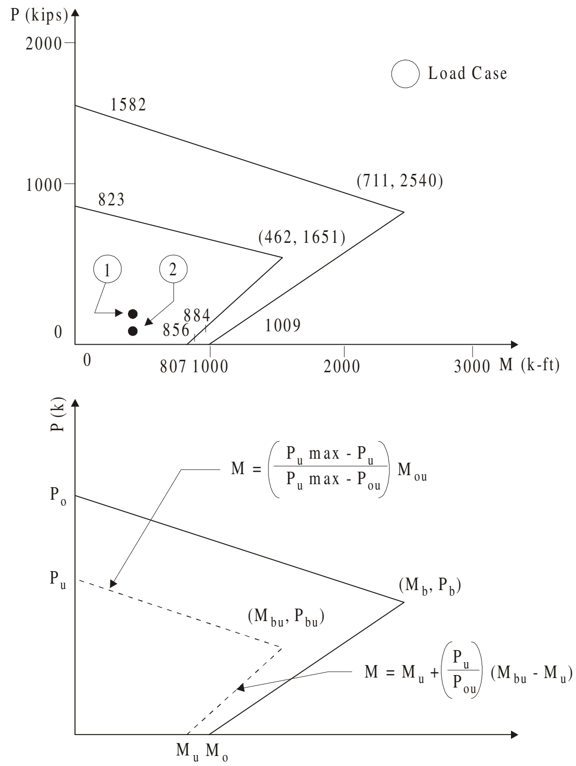


Figure 11-30. Interaction Diagram

C. Load Cases (See Figure 11-30)

Load Case 1:
U = 12D + 1.0E
= $1.42D + 1.0E_h$

Therefore;
U = 1.42(30) + 1.0(400)
= 42.6 kips + 400 k-ft

From Figure 11-30: $P_u = 42.6\text{kips} < P_{bu} = 462\text{kips}$

Thus:

$P_{bu}/(M_{bu} - M_u) = P_u/M_x$
$M_x = (P_u/P_{bu})(M_{bu} - M_u)$
$M_n = M_u + M_x$
$= M_u + (P_u/P_{bu})(M_{bu} - M_u)$
= 807 k-ft + (42.6/462)(1651 - 807)
= 884.8 k-ft Nominal Flexural Moment Strength
Note: 884.8 k-ft > 400 k-ft OK

Note: M/M_{cr} = 884.8/400 = 2.2 > 1.8 OK (recall fully grouted wall)

Load Case 2:

$U = 0.90D + 1.0E$
$U = 0.90(30) + 1.0(400)$
= 27 kips + 400 k-ft

From Figure 11-30: P_u = 27 kips < P_{bu} = 462

Thus:
M_n = 807 + (27/462)(1651 - 807)
= 856 k-ft

Note:

M_n/M_{cr} = 856/400 = 2.14 > 1.8 ....OK

D. Boundary Elements

Section 2108.2.5.6 of the 1997 UBC states that:

"Boundary members shall be provided at the boundaries of shear walls when the compressive strains in the wall exceed 0.0015. The strain shall be determined using factored forces and R_w equal to 1.5"

Note that there is an error in the code since it refers to the obsolete R_w factor, which has been replaced by the R factor in the 1997 UBC. By comparing the values of the new R factor with the old R_w factor, one can conclude that the boundary member requirements should be calculated using an R of 1.1. Since the design forces for the bearing wall were calculated with an R factor of 4.5, the factored loads must be multiplied by 4.5/1.1 = 4.09 in order to determine if the moment capacity of the wall at a maximum compressive strain of 0.0015 is less than that required for boundary members.

To calculate the moment capacity at a maximum compressive strain of 0.0015, we can assume a linear compressive stress-strain relationship for the masonry. So, using a linear strain model, $f_m = 0.75f_m'$ for a strain of 0.0015: See figure 11-31.

_ Must solve for neutral axis (c)

_ Trial and error solution
_ Take moments about plastic centroid

Load Case 1:
U = 12D + 1.0E
= $1.42D + 1.0E_h$
(P_u = 42.6kips and M_u = 400 k-ft)

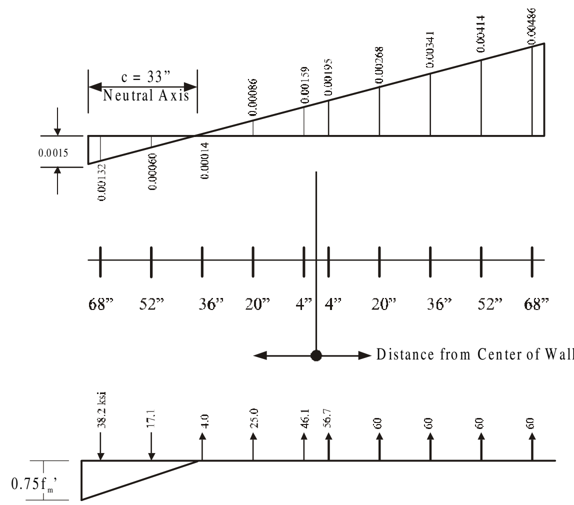


Figure 11-31. Stress/Strain relationship for determining boundary elements in masonry

By trial and error select depth to neutral axis, NA = 33.0 inches (See Figure 11-31 for stress and strain diagrams).

$T = A_s f_s$
= 0.31(4(60) + 56.7 + 46.1 + 25 +4)
= 115.3 kips
$C = A'_s f_s + 0.75f_m' \, cb/2$
= 0.31(38.2 + 17.1)
+ 0.75(1.5)(33)(7.625)(1/2)
= 158.7 kips
C-T = 43.4 kips (P_u = 42.6 kips)…OK

Use: NA = 33.0 inches

Take moments about the center of the wall centroid to determine moment corresponding to $0.75f_m'$. If $4.09M_u$ is less than M_n confinement of vertical steel is not required.

$M_n = A_s f_s$(dist. to Center of Wall)
+ $0.75f'_m(c/2)(L/2-c/3)$

$= 0.31[68(38.2) + 52(17.1) - 36(4.0)$
$-20(25) - 4(46.1) + 4(56.7)$
$+60(20+36+52+ 68)]$
$+ 0.75(1.5)(33/2)[(144/2) - (33/3)]$
$= 441.7$ k-ft $< 4.09M_u = 1636$ k-ft

Thus,
Boundary Elements Required.

E. Shear

1. Shear Demand

Recall : $V_u > \phi V_n$
$V_u > \phi\ (V_m + V_s)$
$V_u = 1.0V_E$
$= 1.0(75$ kips)
$= 75$ kips

2. Shear strength with only CMU (no shear steel)

$V_n = V_m\ (V_s = 0)$
$= C_d A_{mv}(f'_m)^{1/2}$

Where:

$C_d \propto M/Vd$
$d = 12$ ft - $(4/12)$ft $= 11.67$ ft
$V = 75$ kips
$M = 400$ k-ft
$M/Vd = 400/[75(11.67)] = 0.46$ (from Figure 10-26: $C_d = 2.06$)
$A_{mv} = l_w b = 144$ in$(7.625$ in$) = 1098$ in^2

Now:

$V_n \quad = C_d A_{mv}\sqrt{f'_m}$; $C_d = 2.06$
$V_n \quad = 2.06\text{x}1098$ in$^2(1500$ psi$)^{1/2}/1000$ lb/k
$\quad = 87.6$ kips
$V_u \quad > \phi V_n$
$\phi V_n \quad = 0.60(87.6$ kips)
$\quad = 52.6$ kips
$V_u \quad = 75 > 52.6$ ...NG shear reinforcement required

3. Design shear reinforcement to carry total shear (at least majority, authors preference)

$V_u = \phi V_n = \phi V_s$ ......$(V_m = 0)$
$V_u = A_{mv}\rho_n f_y \phi$

Recall:
$\rho_n = V_u/A_{mv}f_y\phi$
$= 75$ kips$/(1098\text{in}^2)(60\ \text{k/in}^2)(0.60)$
$= 0.0019$
Now:
$A_v = 0.0019(12$ in$)(7.625$ in$)$
$= 0.174$ in^2/ft
USE: # 5 @ 16 in. o.c.
$(A_v = 0.23$ in^2/ft > 0.174 in^2/ft)

Thus, the steel can carry all the shear

4.Shear strength of steel only:

$$\phi V_s = \rho_n A_{mn} f_y \phi$$
$$= \frac{0.23(1098\text{in}^2)(60\text{ksi})(0.60)}{(12\frac{\text{in}}{\text{ft}})(7.625\text{in})} = 99.36\text{kips}$$

5.Bottom (L_w) of wall
Shear strength of steel only with ϕ=0.85

$$V_s = 99.36\left(\frac{0.85}{0.60}\right)$$

$= 140.76$ kips > 75 kips OK

Table 11-10. Total Design Base Shear for 3-Story Building Wood Structural Panel Bearing Wall System

Notes	Item/Description	Total Design Base Shear (V) Seismic Zone and Factor				
		1	2A	2B	3	4
		0.075	0.15	0.20	0.30	0.40
1	Cv	0.18	0.32	0.40	0.54	0.64Nv
	I	1.0	1.0	1.0	1.0	1.0
	R	5.5	5.5	5.5	5.5	5.5
2	T EQ. 10-10E	0.256	0.256	0.256	0.256	0.256
	Ca	0.12	0.22	0.28	0.36	0.44Na
3	Nv	-	-	-	-	1.2
3	Na	-	-	-	-	1.0
4	V EQ. 11-10A	0.128W	0.227W	0.284W	0.384W	0.545W
4	V EQ. 11-10B	0.055W*	0.10W*	0.127W*	0.164W*	0.20W*
4	V EQ. 11-10C	0.013W	0.024W	0.031W	0.039W	0.048W
4	V EQ. 11-10D	-	-	-	-	0.070W

Notes: 1. Soil profile type D
2. $T = Ct\ (h_n)3/4 = 0.256$ sec
For $Ct = 0.020$
h_n – 30 feet
3. Seismic source B
Closest distance to seismic source = 5km
4. * = Governs

Table 11-11. Total Design Base Shear for 3- Story Building Masonry Shear Wall Bearing Wall System

Notes	Item/Description	Total Design Base Shear (V) Seismic Zone and Factor				
		1	2A	2B	3	4
		0.075	0.15	0.20	0.30	0.40
1	Cv	0.18	0.32	0.40	0.54	0.64Nv
	I	1.0	1.0	1.0	1.0	1.0
	R	4.5	4.5	4.5	4.5	4.5
2	T EQ. 10-10E	0.256	0.256	0.256	0.256	0.256
	Ca	0.12	0.22	0.28	0.36	0.44Na
3	Nv	-	-	-	-	1.2
3	Na	-	-	-	-	1.0
4	V EQ. 11-10A	0.156W	0.278W	0.347W	0.469W	0.67W
4	V EQ. 11-10B	0.067W	0.122W*	0.156W*	0.20W*	0.244W*
4	V EQ. 11-10C	0.013	0.024W	0.031W	0.039W	0.048W
4	V EQ. 11-10D	-	-	-	-	0.085W

Notes: 1. Soil profile type D
2. * = Governs

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