Chapter 8

Seismic Design of Floor Diaphragms

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Abstract: This chapter surveys the seismic behavior and design of floor and roof diaphragms. Following some introductory remarks, a classification of diaphragm behavior is presented in Section 8.2, and a discussion on the determination of diaphragm rigidity in Section 8.3. Potential diaphragm problems are explained in Section 8.4 where examples are provided to clarify the subject. Provisions of major United States building codes for seismic design of diaphragms are summarized in Section 8.5. Finally, in Section 8.6, the current standard procedures for design of diaphragms are presented via their application in a number of realistic design examples

8.1 INTRODUCTION

The primary function of floor and roof systems is to support gravity loads and to transfer these loads to other structural members such as columns and walls. Furthermore, they play a central role in the distribution of wind and seismic forces to the vertical elements of the lateral load resisting system (such as frames and structural walls). The behavior of the floor/roof systems under the influence of gravity loads is well established and guidelines for use in structural design have been adopted [8-1,8-2].

In the earthquake resistant design of building structures, the building is designed and detailed to act as a single unit under the action of seismic forces. Design of a building as a single unit helps to increase the redundancy and the integrity of the building. The horizontal forces generated by earthquake excitations are transferred to the ground by the vertical systems of the building which are designed for lateral load resistance (e.g. frames, bracing, and walls). These vertical systems are generally tied together as a unit by means of the building floors and roof. In this sense, the floor/roof structural systems, used primarily to create enclosures and resist gravity (or out of plane) loads are also designed as horizontal diaphragms to resist and to transfer horizontal (or in-plane) loads to the appropriate vertical elements.

The analysis and design of a floor or roof deck under the influence of horizontal loads is performed assuming that the floor or roof deck behaves as a horizontal continuous beam supported by the vertical lateral load resisting elements (hereafter referred to as VLLR elements). The floor deck is assumed to act as the web of the continuous beam and the beams at the floor periphery are assumed to act as the flanges of the continuous beam (see Figure 8-1).

Accurate determination of the in-plane shears and bending moments acting on a floor diaphragm, and the corresponding horizontal force distribution among various VLLR elements requires a three dimensional analysis that accounts for the relative rigidity of the various elements including the floor diaphragms. Increasingly, this type of analysis is being performed for design and rehabilitation of major buildings that feature significant plan irregularities. In general, however, some assumptions are made on the horizontal diaphragm rigidity and a relatively simple analysis is performed to determine distribution of lateral forces. Obviously, the accuracy of the results obtained depends on the validity of the assumptions made. In addition, the behavior of certain floor systems such as plywood, metal deck, and precast concrete diaphragms are difficult to model analytically due to their various attachments. In some cases testing may be required to establish the strength and stiffness properties of such systems.

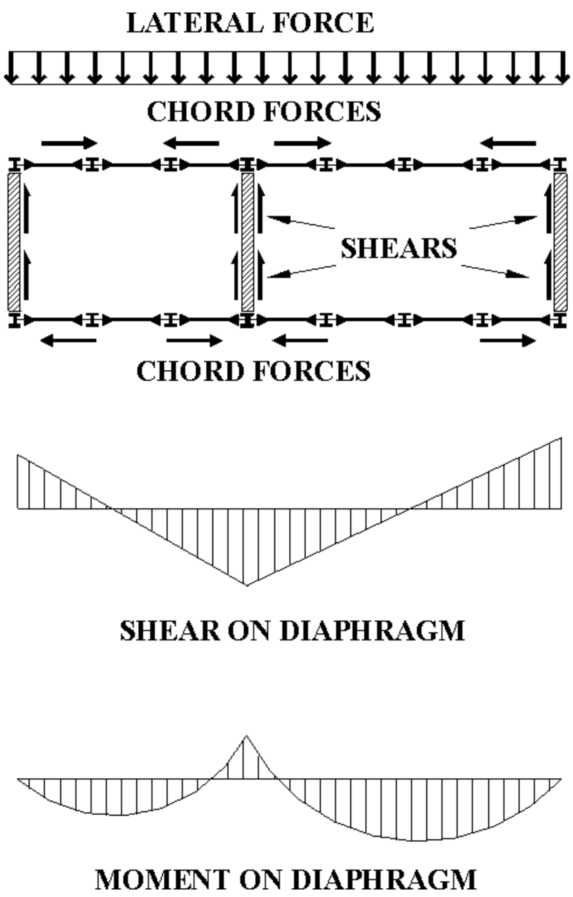


Figure 8-1. Design forces on a diaphragm

While for the great majority of structures, simplified analysis procedures result in a safe design, studies indicate that neglecting the real behavior of floor diaphragms can sometimes lead to serious errors in assessing the required lateral load resistance capacities of the VLLR elements(8-3, 8-4, 8-5).

This chapter addresses the major issues of seismic behavior and design of diaphragms. It starts by classification of diaphragm behavior in Section 8.2, and a discussion on the determination of diaphragm rigidity in Section 8.3. Potential diaphragm problems are explained in Section 8.4 where examples are provided to clarify the subject. Provisions of major United States building codes for seismic design of diaphragms are summarized in Section 8.5. Finally, in Section 8.6, the current standard procedures for design of diaphragms are presented via their application in a number of realistic design examples.

8.2 CLASSIFICATION OF DIAPHRAGM BEHAVIOR

The distribution of horizontal forces by the horizontal diaphragm to the various VLLR elements depends on the relative rigidity of the horizontal diaphragm and the VLLR elements. Diaphragms are classified as "rigid", "flexible", and "semi-rigid" based on this relative rigidity.

A diaphragm is classified as rigid if it can distribute the horizontal forces to the VLLR elements in proportion to their relative stiffness. In the case of rigid diaphragms, the diaphragm deflection when compared to that of the VLLR elements will be insignificant. A diaphragm is called flexible if the distribution of horizontal forces to the vertical lateral load resisting elements is independent of their relative stiffness. In the case of a flexible diaphragm, the diaphragm deflection as compared to that of the VLLR elements will be significantly large. A flexible diaphragm distributes lateral loads to the VLLR elements as a series of simple beams spanning between these elements.

No diaphragm is perfectly rigid or perfectly flexible. Reasonable assumptions, however, can be made as to a diaphragm's rigidity or flexibility in order to simplify the analysis. If the diaphragm deflection and the deflection of the VLLR elements are of the same order of magnitude, then the diaphragm can not reasonably be assumed as either rigid or flexible. Such a diaphragm is classified as semi-rigid.

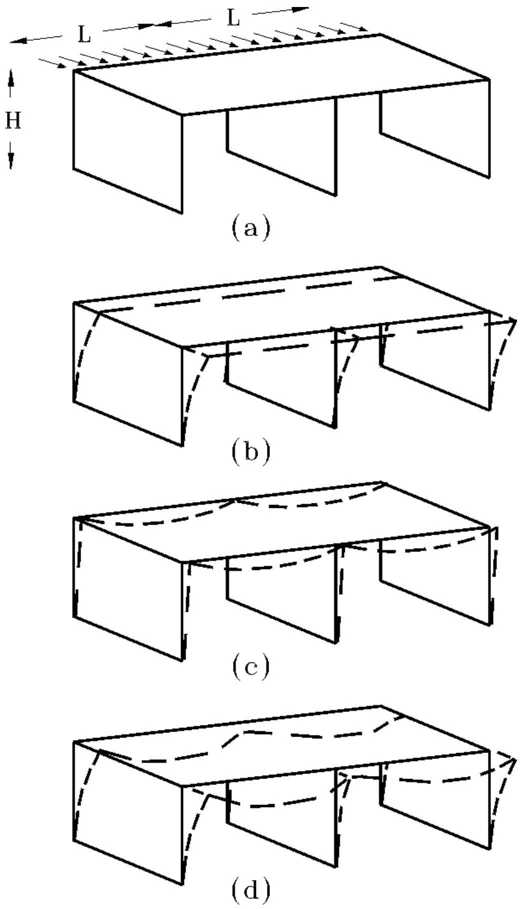


Figure 8-2. Diaphragm behavior. (a) Loading and building proportions. (b) Rigid diaphragm behavior. (c) Flexible diaphragm behavior, (d) Semi rigid diaphragm behavior

Exact analysis of structural systems containing semi-rigid diaphragms is complex, since any such analysis should account for the relative rigidity of all structural elements including the diaphragm. The horizontal load distribution of a semi-rigid diaphragm may be approximated as that of a continuous beam supported on elastic supports. In most cases consisting of semi-rigid diaphragms, assumptions can be made to bound the exact solution without resorting to a complex analysis.

The absolute size and stiffness of a diaphragm, while important, are not the final determining factors whether or not a diaphragm will behave as rigid, flexible, or semi-rigid[(8-3)]. Consider the one-story concrete shear wall building shown in Figure 8-2a. Keeping the width and the thickness of walls and slabs constant, it is possible to simulate rigid, flexible and semi-rigid diaphragms as the wall heights and diaphragm spans are varied. The wall stiffness decreases with an increase in the floor height (H). Similarly, the diaphragm stiffness decreases with an increase in span (L).

The dashed line in Figure 8-2b indicates the deflection of the system under the influence of horizontal forces when the diaphragm is rigid. This can be accomplished by increasing H and decreasing L so that the stiffness of the diaphragm relative to the wall is significantly larger. In such a situation, the deflection of the diaphragm under horizontal loads is insignificant when compared to the deflections of the walls. The diaphragm will move as a rigid body and will force the walls to move together accordingly. The force distribution among the walls will depend only on the relative stiffness of the walls. In Figure 8-2b it is assumed that the applied load and the wall stiffness are symmetric. If this is not the case, in addition to the rigid body translation, the diaphragm will experience rigid body rotation.

Figure 8-2c shows the deflection of the system under the influence of horizontal forces when the diaphragm is flexible. This can be accomplished by decreasing H and increasing L such that the stiffness of the diaphragm when compared to the walls is small. In such a situation, the diaphragm segments between the walls act as a series of simply supported beams and the load distribution to the walls can be determined based on the tributary area of the diaphragm to the wall. Obviously, a flexible diaphragm can not experience the rotation or torsion that occurs due to the rigid body rotation of a rigid diaphragm.

The dashed line in Figure 8-2d indicates the deflection pattern of a semi-rigid diaphragm under the influence of lateral forces. Here the stiffness of the walls and the diaphragm are of the same order. Both wall deflections and diaphragm deflections do contribute to the total system deflection. Determination of exact load distribution among the walls requires a three dimensional analysis of the entire system (including the diaphragm).

8.3 DETERMINATION OF DIAPHRAGM RIGIDITY

In order to estimate the diaphragm rigidity, it is necessary to predict the deflection of the diaphragm under the influence of lateral loads. The various floor and roof systems that have evolved primarily for the purpose of supporting gravity loads do not lend themselves easily to analytical calculation of lateral deflections. Some of the more common floor systems in use today are: (1) cast-in-place concrete; (2) precast planks or Tees with or without concrete topping; (3) metal deck with or without concrete fill and; (4) wood framing with plywood sheathing.

With the single exception of cast-in-place concrete floor system which is a monolithic construction, all the other floor systems mentioned above consist of different units joined together with some kind of connections. In precast concrete construction, adjacent units are generally connected together by welding embedded plates or reinforcing bars. This will help the units to deflect vertically without separation while providing some diaphragm action. The strength and rigidity of such a diaphragm will depend to a great extent on the type and spacing of connections. Analytical computation of deflections and stiffness of such a diaphragm is complex. As an alternative, a bonded topping slab on precast floor or roof can be provided with sufficient reinforcement to ensure continuity and resistance for shear transfer mechanism. In floor systems consisting of metal decks, the deck is welded intermittently to the supports below. Adjacent units of the deck are connected together by means of button punching or welding. Here again, the diaphragm stiffness is directly related to the spacing and type of connections. In the wood construction, the plywood sheathing is nailed directly to the framing members. Again, strength and stiffness depends on the spacing of the nails and whether or not the diaphragm is blocked.

It is general practice to consider the diaphragms made of cast in place concrete, precast with concrete topping, and metal deck with concrete fill as rigid while the diaphragms consisting of precast planks without concrete topping, metal deck without concrete fill, and plywood sheathing as flexible. This classification is valid for most cases. Gross errors in force distribution, however, can occur if the above assumption is used without paying attention to the relative rigidity of the VLLR elements and the diaphragm(8-3, 8-4, 8-5).

Metal deck manufacturers have established test programs to provide strength and deflection characteristics of various metal decks and various connection patterns(8-6, 8-7). Similarly, the Uniform Building Code provides an empirical formula to compute plywood diaphragm deflections and tables to establish the strength of such diaphragms.

8.4 SIGNIFICANT FACTORS AFFECTING DIAPHRAGM BEHAVIOR

Identifying every situation where special attention should be given to the design and detailing of floor diaphragms requires substantial experience and a good amount of engineering judgement. Certain cases, however, more often than not, require special attention and in this section guidelines for identification of such cases are provided.

In general, low-rise buildings and buildings with very stiff vertical elements such as shear walls are more susceptible to floor diaphragm flexibility problems than taller structures.

In buildings with long and narrow plans, if seismic resistance is provided either by the end walls alone, or if the shear walls are spaced far away from each other, floor diaphragms may exhibit the so-called bow action (see Figure 8-3). The bow action subjects the end walls to torsional deformation and stresses. If sufficient bond is not provided between the walls and the diaphragm, the two will be separated from each other starting at the wall corners. This separation results in a dramatic increase in the wall torsion and might lead to collapse.

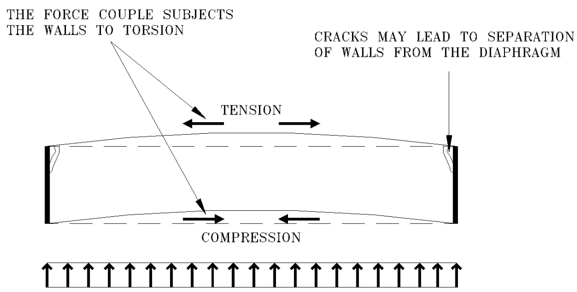


Figure 8-3 A plan showing how the so-called bow action subjects the end walls to torsion

The Arvin High School Administrative Building in California which suffered extensive damage during the Kern County earthquake of July 21, 1952 is a good example in this regard. Schematic plans and elevations of this building are shown in Figure 8-4. An analytical study of this building by Jain(8-8) indicated that the two lowest natural frequencies of the building were close to the fundamental frequencies of the floor and roof diaphragms modeled as simply supported beams. When an analytical model of the building was subjected to a 0.20g constant spectral acceleration, with four translational modes considered, the two diaphragm modes represented 74 percent of the sum of the modal base shears. As documented by Steinburgge (8-9) diaphragm deflections caused a separation between the roof diaphragm and the wall corners at the second story wall located at the west end of the building. This action subjected the wall to significant torsional stresses beyond its capacity.

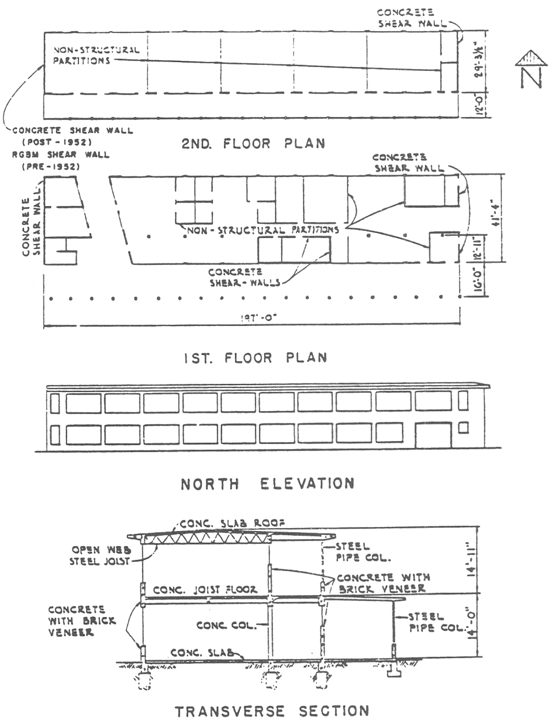


Figure 8-4. Plan and elevation of the Arvin High School Administrative Building [8-8]

Another potential problem in diaphragms can be due to any abrupt and significant changes in a wall stiffness below and above a diaphragm level, or any such changes in the relative stiffness of adjacent walls in passing through one floor level to another (Figure 8-5). This can cause high shear stresses in the floor diaphragm and/or a redistribution of shear forces among the walls.

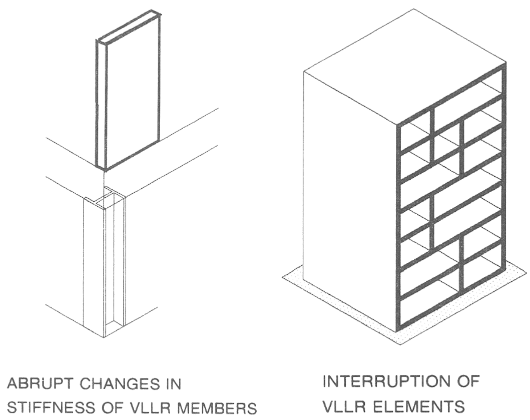


Figure 8-5. Abrupt changes in stiffness and location of VLLR elements can cause drastic redistribution of forces

As an example consider the three story concrete shear wall building shown in Figure 8-6. The concrete floor diaphragms are eight inches thick. A set of static lateral forces of 24 kips, 48 kips and 73 kips are applied at the center of mass of the first, second, and third levels, respectively. The base of the building is assumed to be fixed and the reported results are based on an elastic analysis. An analysis based on a rigid-diaphragm assumption and a finite element analysis considering the un-cracked diaphragm stiffness, yield very close results. However, if we make a simple change in the elevation of the building by moving the opening at the second level, from the wall on line A to the wall on line B (Figure 8-7), the results of the two methods will be markedly different (see Figure 8-8). For example, the rigid diaphragm assumption suggests that the shear force in wall A is reduced from 94.3 kips above the first floor diaphragm to 26 kips below this level, while the finite element model of the building, shows that such a large portion of the shear force is not transferred away from this wall by the floor diaphragm.

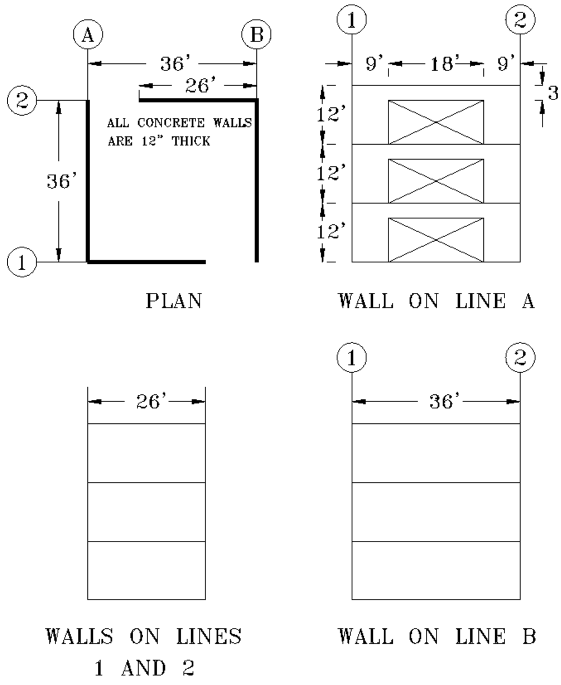


Figure 8-6. Plan and elevation of a simple three story shear wall building (Note the uniform stiffness along the height of walls on lines A and B.)

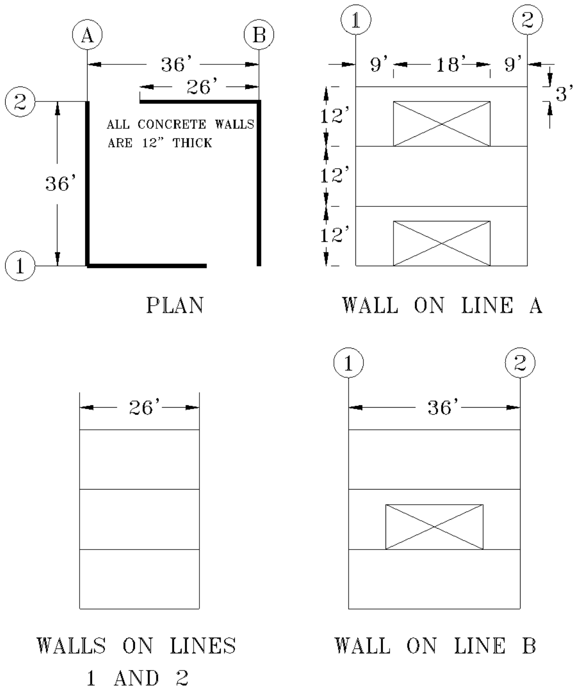


Figure 8-7. Altered plan and elevation of the three story shear wall building (Note the abrupt change of stiffness along the height of walls on lines A and B.)

In buildings with significant plan irregularities, such as multi-wing plans, L-shape, H-shape, V-shape plans, etc. (Figure 8-9) particular attention should be paid to accurately access the in-plane diaphragm stress at the joints of the wings and to design for them. In this type of buildings, the fan-like deformations in the wings of diaphragm can lead to a stress concentration at the junction of the diaphragms (see Figure 8-10). If these stress concentrations are not accounted for, serious problems can arise. For the case of reinforced concrete diaphragms, it is recommended to limit the maximum compressive stresses to $0.2f'_c$. Alternatively, special transverse reinforcement can be provided**.** In some cases the diaphragm stresses at the junctions may be so excessive that a feasible diaphragm thickness and reinforcement can not be accommodated. In these cases the wings should be separated by seismic joints. One example for this type of problems was provided by the West Anchorage High School Building in Anchorage, Alaska, which suffered severe damage during the

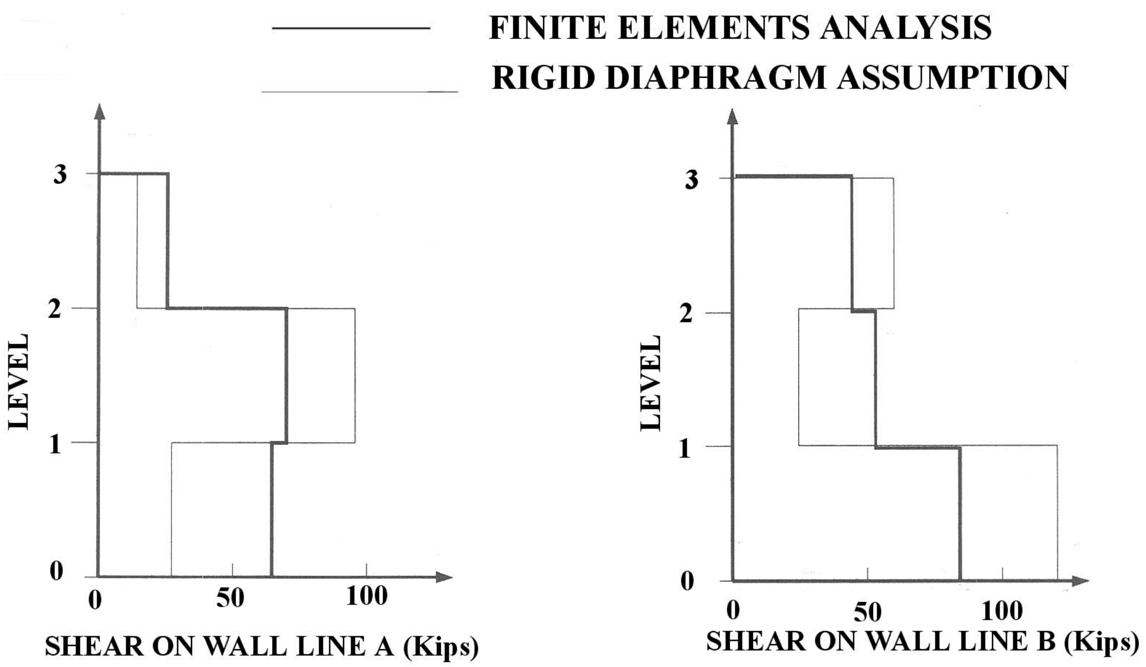


Figure 8-8. Computed shears of walls on lines A and B

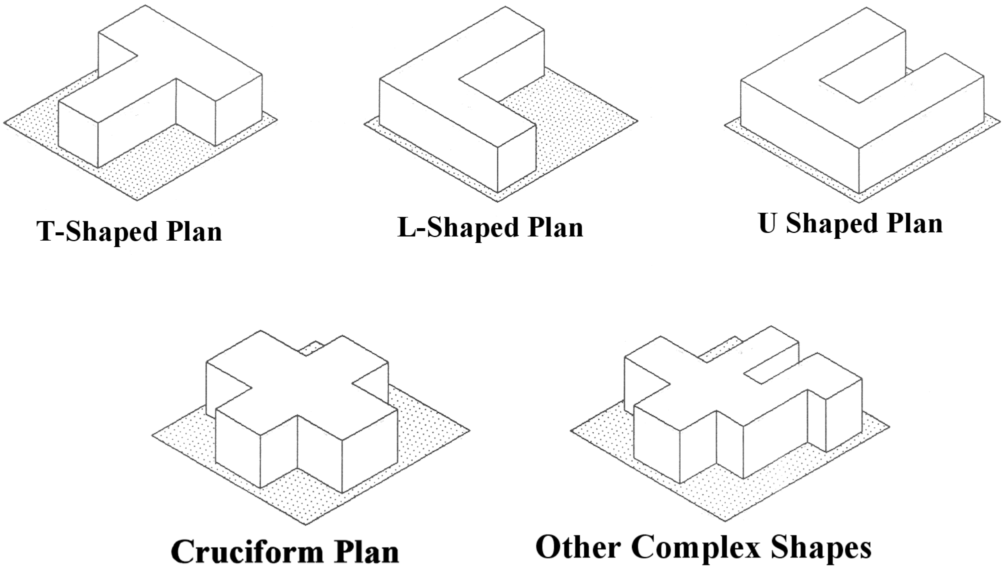


Figure 8-9. Typical plan Irregularities

Alaskan earthquake of March 27, 1964 (see Figure 6-15).

Other classes of buildings deserving special attention to diaphragm design include those with relatively large openings in one or more of the floor decks (Figure 8-11) and tall buildings resting on a significantly larger low-rise part (Figure 8-12). In the later case, the action of the low-rise portion as the shear base and the corresponding redistribution of shear forces (kick-backs) may subject the diaphragm located at the junction of the low-rise and high-rise parts (and sometimes a number of floor diaphragms above and below the junction) to some significant in-plane shear deformations.

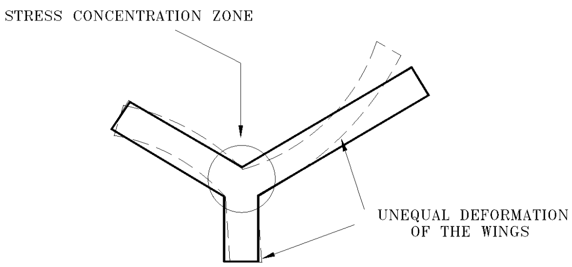


Figure 8-10. Fan-like deformation of wings causes stress concentration at the junction

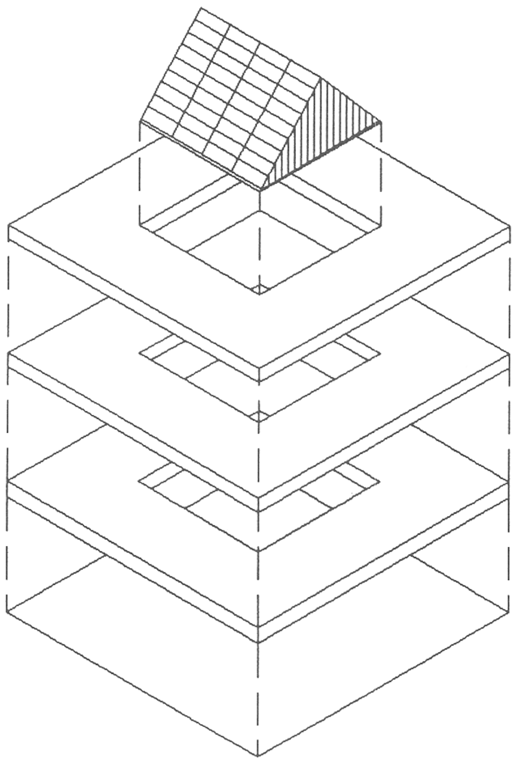


Figure 8-11. Significant floor openings are cause for concern

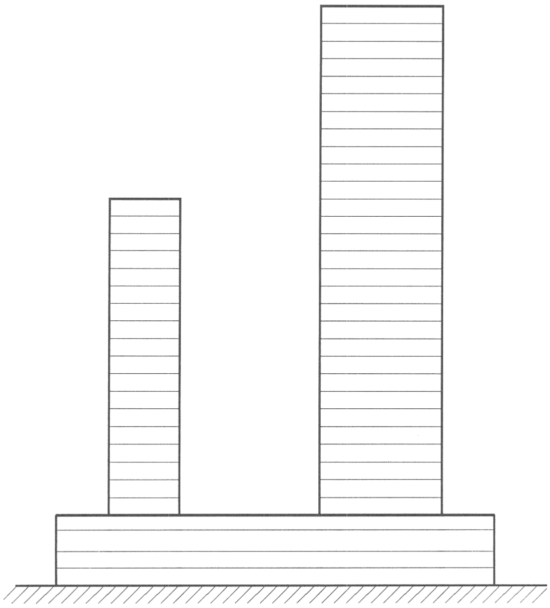


Figure 8-12. Elevation of towers on an expanded low-rise base

8.5 CODE PROVISIONS FOR DIAPHRAGM DESIGN

8.5.1 UBC-97, ASCE 7-95, and IBC-2000 Provisions

Diaphragm design provisions contained in the UBC-97, ASCE 7-95 and IBC-2000 are similar but vary in the degree of detailed information they provide. All these model codes contain a clause limiting the in-plane deflection of the floor diaphragms as follows:

The deflection in the plane of the diaphragm, as determined by engineering analysis, shall not exceed the permissible deflection of attached elements. Permissible deflection shall be that deflection which will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

UBC-97 requires the roof and floor diaphragms to be designed to resist the forces determined in accordance with:

$$F_{px} = \frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} W_i} w_{px} \tag{8-1}$$

The minimum value of F_{px} to be used in analysis is $0.5C_aIw_{px}$. However, it need not exceed $1.0C_aIw_{px}$ where:

C_a = seismic coefficient (see section 5.3)
I = Importance factor (see Section 5.3)
i = Index identifying the ith level above the base
x = Floor level under design consideration
W = Total seismic dead load of the building
F_i = the lateral force applied to level i.
F_t = that portion of the base shear, V, considered concentrated at the top of the structure in addition to F_n
W_i = the portion of W at level i.
w_{px} = the weight of the diaphragm and the elements tributary thereto at level x, including 25% of the floor live load in storage and warehouse occupancies.

UBC-97 makes an exception for buildings of no more than three stories in height excluding basements, with light-frame construction and for other buildings not more than two stories in height excluding basements, diaphragm design forces may be estimated using a simplified procedure as follows:

$$F_{px} = \frac{3.0C_a}{R} w_{px} \tag{8-2}$$

where R is the numerical coefficient representative of the inherent overstrength and global ductility of the lateral-force–resisting system as described in Chapter 5. In the above equation, F_{px} should not be less than $0.5C_aw_{px}$ and need not exceed C_aw_{px}.

ASCE 7-95 requires the floor and roof diaphragms to be designed for a minimum seismic force equivalent to 50% of the seismic coefficient C_a times the weight of the diaphragm. Diaphragm connections can be positive connections, mechanical or welded.

IBC-2000 requires the roof and floor diaphragm to be designed to resist the force F_p as follows:

$$F_p = 0.2 I_E S_{DS} w_p + V_{px} \tag{8-3}$$

where:
F_p = The seismic force induced by the parts.
I_E = Occupancy importance factor (see Section 5.4.2).
S_{DS} = The short period site design spectral response acceleration coefficient (see Section 5.4.6).
w_p= The weight of the diaphragm and other elements of the structure attached to.
V_{px} = The portion of the seismic shear force at the level of diaphragm, required to be transferred to the VLLR elements because of the offsets or changes in stiffness of the VLLR elements above or below the diaphragm.

Notice that vertical distribution of lateral forces in IBC-2000 takes place in accordance with Equations 5-25 and 5-26 (see Section 5.4.13) which do not necessarily conform with the distributions obtained according to the UBC-97 formulas.

IBC-2000 provisions also require that diaphragms be designed to resist both shear and bending stresses resulting from these forces. Ties or struts should be provided to distribute the wall anchorage forces.

Obviously, the floor or roof diaphragm at every level need to be designed to span horizontally between the VLLR elements and to transfer the force F_{px} to these elements (see Figure 8-13a). All contemporary model codes require the diaphragms to be designed to transfer lateral forces from the vertical lateral load resisting elements above the diaphragm to the other VLLR elements below the diaphragm due to offsets in the placement of VLLR elements or due to changes in stiffness of these elements. For example, in Figure 8-13b, the force P_1 has to be transferred by the diaphragm to the VLLR elements below the diaphragm since the VLLR element above the diaphragm has been discontinued at this level. In addition, the force P_2 from the other VLLR element above, has to be redistributed among the VLLR elements below the diaphragm. The diaphragm must be designed to transfer these additional loads.

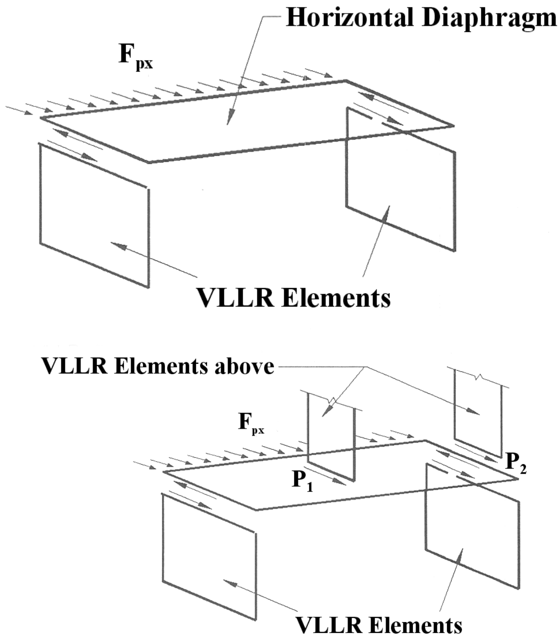


Figure 8-13. Code provisions for diaphragm design

As per UBC-97, additional requirements for the design of diaphragms are as follows:

Diaphragms supporting concrete or masonry walls should be designed with continuous ties between diaphragm chords to distribute the anchorage forces into the diaphragm. Added chords of subdiaphragms may be used to form subdiaphragms to transmit the anchorage forces to the main continuous crossties. The length to width ratio of the wood structural subdiaphragms should not exceed 2½ to 1. Diaphragm deformations should also be considered in the design of supported walls. Furthermore, in design of wood diaphragms providing lateral support for concrete or masonry walls in seismic zones 2, 3, and 4, anchorage should not be accomplished by use of toenails or nails subjected to withdrawal. In addition, wood framing should not be used in cross-grain bending or tension.

For structures in Seismic Zones 3 and 4 having a plan irregularity of type 2 in Table 5-10, diaphragm chords and drag members should be designed considering independent movement of the projecting wings of the structure. Each of these diaphragm elements should be designed for the more severe of the following two conditions:

1. Motion of the projecting wings in the same direction; and
2. Motion of the projecting wings in opposing directions.

This requirement is considered satisfied if a three-dimensional dynamic analysis according to the code provisions is performed.

As a requirement for flexible diaphragms, the design seismic forces providing lateral support for walls or frames of masonry or concrete are to be based on Equation 8-1 and determined with the value of the response modification factor, R, not exceeding 4.0.

8.5.2 ACI 318-95 Provisions

The thickness of concrete slabs and composite topping slabs serving as structural diaphragms used to transmit earthquake forces cannot be less than 2 inches. This requirement reflects current usage in joist and waffle systems and composite topping slabs on precast floor and roof systems. Thicker slabs are required when the topping slab does not act compositely with the precast system to resist the design seismic forces.

A composite cast-in-place concrete topping slab on precast units is permitted to be used as a structural diaphragm provided the topping slab is reinforced and its connections are proportioned and detailed for complete transfer of forces to the elements of the lateral force resisting system. A bonded topping slab is required so that the floor or roof system can provide restraint against slab buckling. Reinforcement is required to ensure the continuity of the shear transfer across precast joints. The connection requirements are to promote provisions of a complete system with necessary shear transfers. Obviously, the cast-in-place topping on a precast floor or roof system can be used without the composite action provided that the topping alone is proportioned and detailed to resist the design forces. In this case, a thicker topping slab has to be provided.

The shear strength requirements are the same as those for slender structural walls (see Chapter 10). The term A_{cv} in the equation for calculating the nominal shear strength refers to the thickness times the width of the diaphragm.

8.6 DESIGN EXAMPLES

As discussed in Chapter 6, it is desirable from the structural point of view to have regular buildings with minimal offset in the location of VLLR elements and without sudden changes in stiffness from floor to floor. Quite often, however, other requirements of the project (such as architectural considerations) control these parameters and the structural engineer is faced with buildings that are considered irregular in terms of seismic behavior and design.

Diaphragm design consists primarily of the following tasks:

1. Determining the lateral force distribution on the diaphragm and computing diaphragm shears and moments at different locations.
2. Providing adequate in-plane shear capacity in the diaphragm to transfer lateral forces to the VLLR elements.
3. Providing suitable connection between the diaphragm and the VLLR elements.
4. Design of boundary members or reinforcement to develop chord forces, and
5. Computing diaphragm deflections, when necessary, to ascertain that the diaphragm is stiff enough to support the curtain walls, etc. without excessive deflections.

In addition, the diaphragm must be designed and detailed for local effects caused by various openings such as those caused by the elevator shafts. Parking structure diaphragms with ramps are a special case of diaphragms with openings. The effect of the ramp attachment to floors above and below the ramp should be considered in lateral force distribution, especially for non-shear wall buildings.

In this section, the current design procedures for seismic design of floor diaphragms are demonstrated by means of four design examples which are worked out in detail. In the first example, a concrete floor diaphragm at the top of a parking level under a two story wood framed apartment building is designed. The second example explains diaphragm design for a four story concrete parking structure, which has setbacks in elevation of the building and the shear walls. In the third example, the metal-deck diaphragm of a three story steel framed office building is designed. Finally, the fourth example, explains the wood diaphragm design for a typical one story neighborhood shopping center.

EXAMPLE 8-1

It is proposed to build a two story wood framed apartment building on top of one story concrete parking. The building will be located in a zone of high seismicity. The concrete floor supporting the wood construction (see Figure 8-14) will be a 14 inch thick, hard rock concrete, flat plate ($f_c{}' = 4000$ lb/in^2). The lateral force resisting system for the concrete parking structure consists of concrete block masonry walls ($f_m{}' = 3000$ lb/in^2). Given that the superimposed dead load from the two story wood framing above is 65 pounds per square foot, design the concrete diaphragm per typical requirements of the modern model codes. Floor to floor height is 10 feet. Assume that the structural analysis of the building has produced a seismic base shear coefficient of 0.293 for strength design purposes (V=0.293W).

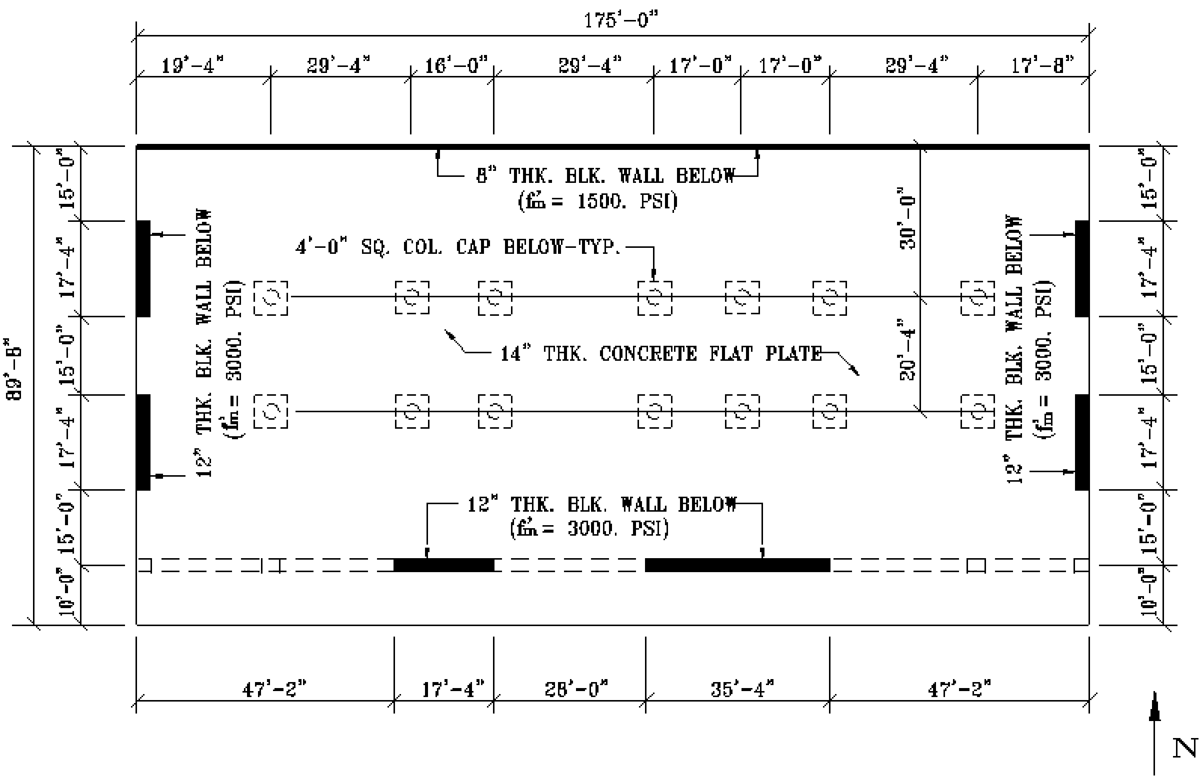


Figure 8-14. Second floor framing plan (Example 8-1)

SOLUTION

- Dead loads and seismic shears:

Superimposed dead load from wood framing above = 65 lb/ft^2
Concrete slab at 150 lb/ft^3 = (14/12)(150) = 175 lb/ft^2
Miscellaneous (M + E + top half of column weights) = 10 lb/ft^2
Total floor weight = (175)(89.66)(65+175+10) = 3922.6 kips

N-S walls:

12-in walls at 124 lb/ft^2 = 4(5)(17.33)(0.124)= 43 kips

E-W walls:

8" wall at 78 lb/ft^2 =
(5)(175)(0.078) = 68.25 kips
12" walls at 124 lb/ft^2 =
(5)(17.33+35.33)(0.124) = 32.65 kips

The weight of the walls parallel to the applied seismic force does not contribute to the diaphragm shears. However, in general, they are included conservatively in the design of concrete floor diaphragms. In this example, the weight of the walls parallel to the applied seismic force is not included in calculating diaphragm shears.

E-W weight = W_x = 3922.6 + 43 = 3965.6 kips
N-S weight = W_y = 3922.6 +68.25 + 32.65 = 4023.5 kips

- Base shears:

F_{Py} =0.293(3965.6)=1161.9 kips (in y direction)

F_{Px} =0.293(4023.5)=1178.9 kips (in x direction)

- Center of mass (see Figure 8-15):

In computing the location of the center of mass of the walls it is generally assumed that one half of the height of a wall above and below the diaphragm will contribute to the mass of each floor. The parameters needed for determination of the center of mass of the walls are calculated in Table 8-1. Therefore, the center of mass of the walls is located at:

$$x_1 = \frac{\sum xW}{\sum W} = \frac{12{,}703.0}{143.85} = 88.31\,\text{ft}$$

$$y_1 = \frac{\sum yW}{\sum W} = \frac{8{,}564.1}{143.85} = 59.53\,\text{ft}$$

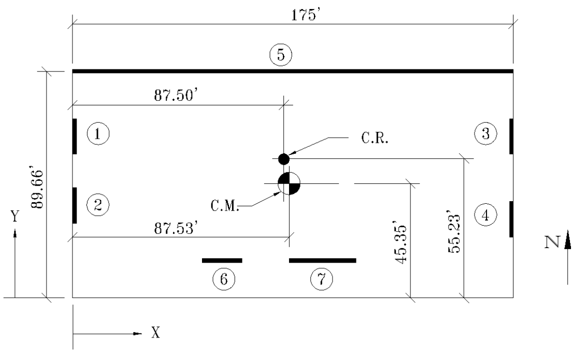


Figure 8-15. Locations of centers of mass and rigidity.

Since the slab is of uniform thickness, the center of mass of the floor coincides with its geometric centroid:

x_2 = 87.50 ft

y_2 = 44.83 ft

Location of the combined center of mass:

$$x_m = \frac{143.9(88.31) + 3922.6(87.5)}{143.9 + 3922.6} = 87.53\,\text{ft}$$

$$y_m = \frac{143.9(59.53) + 3922.6(44.83)}{143.9 + 3922.6} = 45.35\,\text{ft}$$

- Center of rigidity:

For a cantilever wall (see Figure 8-16):

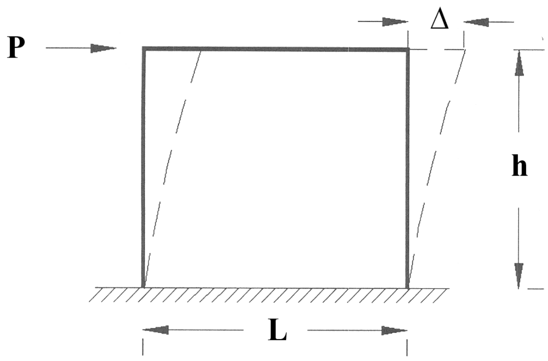


Figure 8-16. Deformation of a cantliever wall panel

$$\Delta = \frac{Ph^3}{3EI} + \frac{1.2Ph}{AG}$$

Denoting wall thickness by t and assuming G = 0.40E for masonry, this relation may be rewritten as:

$$\Delta = \frac{4P(h/L)^3}{Et} + \frac{3P(h/L)}{Et}$$

The relative wall rigidities, R = 1/D, may be computed assuming a constant value of P, say P=1,000,000 pounds. Using the parameters generated in Tables 8-2 and 8-3, the location of the center of rigidity is established as:

$$x_r = \frac{\sum xR_y}{\sum R_y} = \frac{4886.0}{55.84} = 87.50 \text{ ft}$$

$$y_r = \frac{\sum yR_x}{\sum R_x} = \frac{6506.93}{117.8} = 55.23 \text{ ft}$$

- Torsional eccentricity:

$e_x = x_r - x_m = 87.5 - 87.53 \approx 0$ ft

$e_y = y_r - y_m = 55.23 - 45.35 = 9.88$ ft

Table 8-1 Center of Mass Calculations for Example *8*-1

Wall No.	Weight, Lb/ft^2	Length, ft	Area, ft^2	Weight, Kips	Dir.	*x*, ft	*xW*, ft-kips	*y*, ft	*yW* ft-kips
1	124	17.33	86.65	10.74	*y*	0.50	5.37	66.00	708.84
2	124	17.33	86.65	10.74	*y*	0.50	5.37	33.67	361.62
3	124	17.33	86.65	10.74	*y*	174.50	1,874.10	66.00	708.84
4	124	17.33	86.65	10.74	*y*	174.50	1,874.10	33.67	361.62
5	78	175.00	875.00	68.25	*x*	87.50	5,971.88	89.33	6,096.78
6	124	17.33	86.65	10.74	*x*	55.84	559.72	10.00	107.40
7	124	35.33	176.70	21.90	*x*	110.16	2,412.50	10.00	219.00
Σ				143.85			12,703.		8,564.

Table 8-2. Relative Rigidity of the Walls

Wall No.	Height, ft	Length, ft	*H/L*	*E*, lb/in^2	*t*, in.	Δ	*R* = 1/Δ
1	10	17.33	0.5770	3,000,000	11.625	0.0716	13.96
2	10	17.33	0.5770	3,000,000	11.625	0.0716	13.96
3	10	17.33	0.5770	3,000,000	11.625	0.0716	13.96
4	10	17.33	0.5770	3,000,000	11.625	0.0716	13.96
5	10	175.00	0.0571	1,500,000	7.625	0.0150	66.67
6	10	17.33	0.5770	3,000,000	11.625	0.0716	13.96
7	10	35.33	0.2830	3,000,000	11.625	0.0269	37.17

Table 8-3. Center-of-Rigidity Calculations for Example 8-1

Wall No.	Dir.	x	y	R_x	R_y	xR_y	yR_x
1	y	0.50	----	----	13.96	6.98	----
2	y	0.50	----	----	13.96	6.98	----
3	y	174.50	----	----	13.96	2,436.02	----
4	y	174.50	----	----	13.96	2,436.02	----
5	x	----	89.33	66.67	----	----	5,995.63
6	x	----	10.00	13.96	----	----	139.60
7	x	----	10.00	37.17	----	----	371.70
Σ				117.80	55.84	4,886.00	6,506.93

Table 8-4. Wall Shear for Seismic forces in the N-S Direction

Wall No	R_x	R_y	d_x, ft	d_y, ft	Rd	Rd^2	F_v, kips	F_{t-1}, kips	F_{t-2}, kips	$F_{total-1}$ kips	$F_{total-2}$ kips	F_{design} kips
1	0	13.96	-87.00	-----	-1214.52	105,663	294.70	-20.70	20.70	274.00	315.40	315.40
2	0	13.96	-87.00	-----	-1214.52	105,663	294.70	-20.70	20.70	274.00	315.40	315.40
3	0	13.96	87.00	-----	1214.52	105,663	294.70	20.70	-20.70	315.40	274.00	315.40
4	0	13.96	87.00	-----	1214.52	105,663	294.70	20.70	-20.70	315.40	274.00	315.40
5	66.67	0	-----	34.10	2273.45	77,524	0.00	38.80	-38.80	38.80	-38.80	38.80
6	13.96	0	-----	-45.23	-631.41	28,559	0.00	-10.80	10.80	-10.80	10.80	10.80
7	37.17	0	-----	-45.23	-168.20	76,041	0.00	-28.70	28.70	-28.70	28.70	28.70
	Σ											1179.50

Modern codes generally require shifting of the center of mass of each level of the building a minimum of 5% of the building dimension at that perpendicular to the direction of force in addition to the actual eccentricity:

$e_x = 0.05(175) = \pm 8.75$ ft

$e_y = 9.88 \pm 0.05(89.67) = 14.36$ ft or 5.4 ft

- Torsional Moments:

$T_y = F_{Py}\, e_x = 1178.9(\pm 8.75) = \pm 10315.4$ ft-k
$T_{x+} = F_{Px}\, e_{y+} = 1161.9(14.36) = 16{,}684.9$ ft-k

$T_{x-} = F_{Px}\, e_{y-} = 1161.9(5.40) = -6{,}274.2$ ft-k

In-plane forces in the walls due to direct shear are computed from

$$F_{vx} = V_x \frac{R_x}{\sum R_x}$$

$$F_{vy} = V_y \frac{R_y}{\sum R_y}$$

and the in-plane wall forces due to torsion are computed from

$$F_{tx} = T_x \frac{Rd}{\sum Rd^2}$$

$$F_{ty} = T_y \frac{Rd}{\sum Rd^2}$$

where d is the distance of each wall from the center of rigidity. Using these formulas, the wall forces for seismic force acting in the N-S and E-W directions are calculated and reported in Tables 8-4 and 8-5, respectively. Note that the contribution of torsion, if it reduces the magnitude of the design wall shears, is ignored. The design shear forces are summarized in Table 8-6.

Table 8-5. Wall Shear for Seismic forces in the E-W Direction

Wall No	R_x	R_y	d_x, ft	d_y, ft	Rd	Rd^2	F_v, kips	F_{t-1}, kips	F_{t-2}, kips	$F_{total-1}$ kips	$F_{total-2}$ kips	F_{design} Kips
1	0	13.96	-87.00	-----	-1214.52	105,663	0.00	33.52	12.60	33.52	12.60	33.52
2	0	13.96	-87.00	-----	-1214.52	105,663	0.00	33.52	12.60	33.52	12.60	33.52
3	0	13.96	87.00	-----	1214.52	105,663	0.00	-33.52	-12.60	-33.52	-12.60	33.52
4	0	13.96	87.00	-----	1214.52	105,663	0.00	-33.52	-12.60	-33.52	-12.60	33.52
5	66.67	0	-----	34.10	2273.45	77,524	657.60	594.85	-23.60	594.85	634.00	634.00
6	13.96	0	-----	-45.23	-631.41	28,559	137.70	155.10	6.60	155.10	144.30	155.10
7	37.17	0	-----	-45.23	-168.20	76,041	366.60	413.00	17.50	413.00	384.10	413.00
Σ												1,162.95

Table 8-6. Shear Design Forces (kips)

Wall No	Wall L ft.	E-W Load	N-S Load	Max Load
1	17.33	33.52	315.40	315.40
2	17.33	33.52	315.40	315.40
3	17.33	33.52	315.40	315.40
4	17.33	33.52	315.40	315.40
5	175.00	634.00	38.80	634.00
6	17.33	155.10	10.80	155.10
7	35.33	413.00	28.70	413.00

- Diaphragm design for seismic force in the N-S direction:

The wall forces and the assumed direction of torque due to the eccentricity are shown in Figure 8-17. Using this information, the distribution of the applied force on the diaphragm may be calculated. Denoting the left and right diaphragm reactions per unit length by V_L and V_R, from force equilibrium (see Figure 8-18),

$$V_L\frac{175}{2} + V_R\frac{175}{2} = 1179.5\,Kips$$

or

$$V_L + V_R = 13.48 \qquad (I)$$

from moment equilibrium:

$$\left(\frac{175}{3}\right)\frac{175}{2}V_L + 2\left(\frac{175}{3}\right)\frac{175}{2}V_R = 1179.5(96.25)$$

or

$$V_L + 2V_R = 22.24 \qquad (II)$$

Solving equations I and II for V_L and V_R yields:

$$V_L = 4.72 \text{ k/ft, and}$$
$$V_R = 8.76 \text{ k/ft.}$$

The mid-span diaphragm moment[1] (Figure 8-18) is:

M = 548(87.5) – 19.4(79.66) – 4.72(87.5)(58.33)/2 – 6.74(87.5)(29.17)/2 = 25,758 ft-kips

Check slab shear stress along walls 1 and 2:

L = 17.33 ft, t = 14 inches

Slab capacity without shear reinforcement

$$\phi V_c = \phi(2)\sqrt{f_c'}bt$$

$$= \frac{0.85(2)\sqrt{4000}(14)(17.33)(12)}{1000}$$

[1] The mid-span moment has been used in this example to demonstrate the chord design procedures. This moment, however, is not necessarily the maximum moment. In a real design situation the maximum moment should be calculated and used for the chord design.

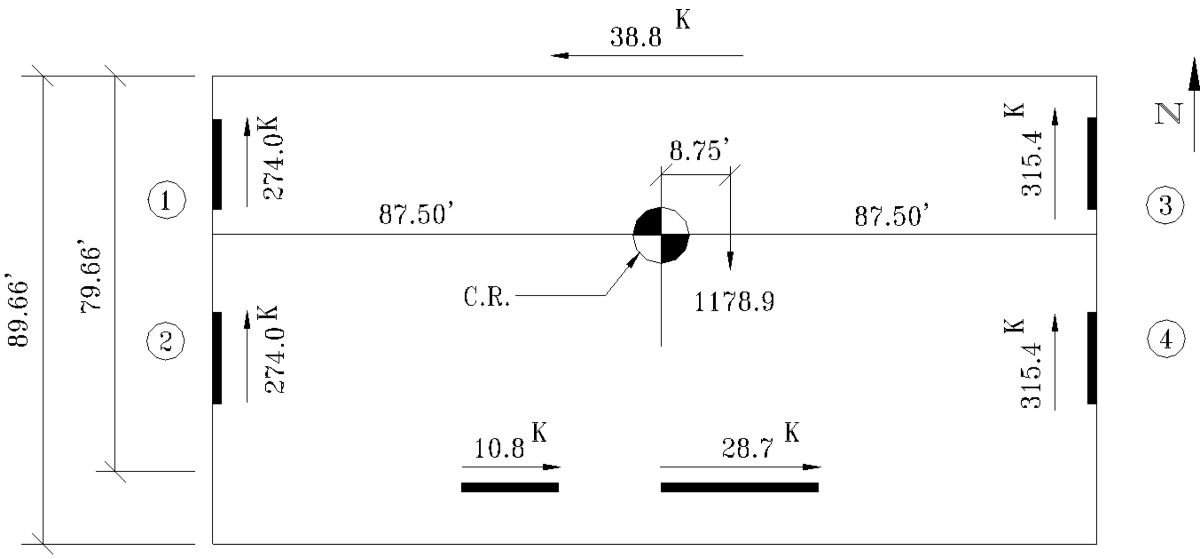


Figure 8-17. Design wall forces for seismic load in the N - S direction

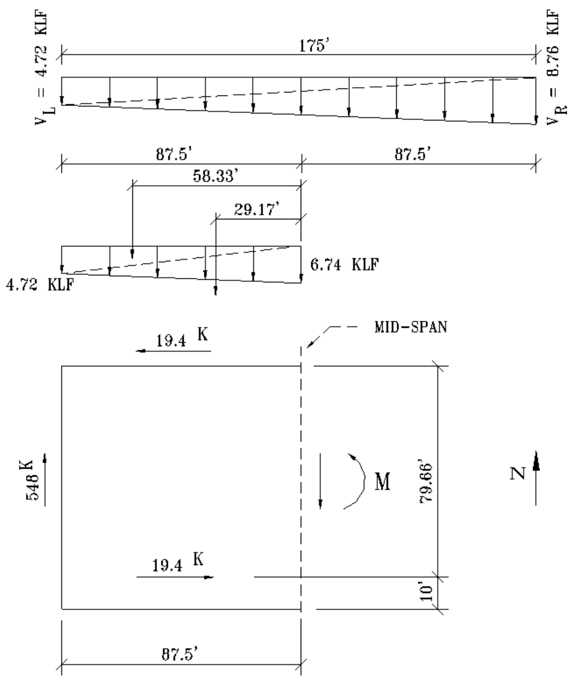


Figure 8-18 Force distribution and diaphragm moments for seismic load in the N-S direction.

= 313 kips ≈ 315.4 O.K.

Chord Design:

$$T_u = \frac{M^1}{d} = \frac{1.0(25{,}758)}{(89.66-4.0)} = 301\,kips$$

$$A_s = \frac{T_u}{\phi f_y} = \frac{301}{0.9(60)} = 5.57\,in^2$$

Provide 6#9 chord bars (A_s = 6.0 in^2) along the slab edges at the North and South sides of the building. Here, we have assumed that the chord bars will be placed over a 4 ft. strip of the slab.

- Diaphragm design for seismic force in the N-S direction:

A sketch of the wall forces indicating the assumed direction of the torque due to eccentricity is shown in Figure 8-19.

Similar to the N-S direction, the force and moment equilibrium equations may be used to obtain the distribution of lateral force on the diaphragm:

[1] Arguably, strict conformity with the UBC-97 would require this moment to be multiplied by a factor of 1.1 (UBC-97 Sec. 1612.2.1 Exception 2). No such requirement exists, however, in the IBC-2000 which replaces UBC-97.

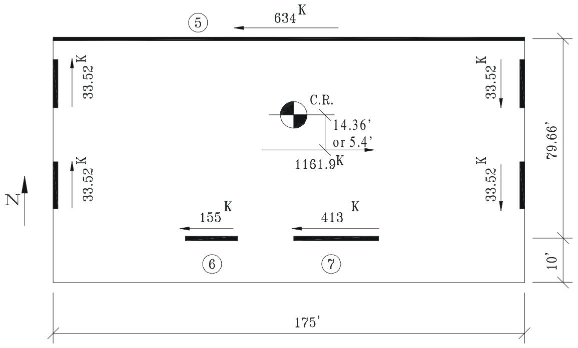


Figure 8-19. Design wall forces for seismic load in the E-W direction

$$V_L \frac{89.66}{2} + V_R \frac{89.66}{2} = 1162.95\, Kips$$

or

$$V_L + V_R = 25.95 \qquad (III)$$

and

$$V_L \frac{89.66}{2}(29.89) + V_R \frac{89.66}{2}(59.77)$$

$$= 1162.95(45.35)$$

or

$$V_L + 2V_R = 39.36 \qquad (IV)$$

solving equations III and IV for V_L and V_R:

V_L = 12.54 k/ft and V_R = 13.41 k/ft

The mid-span diaphragm moment (Figure 8-20):

M = 568(34.83) + 33.52(175) – 12.55(44.83)(29.83)/2 – 12.98(44.83)(14.94)/2 = 12,916 ft-kips

Similarly, diaphragm moments at other locations, including the cantilever portion of the diaphragm can be calculated.

- Check diaphragm shear capacity:

along wall 5:

L = 175 ft, t = 14 in.

$$\varphi V_c = \frac{0.85(2)\sqrt{4000}(14)(175)(12)}{1000}$$

=3,161 kips > 634 O.K.

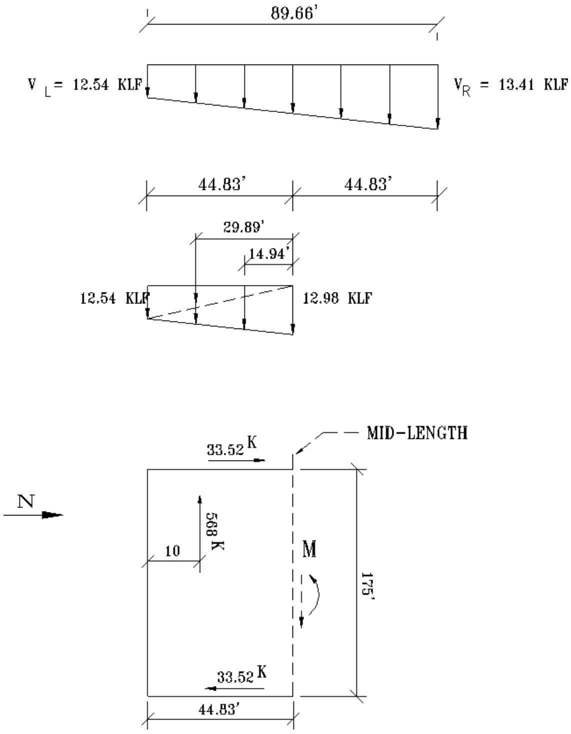


Figure 8-20. Force distribution and diaphragm moments for seismic load in the E-W direction

along wall 6:

L = 17.33 ft, t = 14 in.

$$\varphi V_c = \frac{0.85(2)\sqrt{4000}(14)(17.3)(12)}{1000}$$

= 313 kips > 155 O.K.

along wall 7:

L = 35.33 ft, t = 14 in.

$$\phi V_c = \frac{0.85(2)\sqrt{4000}(14)(35.33)(12)}{1000}$$

= 638 kips > 413 O.K.

Chord Design:

$$T_u = \frac{M}{d} = \frac{12{,}916}{(175.0 - 1.0)} = 74.23 \text{ kips}$$

$$A_s = \frac{T_u}{\phi f_y} = \frac{74.23}{0.9(60)} = 1.37 \text{ in}^2$$

Provide 4#6 chord bars (A_s = 1.76 in^2) along the slab edges at the East and West sides of the building where the maximum chord force occurs.

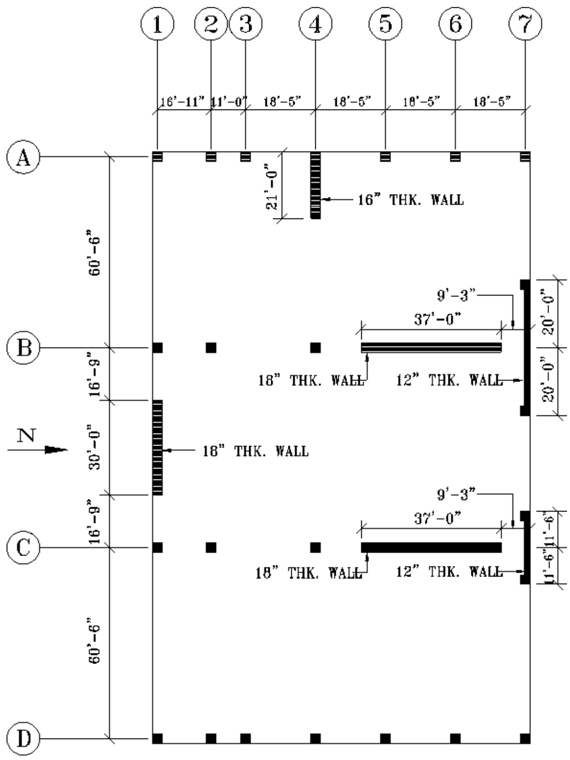


Figure 8-21.Ground floor framing plan (Example 8-2).

EXAMPLE 8-2

Perform a preliminary design the third floor diaphragm of the four story parking structure shown in Figures 8-21 through 8-25. The building is to be located in southern California (UBC seismic zone 4). Access to each floor will be provided from an adjacent parking structure that will be separated by a seismic joint. Typical floor and roof framing consists of a 5½ inches thick post-tensioned slabs spanning to 36 in. deep post-tensioned beams. Typical floor dead load for purposes of seismic design is estimated at 150 pounds per square foot. This includes contributing wall and column weights. Typical floor to floor height is 10 feet. This building is irregular and therefore needs to be analyzed using the dynamic response procedures. Furthermore, the redundancy factor for the building needs to be calculated and applied. For preliminary design purposes only, however, use the UBC-97 static lateral force procedure and ignore accidental torsion. Soil profile type is S_D, I = 1.0, N_a = N_v =1.0. Use f_c' = 5,000 lb/in^2 and F_y = 60,000 lb/in^2.

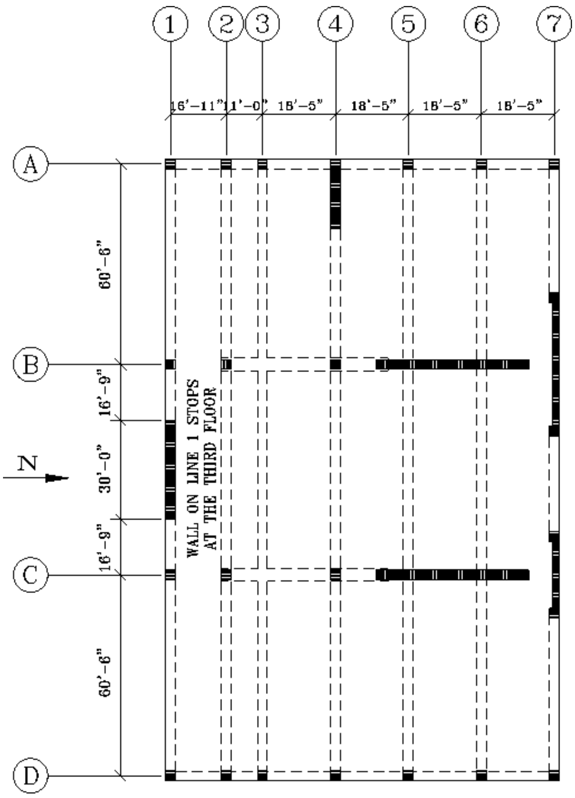


*Figure 8-22.*Second and third floor framing plan *(Example 8-2)*

SOLUTION

- Weight Computations:

Roof Weight = (68')(185')(0.15 k/ft^2)
= 1887 kips

4th Floor Weight = (85')(185')(0.15 k/ft^2)
= 2359 kips
3rd Floor Weight = (104')(185')(0.15 k/ft^2)
= 2886 kips
2nd Floor Weight = (104')(185')(0.15 k/ft^2)
= 2886 kips
Total Weight = 1887 + 2359 + 2(2886)
= 10018 kips

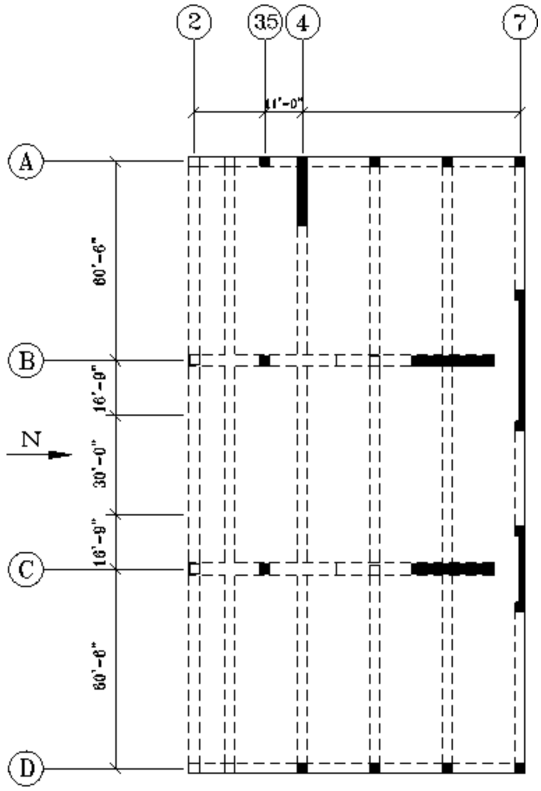


Figure 8-23. Fourth floor framing plan *(Example 8-2)*

- Design Lateral Forces

$T = C_t\,(h_n)^{3/4}$

Take $C_t = 0.02$

$\therefore\ T = 0.02(40)^{3/4} = 0.318$ Sec.

$$Base\ Shear\,(V) = \frac{C_v I}{RT}(W)$$
$$= \frac{0.64(1.0)}{4.5(0.318)}(W)$$
$$= 0.447(W) > (0.11 C_a I)W$$
$$= 0.048W$$
$$> 0.8\frac{Z N_v I}{R}(W) = 0.07W$$
$$> 2.5\frac{C_a I}{R}(W) = 0.244W$$

$\therefore\ V = 0.244\ W = 2444.4$ kips

$$F_x = (V - F_t)\frac{W_x h_x}{\sum W_x h_x}$$

$$F_{px} = \frac{F_t + \sum F_i}{\sum W_i} w_{px}$$

$T = 0.318$ Sec. < 0.7 Sec. $\Rightarrow\ F_t = 0$

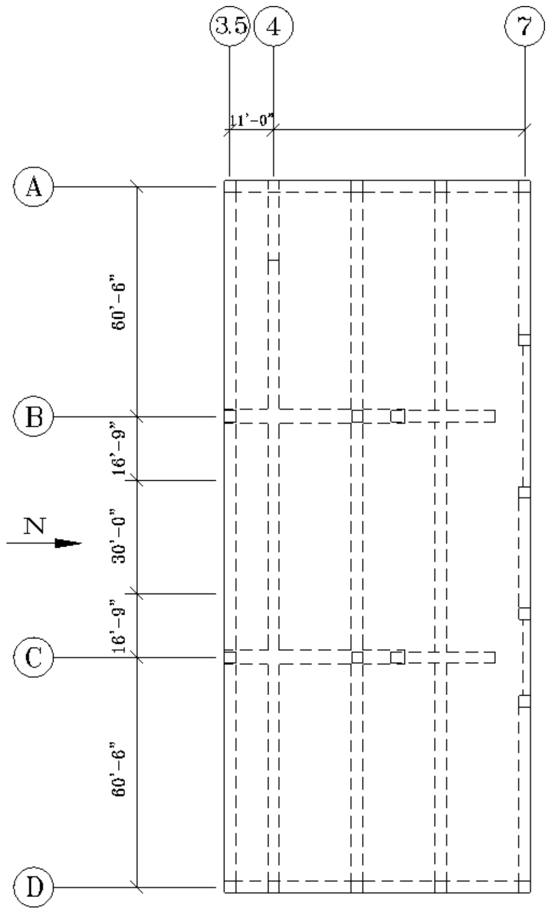


Figure 8-24. Roof framing plan *(Example 8-2)*

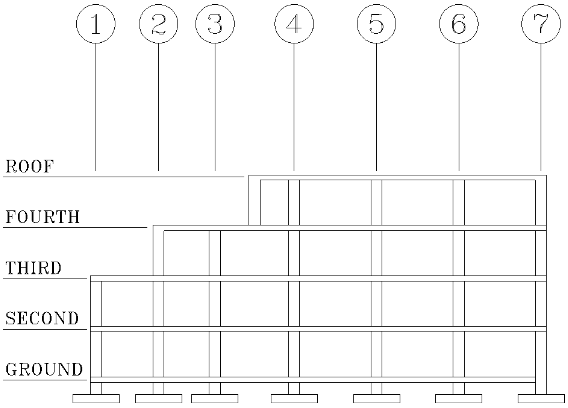


Figure 8-25. A section through the building (Example 8-2)

Values of F_{px} for various floors are calculated in Table 8-7. Concrete diaphragm is assumed to be rigid. The seismic shear forces acting on the walls were obtained by a computer analysis and are shown in Figures 8-26 and 8-27.

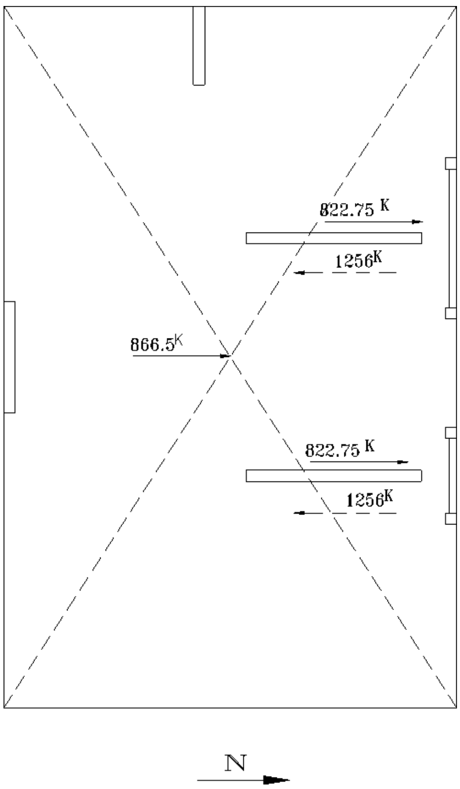


Figure 8-26. Forces on the third floor diaphragm due to N-S seismic loading (Wall shears above the diaphragm are shown with solid arrows while wall shears below the diaphragm are indicated by dashed lines.)

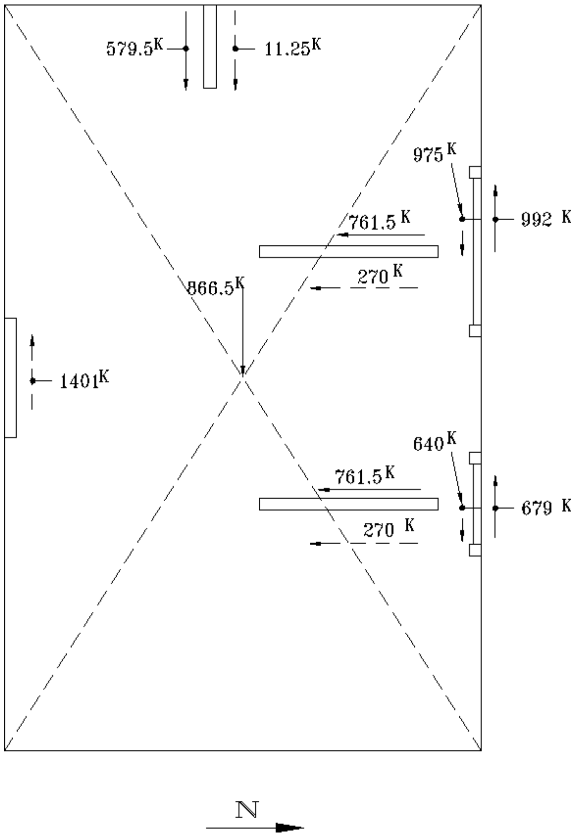


Figure 8-27. Forces on the third floor diaphragm due to E-W seismic loading (Wall shears above the diaphragm are shown with solid arrows while wall shears below the diaphragm are indicated by dashed lines.)

- Diaphragm Design in the N-S Direction:

Net shear forces acting on the walls and the corresponding diaphragm load, shear and moment diagrams are shown in Figure 8-28. Check 8" thick slab shear capacity along the walls on grid lines B and C:

Maximum slab shear = 283.75 kips

Slab capacity without shear reinforcement =

$$\phi V_c = \phi 2\sqrt{f_c'} = \frac{0.85(2)\sqrt{5000}(5.5)(37)(12)}{1000}$$

$$= 294 > 283.75 \text{ kips} \qquad \text{O.K.}$$

Therefore, no shear reinforcement seems to be required by the code.

Chord Design:

Table 8-7. Calculation of Diaphragm Design Forces for Example 8-2

Level	h_x, ft	W_x, Kips	$W_x h_x$	$\frac{W_x h_x}{\Sigma W_i h_i}$	F_x, Kips	ΣF_x Kips	ΣW_i, Kips	$\frac{\Sigma F_i}{\Sigma w_i}$	F_{px}, Kips
Roof	40	1,887	75,480	0.324	792.4	792.4	1,887	0.420	792.4
4th	30	2,359	70,770	0.304	743.0	1,535.4	4,246	0.362	853.1
3rd	20	2,886	57,720	0.248	606.0	2,141.4	7,132	0.300	866.5
2nd	10	2,886	28,860	0.124	303.0	2,444.4	10,018	0.244	704.2
Σ		10,018	232,830	1.00	2444.4				

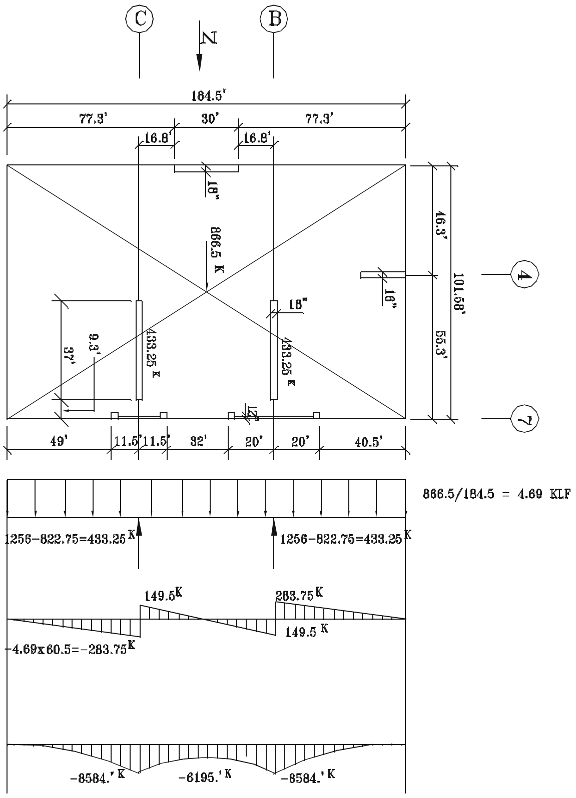


Figure 8-28. Diaphragm loading, shear, and moment diagrams for seismic load in the N-S direction

$$T_u = \frac{M}{d} = \frac{8{,}586}{(101.58 - 1.0)} = 85.4 \text{ kips}$$

$$A_s = \frac{T_u}{\phi f_y} = \frac{85.4}{0.9(60)} = 1.58 \text{ in}^2$$

Therefore provide 3 #7 chord bars (A_s = 1.8 in^2) along slab edges on the North and South sides of the building.

- Diaphragm Design in the E-W Direction:

Net shear forces acting on the walls and the corresponding diaphragm load, shear and moment diagrams are shown in Figure 8-29.

Moment Calculations:

at Section A-A:

$$M_{A-A} = 1{,}401(25.4) - \frac{8.53(25.4)^2}{2}$$
$$= 32{,}833 \text{ ft - kips}$$

at Section B-B:

$$M_{B-B} = 1{,}401(50.8) - 590.6(4.5) - \frac{8.53(50.8)^2}{2}$$
$$= 57{,}505 \text{ ft - kips}$$

at Section C-C:

$$M_{C-C} = 56(25.4) - \frac{8.53(25.4)^2}{2} + \frac{16.1}{37}(1031)(63.5)$$
$$= 27{,}158 \text{ ft - kips}$$

∴Estimated maximum moment[1] = 57,505 ft-k

Chord Design:

$$T_u = \frac{M}{d} = \frac{57{,}505}{(184.5 - 2.0)} = 315 \text{kips}$$

[1] A more accurate value of the maximum moment may be obtained by reading the moment diagram plotted to a larger scale.

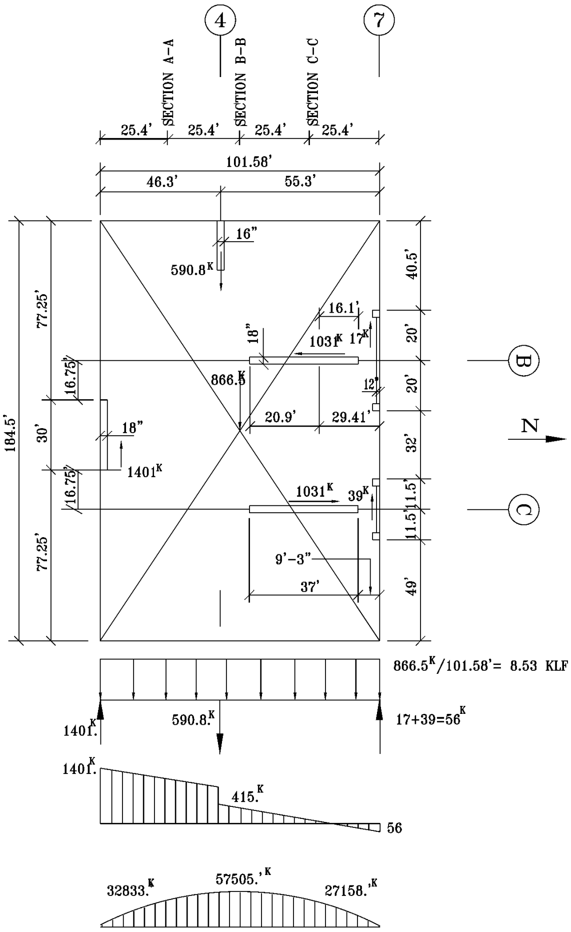


Figure 8-29. Diaphragm loading, shear, and moment diagrams for seismic load in the E-W direction

$$A_s = \frac{T_u}{\phi f_y} = \frac{315}{0.9(60)} = 5.83\text{in}^2$$

Therefore provide 6 #9 chord bars (A_s= 6.0 in^2) along slab edges on the east and west sides of the building

$$C_u = T_u$$

Compression C_u to be resisted by edge beam and concrete slab. Check 5½-in.-thick slab shear capacity along the wall on line 1: For L = 30 ft, slab capacity without shear Reinforcement is:

$$\phi V_c = \frac{0.85(2)\sqrt{5000}(5.5)(30)(12)}{1000}$$
$$= 238\,\text{kips} \;\; < 1401 \;\; \text{N.G.}$$

for L = 184.5 ft, slab capacity without shear reinforcement is:

$$\phi V_c = \frac{0.85(2)\sqrt{5000}(5.5)(184.5)(12)}{1000} =$$
$$= 1465\,\text{kips} \;\; > 1401 \;\; \text{O.K.}$$

Check the capacity of 30 foot long slab with #4 bars @ 18 inches, at the top and bottom of the slab:

ϕV_c = 238 kips

#4 @ 18" A_s = 0.13 in^2/ ft

ϕV_s = (0.85)(2×0.13)(60)(30 ft) = 398 kips

ϕV_n = 398 + 238 = 636 kips < 1401 kips

Drag struts are needed to transfer the difference (1401 - 636 = 765 kips).

- Design of Drag Struts (see Figure 8-30):

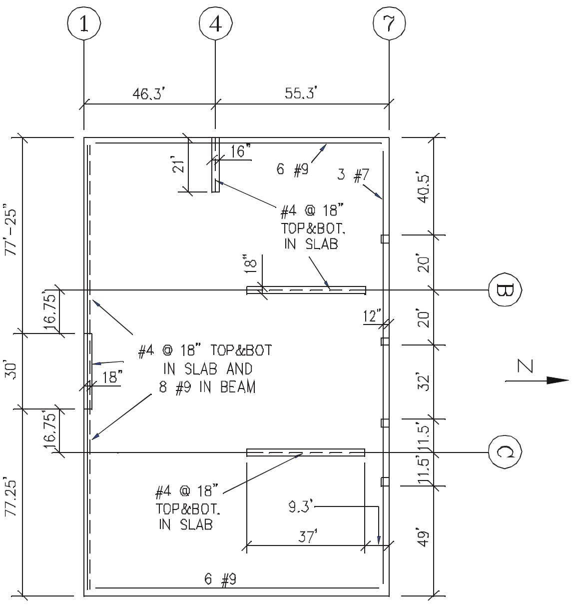


Figure 8-30. Diaphragm chord, drag, and shear reinforcement

The two beams along the Grid line 1 may be designed to transfer the slab shear into the walls:

$$A_s = \frac{\left(\frac{765}{2}\right)}{(0.9)(60)} = 7.08\,\text{in}^2$$

$\therefore$ Provide 8 #9 bars (A_s = 8.0 in^2) in the beams for seismic shear transfer.

Drag strut length provided = 2(77.3) = 154.6 ft

Capacity of slab along drag strut

$$= \frac{0.85(2)\sqrt{5000}(5.5)(154.6)(12)}{1000}$$
$$= 1228\text{ kips} \quad > 693 \qquad \text{O.K.}$$

Check shear capacity of 5½-in. thick slab at the wall on grid line 4 to carry 590.8/2 = 295.4 kips of shear (notice that slab occurs on both sides of the wall):

$$\phi V_c = \frac{0.85(2)\sqrt{5000}(5.5)(21)(12)}{1000}$$
$$= 167\text{ kips} \quad < 295.4 \qquad \text{N.G.}$$

Therefore Shear reinforcement is required. Using #4 bars @ 18 inches at the top and bottom of the slab:

ϕV_s = (0.85)(2×0.13)(60)(21) = 278 kips

ϕV_n = 167 + 278 = 445 kips > 295.4 O.K.

Therefore drag struts are not required. It can be realized by observation that the slab shear capacity along the walls on the grid line 7 is sufficient. Check the shear capacity of the slab along the cross walls on grid lines *B* and *C*. Here again, slab occurs on both sides of the wall:

$$\phi V_c = \frac{0.85(2)\sqrt{5000}(5.5)(37)(12)}{1000}$$
$$= 294\text{ kips} < \left(\frac{1031}{2}\right) = 515.5 \qquad \text{N.G.}$$

Therefore shear Reinforcement is required. Try #4 bars @ 18 inches at the top and bottom of the slab:

ϕV_s = (0.85)(0.13×2)(60)(37) = 490 kips

ϕV_n = 294 + 490 =784 kips > 515.5

Therefore drag struts are not required.

EXAMPLE 8-3

Design the roof diaphragm of the three story steel framed building shown in Figure 8-31. The building is supported on the top of a one story subterranean concrete parking structure. The parking structure deck may be considered as the shear base for the steel structure. The lateral load resisting system for the steel building consists of moment resisting frames in both directions. Beams and columns which are not part of the lateral system are not shown in Figure 8-31. The floor construction consists of 3 1/4 inches of light-weight concrete on the top of a 3 inch deep, 20 gage, metal deck. The maximum spacing of floor purlins is 10 feet. Mechanical equipment is located on the roof, west of grid line D. The roof construction west of grid line D consists of 4 1/2 inches of hard rock concrete on the top of a 3 inch deep, 18 gage, metal deck. The maximum spacing of the roof purlins is 8 feet. The roof construction east of grid line D is similar to the typical floor construction.

The estimated total dead loads for seismic design are 100 psf at the typical floors, 200 psf at the mechanical areas of the roof, and 70 psf elsewhere on the roof. The building is located in area of high seismicity. A three dimensional computer analysis of the building has resulted in a working stress level (WSD) roof diaphragm design force of 364.8 kips in the N-S and E-W directions. The distribution of the roof diaphragm shear among the moment-reistant steel frames are shown in Figures 8-32 and 8-33.

SOLUTION

- Diaphragm Design in the E-W Direction

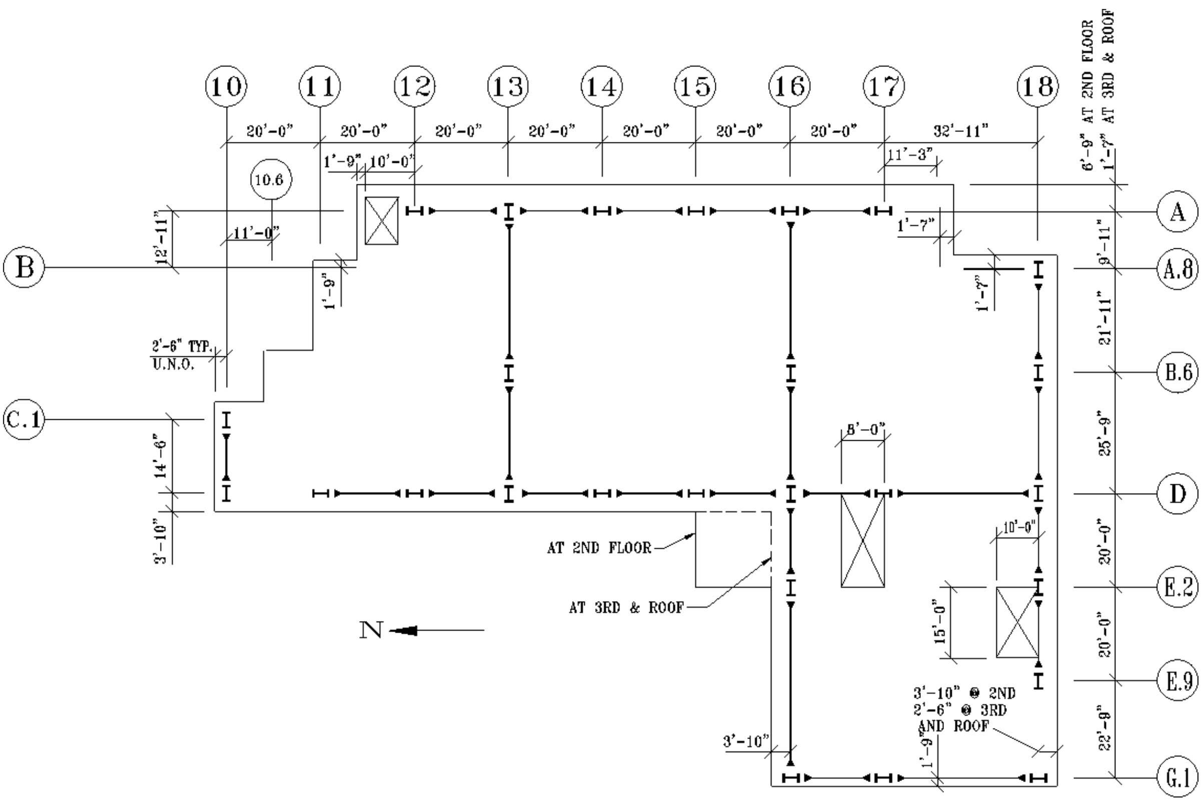


Figure 8-31. Typical floor framing plan for building of Example 8-3 (Opening shown exist on second and third floors only)

The design lateral force of 3604.8 kips is distributed along the roof in the same proportion as the mass distribution at this level. This loading pattern and the corresponding diaphragm shear diagram are shown in Figure 8-34. The maximum diaphragm shear per linear foot occurs at grid line 10 and is equal to:

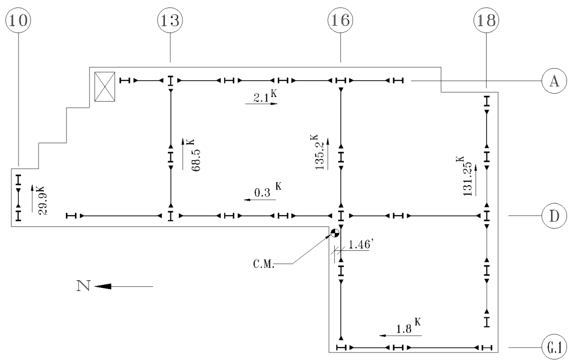


Figure 8-32. Frame shears for E-W seismic loading

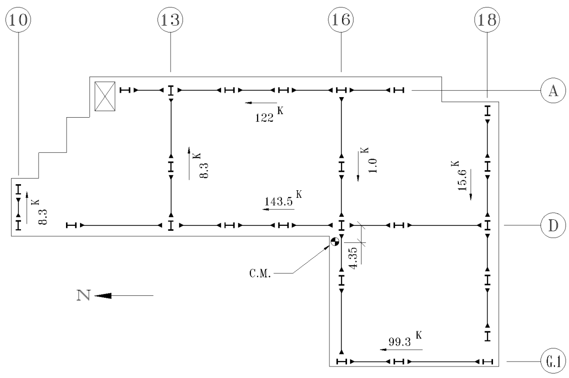


Figure 8-33. Frame shears for N-S seismic loading

$$v = \frac{29.9\ \text{kips}}{(3.8 + 14.5 + 2.5)\,\text{ft}} = 1.44\ \text{k/ft}$$

This value, has to be compared with the allowable shear values supplied by the metal deck manufacturer. For example, if a Verco 20 gage, W3 Formlok deck with 3 1/4 light-weight

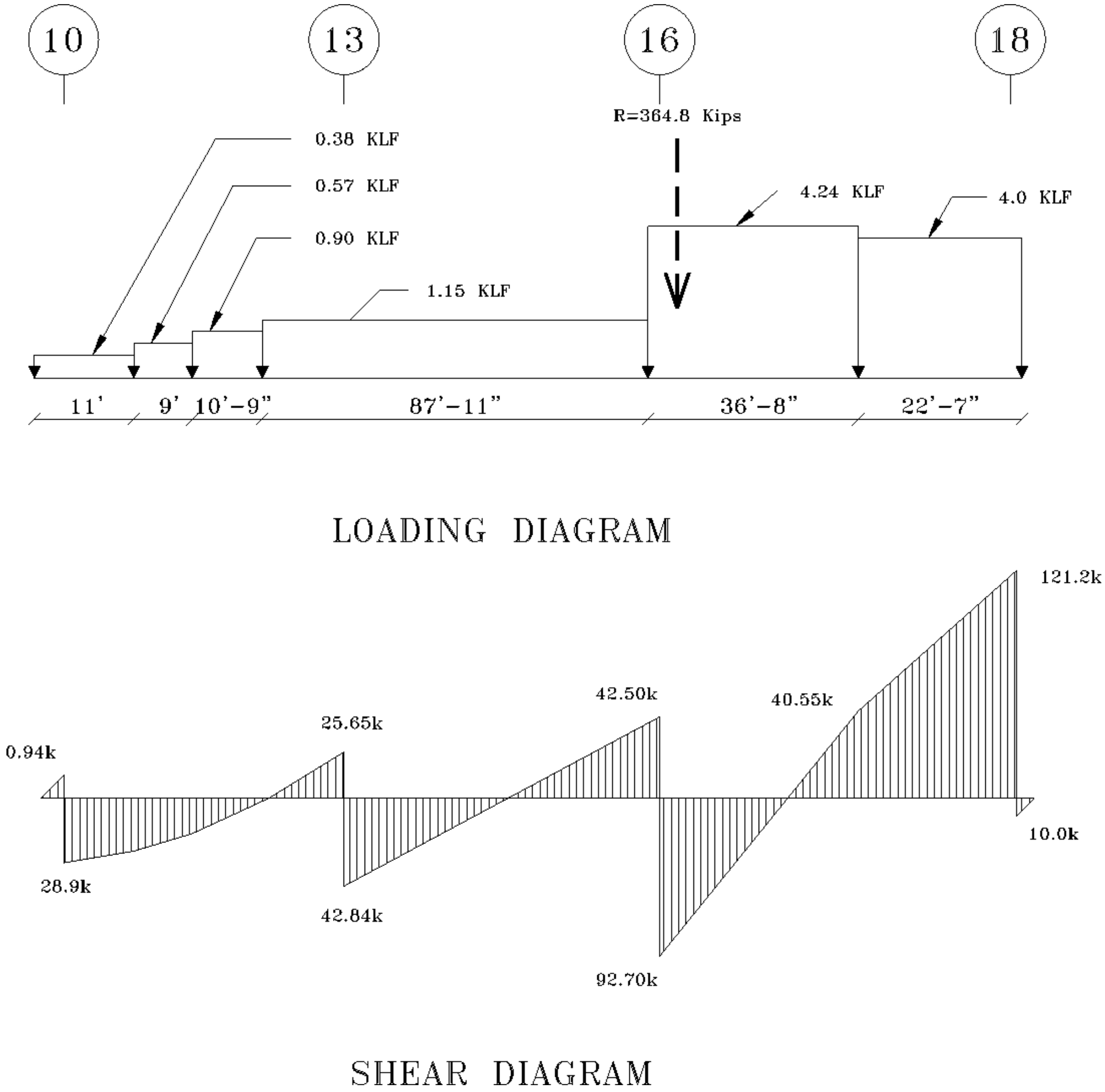


Figure 8-34. Diaphragm loading and shear diagrams for the E-W seismic loading

concrete fill and puddle welds in every flute is used, the allowable shear would be 1.74 kips compared to the required value of 1.44 kips (see Figure 8-35).

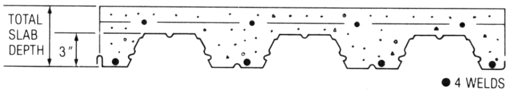
Check diaphragm chord requirements:

As mentioned earlier in this Chapter, the frame beams at the perimeter of the building will act as chord members or flanges of the diaphragm. To get a handle on the magnitude of the chord forces, diaphragm moments are computed at various sections. The transverse shear forces (in the N-S frames) are small and hence, are ignored in this analysis.

Moment at grid line 13

$$= 29.9(60) - 0.38(11)(57) - 0.57(9)(47) - 0.90(10.75)(37.125) - 1.15(31.75)^2/2$$
$$= 375.8 \text{ kips-ft}$$

W3 FORMLOK LIGHT WEIGHT CONCRETE (110 pcf)
GALVANIZED



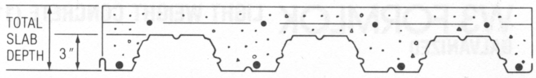
ALLOWABLE SUPERIMPOSED LOADS (Lbs./Sq. Ft.), DIAPHRAGM SHEAR VALUES (q) (Lbs./L.F.) AND FLEXIBILITY FACTORS (F)

TOTAL SLAB DEPTH & CONCRETE WT. psf	DECK GAGE & WT. psf	NUMBER OF SPANS / q / F	SPAN 8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"	13'-0"	13'-6"	14'-0"	14'-6"	15'-0"
6¼" 43.5 2 HOUR FIRE RATING	22 1.9	1	309	238	213	191	172	155	141	128	116	106	97	88	81	74	68
		2	309	279	254	232	172	155	141	128	116	106	97	88	81	74	68
		3	309	279	254	232	213	155	141	128	116	106	97	88	81	74	68
		q	1780	1760	1740	1725	1710	1695	1680	1670	1660	1650	1640	1630	1620	1615	1610
		F	.47	.48	.48	.49	.49	.50	.50	.50	.50	.51	.51	.52	.52	.52	.52
	21 2.1	1	341	309	281	215	194	176	160	145	133	121	111	102	93	86	79
		2	341	309	281	256	235	176	160	145	133	121	111	102	93	86	79
		3	341	309	281	256	235	217	201	145	133	121	111	102	93	86	79
		q	1810	1790	1770	1750	1730	1715	1700	1685	1670	1660	1650	1640	1630	1620	1610
		F	.43	.44	.44	.45	.45	.46	.46	.47	.47	.48	.48	.48	.48	.49	.49
	20 2.3	1	357	323	294	268	205	186	169	154	141	129	118	108	100	92	84
		2	357	323	294	268	246	227	169	154	141	129	118	108	100	92	84
		3	357	323	294	268	246	227	210	195	141	129	118	108	100	92	84
		q	1830	1805	1780	1760	1740	1720	1700	1690	1680	1665	1650	1640	1630	1620	1610
		F	.42	.43	.43	.44	.44	.45	.45	.45	.45	.46	.46	.47	.47	.47	.47
	19 2.7	1	400	372	338	309	284	262	201	183	168	154	142	131	121	111	103
		2	400	372	338	309	284	262	242	225	168	154	142	131	121	111	103
		3	400	372	338	309	284	262	242	225	209	196	142	131	121	111	103
		q	1890	1860	1830	1805	1780	1760	1740	1725	1710	1695	1680	1670	1660	1645	1630
		F	.37	.38	.38	.39	.39	.40	.41	.41	.41	.41	.42	.42	.42	.43	.43
	18 2.9	1	400	400	369	338	310	286	264	241	187	172	158	146	135	125	116
		2	400	400	369	338	310	286	264	241	229	214	158	146	135	125	116
		3	400	400	369	338	310	286	264	241	229	214	200	188	176	125	116
		q	1940	1905	1870	1845	1820	1795	1770	1750	1730	1715	1700	1690	1680	1665	1650
		F	.34	.35	.35	.36	.36	.37	.37	.38	.38	.39	.39	.39	.39	.40	.40
	16 3.5	1	400	400	400	400	370	341	316	293	273	255	197	182	169	157	146
		2	400	400	400	400	370	341	316	293	273	255	239	224	211	157	146
		3	400	400	400	400	370	341	316	293	273	255	239	224	211	199	188
		q	2070	2025	1980	1945	1910	1880	1850	1830	1810	1790	1770	1750	1730	1715	1700
		F	.28	.29	.30	.31	.31	.32	.32	.32	.32	.33	.33	.34	.34	.34	.34
7¼" 52.7 3 HOUR FIRE RATING	22 1.9	1	315	280	250	224	202	182	165	150	136	124	113	103	94	86	79
		2	364	329	299	224	202	182	165	150	136	124	113	103	94	86	79
		3	364	329	299	273	202	182	165	150	136	124	113	103	94	86	79
		q	2100	2080	2060	2040	2020	2010	2000	1985	1970	1960	1950	1945	1940	1930	1920
		F	.40	.41	.41	.41	.41	.42	.42	.42	.42	.43	.43	.43	.43	.44	.44
	21 2.1	1	400	314	281	253	228	206	187	170	155	142	130	119	109	100	92
		2	400	363	330	302	228	206	187	170	155	142	130	119	109	100	92
		3	400	363	330	302	277	255	187	170	155	142	130	119	109	100	92
		q	2130	2125	2080	2060	2040	2025	2010	1995	1980	1970	1960	1950	1940	1935	1930
		F	.37	.38	.38	.38	.38	.39	.39	.40	.40	.40	.40	.40	.40	.41	.41
	20 2.3	1	400	380	345	266	240	218	198	180	165	150	138	126	116	107	98
		2	400	380	345	316	290	218	198	180	165	150	138	126	116	107	98
		3	400	380	345	316	290	267	247	180	165	150	138	126	116	107	98
		q	2150	2120	2090	2070	2050	2035	2020	2005	1990	1980	1970	1960	1950	1940	1930
		F	.35	.36	.36	.37	.37	.38	.38	.38	.38	.39	.39	.39	.39	.39	.39
	19 2.7	1	400	400	397	363	333	258	235	214	196	180	166	153	141	130	120
		2	400	400	397	363	333	307	285	214	196	180	166	153	141	130	120
		3	400	400	397	363	333	307	285	264	246	180	166	153	141	130	120
		q	2210	2180	2150	2125	2100	2080	2060	2040	2020	2010	2000	1985	1970	1960	1950
		F	.31	.32	.32	.33	.33	.34	.34	.34	.34	.35	.35	.35	.35	.36	.36
	18 2.9	1	400	400	400	396	364	335	260	238	218	201	185	171	158	146	135
		2	400	400	400	396	364	335	310	288	218	201	185	171	158	146	135
		3	400	400	400	396	364	335	310	288	268	251	235	171	158	146	135
		q	2260	2225	2190	2160	2130	2110	2090	2070	2050	2035	2020	2005	1990	1980	1970
		F	.29	.30	.30	.31	.31	.31	.31	.32	.32	.33	.33	.33	.33	.33	.33
	16 3.5	1	400	400	400	400	400	399	370	343	320	248	229	212	197	183	170
		2	400	400	400	400	400	399	370	343	320	299	280	212	197	183	170
		3	400	400	400	400	400	399	370	343	320	299	280	263	247	183	170
		q	2380	2340	2300	2265	2230	2200	2170	2145	2120	2100	2080	2065	2050	2035	2020
		F	.25	.26	.26	.26	.26	.27	.27	.28	.28	.28	.28	.29	.29	.29	.29

Figure 8-35. A Verco Formlok diaphragm design table (reproduced with permission of Verco Manufacturing Company, Benicia, California)

W3 FORMLOK NORMAL WEIGHT CONCRETE (145 pcf)

GALVANIZED



ALLOWABLE SUPERIMPOSED LOADS (Lbs./Sq. Ft.), DIAPHRAGM SHEAR VALUES (q) (Lbs./L.F.) AND FLEXIBILITY FACTORS (F)

● 4 WELDS

TOTAL SLAB DEPTH & CONCRETE WT. psf	DECK GAGE & WT. psf	NUMBER OF SPANS q F	8′-0″	8′-6″	9′-0″	9′-6″	10′-0″	10′-6″	11′-0″	11′-6″	12′-0″	12′-6″	13′-0″	13′-6″	14′-0″	14′-6″	15′-0″
6½″ 60.4 1 HOUR FIRE RATING	22 1.9	1	266	235	209	186	166	149	134	120	108	97	87	79	71	64	57
		2	322	291	209	186	166	149	134	120	108	97	87	79	71	64	57
		3	322	291	265	242	166	149	134	120	108	97	87	79	71	64	57
		q	2430	2410	2390	2375	2360	2345	2330	2315	2300	2290	2280	2265	2270	2260	2250
		F	.34	.35	.35	.36	.36	.36	.36	.36	.36	.37	.37	.37	.37	.37	.37
	21 2.1	1	356	266	236	211	189	170	153	138	125	113	102	93	84	76	69
		2	356	322	293	211	189	170	153	138	125	113	102	93	84	76	69
		3	356	322	293	267	246	170	153	138	125	113	102	93	84	76	69
		q	2460	2435	2410	2390	2370	2355	2340	2330	2320	2305	2290	2280	2270	2265	2260
		F	.32	.33	.33	.33	.33	.33	.33	.34	.34	.34	.34	.34	.34	.35	.35
	20 2.3	1	373	337	250	224	201	181	163	147	133	121	110	100	90	82	74
		2	373	337	306	280	201	181	163	147	133	121	110	100	90	82	74
		3	373	337	306	280	257	181	163	147	133	121	110	100	90	82	74
		q	2480	2455	2430	2405	2380	2365	2350	2335	2320	2310	2300	2290	2280	2270	2260
		F	.31	.31	.31	.32	.32	.32	.32	.33	.33	.33	.33	.33	.33	.34	.34
	19 2.7	1	400	388	353	322	239	216	196	178	162	147	134	123	112	103	94
		2	400	388	353	322	296	216	196	178	162	147	134	123	112	103	94
		3	400	388	353	322	296	273	252	178	162	147	134	123	112	103	94
		q	2540	2510	2480	2455	2430	2410	2390	2370	2350	2340	2330	2315	2300	2290	2280
		F	.27	.28	.28	.29	.29	.29	.29	.30	.30	.30	.30	.30	.30	.30	.30
	18 2.9	1	400	400	385	352	323	241	219	199	181	166	152	139	127	117	107
		2	400	400	385	352	323	298	276	199	181	166	152	139	127	117	107
		3	400	400	385	352	323	298	276	256	238	223	152	139	127	117	107
		q	2590	2555	2520	2490	2460	2440	2420	2400	2380	2365	2350	2335	2320	2310	2300
		F	.25	.26	.26	.27	.27	.27	.27	.28	.28	.28	.28	.28	.28	.29	.29
	16 3.5	1	400	400	400	400	385	355	329	248	227	208	191	176	162	150	139
		2	400	400	400	400	385	355	329	305	284	266	191	176	162	150	139
		3	400	400	400	400	385	355	329	305	284	266	249	234	213	150	139
		q	2710	2670	2630	2595	2560	2530	2500	2475	2450	2430	2410	2395	2380	2365	2350
		F	.22	.22	.22	.23	.23	.23	.23	.24	.24	.24	.24	.25	.25	.25	.25
7½″ 72.5 2 HOUR FIRE RATING	22 1.9	1	311	275	244	217	194	173	155	140	125	113	101	91	82	73	66
		2	378	275	244	217	194	173	155	140	125	113	101	91	82	73	66
		3	378	342	311	217	194	173	155	140	125	113	101	91	82	73	66
		q	2910	2890	2870	2850	2830	2820	2810	2795	2780	2770	2760	2755	2750	2740	2730
		F	.29	.29	.29	.30	.30	.30	.30	.30	.30	.30	.30	.30	.30	.31	.31
	21 2.1	1	350	310	276	246	221	198	178	161	145	131	119	107	97	88	79
		2	400	377	276	246	221	198	178	161	145	131	119	107	97	88	79
		3	400	377	343	313	221	198	178	161	145	131	119	107	97	88	79
		q	2940	2915	2890	2870	2850	2835	2820	2805	2790	2780	2770	2760	2750	2745	2740
		F	.27	.27	.27	.27	.27	.28	.28	.28	.28	.28	.28	.28	.28	.29	.29
	20 2.3	1	400	328	292	261	234	210	190	171	155	140	127	115	105	95	86
		2	400	395	359	261	234	210	190	171	155	140	127	115	105	95	86
		3	400	395	359	328	301	210	190	171	155	140	127	115	105	95	86
		q	2950	2925	2900	2880	2860	2845	2830	2815	2800	2790	2780	2770	2760	2750	2740
		F	.26	.26	.26	.27	.27	.27	.27	.27	.27	.27	.27	.27	.27	.28	.28
	19 2.7	1	400	400	400	310	279	252	228	207	188	171	156	142	130	119	109
		2	400	400	400	377	346	252	228	207	188	171	156	142	130	119	109
		3	400	400	400	377	346	319	295	207	188	171	156	142	130	119	109
		q	3020	2990	2960	2935	2910	2890	2870	2850	2830	2815	2800	2790	2780	2770	2760
		F	.23	.24	.24	.24	.24	.24	.24	.24	.25	.25	.25	.25	.25	.25	.25
	18 2.9	1	400	400	400	400	352	280	254	231	211	193	176	161	148	135	124
		2	400	400	400	400	378	348	254	231	211	193	176	161	148	135	124
		3	400	400	400	400	378	348	322	299	211	193	176	161	148	135	124
		q	3070	3035	3000	2970	2940	2920	2900	2880	2860	2845	2830	2815	2800	2790	2780
		F	.21	.22	.22	.22	.22	.23	.23	.23	.23	.23	.23	.23	.23	.24	.24
	16 3.5	1	400	400	400	400	400	400	363	288	263	242	222	204	188	174	160
		2	400	400	400	400	400	400	384	356	263	242	222	204	188	174	160
		3	400	400	400	400	400	400	384	356	332	310	290	204	188	174	160
		q	3190	3150	3110	3075	3040	3010	2980	2955	2930	2910	2890	2875	2860	2845	2830
		F	.18	.19	.19	.19	.19	.20	.20	.20	.20	.20	.20	.21	.21	.21	.21

Figure. 8-35 (continued)

Chord force at grid line 13
$= 375.8/57.58 = 6.52$ kips

Moment at grid line 16
$= 29.9(120) - 0.38(11)(137) - 0.57(9)(107) - 0.90(10.75)(97.125) - 1.15(87.92)(47.76) - 4.24(3.8)^2/2 + 68.5(60)$
$= 777.2$ k-ft

Chord force at grid line 16
$= 777.2/57.58 = 13.5$ kips

Similarly, diaphragm moments and chord forces can be computed at other locations. In design of beams and the beam-column connections, these chord forces must be considered. The metal deck-beam welds must be verified to be able to develop the chord forces in addition to their shear transfer capability.

- Diaphragm Design in the N-S Direction

Here again, the applied lateral force of 364.8 kips is distributed in proportion to the mass distribution (see Figure 8-36). Diaphragm shears and moments at any location can be computed similar to the east-west seismic analysis. For example,

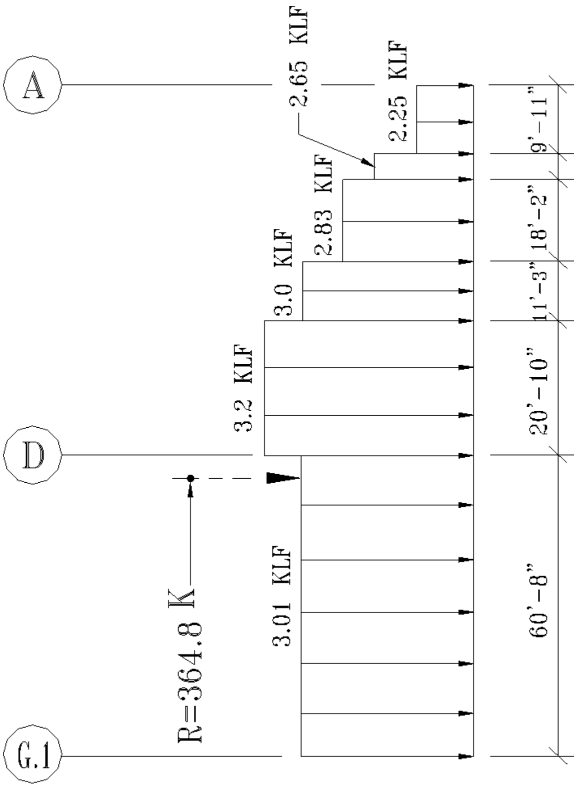


Figure 8-36. Diaphragm loading diagrams for the N-S seismic loading

diaphragm shear at grid line G.1

$$= \frac{99.3 - 3.01(1.75)}{59.25} = 1.59 \text{ kips/ft}$$

diaphragm shear at grid line D

$$= \frac{3.01(60.67) - 99.3}{59.25} = 1.40 \text{ kips/ft}$$

Both of the above computed diaphragm shears are less than the allowable shear value of 3.07 kips per linear foot for a Verco 18 gage, W3 Formlok deck with puddle welds in all flutes. As an example of diaphragm moment calculations, we compute the diaphragm moment at grid line D:

diaphragm moment at grid line D

$$= 99.3(58.92) - \frac{3.01(60.67)^2}{2}$$

$$= 311 \text{ ft - kips}$$

Chord force at grid line D

= 311.05/52.92 = 5.87 kips

To complete this design, diaphragm moments should be computed at a few other locations on the diaphragm, in order to establish the maximum moment, and the corresponding maximum chord force. The beams along grids 16 and 18, near grid line D may be designed to carry these chord forces.

EXAMPLE 8-4

The ground floor and roof plans of a one story neighborhood shopping center which is being planned for a city in a zone of high seimsicity are shown in Figure 8-37. The roof framing consists of plywood panelized roof with glue laminated beams and purlins. The roof dead load for the purposes of seismic design calculations is estimated to be 16 pounds per square foot. In addition to the framing weight, this includes allowances for composition roof, insulation, acoustic tile ceiling and a miscellaneous load of 1.5 pounds per square foot. Design the roof diaphragm in accordance with the UBC-97 requirements (IBC-2000 diaphragm design process is virtually the same). Assume $Z = 0.40$, $I = 1.0$, $N_a = N_v = 1.0$, and the S_B soil type.

- Dead load and base shear in the N-S direction

north wall at 75 lb/ft^2 = 75(14/2 + 2)(180)
= 121,500 lb

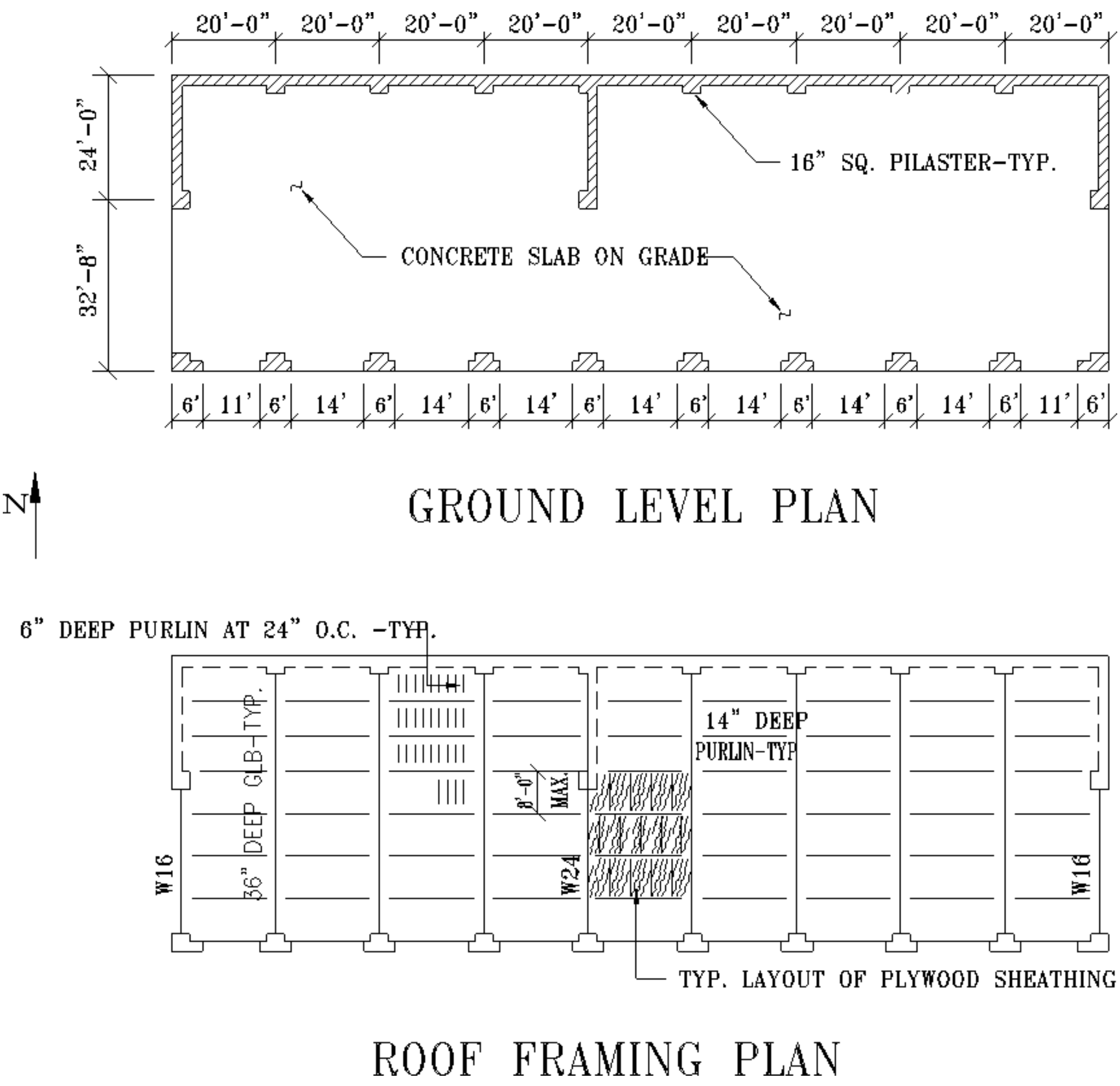


Figure 8-37. Floor plans for building of Example 8-4

pilasters in North wall = 75(14/2)(1.33×8)
= 5,600 lb

pilasters in South piers = 75(14/2)(1.33 ×10)
= 7,000 lb

south piers at 75 lb/ft^2 = 75(14/2+2)(10×6)
= 40500 lb

glass window at 15 lb/ft^2
= 15(14/2 + 2)(7×14 + 2×11) = 16,200 lb

roof at 16 lb/ft^2 = 16(180)(56.67)
= 163,210 lb

total dead load = 121,500 + 5,600 + 7000 + 40500+16,200 + 163,210 = 354,010 lb

Because this is a one story light-weight structure, we can use the simplified method according to UBC-97 section 1629.8.2. Notice that for flexible diaphragms providing lateral support for masonry, an R value of 4.0 must be used (UBC-97 section 1633.2.9.3)

$$Base\ Shear\,(V) = \frac{3.0C_a}{R}W$$

$$F_{px} = \frac{3.0C_a}{R}W_{px}$$

$$F_{px} = \frac{3.0(0.4)}{4.0}W_{px} = 0.30W_{px}$$

$$= 0.30(354{,}010)$$

$$= 106{,}203\,\text{lb} \quad \text{in N} - \text{S direction}$$

This value, however, is intended for strength design purposes. To convert it to the corresponding working stress design value, we divide it by a load factor of 1.4.

$$F_{px(WSD)} = \frac{106{,}203}{1.40} = 75{,}859\,\text{lb}$$

- Diaphragm design in the N-S direction (see Figure 8-38):

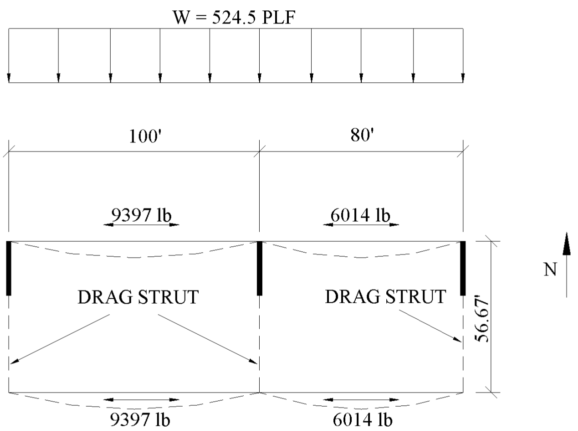


Figure 8-38. Chord forces for the N-S seismic loading based on flexible diaphragm assumption.

The diaphragm is assumed to be flexible. Therefore, in both directions, the wall loads will be based on the tributary diaphragm areas.

$$\text{N} - \text{S diaphragm load} = \frac{75{,}859\,\text{lb}}{180\,\text{ft}}$$

$$= 421 \text{ lb/ft}$$

East wall:

diaphragm shear = 421(80/2)
= 16,840 lb

diaphragm unit shear = 16,840/56.67
= 297 lb/ft

force in the drag strut = 297(32.67)
= 9,703 lb

Center Wall:

east side shear = 421(80/2) = 16,840 lb

diaphragm unit shear = 16,840/56.67
= 297 lb/ft

west side shear = 421(100/2) = 21,050 lb

diaphragm unit shear = 21,050/56.67
= 372 lb/ft

force in the drag strut =
(297 + 372)(32.67) =21,856 lb

West Wall:

diaphragm shear = 421(100/2) = 21,050 lb

diaphragm unit shear = 21,050/56.67
= 372 lb/ft

force in the drag strut = 372(32.67)
= 12, 153 lb

Diaphragm plywood requirements: Per UBC-97 Table 23-II-H (or similarly from IBC-2000 Table 2306.3.1), use $^3/_8$-in. Structural 1 wood panel diaphragm, blocked, 8d nails at $2^1/_2$-in. on center at the boundaries and continuos panel edges, 8d nails at 4 in. on center at other panel edges, and 12 in. on center on intermediate framing members. Allowable diaphragm shear is 530/1.4= 378 lb/ft which is greater than the maximum demand of 372 lb/ft.

Chord Design (see Figure 8-38):

for the 100 ft span:

$$M = \frac{421(100)^2}{8} = 526{,}250\ \text{ft - lb}$$

$$d = 56.67 - \frac{8}{12} = 56.0\ \text{ft}$$

$$C \text{ or } T = \frac{536{,}250}{56.0} = 9{,}397\ \text{lb}$$

for the 80 ft span:

$$M = \frac{421(80)^2}{8} = 336{,}800\ \text{ft - lb}$$

$$d = 56.67 - \frac{8}{12} = 56.0\ \text{ft}$$

$$C \text{ or } T = \frac{336{,}800}{56.0} = 6{,}014\ \text{lb}$$

Provide horizontal reinforcement as chord reinforcement in the North wall at the roof level. The maximum required area of steel is:

$$A_s = \frac{9{,}397}{1.33(24{,}000)} = 0.30\,\text{in}^2$$

Therefore a #5 continuous horizontal bar may be used typically (A_S = 0.31 in^2). A chord member is also required on the south side of the diaphragm. Alternatively, a timber chord member may be designed and used. Since the required chord area is small, one can design the edge purlin to act as a chord. Bolt purlin to the piers and provide metal strap across the beams for continuity of the chord.

Design of drag struts: The steel beams may be designed to act as drag struts to transfer the drag force from the steel beam to the block walls (see Figure 8-38). Diaphragm shear is transferred from plywood to the drag strut by means of the nailer as shown in Figure 8-39. The nailer is bolted to the drag strut. The plywood sheathing is nailed to the nailer. The drag strut force is transferred to the wall by means of the steel angle shown in Figure 8-40. The steel angle is welded to the steel beam and bolted to the wall. A wood ledger is used to transfer the diaphragm shear from the plywood to the wall, and to attach purlins to the wall.

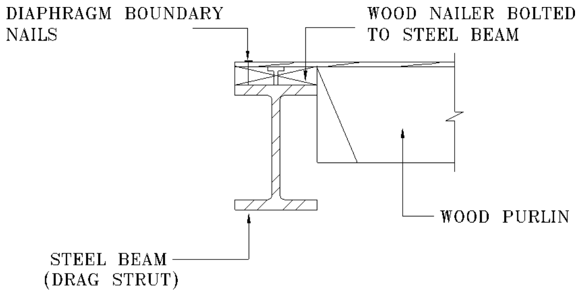


Figure 8-39. Typical detail for transfer of shear from plywood to the drag strut

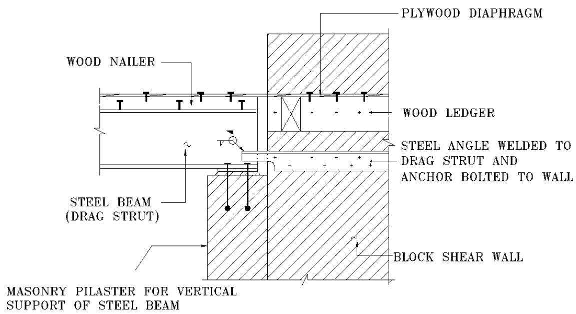


Figure 8-40. Typical detail for transfer of force from drag struts to a block shear wall

- Dead load and base shear in the E-W direction:

east and West walls at 75 psf =
75(14/2+2)(2)(24) + 75(14/2)(24) = 45,000 lb

pilasters at 75 psf = 75(14/2)(16/12)(3)
= 2,100 lb

glass windows at 15 psf =
15(14/2 + 2)(2)(32.67) = 8,821 lb

roof at 16 psf = 16(180)(56.67) = 163,210 lb

total dead load = 45,000 + 2,100 + 8,821 + 163,210 = 219,131 lb

$$F_{px} = 0.30W_{px} = 0.3(219{,}131) = 65{,}739 \text{ lb}$$

$$F_{px(WSD)} = \frac{65{,}739}{1.40} = 46{,}957 \text{ lb}$$

- diaphragm design in the E-W direction (see Figure 8-41):

North wall:

$$\text{E} - \text{W diaphragm load} = \frac{46{,}957 \text{ lb}}{56.67 \text{ ft}}$$
$$= 829 \text{ lb/ft}$$

$$\text{diaphragm shear} = 829 \times \frac{56.67}{2} = 23{,}490 \text{ lb}$$

effective length of diaphragm = 180 ft

$$\text{diaphragm unit shear} = \frac{23{,}490}{180}$$
$$= 131 \text{ lb/ft} < 378 \text{ lb/ft}$$

Therefore plywood requirements specified for N-S seismic is adequate along this wall.

South wall:

$$\text{diaphragm shear} = 829 \times \frac{56.67}{2}$$
$$= 23{,}490 \text{ lb}$$

Length of diaphragm in direct contact with the wall is 10×6ft = 60 ft. However, the south-side edge purlins, which were also designed and detailed as the chord for N-S seismic, will act as drag members along the south wall. Therefore, diaphragm shear = 23,490/180=131 < 378 lb/ft. Hence, previously specified plywood detailing will be adequate. Push or pull at the wall in a typical drag strut is

T = (131 lb/ft)(14/2 ft) = 917 lb.

The edge purlin and its bolting to the wall must be verified for the above force.

Chord design:

diaphragm span = 56.67 ft

$M = 829(56.67)^2/8 = 332{,}791$ ft-lb

$d = 180 - 8/12 = 179.33$ ft

C or $T = 332{,}791/179.33 = 1{,}856$ lb

The chord force is small. Hence, the steel beam and the horizontal reinforcement in the block wall will work as chord members.

- Diaphragm deflections:

The span to width ratio of the diaphragm in both directions is less than 4. Therefore, deflection is not expected to be a problem. However, if a deflection check is necessary, a simple procedure described in the Timber Construction Manual(8-14) or formula 23-1 of the IBC-2000 may be used to estimate diaphragm deflections.

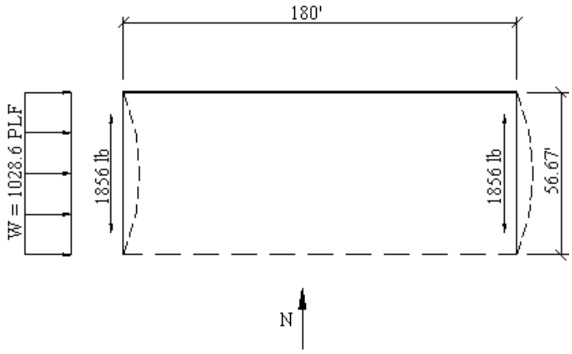


Figure 8-41. Chord forces for E-W seismic loading

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