

TILT-UP CONCRETE STRUCTURES

Reported by ACI Committee 551

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Tilt-up concrete construction is commonly used in low-rise building construction. This report discusses many of the items that should be considered in planning, designing, and constructing a quality tilt-up project. Major topics discussed include design, construction planning, construction, erection, and finishes.

Keywords: Analysis; box type system; composite construction; connections; cranes (hoists); diaphragms (concrete); earthquake resistant structures; erection; finishes; inserts; lifting hardware; load bearing walls; moments; parting agents; panels; rigging; roofing; sandwich structures; stability; strongback; structural design; tilt up construction.

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ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction and in preparing specifications. References to these documents shall not be made in the Project Documents. If items found in these documents are desired to be a part of the Project Documents, they should be phrased in mandatory language and incorporated into the Project Documents.

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CHAPTER I-INTRODUCTION

1.1-Introduction

The technique of site-casting concrete wall panels on a horizontal surface and then lifting or “tilting” them into place is referred to as tilt-up construction. Tilt-up construction uses less forming material than cast-in-place concrete construction and minimizes heavy equipment usage, which results in savings in time, equipment, and manpower. This efficient and cost effective method of construction has been used in the United States since the early 1900’s. Tilt-up has subsequently spread to many

other countries around the world. The American Concrete Institute, recognizing the increasing interest in this type of construction, formed ACI Committee 551 in 1980. The Committee’s mission is to “study and report on the design and construction of tilt-up structures.”

This report is in conflict with ACI 318 in three areas. The first conflict is found in [section 2.7.5](#) and deals with the distribution of concentrated loads. The second conflict is found in [section 2.10.5](#), which discusses typical connection details between the panel, foundation, and slab-on-grade. The third conflict concerns the amount of shrinkage and temperature reinforcement required in a tilt-up panel and is found in [section 2.15.5](#). At the time this report was prepared, these three conflicts were being discussed with Committee 318 in an effort to eliminate them.

1.2-Definition

The definition for precast concrete found in ACI 116R is “concrete cast elsewhere than its final position,” and includes tilt-up concrete. A more specific definition of tilt-up construction is “a construction technique of casting concrete elements in a horizontal position at the jobsite and then tilting and lifting the panels to their final position in a structure.”

1.3-History

In 1909, Aiken¹ described an innovative method of casting panels on tilting tables and then lifting them into place by means of specially designed mechanical jacks. This technique was used for constructing target abutments, barracks, ammunition and gun houses, a mess hall, factory buildings, and churches (see Figs. 1.1 - 1.4).

In the mid-1950s, Collins²⁻⁴ wrote three volumes devoted to the entire process of tilt-up. These publications were *Design of Tilt-Up Buildings*,² *Manual of Tilt-Up Construction*,³ and *Building with Tilt-Up*.⁴ During this same time period, tilt-up concrete construction began to gain nationwide acceptance as techniques were refined. California led the way and Sun Belt states were quick to follow. Since that time, tilt-up buildings have been constructed in every state in the United States, and in other countries around the world.



Fig. 1.1-Messhall, Camp Perry, Ohio

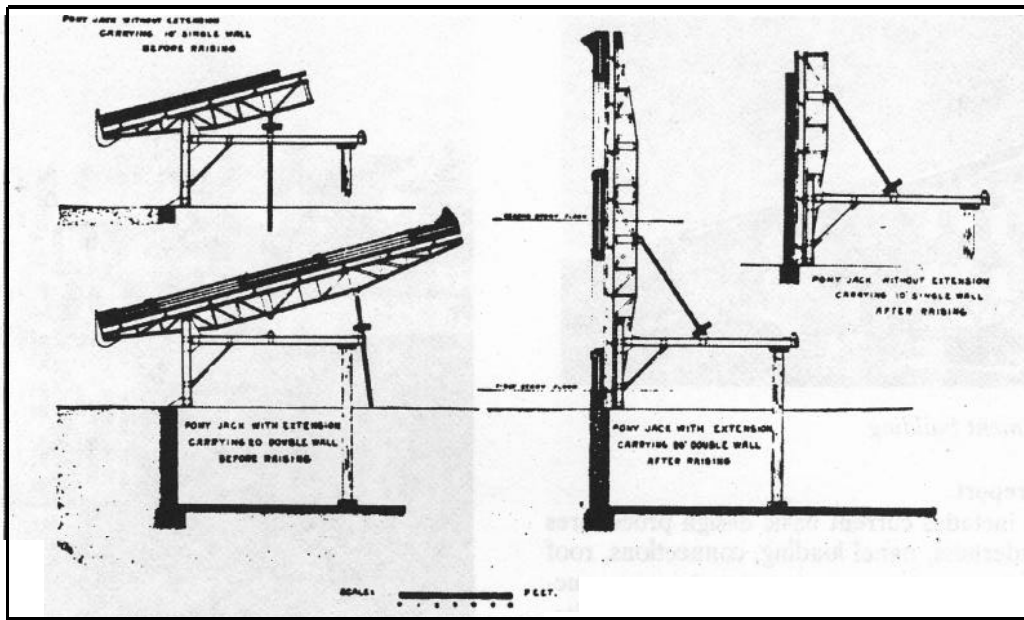


Fig. 1.2-Wall raising jack

1.4-Advantages

There are many advantages in tilt-up construction for low and even mid-rise buildings, including industrial plants, warehouses, office buildings, residential buildings, and commercial shopping centers. Examples of these types of buildings are shown in Figures 1.5 to 1.11. Some of these advantages are:

- 1) Elimination of expensive formwork and scaffolding
- 2) Fast, economical construction cycle time — from initial grading to move-in
- 3) Lower insurance rates that are typical for non-combustible construction
- 4) Wide variety of exterior finishes such as colored concrete, exposed aggregate, graphic painting and form liner finishes
- 5) Easily modified structures for building expansion
- 6) Durable, long-life and low maintenance building

Perhaps the greatest advantage of tilt-up is the ease and speed of construction. Panels can be tilted with high capacity mobile cranes and braced in less than ten minutes. It is possible to construct the complete building shell, from foundation through the roof, for a 100,000 ft² warehouse with office space in 30 days or less.

1.5-Disadvantages

- 1) Certain architectural treatment may become costly because of the construction techniques
- 2) Lack of availability of qualified personnel and contractors
- 3) Weight of the panels on certain soils
- 4) Available space to cast panels
- 5) Temporary bracing during construction
- 6) Availability of lifting equipment
- 7) Structural integrity requires careful consideration

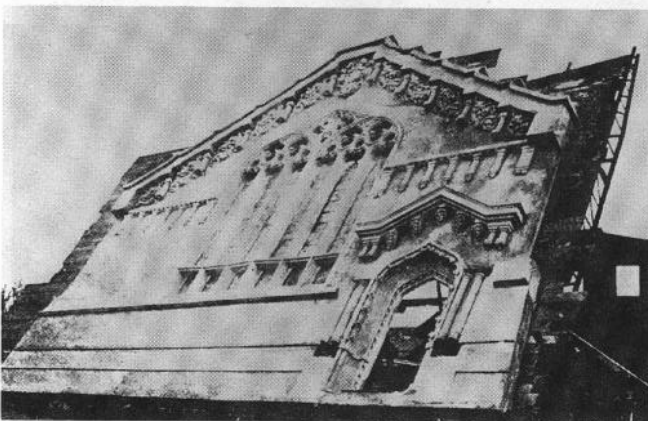


Fig. 1.3-Tilting front wall of Zion Methodist Church

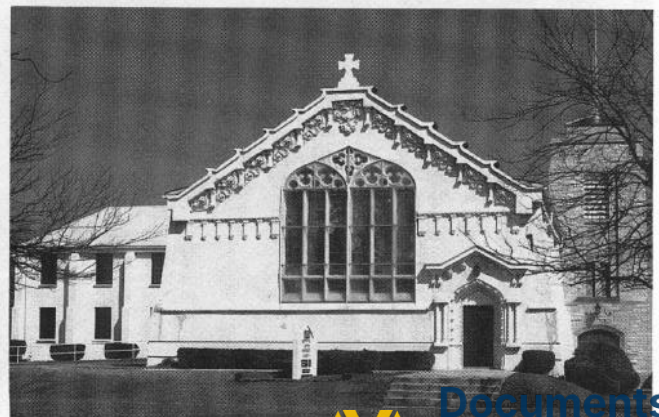


Fig. 1.4-Zion Methodist Church in 1987



Fig. 1.5-Apartment building

1.6-Scope of report

This report includes current basic design procedures relating to slenderness, panel loading, connections, roof diaphragms, lifting analysis, temporary bracing, construction planning, construction procedures at the jobsite, erection, and safety procedures, along with a discussion of concrete mixture proportions and methods and types of finishes. Because of the concern for energy conservation a section devoted to the construction of insulated sandwich wall panels is also included.

Following the recommendations contained in this report will reduce the need for experimenting at the jobsite. The five steps of design, planning, construction, erection, and creating finishes are crucial to a successful tilt-up project. With ample preplanning between the owner, contractor, concrete subcontractor, erection subcontractor, accessory suppliers, and architect/engineer, and close adherence to the ideas and suggestions in this report, tilt-up concrete construction can provide a quick, economical, and versatile method of constructing low and mid-rise buildings.



Fig. 1.6-Condominium

CHAPTER 2-DESIGN

2.1—General

2.1.1 Slenderness — Tilt-up buildings are typically low-rise structures of four stories or less in height, with the majority being one and two stories. Wall panels for these buildings are generally designed as load-bearing beam-columns spanning vertically between the ground floor and the roof, or intermediate floors. Typically, these panels support vertical gravity loads in combination with lateral loads such as wind, seismic, or earth pressures. Often the panels are very slender; slenderness ratios of



Fig. 1.7-Industrial building

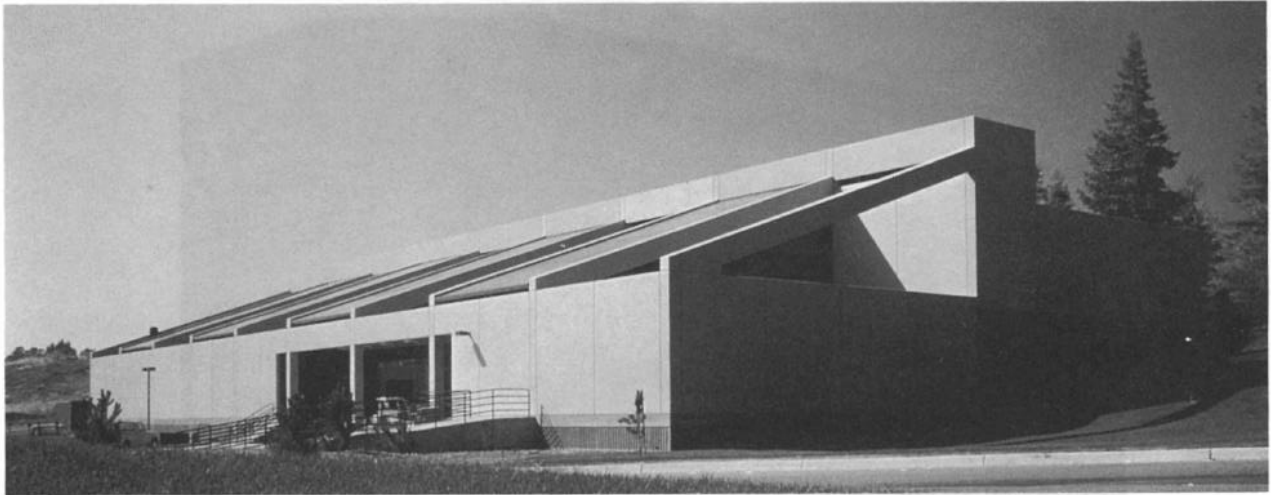


Fig. 1.8-Service building



Fig. 1.9-Warehouse



Fig. 1.10-Office building



Fig. 1.11-Shopping center

l_u/r of 140 to 200 are common. Bending moments due to applied loads can be magnified significantly by the effect of an axial load on the deflected shape. This increase in moment is generally referred to as the P-delta moment. P-delta magnification makes it necessary to take proper account of out-of-plane deflections.

2.1.2 Panel thickness — Panel thickness is often specified to conform to dressed lumber sizes, however, any thickness can be used. Thickness of 5% to 9% in. are commonly used.

2.1.3 Concrete — Either normal-weight or lightweight concrete can be used in tilt-up concrete panels. Because of exposure to weather and early loading during the erection process, concrete compressive strength of at least 3000 psi at 28 days is commonly specified.

2.2-Analysis

Slender tilt-up concrete walls must be analyzed as beam-columns. Design provisions in ACI 318 are applicable to walls where the slenderness ratio (l_u/r) is less than 100. This is approximately equivalent to a height-to-thickness ratio (l_u/h) of 30. Tilt-up walls will often exceed this limitation with l_u/h ratios of 40 to 50 or more. These are permitted by ACI 318, but only where a detailed structural analysis, including long term effects, shows adequate strength and stability.

Several methods have been proposed for computing the load carrying capacity of tilt-up concrete wall panels. In 1974 the Portland Cement Association published a design aid for tilt-up load bearing walls.⁵ A series of design charts were produced based on a detailed computer analysis. Coefficients to determine the maximum axial loadings were given for several combinations of section thickness, reinforcing steel areas, lateral loading,

panel height, and concrete strength.

Other variations of the design charts, including an expanded version of the PCA publication in 1979⁶ were produced which made it easier to consider special loading conditions or variations in section properties. These require some interpolation and extrapolation.

Most designers prefer a simplified analysis method that gives reasonably accurate but conservative results. Such a method is provided by the Structural Engineers Association of Southern California (SEAOSC) in the "Yellow Book," *Recommended Tilt-Up Wall Panel Design*⁷ and the "Green Book," *Test Report on Slender Walls*.⁸ These and other methods of approximate analysis⁹ are used to compute the bending stiffness of the concrete section from which maximum panel deflection and, thus, P-delta moments can be obtained. It is left to the designer to select the rational method of analysis that best suits his own needs. In [Section 2.6](#), an example problem is solved using three design methods.

2.3-Loads

2.3.1 Vertical loads — Tilt-up panels commonly support roof and floor joists. Joist spacing is usually five feet or less and the joist loads are considered as a uniformly distributed load on the panel. In most cases, these loads are applied at an eccentricity from the centerline axis of the panel.

Even if loads are intended to be concentric, a minimum eccentricity of one-third to one-half the panel thickness is generally used for design where the effect is additive to the lateral load, and zero where a reduction of total moment would otherwise occur. Eccentricity at the bottom of the panel is generally assumed to be zero.

2.3.2 Lateral loads — Usually wind pressures, as

specified by the local building code, control the design, although seismic accelerations are controlling in some areas.

Sometimes panels are required to resist lateral pressure due to soil in combination with vertical loads. These lateral loads can be significant and may limit the vertical span of the panel unless stiffening ribs are used for additional strength.

2.2.3 Self weight — The effect of panel self weight on the moment magnification can be approximated by assuming that a portion of the total weight acts at the top as a concentric axial load. For solid panels, the critical section for bending occurs at or above mid-height. It is therefore conservative to use one-half of the total panel weight. For panels with large openings, the location of the critical section will change and engineering judgment is required in determining the effect of P-delta. Changes in panel stiffness and application of loads will each affect the location of maximum design moment.

2.3.4 In-plane shear — In-plane shear forces on tilt-up panels can be significant for long-narrow buildings in seismic zones. These shear forces can result in significant panel overturning moments and increase the section and reinforcing requirements for panels with large openings and narrow legs. Horizontal reinforcement in the panels may be especially critical.

2.3.5 Load combinations — The following load combinations should be investigated. Lateral loads due to wind, earth, or seismic forces are usually predominant in determining the design moment:

- $U = 1.4D + 1.7L$
- $U = 0.9D + 1.3W$
- $U = 0.9D + 1.3(1.E)$
- $U = 1.4D + 1.7L + 1.7H$
- $U = 0.75(1.4D + 1.7L + 1.7W)$
- $U = 0.75[1.4D + 1.7L + 1.7(1.E)]$

2.4-Design bending moment

The design bending moment is the combined result of several effects including:

- Lateral loads
- Vertical loads applied at some eccentricity
- Initial out-of-straightness
- P-delta effects produced by vertical load

The maximum bending moment will usually occur at about mid-height of wall for panels spanning vertically. For panels with large vertical loads or large eccentricities the maximum bending moment may occur at a location other than mid-height.

The common practice is to combine the effects of all applied loads to obtain a maximum applied or primary moment acting on the panel. The P-delta moment is added separately as follows:

$$M_u = M_a + P_u \Delta$$

$$\begin{aligned} M_u &= \text{total maximum moment} \\ M_a &= \text{applied moment} \\ P_u \Delta &= \text{P-delta moment} \end{aligned}$$

Calculations for the P-delta moment are difficult in that they require a determination of the panel bending stiffness. The nonlinear properties of the concrete section make it difficult to precisely calculate the bending stiffness so approximate values are used. Effects of creep are typically ignored because of the transient nature of the predominant lateral loads. Where heavy vertical loads with large eccentricities are resisted, creep effects need to be considered.

2.5-Bending stiffness

Adequate bending stiffness is necessary for tilt-up panels in order to minimize out-of-plane deflections and the resulting P-delta moments. The bending stiffness of a reinforced concrete section varies with the following:

- Geometry of the concrete section
- Concrete modulus of elasticity
- Flexural strength of concrete
- Amount, grade and location of reinforcing steel
- Axial compression force
- Extent of cracking

Unless tilt-up panels are subjected to unusually large vertical loads, the bending component is generally dominant. Computer analysis of reinforced concrete sections has shown that for factored axial loads less than about 5 percent P_o , the bending stiffness is almost independent of curvature after flexural cracking has occurred. In actual load tests conducted by the Southern California Chapter of ACI and the Structural Engineers Association of Southern California⁸ and by the Portland Cement Association (unpublished report) the panels tested were capable of supporting additional lateral load after cracking and after first yield of the reinforcement (see Fig. 2.1). In the ultimate design state the concrete flexural cracking stress will be exceeded along most of

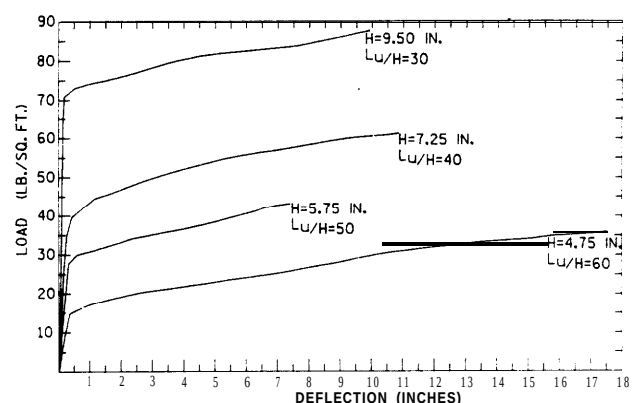


Fig. 2.1-Deflection at mid-height of panel

the height of slender walls, and the cracked section stiffness is commonly used as a reasonably accurate but conservative approximation of the actual stiffness.

The design methods in References 6, 8 and 9 use the cracked section stiffness with modification to account for the effect of axial compression. The reader is referred to these publications for further detail.

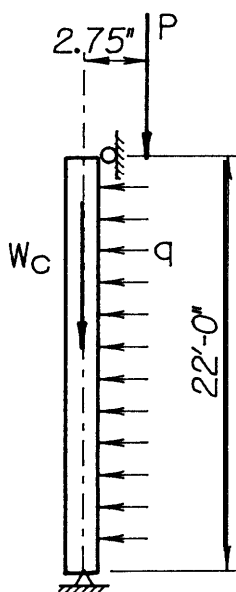
Tilt-up wall panels are primarily bending members and as such are governed by the maximum and minimum reinforcing requirements of ACI 318 for flexural reinforcement.

2.6-Example

The design of a typical tilt-up panel for vertical and lateral loads follows. The panel reinforcement is determined by three different methods for comparative purposes. For additional design information and examples see: *The Tilt-Up Design and Construction Manual*.¹⁰

Tilt-Up Panel Design

Sample Problem



P ; Dead Load = 0.4 k/ft

P ; Live Load = 0.32 k/ft

$q = 20 \text{ psf}$

$w_c = 70 \text{ psf}$ ($5\frac{1}{2} \text{ in. panel}$)

$f'_c = 4 \text{ ksi}$ $f_y = 60 \text{ ksi}$

$h = 5\frac{1}{2} \text{ in.}$ $d = 2.75 \text{ in.}$

Determine required reinforcement

Load Case: $U = 0.75(1.4D + 1.7L + 1.7W)$

Roof load $P_u = 0.75 (1.4 \times 0.4 + 1.7 \times 0.32) = 0.83 \text{ k/ft}$

Self wgt $w_{cu} = 0.75 \times 1.4 \times 70 = 74 \text{ psf}$

Wind $q_u = 0.75 \times 1.7 \times 20 = 25.5 \text{ psf}$
 $P'_u = 0.83 + 0.074 \times 22/2 = 1.64 \text{ k}$
 $\phi = 0.9 - 2 \times 1.64 / (4 \times 5.5 \times 12) = 0.89$
 Ignore initial deflection

$$P_u / \phi = 0.83 / 0.89 = 0.93^k$$

(self weight of wall included in table)

$$q_u / \phi = 25.5 / 0.89 = 28.65 \rightarrow \text{say } 30 \text{ psf}$$

From Table A2:

Required Coefficient =

$$\frac{P_u / \phi}{b h f'_c} = \frac{0.93}{12 \times 5.5 \times 4} = 0.0035$$

$$k l u / h = \frac{1.0 \times 22 \times 12}{5.5} = 48$$

For $\rho = 0.25$, Coef. = $0 + 2/10 (0.007) = 0.0014$

$\rho = 0.50$, Coef. = $0.007 + 2/10 (0.012) = 0.0094$

Required

$$\rho = 0.25 + (0.0035 - 0.0014) / (0.0094 - 0.0014) \times 0.25 = 0.316$$

$$A_s = 0.316 / 100 \times 5.5 \times 12 = 0.21 \text{ in.}^2 / \text{ft}$$

SEASOC Method (Ref. 8)

$$P'_u = 0.83 + 0.074 \times 22/2 = 1.64 \text{ k/ft}$$

try

$$A_s = 0.22 \text{ in.}^2 / \text{ft}$$

$$A'_s = (0.22 \times 60 + 1.64) / 60 = 0.247$$

$$a = (1.64 + 0.22 \times 60) / (0.85 \times 3 \times 12) = 0.485 \text{ in.}$$

$$c = 0.485 / 0.85 = 0.57 \text{ in.}$$

$$M_y = 0.247 \times 60 [2.75 - (0.485/2)] = 37.16 \text{ in.-k}$$

$$= 3.10 \text{ ft-k}$$

$$\phi M_y = 0.89 \times 3.10 = 2.76 \text{ ft-k}$$

$$E_c = 3,120 \text{ ksi } n = 9.3$$

$$I_{cr} = 9.3 \times 0.247 (2.75 - 0.57)^2 + 12 \times 0.57^3 / 3$$

$$= 11.6 \text{ in.}^4$$

$$\Delta = (5 \times 37.16 \times 22^2 \times 12^2) / (48 \times 3120 \times 11.66)$$

$$= 7.4 \text{ in.}$$

$$M_u = (25.5 \times 22^2) / (8 \times 1000) + 0.83 \times 2.75 / 24 + 1.64 \times 7.4 / 12 = 2.65 \text{ ft-k} < 2.76$$

$$A_s = 0.22 \text{ in.}^2 \text{ ft is OK}$$

Table A2 — Load capacity coefficients of tilt-up concrete walls* ($h = 5\frac{1}{2}$ in. and $q_u/\phi = 30$ or 45 psf)

*Observe the direction of ultimate transverse loads (q_u) and note the bending moments due to transverse loads are additive to those caused by the axial loads (Sec. 2.4). A dash indicates that the wall panel cannot sustain any load.

**Walls with slenderness ratios, kl_u/h , greater than 50 are not recommended.

† This column gives the values of the slenderness ratios above which the walls have negligible load-carrying capacity.

Weiler Method (Ref. 9)

Applied Loading:

$$P_u' = 0.83 + (74/1000) \times 22/2 = 1.64 \text{ k/ft}$$

$$M_a = (25.5/1000) \times (22^2/8) + (0.83 \times 2.75)/(12 \times 2) = 1.64 \text{ ft-k/ft}$$

$$P_u'/\phi = 1.85 \quad M_a/\phi = 1.85$$

Try

$$A_s = 0.22 \text{ in.}^2$$

$$A_s' = (0.22 \times 60 + 1.85)/60 = .251 \text{ in.}^2$$

$$\rho' = 0.251/12 \times 2.75 = 0.0076$$

$$\rho'n = 8.06 \times 0.00076 = 0.0613$$

$$k'd_r =$$

$$[\sqrt{(0.0613)^2 + 2 \times 0.0613} - 0.0613] \times 2.75 = 0.809$$

$$M_y = 0.251 \times 60 (2.75 - 0.809/3) = 37.4 \text{ in.-k}$$

or 3.11 ft-k

$$\text{Stiffness } EI = M/\theta = (37.4)/\{60/[29000(2.75 - 0.0809)]\} = 35048 \text{ k-in.}^2$$

$$\text{Bending } k_b = (\pi^2 \times 35048/22^2 \times 12^2) = 4.96 \text{ ft-k/ft}$$

Stiffness

$$\text{Magnifier } \delta = 1/(1 - 1.85/4.96) = 1.59$$

$$\text{Magnif. } M_u = M_a/\phi \delta = 1.85 \times 1.59 = 2.94 \text{ ft-k}$$

Mom't

$$\text{Resisting } M_n = (0.251 \times 60)/12 (2.75 - (0.251 \times 60)/12) = 3.22 > 2.94 \text{ ft-k}$$

Mom't

$$\text{Min. } A_s = 0.21 \text{ in.}^2/\text{ft}$$

2.7-Special design considerations

2.7.1 General — Simplified design and analysis techniques, when used with engineering judgment, are satisfactory for the majority of tilt-up concrete panel conditions and configurations. However, special design considerations may be required along with a more detailed elastic analysis where simplified analysis assumptions are too conservative and not applicable.

2.7.2 Continuity — Simplified techniques in the design methods discussed generally assume the panel is pinned at points of support, typically at the floor slab and roof diaphragm. Often additional attachment at the footing or

at intermediate floors provides some degree of continuity. This continuity may be included in a thorough elastic analysis with careful consideration of the foundation and slab connections and lateral movement of the roof diaphragm.

2.7.3 Openings — Openings constitute the most typical special condition which must be considered in panel design. Careful panel joint location selection can minimize the effect of openings. Since tilt-up panels are able to redistribute loads well, single openings with a maximum dimension of two feet or less are generally ignored analytically unless located at areas of maximum stress in the panel. For these openings, diagonal corner reinforcement (two #5 x 4 ft long bars or reinforcement of equivalent area) is used to limit development of cracks as shown in Fig. 2.2.

Where larger openings occur, such as personnel doors, the horizontal and vertical loads applied over the width of the opening are generally distributed equally to vertical panel segments on each side of the opening. These segments are then designed for the increased vertical loads and moments uniformly distributed over the section. For these cases, reinforcing bars are commonly placed along each side of the opening (vertical and horizontal) in addition to the #5 diagonal corner bars. The horizontal and vertical bars should be #5 as a minimum and should extend at least two ft beyond the limits of the opening (see Fig. 2.3).

Where major openings in panels occur, such as at overhead doors, the horizontal and vertical loads are also distributed to segments on each side of the openings. These panel segments are then designed as beam columns extending the full height of the panel. In some cases design loads may be substantial. Items to consider in the design are:

- Use of additional reinforcement on both faces of the vertical and/or horizontal panel segments and use of closed ties
- Effective width of the segment
- Bearing stress limitations at base of panel
- Need for thickened pilasters
- Design of the panel above the opening for out-of-plane forces
- In-plane shear and frame action
- Possible need for strong-backing during erection (see Section 2.11)

Because the panel reinforcement around these openings, as determined by analysis, is often considerable, added crack control reinforcement may not be necessary (see Fig. 2.4).

2.7.4 Isolated footings — Simplified design analysis assumes continuous support, however, tilt-up panels may be used to span between isolated footings, pile caps, or caissons. This special case is similar to conditions of load concentrations or large openings in that the effective panel width is reduced. If pilasters are not used and the

bottom of the panel is laterally supported by the floor slab, the total horizontal load may be assumed to act on the width of the vertical resisting elements. For design assistance for this condition see the Portland Cement Association publication *Tilt-Up Load Bearing Walls*.⁶

Special attention should begin to the horizontal reinforcement at the bottom of panel. This reinforcement resists panel shrinkage and thermally induced stresses in addition to flexural requirements, and should be developed at the edge of the foundation using hooks if required (see Fig. 2.5).

2.7.5 Concentrated loads — Concentrated loads on panels constitute a special condition which could invalidate assumptions of simplified design techniques. A series of concentrated loads such as roof or floor joists or purlins along a panel, are usually considered uniform for design purposes. Where reactions from major elements (such as in large beams or girders) produce load concentrations, the panel analysis must account for this effect.

ACI 318 allows a load concentration to be distributed to a width equal to the actual bearing width plus four times the panel thickness at the point of load. However, Committee 551 believes that the effective width to resist this concentrated load in tilt-up panels should be the width of bearing, plus a width described by sloping lines of one horizontal to two vertical on each side of the bearing to the critical design section in question (see Fig. 2.6). This vertical panel segment should be analyzed to provide suitable reinforcement for the full height of the panel. The horizontal load is considered to be that acting on the width of the segment in question. Special care is required when heavy loads occur at edges of panels (see Fig. 2.7). In this case the effective panel width is the width of bearing plus twice the distance from bearing edge to edge of the panel.

Panel reinforcement may be required in this vertical segment on both faces. Closed ties are required if reinforcement functions as compression steel.

Where load concentrations exceed the capacity of the panel segment, a pilaster or thickened panel segment may be used. A minimum of 3/12 in. added thickness is recommended to facilitate construction. Closed ties may be required. Where a pilaster is used its increased stiffness relative to the panel will attract a higher proportion of horizontal load than that acting on the remainder of the panel.

2.7.6 In-plane shear — Tilt-up panels are generally used as shear walls for building stability. Analysis of the panels should include in-plane shear stresses, panel stability, and floor and roof diaphragm connections. If panels must be connected to adjacent panels for stability, it is suggested that they be connected in groups with as few panels as needed to satisfy overturning requirements. See expanded discussion in Sections 2.8 and 2.10.

2.8-Building stability

2.8.1 General — Because tilt-up panels are

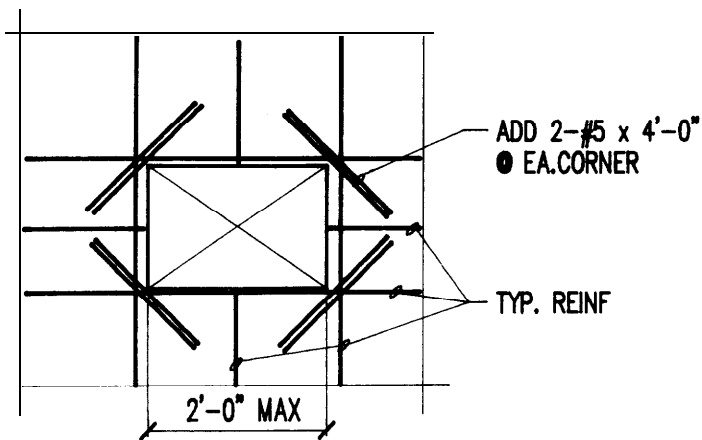


Fig. 2.2-Typical reinforcement at small openings

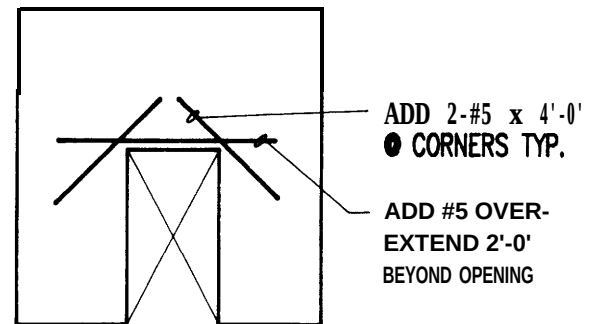


Fig. 2.3-Typical reinforcement at personnel door

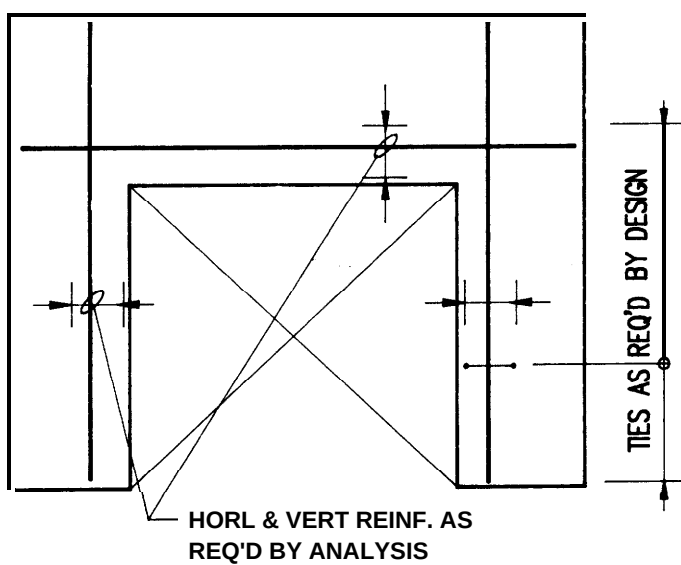


Fig. 2.4-Typical reinforcement at major openings

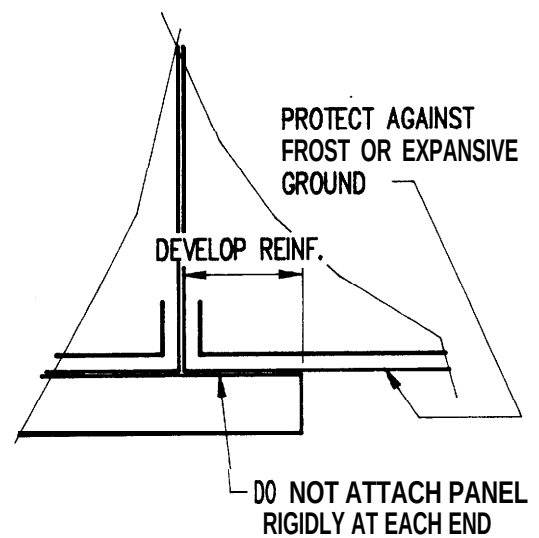


Fig. 2.5-Typical reinforcement at isolated foundation

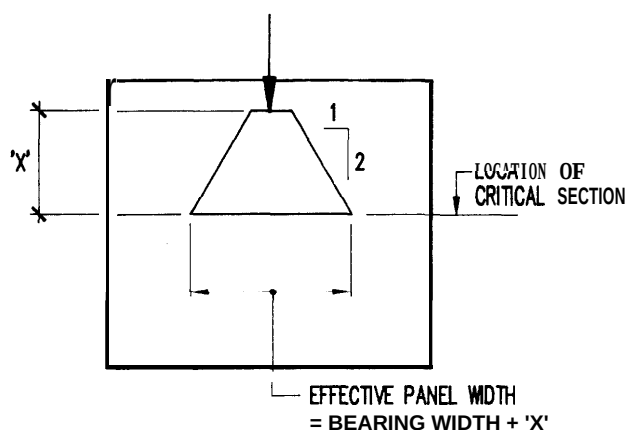


Fig. 2.6-Distribution of concentrated load

mid-rise, lateral loads and building stability sometimes do not receive sufficient attention in their design. Due to the special nature of these structures, designers and constructors need to be aware of the bracing requirements necessary to insure a stable, safe structure during construction and for the life of the building.

2.8.2 Structural systems

2.8.2.1 Box-type system—Tilt-up buildings are predominantly classified and designed as box-type structures. Box-type structural systems carry loads through sets of planes. As such, lateral forces are resisted by the roof and floor systems resist lateral forces in the horizontal plane, and the wall panels act as shear walls in the vertical plane.

Wall panels, therefore, must support both gravity and lateral loads while also providing lateral stability to the structure. These types of structures are not stable until all structural elements are in place and connected. Special attention must be given to stability during construction. This means that wall panel erection braces should not be removed until the deck diaphragm is completely fastened to the structural systems, and all other permanent connections have been installed as detailed on the plans.

2.8.2.2 Rigid frame — Some tilt-up buildings are constructed using independent moment resisting rigid frames to resist all lateral loads. This sometimes is done when the panels are non-loadbearing and used mainly as a curtain wall system. This framing system allows for easy future expansion.

2.8.2.3 Combined system — Sometimes the box and rigid frame systems are combined with rigid frames resisting the lateral loads in one direction, while the wall panels resist the lateral loads as shear walls in the other direction. Other combinations are possible.

2.8.3 Lateral bracing systems — Tilt-up panels usually rely on the roof and floor levels for lateral support. This support can be accomplished in many ways. Typically, the floor or roof deck is designed and used as a structural diaphragm. This is an efficient and economical bracing system. Many common construction materials may be

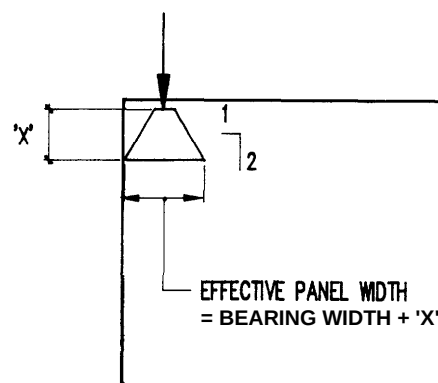


Fig. 2.7-Distribution of concentrated load near edge

used to construct structural diaphragms such as cast-in-place or precast concrete, steel deck, or plywood sheathing. Traditional 'x'-bracing at the roof level can also be used. Either type can be used in conjunction with rigid frames or shear walls. The bracing system that is used should be noted on the plans to aid the constructor.

2.8.4 Diaphragms— Typically, diaphragms are analyzed as large plate girders lying in the plane of the floor or roof, spanning horizontally between vertical shear resisting elements. The deck functions as the web to resist shear forces while perimeter or chord members function as flanges to resist the bending moment. Although this analysis is approximate, it is sufficiently accurate for most structures of this type.

The distribution of shear forces to the shear walls depends on the stiffness of the diaphragm. Diaphragms can be divided into five groups based on their shear stiffness moduli, G' , expressed in kips/in. of deflection.¹¹⁻¹³ Typical diaphragm spans are indicated in the following paragraphs. Often these spans are limited by the aspect ratio (spans to width) of the diaphragm.

2.8.4.1 Very flexible— Very flexible diaphragms with an effective shear stiffness of less than 6.7 kips/in. such as straight and conventional diagonally sheathed wood diaphragms, will distribute forces in direct proportion to the tributary area supported. These types of diaphragms should not be used to support tilt-up concrete walls.

2.8.4.2 Flexible— Flexible diaphragms, with an effective shear stiffness of 6.7 kips/in. to 15 kips/in., such as special diagonal wood sheathing, plywood sheathing, and some lightly fastened light gauge steel decks, will distribute lateral forces in proportion to the tributary area supported. The span of flexible diaphragms is usually limited to a maximum of 200 ft when supporting tilt-up concrete walls, unless diaphragm deflections are calculated.

2.8.4.3 Semi-flexible — Semi-flexible diaphragms, with an effective shear stiffness of 15 to 100 kips/in., such as plywood sheathing and moderately fastened medium gauge steel decks, will distribute lateral forces in proportion to the tributary area supported.

in proportion to the tributary area supported. Most of the steel roof decks commonly in use fall into this category. The span of semi-flexible diaphragms is usually limited to a maximum of 400 ft when supporting tilt-up concrete walls unless diaphragm deflections are calculated.

2.8.4.4 Semi-rigid — Semi-rigid diaphragms, such as some heavily fastened heavy gauge steel decks, with an effective shear stiffness of 100 to 1000 kips/in., can exhibit large deflections under load yet will distribute the loads in proportion to the relative stiffness of the vertical shear elements. The span of semi-rigid diaphragms is usually not limited.

2.8.4.5 Rigid — Rigid diaphragms, with an effective shear stiffness greater than 1000 kips/in., such as cast-in-place concrete decks, will distribute lateral forces in direct proportion to the relative stiffness of the vertical shear elements. The span of rigid diaphragms is not limited.

2.8.4.6 Diaphragm connections — The strength and performance of the entire diaphragm bracing system is dependent on the use of proper details for connecting the diaphragm to the structural framing system and to the vertical shear elements such as the tilt-up wall panels. Proper edge distances between the structural fastener and edge of the diaphragm must be detailed to prevent the fastener from tearing loose. Because of the importance of diaphragm connections, they should be inspected if the deck is designed to function as a diaphragm.

2.8.4.7 Diaphragm opening — Openings in diaphragms should be framed with some type of structural member of sufficient strength to carry the required forces around the opening.

2.8.5 Lateral deflection — In addition to strength, the bracing systems must be stiff enough to limit lateral deflections to a range where the vertical elements will not be damaged. Lateral deflections are generally not a problem with at-grade tilt-up buildings because panels sitting on grade beams are essentially pin-ended and able to rotate about their base. When panels extend below the floor slab such as at truck back-up doors, a certain degree of fixity exists at the floor level. If the top of the wall is pushed outward or pulled inward by large deflections of the bracing system, high flexural stresses can develop in the panel at the floor slab causing cracking or crushing. Based on guidelines provided in *Seismic Design for Buildings*, it is suggested that diaphragm deflection be limited to:

$$\Delta = l^2/24h$$

where

Δ = lateral deflection in inches

l = unsupported height of panel in feet

h = thickness of wall panel in inches

Tilt-up panels at loading docks are usually designed as a pinned condition at the roof and floor lines. This

deflection, therefore, should be limited to control bending stresses and possible cracking of the panel near the floor level.

2.8.6 Stability — Wall panels function as supports for the horizontal bracing. They must, therefore, be designed to resist in-plane shear forces. Typically, the shear stresses are very low. However, because the structure is made up of many individual panels, the sliding resistance and overturning stability of each panel must be calculated to insure an adequate margin of safety. Many times in regions of low or no seismic risk, the weight of the panels is sufficient to safely resist these forces, requiring no connection to the foundation other than grout for a uniform continuous bearing. When greater stability is required due to loading and to moderate or high seismic risk, the panels may be interconnected or connected to the foundation and/or floor slab to provide the required resistance.

2.8.7 Joints — To limit the effects of building movements due to thermal and shrinkage forces on large structures, structural expansion/contraction joints are used. These joints must be carefully located in a diaphragm supported structure to maintain lateral stability.

2.8.8 Summary — The stability and safety of a tilt-up structure depends on interaction of many different parts of the structure. The structural system used to furnish permanent stability of the structure should be clearly noted on the design drawings so that there is no question when the temporary panel erection braces may be removed.

2.9-Tolerances

2.9.1 General — Until tolerances have been established specifically for tilt-up construction, it is the recommendation of the Committee 551 that the tolerances for precast nonprestressed elements in *Standard Tolerances for Concrete Construction and Materials* (ACI 117), be used for tilt-up elements.

The Building and Construction Industry Division of the Department of Labour in Australia has developed a code of practice for tilt-up construction.¹⁴ Their recommended maximum fabrication tolerances are:

Length and Height:

Up to 3m (10 ft)	+0, - 10mm (3/8 in.)
3m to 6m (10 ft to 20 ft)	+0, - 12mm (1/2 in.)
Over 6m (20 ft)	+0, - 15mm (5/8 in.)

Thickness:

Overall	±5mm (3/16 in.)
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Straightness (deviation from intended line):

Up to 3m (10 ft)	+10mm (3/8 in.)
3m to 6m (10 ft to 20 ft)	±15mm (5/8 in.)
6m to 12m (20 ft to 40 ft)	±20mm (3/4 in.)



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Skewness (measure as tolerance in length of diagonal):

Up to 3m (10 ft)	$\pm 10\text{mm}$ ($\frac{3}{8}$ in.)
3m to 6m (10 ft to 20 ft)	$\pm 15\text{mm}$ ($\frac{5}{8}$ in.)
6m to 12m (20 ft to 40 ft)	$\pm 20\text{mm}$ ($\frac{3}{4}$ in.)

2.10-Connections

2.10.1 General - Tilt-up panels are generally incorporated into the overall building structural system supporting both vertical and horizontal loads, and also serve as external cladding. Connections must therefore be designed to adequately transmit forces from the roof and floor systems to the foundations.

In addition to the strength requirements, connections should be detailed to provide a degree of ductility for relief of temperature and shrinkage stresses, and for seismic energy absorption.

2.10.2 Types of connections - Details for connecting structural components to tilt-up panels are difficult to standardize. Variations in the type of roof and floor systems, combined with a designer or contractor's own preferences, have resulted in a wide variety of connection types. Before making a decision on connection types, an investigation of local practices should be made.

The following discussion will highlight some of the common features of connections and illustrate typical examples of details. For a more complete discussion on connections see the Portland Cement Association publication *Connections for Tilt-Up Wall Construction*.¹⁵

Connections used in tilt-up construction can be categorized into four main groups:

- Welded embedded metal
- Embedded inserts
- Drilled-in anchors
- Cast-in-place concrete

Welded embedded metal is the most common tilt-up connection. Typically, a steel angle or plate with anchors is cast into the panel. Connections are made by field welding to the exposed metal surfaces. These connections are sufficiently strong for most applications, are fast and inexpensive, and can be designed with reasonable ductility. Care should be taken not to overheat and spall concrete surrounding embedded items during field welding. Smaller sized welds with multiple passes are generally preferred.

Embedded inserts such as the ferrule loop allow bolted connections to be made directly. These metal to metal connections are reasonably ductile and they eliminate the need for field welding.

Drilled-in anchors (post installed) are inserted into holes drilled in hardened concrete. Anchorage of the steel insert to the concrete is provided by mechanical means or bond with a chemical adhesive. For indepth information on drilled-in anchors see ACI 355.

Mechanical anchorage is obtained by radial expansion

of the anchor. A force is applied on the walls of the hole by tightening the anchor to a specified torque. To insure capacity of the mechanical anchor proper, hole size and torque are necessary. The designer should investigate the cyclic load characteristics of any mechanical anchor which will be subject to seismic loads or heavy vibration.

Chemical anchors rely on adhesives which cure and bond with the surrounding concrete. Torque of the nut is not required to develop anchorage strength. The designer should investigate the effects of corrosive environments when using chemical anchors.

Powder-actuated fasteners are not recommended for structural applications.

Cast-in-place concrete connections are commonly used to connect wall panels to the slab-on-grade floor. Cast-in-place connections are also made by casting infill sections between erected panel components with overlapping reinforcement projecting from the ends of the panel (see Fig. 2.8). Cast-in-place connections between panels create restraint to thermal and shrinkage volume changes, and can result in cracking of the panel. In an earthquake, however, ductility after yielding of the reinforcement can be attained. Cast-in-place connections between panels are not common and the committee does not recommend their use as a general practice.

2.10.3 Roof and floor connections

2.10.3.1 Seat for steel joist - One type is a pocket recessed in the panel with an anchored angle seat (see Fig. 2.9). This connection must carry vertical gravity loads and transverse loads due to out-of-plane and in-plane wind or seismic forces. The steel joist is commonly field welded to the seat.

An alternate is a flat steel plate embedded flush with the panel face and with stud anchors embedded in the concrete (see Fig. 2.10). An angle seat is welded on after the panel is cast. Note that in both cases it is desirable to avoid projections beyond the surface of the panel to allow for easy screeding and finishing, or for stack casting one panel on top of another.

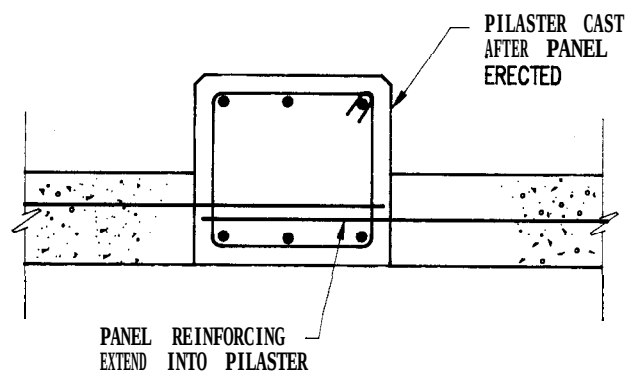


Fig. 2.8-Cast-in-place pilaster