

Analysis, Design, and Construction of Tilt-up Wall Panel

by

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(ABSTRACT)

The idea of tilt-up construction started in America in the early 1900's. In the beginning, this technique was mainly used on structures such as industrial warehouses and factories. However, recent developments and improvements in tilt-up construction technique and accessories have enabled this building method to be applied to many architecturally appealing offices and residential structures.

There are many details the design-build team must consider to ensure the success of a tilt-up project. The floor slab must be designed for panel casting and to withstand the loading of the mobile crane which will be used to lift the panel. The crane capacity affects the panel size and weight. Proper curing and bondbreaker application are very important to reduce bonding and to ensure clear cleavage between concrete surfaces. Changing rigging configuration consumes expensive crane time and must be reduced to minimum possible. The availability of ground-release quick connect/disconnect tilt-up hardware improves workers safety and speeds up the erection process substantially.

Although the stress analysis of simple wall panels during erection can be done by hand, panels with more complicated geometry or with openings, are more efficiently analysed with a computer. Many manufacturers have technical services to help in the design of insert layout so that the concrete will not be over stressed when the panel is tilted into position. After the panel is plumbed, it is braced temporarily before the final connection is made. For in-place loading there are now design aids available which ease the design process. When properly designed and built, tilt-up has proved to be a fast, efficient, and economical building construction technique.

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1.0 INTRODUCTION

The purpose of this thesis is to present the current status of tilt-up wall panel construction by evaluating the available literature and synthesizing the information in a state-of-the art report. Tilt-up construction commonly refers to the technique of precasting wall panels horizontally at the jobsite and then tilting them to a vertical position to become part of the building. Tilt-up is an innovative method of building construction; the concept was first put into practice in 1900-1912 when Col. Robert H. Aiken constructed a tilt-up building in Des Moines, Iowa. In addition to that, Aiken had also built a tilt-up concrete mess hall at Camp Perry, Ohio in 1908 (Aiken,1909). Many developments and improvements have taken place in tilt-up construction technique and today tilt-up offers a competitive alternative to conventional building construction.

When properly planned, designed and constructed, tilt-up construction has proved to be fast, efficient, and economical for many structures. The Atlantic Winter Fair Complex 1981 in Halifax, Nova Scotia, Canada was completed in less than seven months and within budget. The complex consists of four interconnected buildings with a total area of about 142,500 square feet. Tilt-up panels and jobsite precast K-frames were used to ensure that the project would be completed on schedule (Parker and Giffin, 1986).

Another example of a successful tilt-up project which not only met the demanding schedule but also produced an architecturally pleasing finish is the recently completed two story Grande

Boulevard Mall in Florida. It has an area of 225,000 square feet with reveal resembling cut stone finish on the wall panels. The owner saved approximately one million dollars for using the tilt-up panels instead of real stone. The entire project was completed in less than 12 months and has been dominated for several awards (Varon, 1986).

There are numerous other successful tilt-up projects which demonstrate the versatility and appeal of this construction technique. With further developments and with more tilt-up projects being completed successfully, this technique will become more popular among builders and designers. Tilt-up was commonly used on industrial buildings at the beginning; however, it is gaining strength in other types of structures such as residences, retail stores, shopping centers and offices. Nevertheless some designers are still reluctant to take advantage of this technique, thinking that it is either not practical enough or too complicated to construct. In fact this need not be the case although it is true that tilt-up construction requires much more communication and coordination between various parties involved (the designer, the supplier, and the contractor); as we will see in this paper.

Unlike other precast members, wall panels in a tilt-up project need not be transported over the highway to the building site. Therefore their size and weight are not limited by the power of the hauling truck or the traffic regulations on the highway. Besides, tilt-up wall panels have good fire rating and thus allow maximum use of the site when locating the building with respect to property lines. Because of greater mass, the requirement on thermal resistance (R) of tilt-up walls is about 30% less than lightweight walls. The greater mass and better insulation of tilt-up walls resulted in a more energy efficient building (Spears, 1980). In addition to that, in-place tilt-up walls very often act as shear walls resisting lateral forces from wind and earthquake.

Tilt-up construction has other advantages too. Because the panels are cast horizontally, it is easier for the crew to place, finish, and cure the concrete. Besides, it requires no rented scaffolding and relatively less formwork.

Some designers feel that the building size must be at least 40,000 square feet in order to have sufficient number of repetitive panels to attain the economy of tilt-up construction. However, there are many smaller size buildings which have been completed economically by tilt-up construction.

Tilt-up construction requires some special precautions and planning which may not be necessary in other building techniques. In Chapter 2 of this paper, we will first look at the building site preparation, including subgrade and floor slab, and various factors which need to be considered in the selection of the building site. We will then consider in Chapter 3 the guidelines for panel design, selection of suitable bondbreaker, and typical panel casting procedures. The hardware which is commonly used in tilt-up panels such as inserts, strongbacks, lifting plates, and braces as well as the panel erection procedure will be addressed in Chapter 4. Chapter 5 discusses various aspects which have to be considered in designing the connections and gives some typical connections which have been used successfully in connecting tilt-up panels.

The stresses induced in the panel during erection have to be analysed so as not to overstress the concrete. Hand computations will often give satisfactory results for simple solid panels. However, panels of more complicated shape or with openings are best handled by computer. The stress analysis of a wall panel by hand computation and by computer using the beam method is discussed in Chapter 6.

Chapter 7 presents various design charts and tables developed by a number of professional organizations and institutions to ease the work of designing tilt-up structures. It is to the advantage of the designer to make use of these design charts and tables whenever possible.

Finally, Chapter 8 summarizes the material presented and gives concluding remarks and recommendations on tilt-up wall panel construction.

2.0 BUILDING SITE, SUBGRADE, AND FLOOR SLAB PREPARATION

There are some important aspects of tilt-up construction the contractor and the designer must look into so as to enable the building under consideration to achieve the economy, efficiency, and speed of this technique of building construction. These include determining if the proposed site is accessible by the crane, the suitability of the surrounding terrain for crane operation, the uniformity of the subgrade, and any adjacent construction in progress.

2.1 Building Site

The design-build team must first investigate to see if the proposed building site is accessible to the mobile crane which will be used to lift the panels. The crane has to be transported over the streets and bridges, and it must have the capacity to lift the heaviest panel anticipated. Attention should also be given to any overhead telephone and power lines which may be encountered. Besides that, the terrain surrounding the site must be suitable for crane operation (Courtois, 1980),

there must be enough space for casting the panels outside the building if for some reason the floor slab can not be used as the casting surface, and has sufficient area to hoist the equipment necessary for the tilt-up construction.

2.2 Subgrade Preparation

After the site is cleared of trees, grubbed, and graded, it comes the time for preparing the subgrade for the building. It is important that the subgrade be compacted thoroughly and uniformly so as to distribute the load more evenly.

The concrete floor slab is usually rigid enough so that the loads on the floor are spread over large areas of the subgrade and consequently the pressures on the subgrade are very low. Therefore, rather than having a strong support from below, it is more important to have a uniform subgrade with no abrupt changes in the degree of support or subgrade stiffness. This will help to ensure that the slab will not crack structurally when the crane is using it as the working surface to lift and set the wall panels (Spears, 1980). It should also be born in mind that it is best to install the utility pipes for water, electricity, etc. at this stage before the floor slab is cast.

2.3 Floor Slab Preparation

Very often, the floor slab not only serves as the working surface for the crane, but is also the casting surface for the panels. The slab must be of such a thickness as to withstand the stresses induced by the motor crane, the loading and abrasion of forklift trucks in the case of an industrial facility, without too much deterioration in its lifetime.

Both compressive and flexural stresses are produced when a slab is loaded. Flexural stress is usually more critical and therefore the flexural strength of the concrete usually determines how thick the slab should be. The allowable tensile stress for concrete floor design ranges from 300 to 350 psi. (Spears, 1980). Reinforcing steel is used to control shrinkage cracking and volume changes due to variation of temperature and moisture content. Reinforcing steel may also be used to increase the flexural strength of the slab.

When the concrete floor slab of the building is used for the casting of tilt-up panels, it is essential that the surface of the slab is level, smooth, and free of trowel marks. Two to three hard trowelings are recommended to obtain high density, wear resistant finishes. A smooth finished surface is important if the exterior surface of the panel is to be smooth, or if painting is to be done. In addition to that, a smooth casting surface will help to prevent mechanical bond and provide a clear separation between concrete surfaces when the panel is lifted (McKinney, 1980). The floor slab surface is the 'mirror' of the tilt-up panel surface, any imperfect marks, uncontrolled cracking, and joints will be reproduced in reverse on the finished surface of the panel. Hence, the design-build team must make it a point to locate all control and construction joints in the floor so that they occur between panels, whenever possible.

The openings in the floor, necessary for interior columns, piping, and utilities can be formed by a three quarter inch thick of skin concrete coat over a sand fill at the locations required. The concrete coat can be knocked out at a later date.

After the concrete for the floor slab is placed, compacted, consolidated, and trowelled, the slab must be cured. Curing must be done immediately following the final hard trowelling. In extremely hot weather condition, some contractors prefer to fog-spray the concrete before the curing compound is applied (Spears, 1980).

3.0 GUIDELINES FOR PANEL DESIGN, BONDBREAKER SELECTION, AND PANEL CASTING

There are some general guidelines which designers must consider when choosing the wall panels for a particular tilt-up structure. Observing these guidelines can avoid a lot of unnecessary trouble, delays, and extra costs.

3.1 Panel Size and Weight

The size and weight of the wall panel is limited by the capacity of the lifting crane available. Many contractors recommend the capacity to be at least two to three times the heaviest panel in the entire structure for smooth and speedy handling and erection (Courtois, 1980). However, the capacity is also a function of boom length and its angle of inclination (Waddell, 1974). One of the charts developed to show this relationship is given in Figure 1. The lifts available decreases with

boom length, and increases with angle of inclination with the ground. Therefore, the designer should also pay attention to the panel which requires the longest reach.

3.2 *Panel Shape*

Solid rectangular panels are easiest to design and construct, and they rarely pose a problem to lifting insert layout and lifting stress analysis. However, there are also times when rectangular or even circular openings need to be formed in the panel to achieve a certain purpose. Experienced engineers usually find the following guidelines useful in selecting the panels (Varon, 1986).

1. The economy of tilt-up construction partially lies in the repetition of shapes and details of wall panels. Hence, every effort should be made to avoid too much variation in panel width and size. Unequal panel size and width will undoubtedly affect stack casting, crane capacity needed, and may require the change in lift rigging more than is necessary.
2. The width of panel segments alongside an opening should not be too narrow (see Figure 2a). Generally, 24 in. is the minimum allowed. Openings should be located in the middle of the panel and every effort should be made to avoid sharing an opening between two adjacent panels. Sharing of an opening not only make it difficult for the contractor to match the windowsill, the window may actually crack and the door jammed due to possible differential panel movement.
3. Avoid panels which may be unstable when erected, or laterally unbalanced as shown in Figure 2b. The shape shown also poses a problem during erection, the concrete may be overstressed if the leg is too narrow and the panel is heavy. When such a panel is inevitable, strongbacks and shores (see Figure 2c) are usually attached to the panel during tilting, espe-

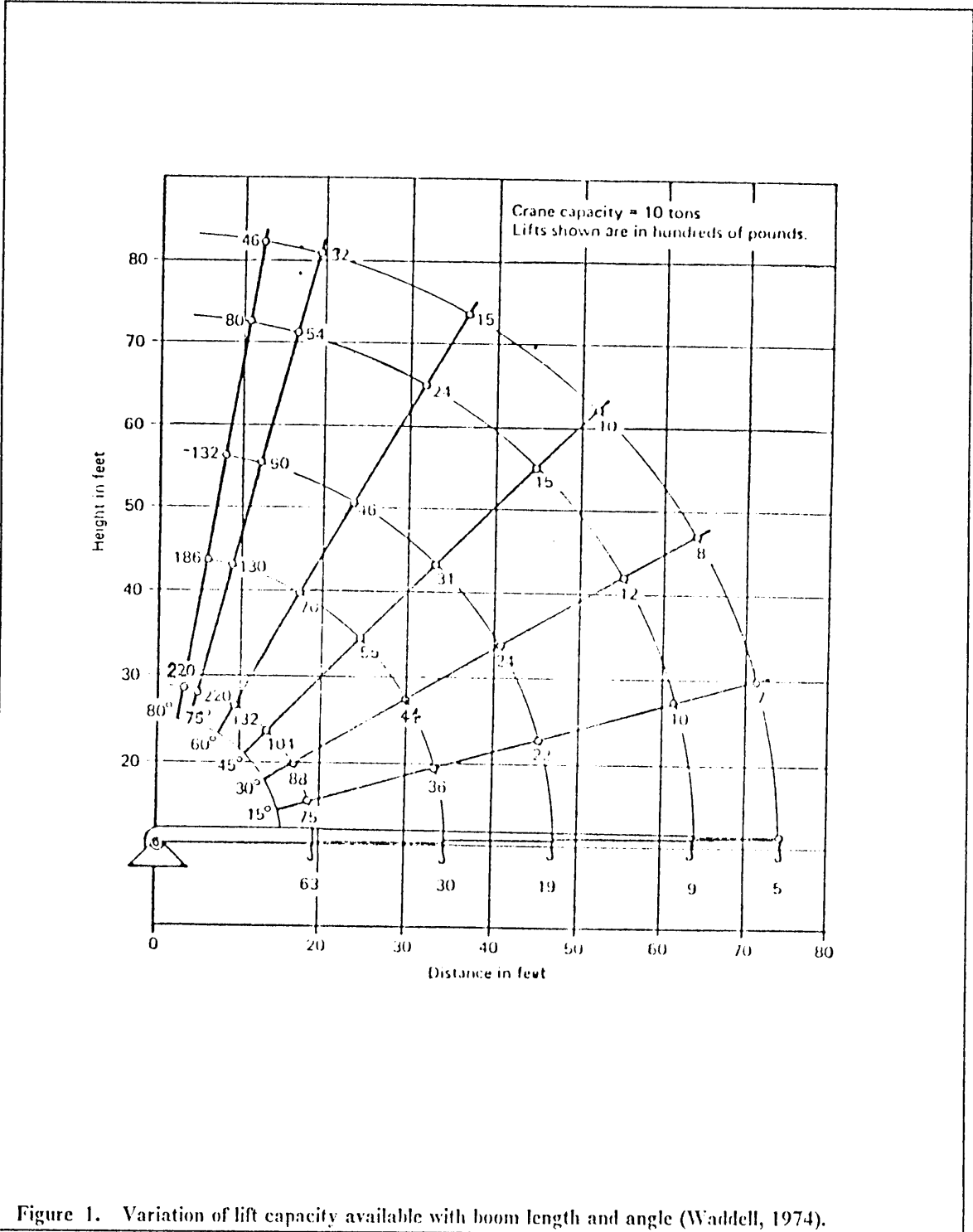
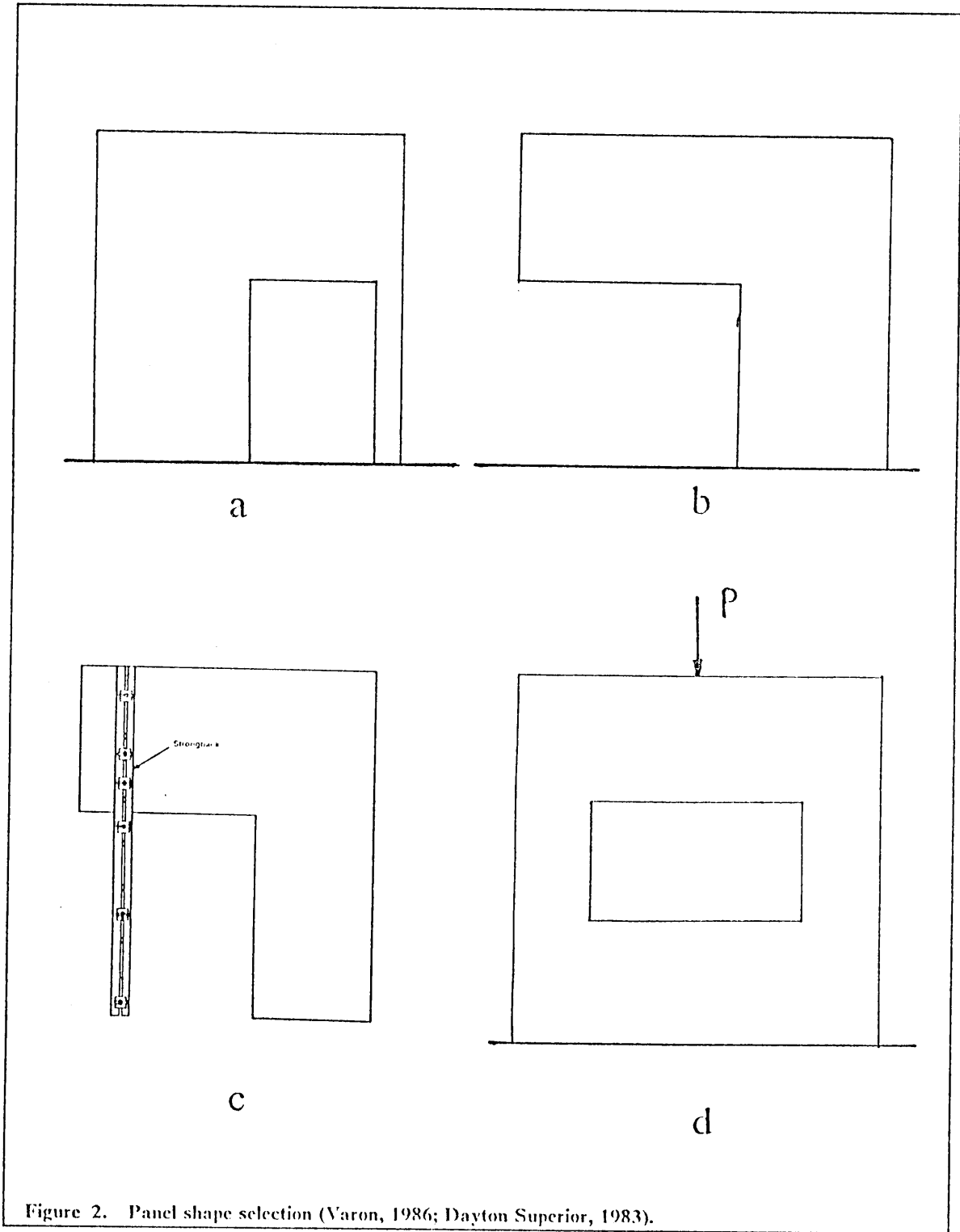


Figure 1. Variation of lift capacity available with boom length and angle (Waddell, 1974).



cially for those with the total widths of the legs less than half the overall panel width. The strongbacks can be made of two 2-inch timber or steel channels. The timber or channels are arranged back to back with a slot in between which allows the channels to be fastened to the panel. The shores are usually of the same thickness as the panel. This additional effort usually delays the tilt-up operation and the concrete surface near the inserts may be damaged (Waddell, 1974).

4. Avoid having an opening at a place where heavy gravity or lateral load has to be resisted (see Figure 2d). If the opening is long and inevitable, then the lintels should be designed to support beam or girder reaction which may require additional panel thickness and reinforcing steel.
5. Reveals, horizontal slots and rustication strips are sometimes used to improve the panel appearance. However, such features also tend to weaken the concrete panel and should be considered carefully. Since the maximum design moment usually occurs at the mid-height of the panel, it is preferable to put the reveal near the top portion of the panel rather than in the middle. Also, vertical indentations tend to have less effect on the strength of the panel and should be preferred whenever possible (see Figure 3).
6. If any projections are to be formed on the panel, it should be cast on the top surface. Attempts to form projections on the surface of the panel facing down is difficult and would require a lot of formwork.
7. When deciding the shape of the corner panels, attention should be given to the method of joint construction which will be employed later. Square corners are easier to form than mitered joints (see Figure 4). Mitered joints require special fabrication techniques to avoid uneven finishes and they chip more easily.

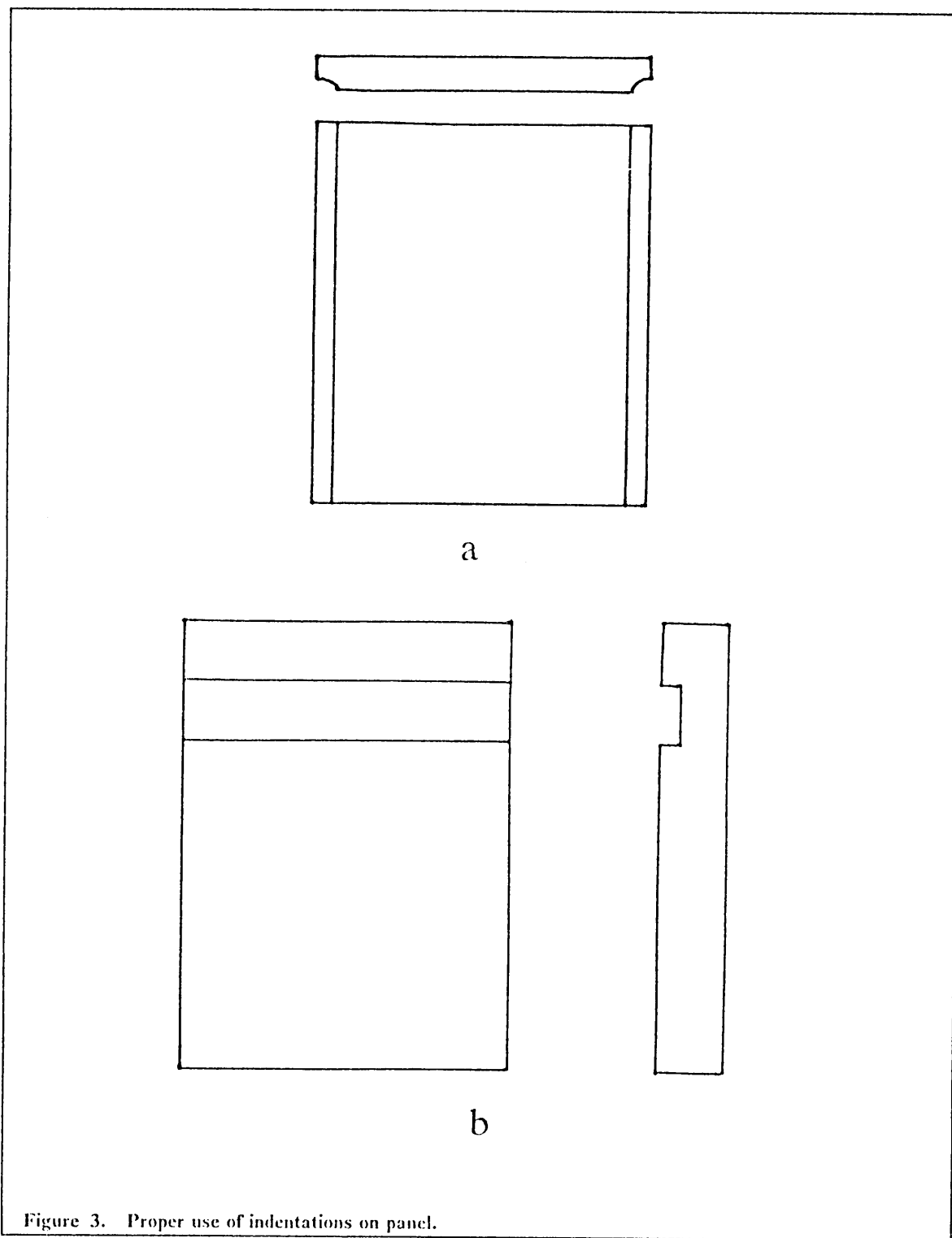


Figure 3. Proper use of indentations on panel.

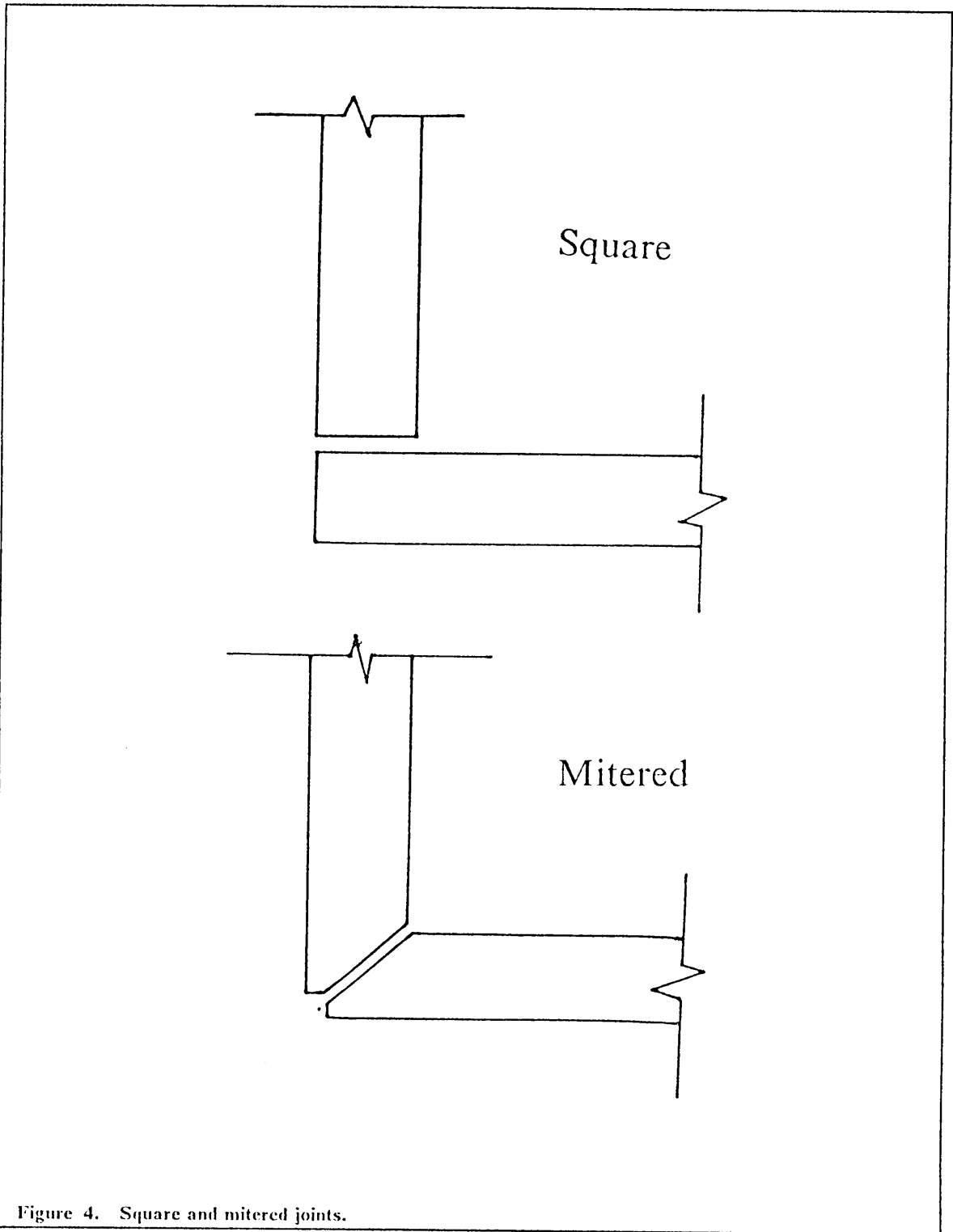


Figure 4. Square and mitered joints.

3.3 *Bondbreaker*

One of the most important and crucial factors to a successful tilt-up project is the proper use and application of the curing compound and bondbreaker. The bondbreaker prevents or reduces the mechanical bond between the casting surface and wall panel, and between panels themselves if stack cast. It helps to produce a clear cleavage during lifting.

There are three basic types of bondbreaker:

1. Synthetic petroleum hydrocarbon resin solutions
2. Solutions of waxes with metallic soaps
3. Solutions of organic esters and silicones.

The resin and wax/soap type of bondbreaker prevent the contact between concrete surfaces by providing a film between them. The ester/silicon type reacts with the concrete surfaces of casting slab and wall panel to prevent set and thus formation of bond. The ester as well as the soap type can be applied while the concrete surface is still damp thus reducing early moisture loss whereas the resin type can only be applied when the bleed water has disappeared totally. However, the soap/wax type tend to leave residues on the concrete surface and interfere with painting and other adhesive applications, while the resin type will oxidize in due time and leave little or no residue.

In the early days lard, chalk, and castor oil were used to separate panels. Since then, consistent research has created bond breaking compounds that are inexpensive, effective, and reliable.

Most contractors prefer to use one compound which will serve both as the curing agent and bondbreaker. This helps to avoid the use of wrong chemical for either of the functions and there is no problem of incompatibility between two compounds (Thraikill, 1986).

Weather conditions are another factor which has to be considered when selecting a bondbreaker. Some products can be washed away by rain; others may oxidized in a few days as

noted before, especially during hot season. In those circumstances, the bondbreaker must be replaced before the panel is cast (Thraikill, 1986).

The surface finish and any architectural treatment on the wall panel and slab also affect selection of bond breaker. Bondbreaker of ester and soap type leave stains or will discolor the surface and should not be used on panels that will be left exposed. If the panel is to be painted, the bondbreaker must be of the type that does not transfer to the concrete surface or that will oxidize easily, such as the resin type. If such a bondbreaker is not available, the coating can be removed by sandblasting, steam cleaning or chemical cleaning before painting (PCA Publication, 1970).

Because the ester and soap type of bondbreaker leave a layer of residue which affects the adhesion of flooring materials like the resilient floor tiles, care should be taken so as not to over spray the casting slab (Courtois, 1980).

Since bondbreaker will damage the bond between reinforcing bars and concrete, it should be applied before the bars are in place. If the panel is to have a broom finish, it usually requires a little more bondbreaker since it has greater surface area than smooth panels. The supplier of bondbreaker is usually able to offer some assistance on the recommended rate of coverage in this case. During hot and windy condition, it is a good practice to follow up the application of bondbreaker with a fog coat of water. The water will fill up the pores which will otherwise draw the bondbreaker into them and the coating on the surface will then be reduced (Courtis, 1980).

The bondbreaker is usually applied in two passes at right angle to each other about 15 minutes apart (PCA, Tilt-up Concrete Buildings, 1982). This helps to prevent thin spots in the coverage. For best results, it should be sprayed with a straight back-and-forth walking motion until the whole area is covered in parallel sweeps. Walking on the surface should be avoided until the coating is dry. To test the effectiveness of the bondbreaker, water can be sprinkled on the surface. If the water spreads and soaks in, an additional light spray is necessary (Burke Co., 1984).

3.4 Panel Casting

Since the floor slab is used as the casting surface in most tilt-up projects, it should be placed as soon as possible. After the slab has been placed, cured, has attained the required strength, and cleaned of any dirt, it is ready as the working surface. The wall panels should be cast as close to their final positions as possible. First of all, the panels are chalked off directly on the slab. After that, the edge forms are constructed along with any windows, doors, or other openings that will not stay in the panel. Windows can be formed by casting directly against the steel sash. Smaller blockouts are achieved by using short sections of tubes, pipes, or tin cans. However, they must be galvanized to prevent rusting if they are to be left in the panels. The edge forms should either be securely bolted or weighted down to prevent movement (PCA Publication, 1970). There are four ways of doing this (Burke, 1984):

1. By bolts drilled into the floor for anchorage.
2. By nails driven into the floor with power actuated tools.
3. By gluing edge forms on wooden plates to the floor surface.
4. By bracing edge forms against each other.

Also, it is important to make sure that the forms are aligned square. A three quarter inch chamfer strip can be included at the edge to prevent marring of slab surface during panel erection.

When the edge forms and the items which are not supposed to stay in the panel are completely installed, a second layer of bondbreaker compound should be applied to the casting surface and the forms. The first layer of the compound was applied to the casting surface as the curing agent before the forms are constructed. When the second coat of the bondbreaker has dried, it will be the time to place the reinforcing steel, inserts, and any other items which need to be embedded.

The door and window frames that will remain in the panel should also be placed at this stage (McKinney, 1980).

One of the functions of the reinforcement is to control shrinkage cracking and volume changes due to temperature and moisture variation. Most building codes require reinforcement in the range of 0.10 to 0.25 percent of the cross sectional area of the concrete (Spears, 1980). For example ACI 318-83 (14.3.2 and 14.3.3) require the minimum vertical and horizontal reinforcement of 0.12 and 0.20 percent respectively, for deformed bars not larger than #5 and having a yield strength of at least 60,000 psi. Most contractors find that #4 deformed bars at 16 inches on center each way will fulfill the minimum reinforcement requirement in a 6 inches thick panel. The reinforcement as mentioned above is about 0.208 percent.

Welded wire fabric can also be used to achieve the purpose of minimum reinforcement. Additional reinforcement in the panel may be required if the panel is being designed as load bearing unit, or because certain portions of the panel will be overstressed during lifting. More rebars are also required around openings, especially to prevent diagonal cracking (see Figure 5). ACI 318-83 (14.3.7) requires at least two #5 bars around all window and door openings. The bars must be extended beyond the corners of the opening by at least their development length and greater than 24 inches. In any case, it is important to place the rebars on some kind of support or chair (usually galvanized or plastic coated) to keep it in the correct place. The reinforcement is usually placed at/near the middle of the panel thickness and rarely exceeds two layers. Various inserts for lifting, bracing, and possibly strong back are placed next. The inserts must be securely tied to the rebars as shown in Figure 6 to prevent them from being displaced during concrete placing. Lifting inserts must be located at the exact position as shown in the drawings. If that is not possible because of obstruction of rebars or some other reasons, the designer must be notified to locate an alternate spot. Bracing and strong back insert locations are not as critical and may be placed within 6 in. of the position shown on the drawings (McKinney, 1980). The inserts must be set perpendicular to the panel surface to ensure proper bearing and tightening of the bolt which will minimize the bending of the bolt and crushing of the surrounding concrete which may then lead to insert failure during erection. The inserts are usually placed at a set back of three-eighth to half an inch below

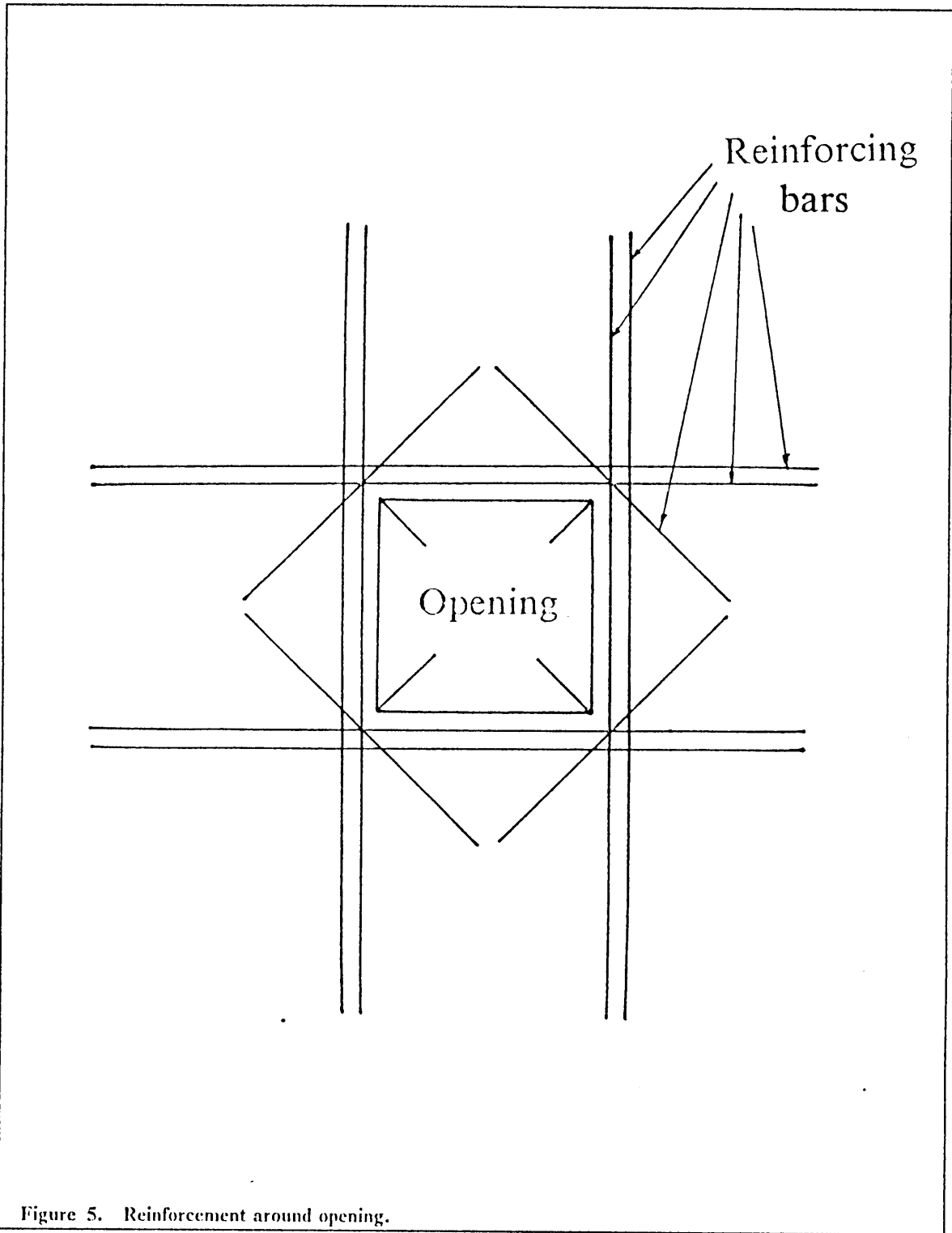


Figure 5. Reinforcement around opening.

the finished panel surface depending on the recommendation of the manufacturer (see Figure 7). Under no circumstance should an insert be welded to rebars because welding causes embrittlement and can produce a sudden, brittle failure (Dayton Superior, 1983).

Before the concrete is placed, the forms must be moistened so that they will not absorb water from the concrete and swell. All inserts must be cleaned and the proper plugs or slotted lagstud (which mask out the concrete) are installed. Care must be taken to see that the inserts and the plugs are not disturbed during the placement of the concrete. The panel should be cured as soon as possible, after the concrete has been properly compacted and consolidated. Delay in application of curing coat may cause significant moisture loss and affect surface quality of concrete, especially in hot weather. A chart by Burke Co. showing various factors affecting the rate of evaporation of surface moisture is shown in Figure 8. From the chart, it can be seen that if the air and concrete temperature are increased by 20 degree F , or if the wind velocity is increased by 10 mph, the rate of evaporation can be increased several fold.

Sometimes the panels are stack-cast because of lack of casting area. A second coat of bondbreaker on the panel is required since the previous panel is now serving as the casting surface. When the lower panel has an opening, it should be filled with sand, wetted down, and then a thin coat of plaster is cast over the fill. It should be finished to match the panel and then the bondbreaker is applied (Courtois, 1980).

The panels are usually cured for about 10 days and then the forms are stripped (Hodges, 1980). However, the contractor should not attempt to erect the panels until the concrete has attained its required strength.

Some contractors find it more convenient to delay the construction of foundation/footing until the panels are cast, if the floor slab is used as the casting surface. This will enable the mixer truck to reach and to move freely around the casting area. Also, it is useful to plan ahead the casting locations of all corner panels.

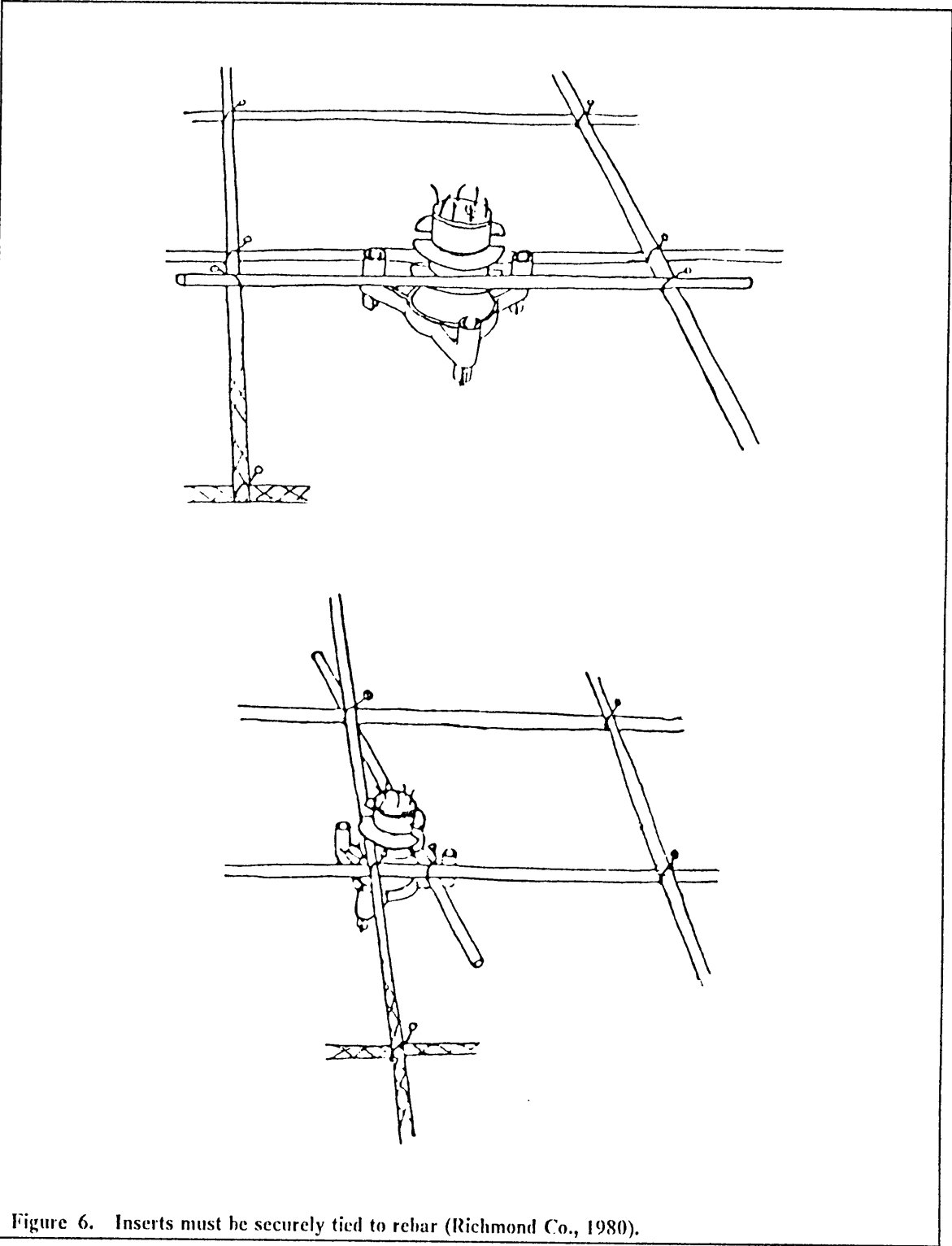
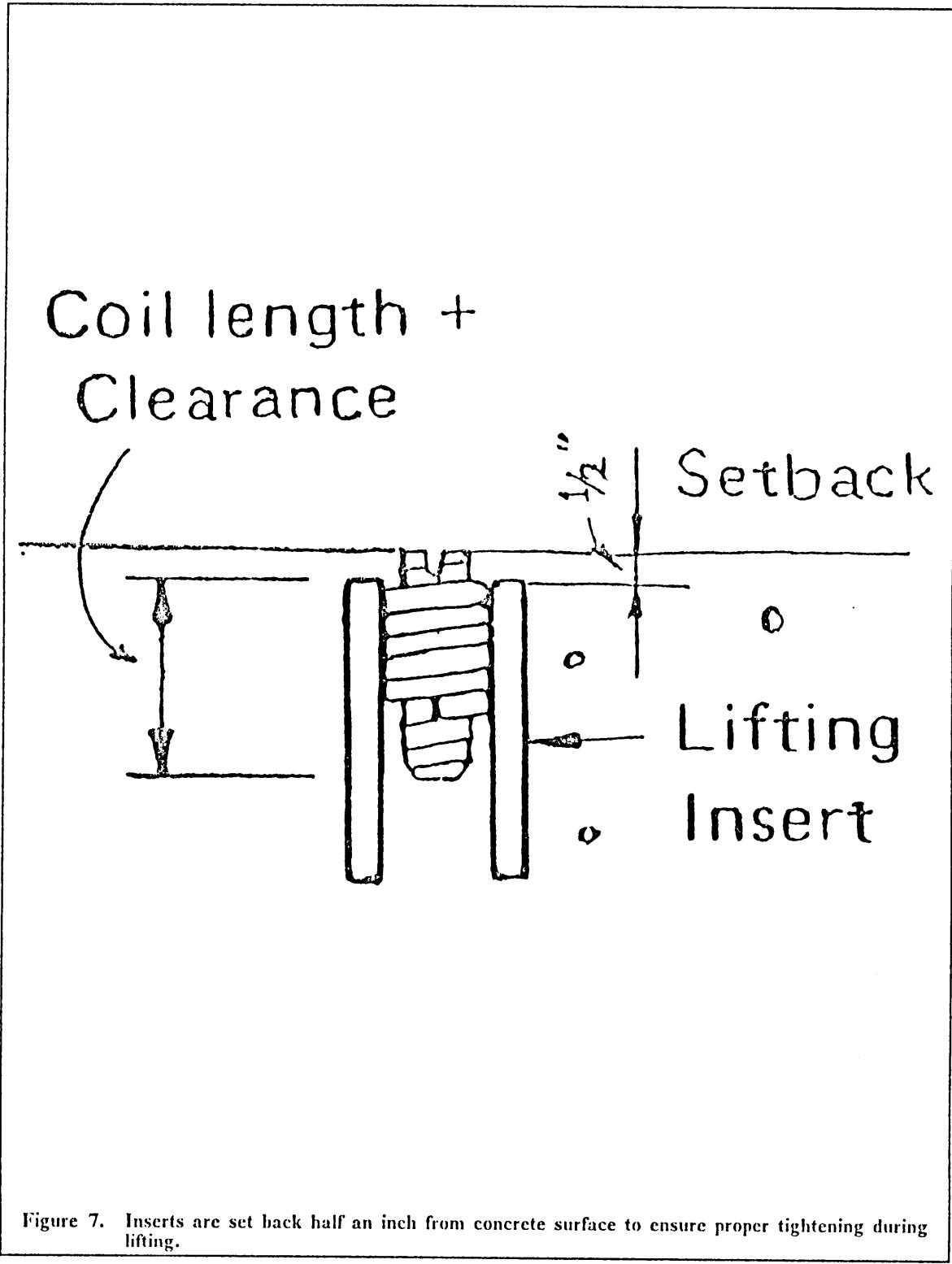


Figure 6. Inserts must be securely tied to rebar (Richmond Co., 1980).



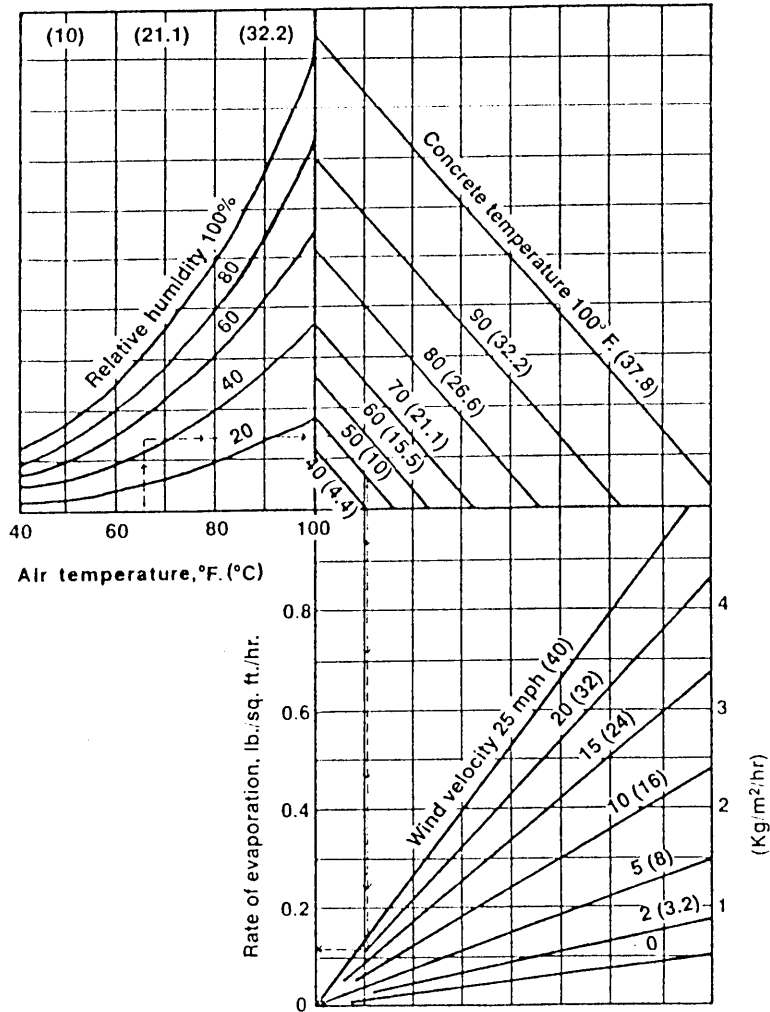


Figure 8. Factors affecting rate of evaporation from concrete surface (Burke Co., 1984).

4.0 TILT-UP HARDWARE, RIGGING, AND ERECTION OF WALL PANEL

4.1 *Hardwares and Accessories*

There are a number of items specifically pertinent to tilt-up construction such as inserts, braces, lifting plates, and various kinds of filler plugs. Proper selection and use of these accessories during various stages of tilt-up construction will help to ensure their performance and yield the result desired.

4.1.1 Inserts

Inserts can be classified in two different ways:

1. Based on the position they are placed on the wall panel, inserts can be classified as either face insert or edge insert. The inserts which position the bolts in the face of the panel are called face

inserts while the edge inserts are used to position the bolts at the edge of the panel. An example of a face and an edge insert is shown in Figure 9a.

2. Based on the number of helix coils and therefore the number of bolts required to attach the lifting plate to the insert(s), they can be classified as either single or double pickup inserts. An example of a double pickup insert is shown in Figure 9b. The standard spacing between the coils of a double pickup insert is 12 in. on centers.

Each insert is designed with a specific safe working load in shear and tension. Larger size inserts will have higher safe working load. The capacities of inserts are usually verified by manufacturers by physical testing. The insert's tensile strength can also be predicted with reasonable accuracy by various empirical formulae (Richmond Co., 1980). External tension loads on the inserts are resisted through the lifting hardware. External shear loads, however, must be opposed by shear and 'heel and toe' forces. A free body diagram of an insert with lifting plate under shear load is shown in Figure 10 on page 26 (Richmond Co., 1980).

$F_h = \text{horizontal force component}$

$R_h = \text{opposing shear}$

$P_{brg} = \text{heel and toe forces}$

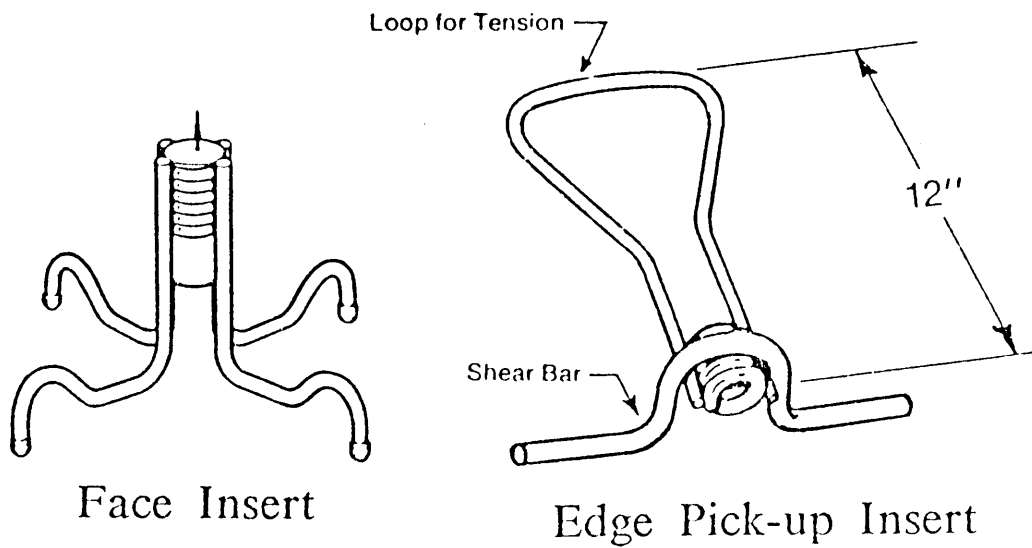
Summing moments about point a:

$$F_h e - P_{brg}(l - x) = 0 \quad (a)$$

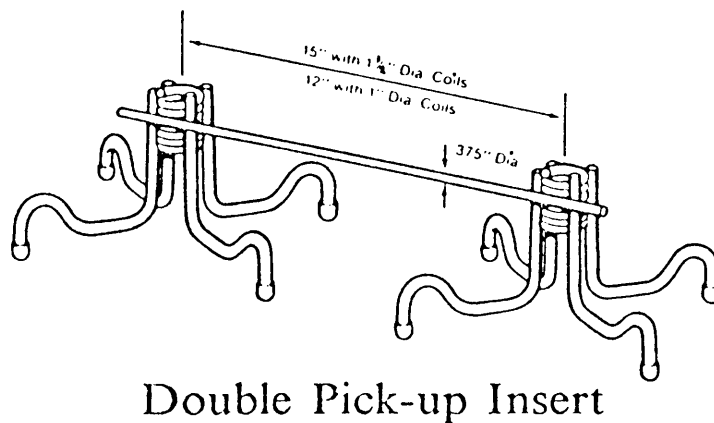
Summing forces in the vertical direction:

$$T - P_{brg} = 0$$

or $P_{brg} = T$

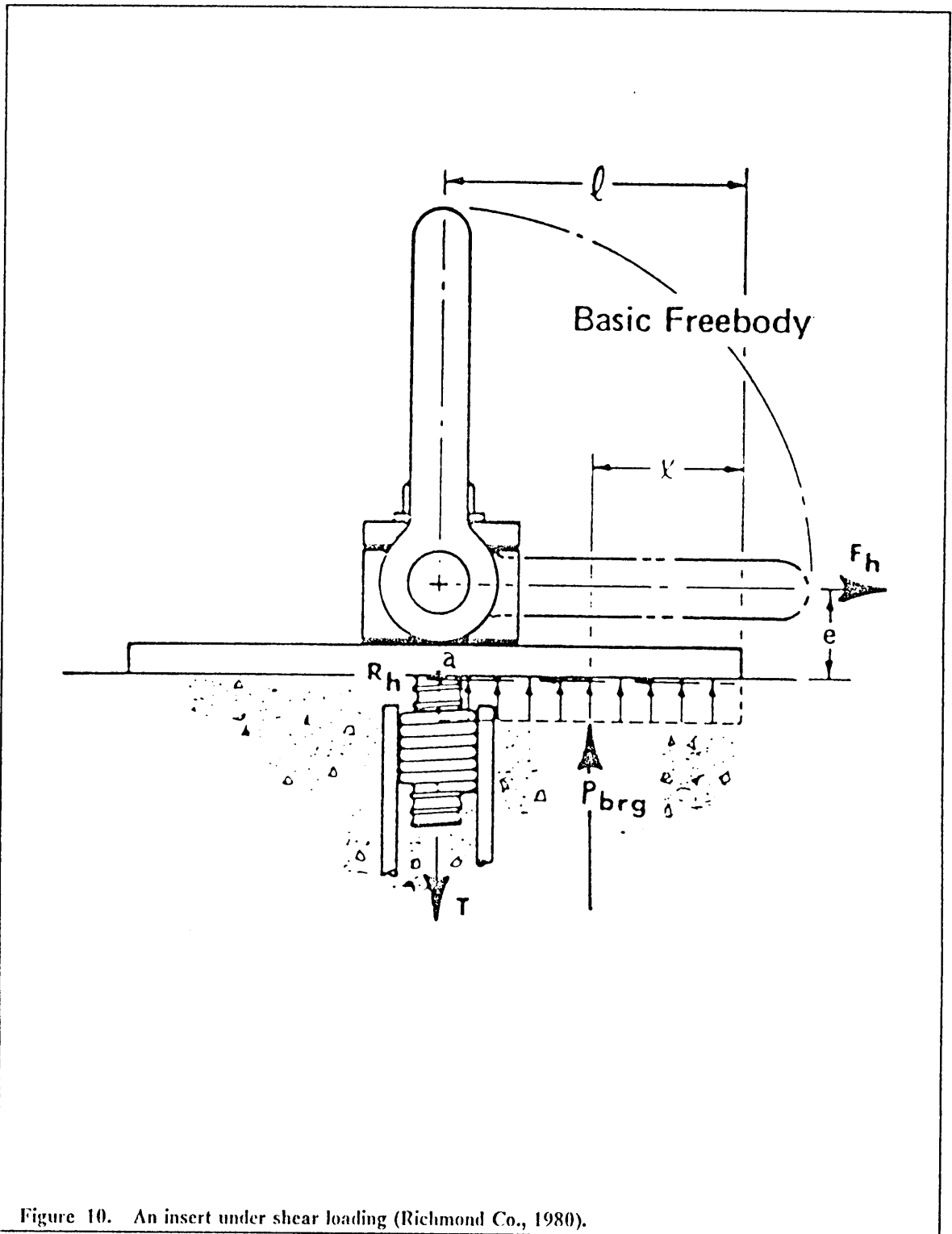


a



b

Figure 9. Pickup insert classification.



Substitute equation for P_{brg} into (a) we get:

$$F_h e = T(l - x)$$

$$\text{or } T = \frac{F_h e}{(l - x)}$$

where $(l - x)$ is the distance from the bolt center line to the center of pressure, P_{brg} . If a uniform distribution of pressure P_{brg} is assumed, then

$$x = \frac{l}{2}$$

and therefore

$$l - x = \frac{l}{2}$$

Equation for T then becomes:

$$T = 2F_h \frac{e}{l}$$

where T=tension in the bolt. It is important that the bolt holes in the lifting plate match bolt diameters, otherwise slippage may occur and both the concrete as well as the hardware will be subjected to dynamic loading.

Inserts can fail in several possible ways when loaded. Basically, there are 4 different modes of insert failure (Dayton Superior, 1983):

1. The insert is pulled out of concrete because of bond failure between insert and concrete (see Figure 11a). This usually occurs in low strength concrete.

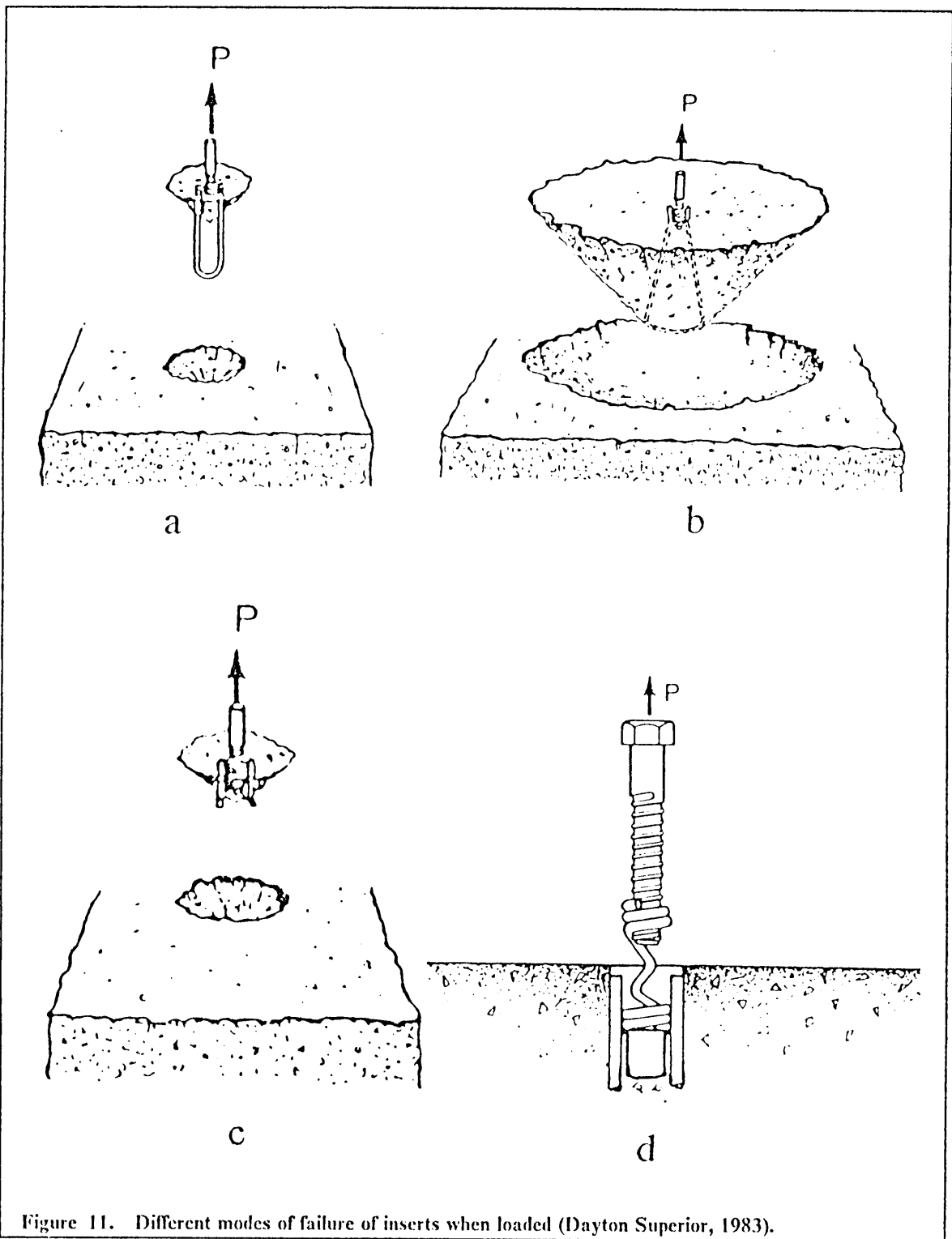
2. The insert together with a cone of concrete may be pulled out of the panel (see Figure 11b). This type of failure will occur when the tensile strength of the 'shear cone' is less than the strength of the insert.
3. The insert may break at a point just below the coil, pulling out a small cone of concrete with it (see Figure 11c). This usually occurs in high strength concrete when the insert is overloaded.
4. The coil of the insert may unwind if the bolt does not extend about $\frac{3}{4}$ in. beyond the bottom of the coil (see Figure 11d).

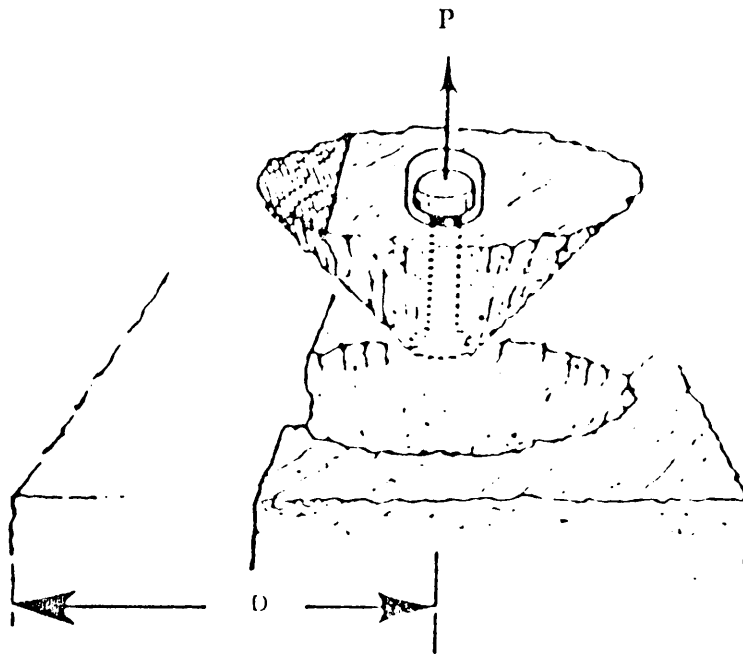
In addition to the guidelines for placing the inserts in the panel as discussed in Chapter 3 on panel casting procedures, many manufacturers also recommend a minimum distance an insert may be placed from a free edge or opening. The minimum distance varies slightly from supplier to supplier, usually in the neighborhood of 15 in. If this minimum distance is not observed, the working load of the insert may be greatly reduced because the effective area of the shear cone will be diminished, as shown in Figure 12 on page 30 (Dayton Superior, 1983).

Other than lifting inserts, there are also inserts for bracing and strongback applications. Special purpose inserts such as the inverted slab bracing insert is also available. It is used to place the anchorage coil at the bottom of the poured concrete so that it is available for fastening when bracing is to be done on the 'down' side of the panel.

4.1.2 Plug

There are several types of plugs which are used to mask out the concrete and prevent grout from leaking into the coil during concrete placement. The plug has to penetrate at least $1\frac{1}{4}$ in. beyond the bottom of the coil to ensure enough clearance during lifting. This will prevent 'bottoming' and allow the bolt to be tightened properly.





D = Insert Minimum
Edge Distance

Figure 12. Insert placed too near to free edge, causing a reduction in effective shearcone (Dayton Superior, 1983).

Some plugs have a flexible tail at the top which springs back after screeding and allow the insert to be located easily. There are also filler plugs available for filling the voids left by inserts to reduce the patching time required and grout usage.

4.2 *Lifting Plate*

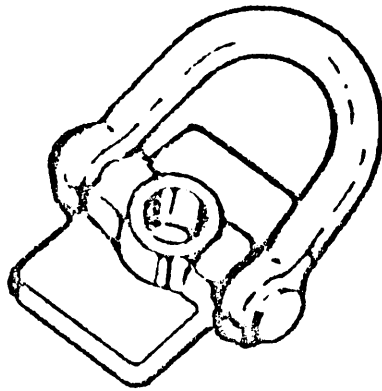
Various lifting plates and lifting angles are available from different suppliers. The lifting plate is used to connect the cable to the insert during erection. A typical lifting plate and a lifting angle are shown in Figure 13 on page 32. The lifting plate is used in connection with the single pickup coil insert, and the lifting angle is used with the double pickup face insert. There are also lifting plates for edge inserts.

For panels with exposed aggregate cast face up, special care must be taken to prevent the plate from damaging the surface. Usually a grout pad is used to provide a bearing surface and a longer bolt will be required in this case. See Figure 14 on page 33.

4.3 *Braces*

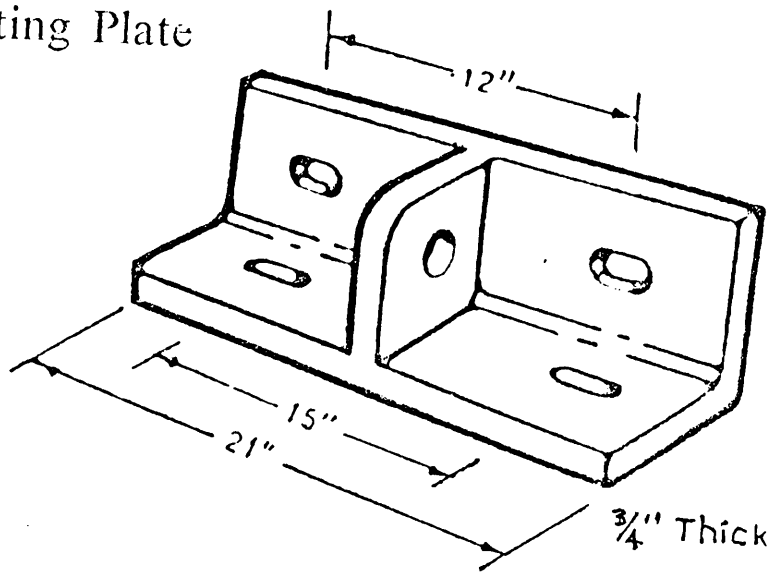
Braces are used to support the wall panels temporarily after they are tilted into place. At least two braces per panel are required to achieve stability. Braces come in several sizes and lengths to accommodate a wide range of wall widths and heights. A tilt-up panel of 50 feet or even higher is not uncommon nowadays. The brace spacing or the number of braces per panel required depends on the panel height, the brace capacity (and the length it is extended), and the lateral load the panel will be subjected to before it is being permanently joined to the remaining components of the

Lifting Angle



a

Lifting Plate



b

Figure 13. Lifting plate and lifting angle.

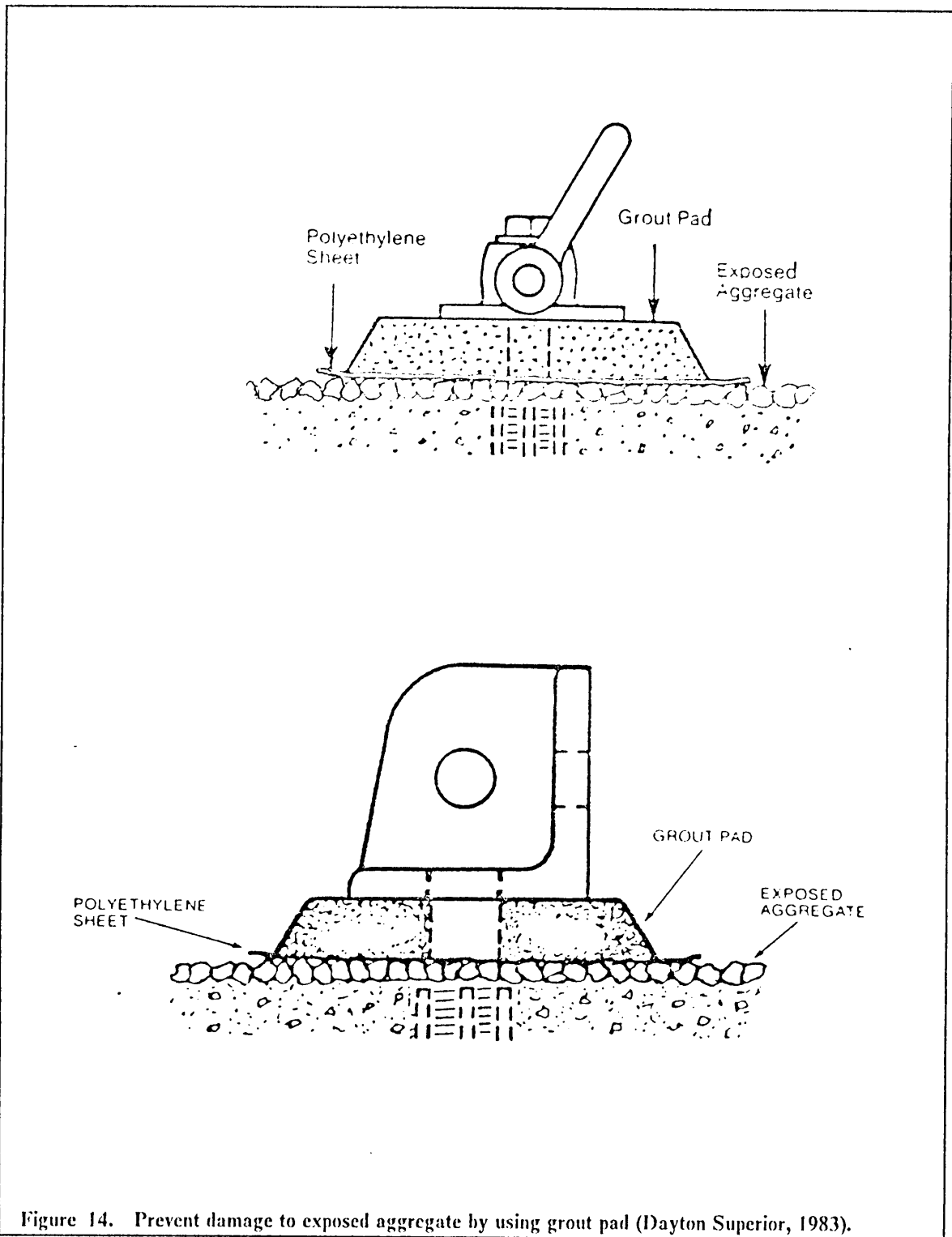


Figure 14. Prevent damage to exposed aggregate by using grout pad (Dayton Superior, 1983).

structure. The lateral load considered in this case is usually the wind load and its design value depends on the wind velocity. A chart given by the Burke Company showing the relationship between pressure and wind velocity is shown in Figure 15 on page 35. The chart was developed based on the Occupational Safety and Health Act (OSHA) standard. The minimum wind load required by OSHA is 10 psf which corresponds to a wind velocity of about 45 mph. However, it should be noted that wind velocity also varies with geographical location and seasonal conditions. The amount of force exerted by wind will be greater if it is accompanied by rain, snow, or dust and sand (Burke Co., 1984). Therefore the designer must be aware of any additional requirements recommended by local agencies.

The supplier usually has charts or tables to assist the designer in the selection of brace spacing. One such table by Richmond Co. is shown in Table 1 on page 36. (Refer to Figure 16 on page 37 for the physical meaning of brace working dimension mentioned in the table). These charts or tables are usually based on the assumption of 10 psf of wind load. For values of wind load greater than 10 psf, the technical service department of the supplier is usually available for assistance in determining the bracing requirements. The panel should be braced at least 12 in. from any free edge or opening. Brace spacing should not be increased when there are openings on the panel because of possible suction at the back (Burke Co., 1984). The manufacturers also specify the ways braces should be connected to the wall panel and floor slab, as shown in Figure 16a and b. Note that there are some differences in the working dimensions between companies, therefore braces should be connected in the manner suggested by its supplier.

Most braces have telescoping capability for quick and easy adjustment. For high walls, lateral and knee bracing are also required. See Figure 17 on page 38. Knee bracing prevents buckling and its position varies slightly with supplier's recommendation. The knee brace should make an angle of approximately 90 degrees with the pipe brace. Lateral braces must be continuous and must be tied to an end brace at the end of each line or preferably after every 4 to 6 panels (McKinney, 1980). For the braces to develop their working load, they must be arranged so that they appear to be perpendicular to panel surface in the plan view as shown in Figure 18 on page 39.

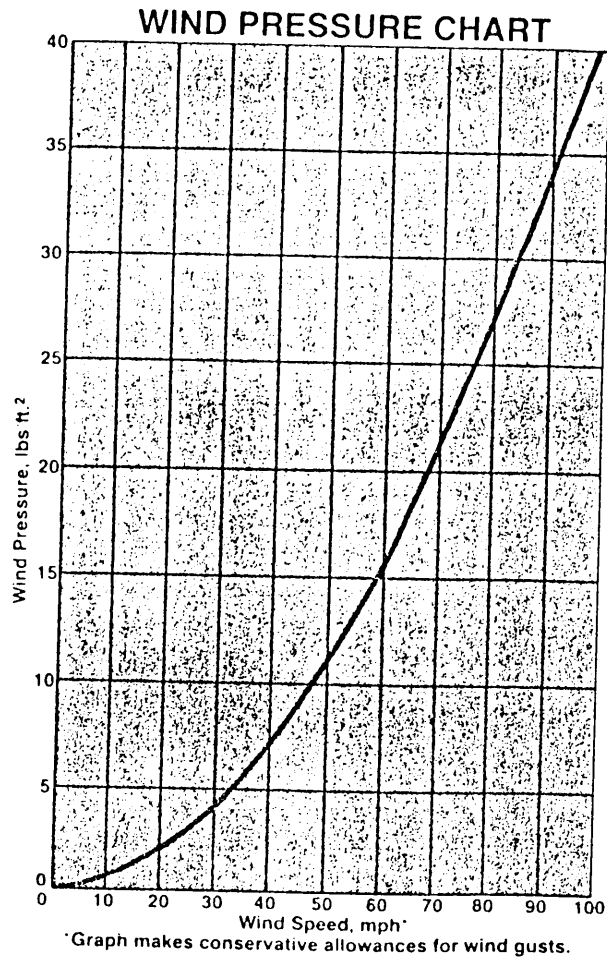


Figure 15. Variation of lateral pressure with wind velocity (Burke Co., 1984).

Table 1. A Brace Selection Table: Richmond Co.

MAXI - BRACE DATA CHART

PANEL HEIGHT (H)*	BRACE WORKING DIMENSIONS			BRACE SPACING **	
				2:1	1.5:1
	0.667H	0.385H	0.770H	SAFETY FACTOR	
LONG MAXI - BRACE					
51'	34'-0"	19'-7"	39'-3"	3'-1"	4'-2"
50'	33'-4"	19'-3"	38'-6"	3'-8"	4'-10"
49'	32'-8"	18'-10"	37'-8"	4'-3"	5'-8"
48'	32'-0"	18'-5"	36'-11"	5'-0"	6'-8"
47'	31'-4"	18'-1"	36'-2"	5'-9"	7'-9"
46'	30'-8"	17'-8"	35'-5"	6'-4"	8'-6"
45'	30'-0"	17'-4"	34'-7"	7'-2"	9'-7"
44'	29'-4"	16'-11"	33'-10"	8'-2"	10'-11"
43'	28'-8"	16'-6"	33'-1"	9'-3"	12'-4"
42'	28'-0"	16'-2"	32'-4"	9'-6"	12'-6"
41'	27'-4"	15'-9"	31'-0"	9'-9"	13'-0"
40'	26'-8"	15'-4"	30'-9"	10'-0"	13'-4"
39'	26'-0"	15'-0"	30'-0"	10'-3"	13'-8"
38'	24'-4"	14'-8"	29'-3"	10'-6"	14'-0"
37'	24'-8"	14'-3"	28'-6"	10'-9"	14'-5"
36'	24'-0"	13'-10"	27'-8"	11'-1"	14'-9"
35'	23'-4"	13'-5"	26'-11"	11'-5"	15'-0"
34'	22'-8"	13'-1"	26'-2"	11'-9"	15'-0"
33'	22'-0"	12'-8"	25'-4"	7'-1"	9'-5"
32'	21'-4"	12'-4"	24'-7"	7'-8"	10'-3"
31'	20'-8"	11'-11"	23'-10"	8'-8"	11'-7"
INTERMEDIATE MAXI - BRACE					
42'	28'-0"	16'-2"	32'-4"	6'-5"	8'-7"
41'	27'-4"	15'-9"	31'-6"	6'-11"	9'-3"
40'	26'-8"	15'-4"	30'-9"	7'-4"	9'-10"
39'	26'-0"	15'-0"	30'-0"	8'-0"	10'-8"
38'	25'-4"	14'-8"	29'-3"	8'-8"	11'-7"
37'	24'-8"	14'-3"	28'-6"	9'-5"	12'-7"
36'	24'-0"	13'-10"	27'-8"	10'-5"	13'-10"
35'	23'-4"	13'-5"	26'-11"	11'-5"	15'-0"
34'	22'-8"	13'-1"	26'-2"	11'-9"	15'-0"
33'	22'-0"	12'-8"	25'-4"	12'-1"	15'-0"
32'	21'-4"	12'-4"	24'-7"	6'-6"	8'-9"
31'	20'-8"	11'-11"	23'-6"	7'-11"	10'-6"
STANDARD MAXI - BRACE					
30'	20'-0"	11'-6"	23'-1"	10'-0"	13'-5"
29'	19'-4"	11'-2"	22'-4"	10'-9"	14'-5"
28'	18'-8"	10'-9"	21'-6"	5'-0"	6'-8"
27'	18'-0"	10'-4"	20'-9"	5'-9"	7'-9"
26'	17'-4"	10'-0"	20'-0"	6'-6"	8'-8"
25'	16'-8"	9'-7"	19'-3"	7'-4"	9'-9"
24'	16'-0"	9'-2"	18'-6"	8'-4"	11'-1"
23'	15'-4"	8'-10"	17'-8"	9'-5"	12'-6"
22'	14'-8"	8'-5"	16'-11"	10'-7"	14'-1"
21'	14'-0"	8'-1"	16'-2"	11'-11"	15'-0"
20'	13'-4"	7'-8"	15'-4"	13'-4"	15'-0"
19'	12'-8"	7'-3"	14'-7"	14'-0"	15'-0"
18'	12'-0"	6'-11"	13'-10"	14'-10"	15'-0"
LOW-WALL MAXI - BRACE					
16'	10'-8"	6'-2"	12'-4"	12'-6"	15'-0"
15'	10'-0"	5'-9"	11'-7"	13'-4"	15'-0"
14'	9'-4"	5'-5"	10'-9"	14'-3"	15'-0"
13'	8'-8"	5'-0"	10'-0"	15'-0"	15'-0"
12'	8'-0"	4'-7"	9'-3"	15'-0"	15'-0"
11'	7'-4"	4'-3"	8'-6"	15'-0"	15'-0"
10'	6'-8"	3'-10"	7'-8"	15'-0"	15'-0"

☐ KNEE & LATERAL BRACING REQUIRED.
 * Panel Height (H) dimension is from finished floor to top of panel.
 ** Maximum spacing width per brace. Chart is based on a wind loading of 10 lbs. per square foot.

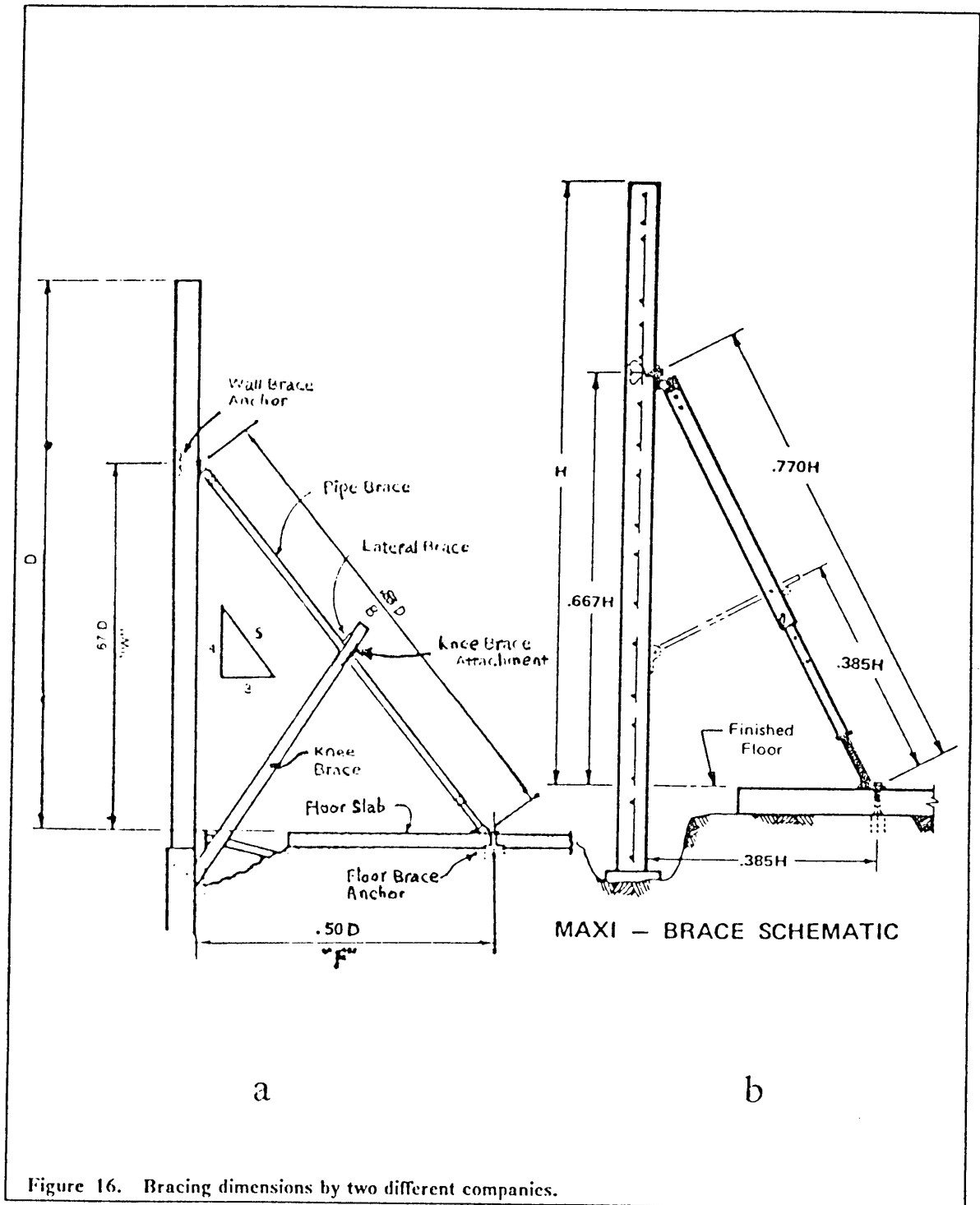


Figure 16. Bracing dimensions by two different companies.

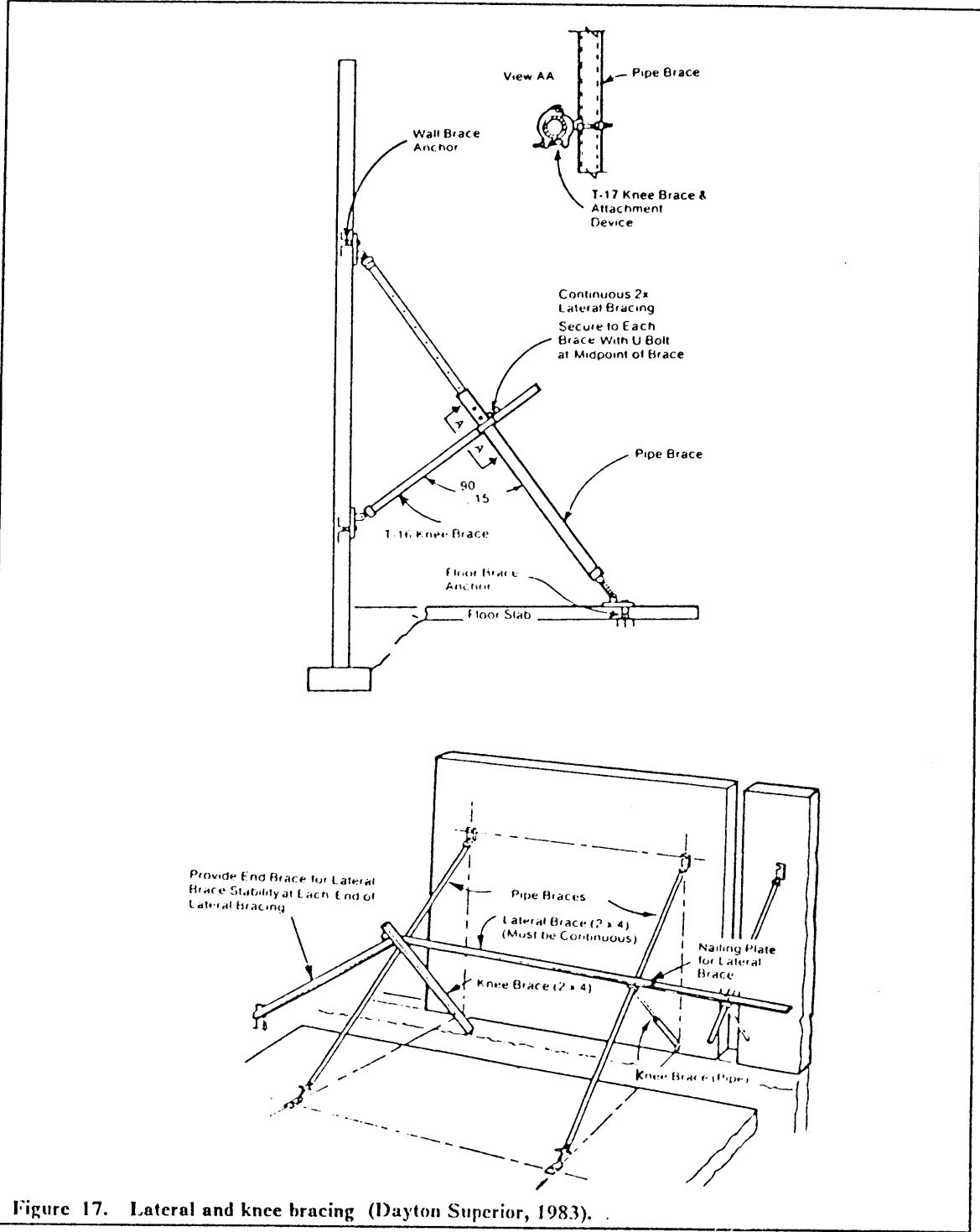


Figure 17. Lateral and knee bracing (Dayton Superior, 1983).

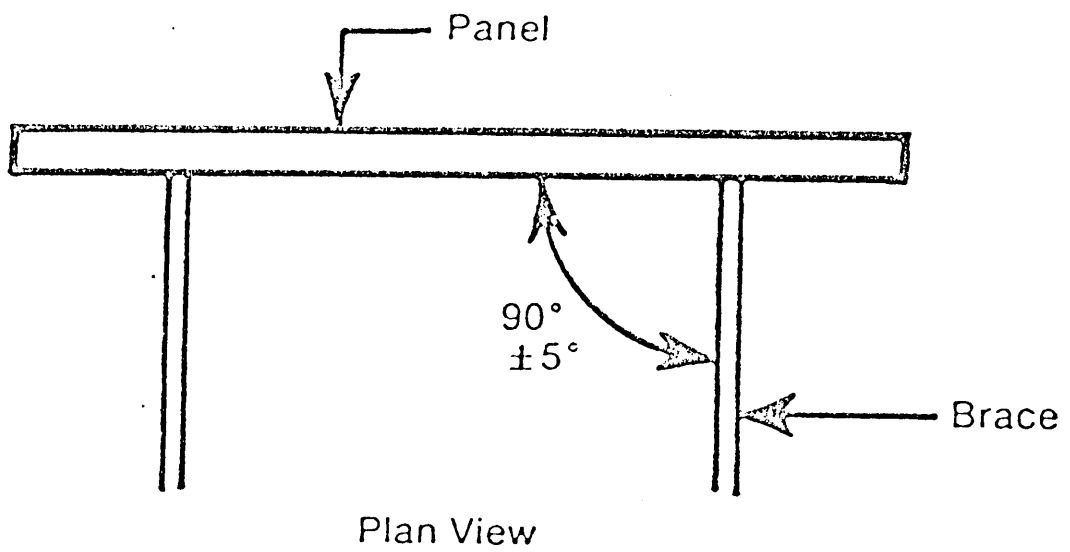


Figure 18. Pipe braces must be perpendicular to panel surface: in the plan view.

Each manufacturer has a complete set of accessories to go with their braces; such as the wall and floor bracket, brace anchor, and inserts. Every attempt should be made not to mix the accessories of one company and use them in conjunction with the products of another company.

4.4 Rigging

Rigging is an important consideration to achieve the speed and economy of tilt-up construction. Mobile crane time is very expensive and every effort should be made to minimize the time loss.

There are two ways of describing the rigging configuration:

1. The 'high' system. For example a 2x4 III rigging has 2 columns and 4 rows of lifting inserts. Similarly, a 6x2 III rigging has 6 columns and 2 rows of inserts. The second number in the designation indicates the number of inserts in the 'high' (top to bottom) direction.
2. The 'R' system. For instance R-23 has 2 rows and 3 columns of inserts and R-14 has 1 row and 4 columns of inserts.

Figure 19 on page 41 shows the basic rigging configurations that are common in tilt-up construction (Richmond Co., 1980). Every rigging configuration needs a specific set of cable/sling, the lifting hardware, as well as the spreader bar to go along with it. The minimum sling length requirement of the manufacturer must be met. This minimum sling length is necessary to avoid excessive side pull on the inserts which may cause inserts to fail or a panel to crack (Kelly, 1986). The distance between inserts that are connected by a single cable usually dictates the minimum length required. Figure 20 on page 43 shows the minimum required sling length in a few typical rigging configurations (Dayton Superior, 1983; McKinney, 1980; Richmond, 1980). This minimum

TYPICAL RIGGING METHODS

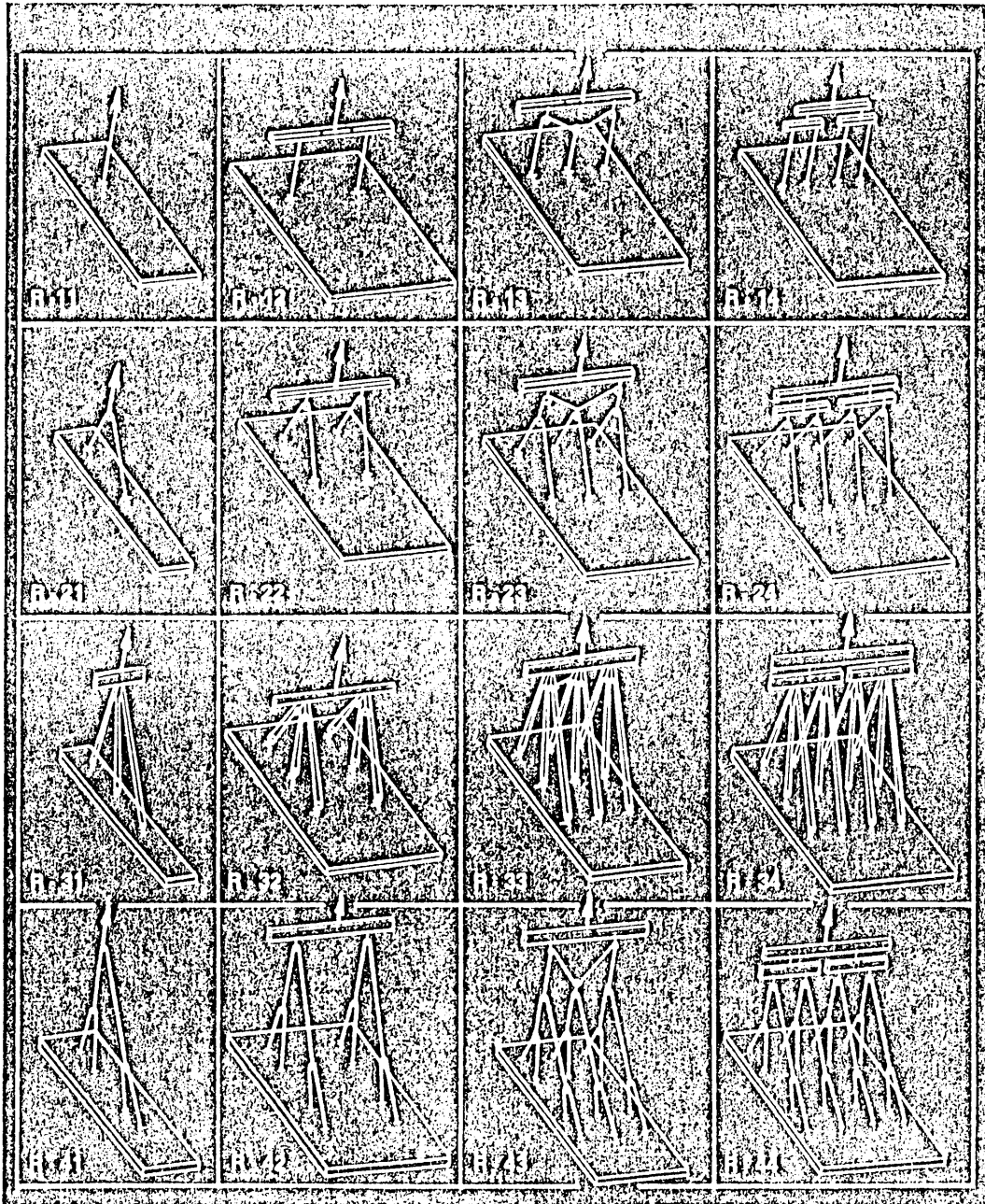


Figure 19. Typical rigging configurations: Richmond Co.

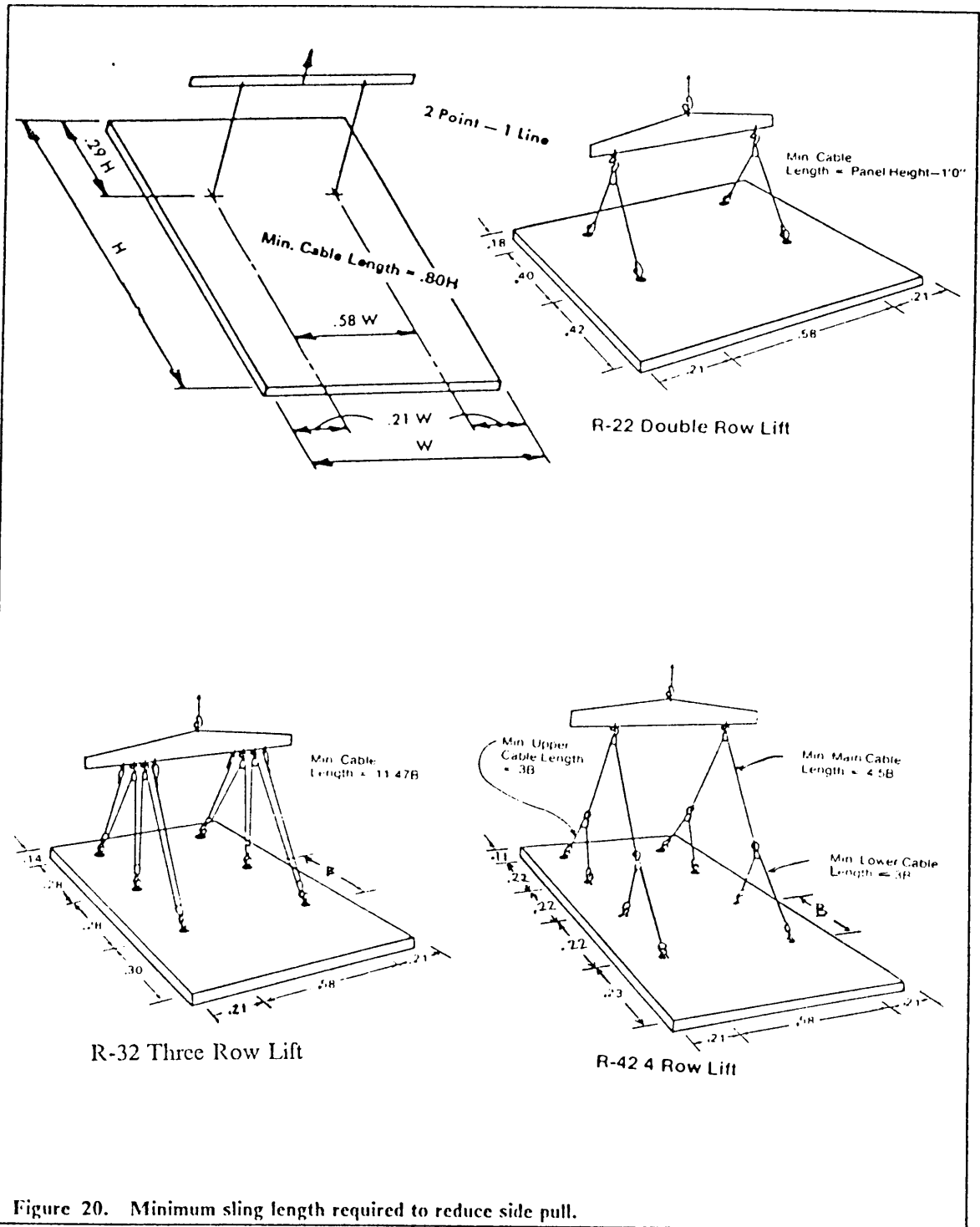
length requirement varies slightly from supplier to supplier and their respective standards must be adhered to.

For speedy and smooth erection of wall panels, changes in rigging must be minimized. Sometimes it is more economical to use more inserts on lighter panels than are required for safe lifting so as to keep the rigging consistent with larger or heavier panels. When rigging changes are absolutely necessary, certain methods may be employed to accomplish the change rapidly as illustrated in Figure 21 on page 44 (Kelly, 1986). From the figure, we can see that a 3 III rigging can be change to a 2 III rigging by leaving the bottom sling idle; and a 2X4 III rigging is changed to a 2X2 III rigging by bringing the bottom pair of sling to the top. It is also advisable to lift all panels requiring one rigging configuration first before a change in rigging is made. This will help avoid having to change it back later and hence save some valuable crane time.

4.5 Erection of Wall Panels

It is a good practice to clear and prepare the building site one or two days before the crane arrives and erection of panels begins. The contractor and crane supervisor must make sure that the crane has access to the panels and thus may require to clean up the area around the panels beforehand. The path that the crane will maneuver during the erection process also has to be cleared so that the crane can proceed smoothly from one panel to another. Special attention has to be paid to overhead power lines and adjacent construction which may pose potential hazards. Debris, form boards, or tools must be removed or they will fall off and possibly injure somebody (Mckinney, 1980).

Lifting of the panels can only be carried out if the concrete has attained its desired strength. Before the lifting begins, braces should be attached to the panel and adjusted to approximate length. This will help to speed up the bracing of panel after it has been lifted. However, this can only be done if the panel is to be braced from the 'upper' face (Mckinney, 1980). Therefore it is advisable



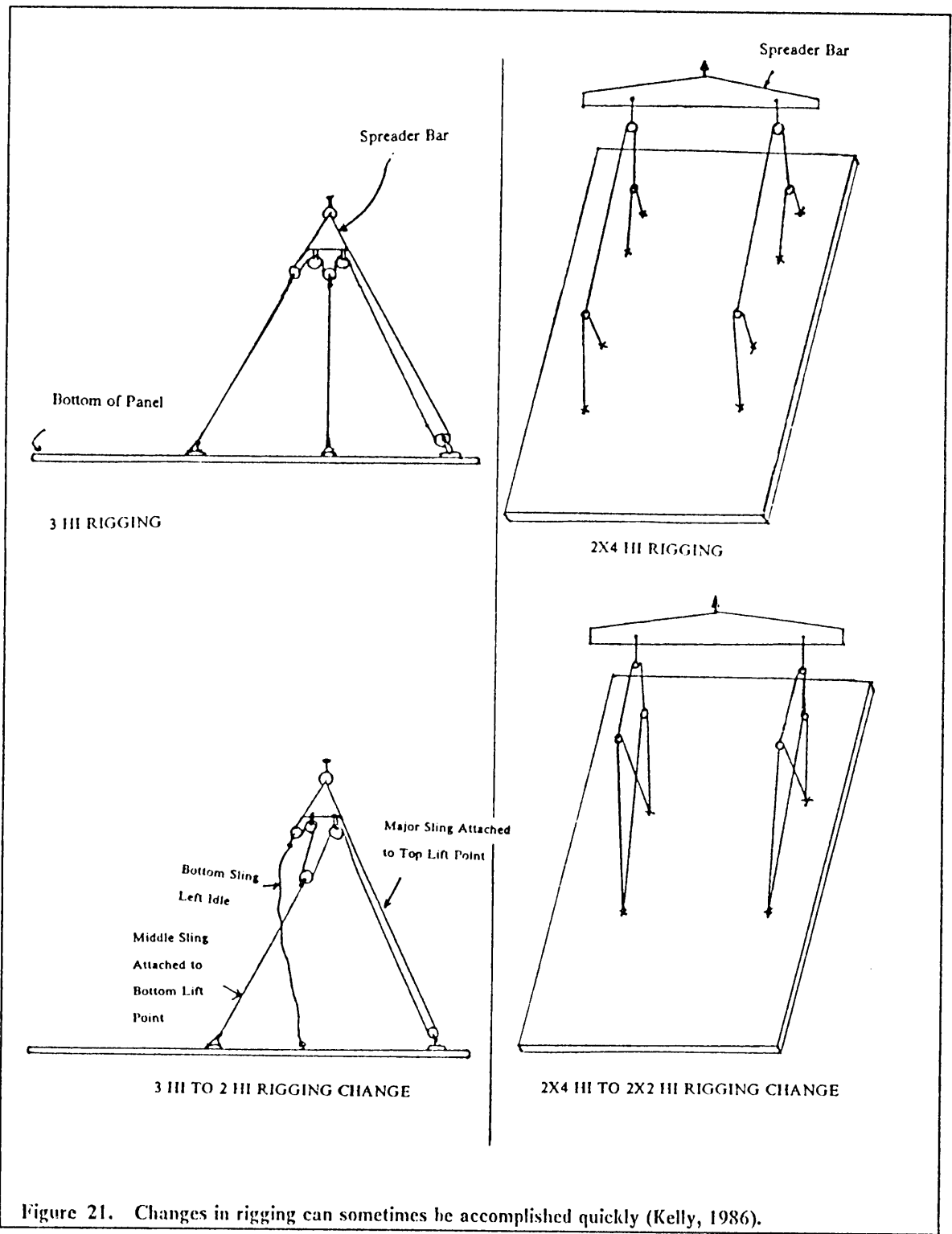


Figure 21. Changes in rigging can sometimes be accomplished quickly (Kelly, 1986).

to lift the inside face of the panel from the inside of the building. If the panel is cast with exterior face up and lifted with the crane outside of the building, braces must be added while the crane is still supporting the panel and thus consume a lot of expensive crane time. Alternatively, anchoring points can be provided outside of the building (Kelly, 1986).

Precise placing position of the panel should also be marked on the footings before lifting begins. Sometimes dowels are installed in the footing to help in guiding the panels and to provide lateral resistance between panel and foundation. Leveling shimpaks, shim strips, or grout pads should also be placed on the footing before lifting to save time. An example of shimpak, and a shimpak on footing are shown in Figure 22 on page 46 (Burke Co., 1984; Dayton Superior, 1983).

It is important to have an experienced crane operator who will not exert too much dynamic force on the panel during lifting. Also, if the crane is working outside of the building, the path to be traveled must be as even as possible to reduce impact loading on inserts. The cable used in the rigging should be of the largest size possible to minimize tension stretch. This is because although thinner cables may have adequate strength, their springiness tends to increase impact loading (Burke Co., 1984).

After a panel is lifted, it is then aligned and plumbed. The use of face lifting inserts will prevent the panel from hanging vertical and therefore is difficult to position the panel on the foundation. A second crane line can be attached to edge lift points at the top of the panel and then the load can be transferred from the previous crane line to the second crane line so that the panel hangs vertically. Alternatively, a plumbing block as shown in Figure 23 on page 47 can be used to achieve the same purpose. The panel is lifted first and then it is lowered so that the bottom rests on the ground. The panel is tipped so that the face rests against the slings and then the plumbing block is fastened around the slings to the top of the panel. (See Figure 23 on page 47 (Kelly, 1986)).

After the panel is plumbed and checked in different directions, braces may then be set. Many contractors use drilled in type anchor in the floor slab. The hardware supplier also provides special inserts for this purpose. Knee and lateral bracing should also be provided according to manufacturer's recommendation. The crane will then be able to release the panel and proceed to lift the

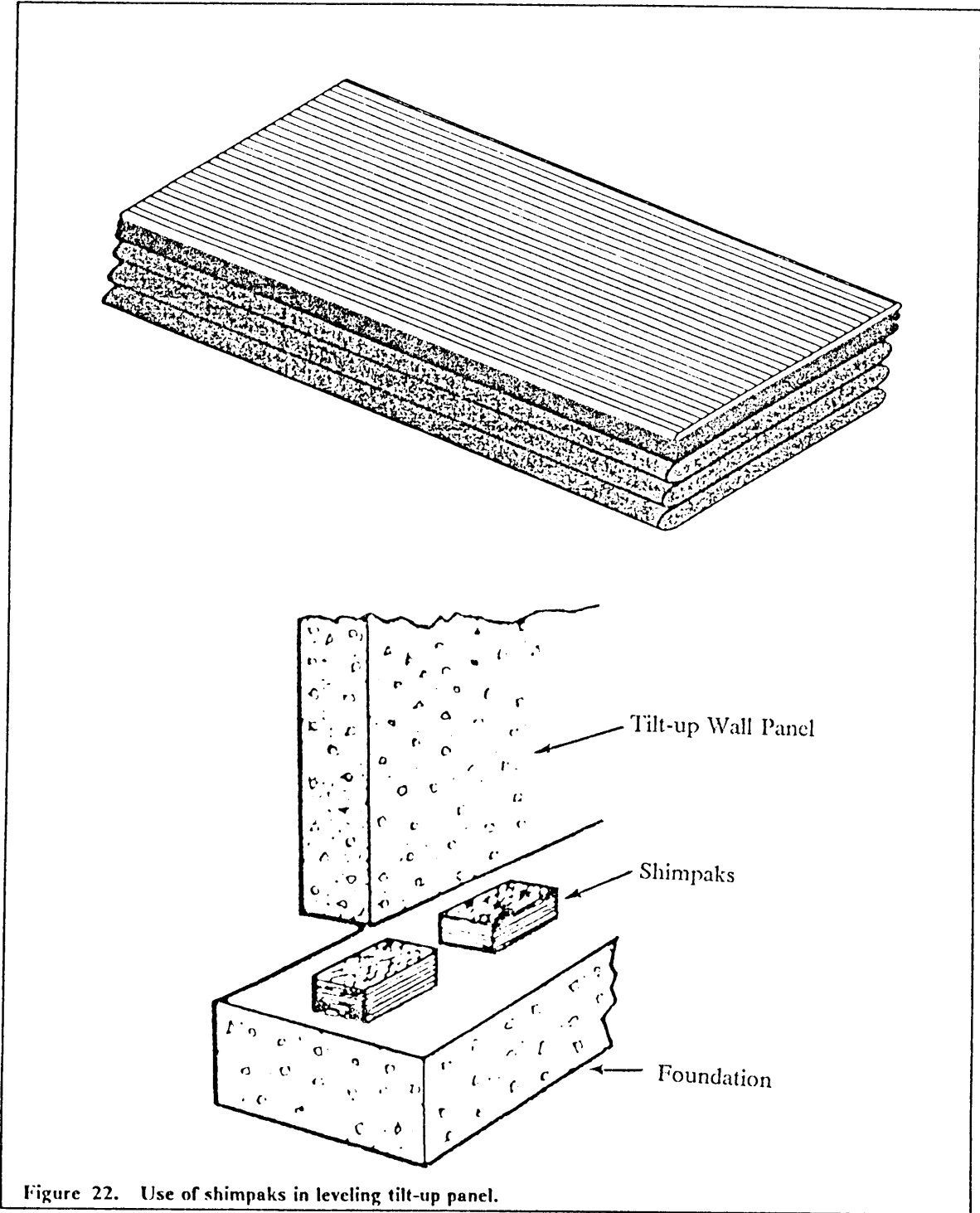


Figure 22. Use of shimpaks in leveling tilt-up panel.

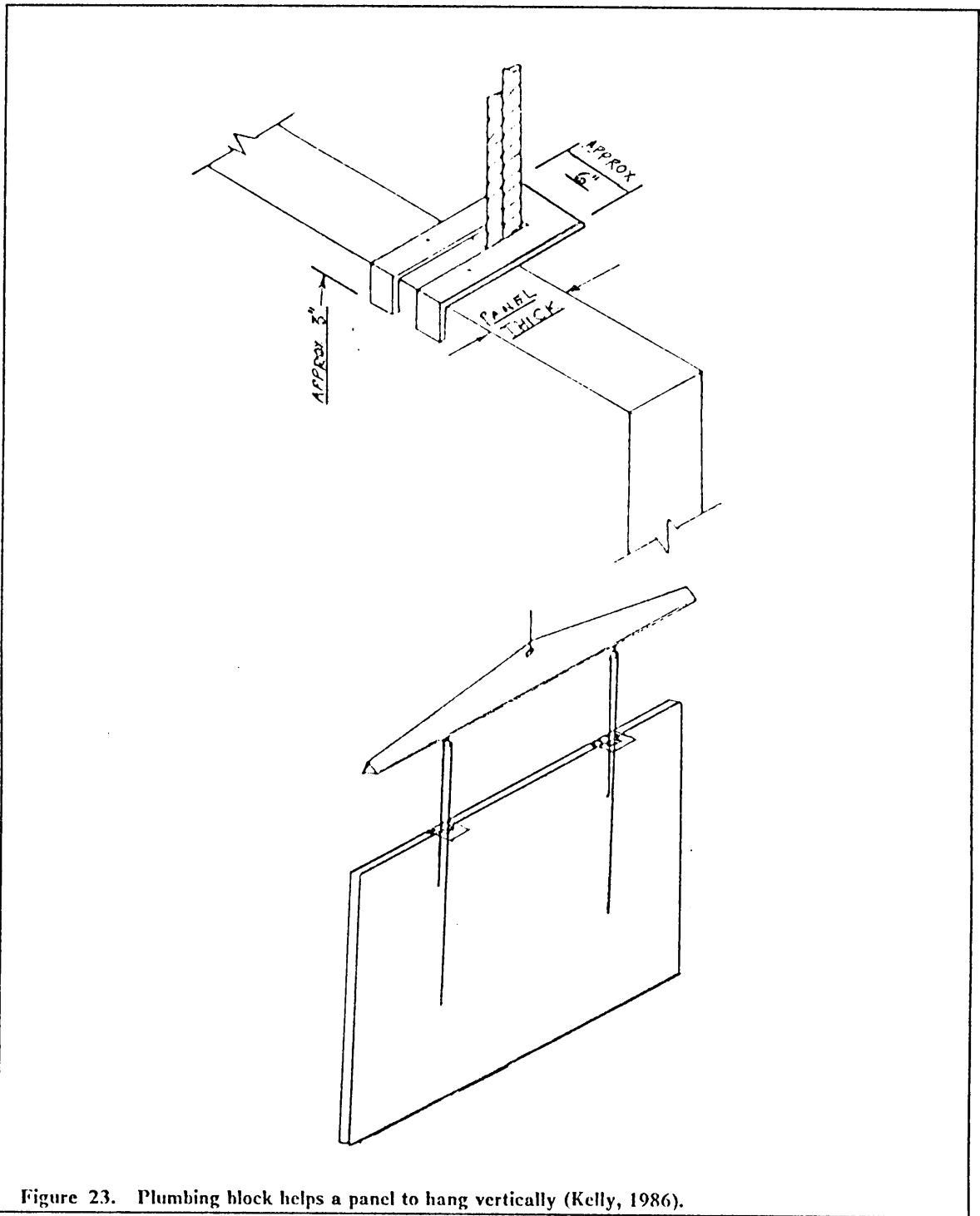


Figure 23. Plumbing block helps a panel to hang vertically (Kelly, 1986).

next one (McKinney, 1980). Each panel should be grouted to provide uniform bearing and a seal to the footing.

Coil lift inserts provide secure and positive attachment. However, it is necessary to climb a ladder to remove the lifting plates. It is also necessary for the contractor to have at least two sets of lifting hardware so that a panel can be set up while the previous one is being tilted into place. To overcome this problem, many hardware suppliers have invented a quick connect/disconnect ground release system to speed up the erection operation and avoid having two sets of lifting hardware. Because there is no ladder climbing, the safety of the workers is also improved. Examples of such systems are the Maxi-Lift system by Richmond Co., Swift Lift system by Dayton Superior, and Super-Lift system by Burke Co. To disengage the lifting hardware, a worker needs only to apply a quick downward force on the release rope and the lifting unit that was fastened to the panel will be ejected from the panel. Many of these hardware accessories are patented by various manufacturers.

There are two basic ways of picking up panels. More commonly, the crane will pick on the interior face of the panel from inside of the building, or pick on the exterior face of the panel from outside of the building. In the type of picking described above, the crane operator has good *visual* control of the lifting devices and rigging. The panel is inclined away from the crane before it reaches vertical position.

In another type of picking, the top portion of the panel is closest to the crane. As the panel is lifted, the lifting devices will be concealed from the crane operator. The crane is usually positioned slightly to the side of the panel. This type of lifting is called *blind* lift. If the crane is positioned directly in front of the panel, it is called suicide lift. The panel will fall upon the crane in case of a concrete, insert, or rigging failure. Therefore it is necessary to use a crane with a lifting capacity of 3 to 4 times the weight of the panel so that the boom can be lowered and the panel is lifted with the crane at a safe distance away (Dayton Superior, 1983). To prevent the crane from walling itself inside the building, it is often necessary to lift the last panel from outside of the building.

5.0 CONNECTIONS

5.1 General Considerations

The tilt-up wall panels have to be connected to each other, and to the structural frame of cast-in-place concrete, steel or masonry. The connections must be able to transmit loads from the roof and floor members into the foundations. They must also be able to take the stresses resulting from shrinkage and creep of concrete as well as temperature variations. There are several important aspects which must be considered when choosing a connection (Waddell, 1974; Weiler, 1986; PCI Manual, 1977).

1. Connections must be structurally sound at both service load and ultimate load, with all the possible load cases considered. Good engineering practice requires that the adjacent concrete members must fail before the connection, i.e. the strength of the structure should be governed by the strength of the structural members rather than by that of the connections. This is normally done by increasing the strength of the connection so that it is about 10% higher than its adjacent members.

2. The connections, preferably, should not be exposed to view. When it is visible, it should be neat, watertight, protected from corrosion and fire, and nonrusting.
3. Tilt-up panels are less precise than plant-cast panels. Therefore proper tolerance must be given when determining the sizes of holes, corbels, etc. as well as erection clearances. The more exact the location where a connection is to be made, the more difficult it is to fabricate and the higher is the cost. In short, the connection must be economical and easy to construct.
4. Effects of volume changes and movement may have to be considered if the panels are erected before they attain their final equilibrium moisture content.
5. Connections should be ductile and have adequate rotational capacity to prevent brittle failure of steel, weld, or concrete.

In seismic regions, the earthquake forces are usually determined as a percent of the weight of the panel and are located at the center of mass. Effects of relative differential movements must be considered. UBC-76 and Seismic Engineers Association of California (SEAOC) require that the connections between panels must accommodate the movements caused by:

$\frac{2.0}{K}$ times story drift by wind

$\frac{3.0}{K}$ times seismic story drift

$\frac{1}{4}$ inch,

whichever is greater, where

K = horizontal force factor. See Table 2 on page 51.

Movements in the connection can be accommodated with slotted or oversized holes. Alternatively, movement can be accommodated by the ability of steel to bend or by providing sliding

Table 2. UBC K-factor for the design of connections allowing lateral movement in seismic zones (PCI Manual, 1977).

UBC – Table No. 23-I – Horizontal force factor K for buildings or other structures⁽¹⁾

TYPE OR ARRANGEMENT OF RESISTING ELEMENTS	VALUE OF K
1. All building framing systems except as hereinafter classified	1.00
2. Buildings with a box system	1.33
3. Buildings with a dual bracing system consisting of a ductile moment resisting space frame and shear walls or braced frames using the following design criteria: a. The frames and shear walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames b. The shear walls acting independently of the ductile moment resisting portions of the space frame shall resist the total required lateral forces c. The ductile moment resisting space frame shall have the capacity to resist not less than 25 percent of the required lateral force	0.80
4. Buildings with a ductile moment resisting space frame designed in accordance with the following criteria: The ductile moment resisting space frame shall have the capacity to resist the total required lateral force	0.67

1 Where wind load would produce higher stresses, this load shall be used in lieu of the loads resulting from earthquake forces.

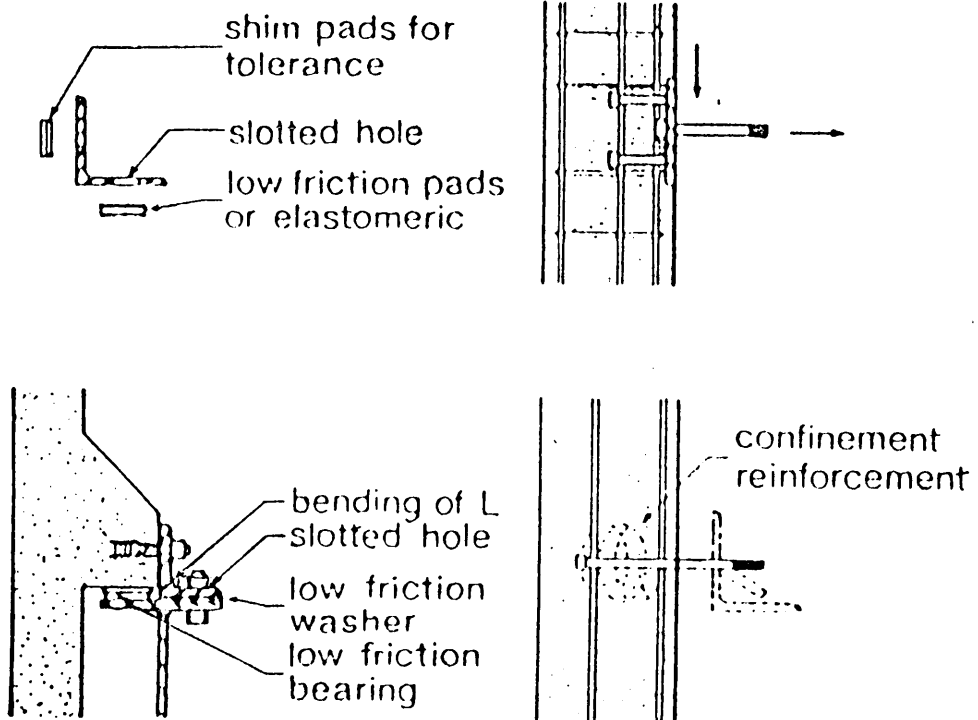


Figure 24. Connection design details allowing lateral movement (PCI Manual, 1977).

and ductility to the joint as given by connections shown in Figure 24 on page 52 (PCI Manual, 1977).

5.2 *Types of Connections*

Connections vary with the type of roof and floor system of the structures as well as the preference of designer and contractor. Roughly, they can be categorized as (Weiler, 1986):

1. Cast-in-place connection. The connection is formed by casting infill concrete sections between panels. The rebars projecting from the panels may overlap and have to be staggered.
2. Welded embedded metal connection. A steel angle or steel plate is anchored and cast into the panel. Connections are then made by field welding.
3. Embedded inserts connection. A bolt is cast into the panel with the head in. Alternatively, a loop insert or ferrule insert can be embedded and then a bolt is screwed into the insert to allow a bolted connection to be made.
4. Drilled-in inserts connection. This is similar to type 3 above except a hole slightly larger than the insert is drilled in the hardened concrete. The insert is then placed into the hole and expanded by screwing a bolt into it. The expanding pressure creates friction and mechanical anchorage necessary for connection.

Both type 3 and type 4 are only suitable for supporting light loads or nonstructural elements. They are cheap and easy to construct. However, they must not be used in seismic zones because of their poor cyclic loading characteristic. There are other types of connections, such as a seat connection, which can also be used to join precast elements.

5.3 Panel to Panel Connection

There is a difference of opinion among designers as to how wall panels should be connected to each other. Panels which are not connected to each other are free to expand and contract thereby reducing stresses set up due to volume changes. It is also believed by some engineers that unconnected panels may perform better during a large earthquake because of better structural damping. However, there are others who suggest that at least two or three welded connections should be made at each panel joint, particularly for those buildings with high and narrow panels and situated in an earthquake zone. In any case, it is necessary to provide all the connections that are required for structural stability. Unconnected panels are less stable under lateral loadings. Therefore, panels should be connected in groups of two or three to resist the overturning forces as shown in Figure 25 on page 55 (Weiler, 1986).

There are several ways panels can be connected to each other. A cast-in-place connection provides a strong and continuous wall for shear resistance. It also requires little water proofing. This can take the form of a column (pilaster) or a 12-inches cast-in-place section. (See Figure 26 on page 56). Where movement at joints is desirable to reduce stresses set up by shrinkage and temperature variation, a bond-preventive material or a coating of bondbreaker can be placed in the joint. Also, if the rebars are extended into the column, tension cracks may result. In this case, the panel can be set free from the column by stopping the horizontal bar just short of the panel edge. Smooth dowels covered with grease, bondbreaker, or wrapped in felt are then extended from the panel into the column (Waddell, 1974). Sometimes difficulty may be encountered in placing the concrete for a cast-in-place column because of congestion of rebars. To eliminate any possible void, immersion vibrators must be used to properly consolidate the concrete. The technique of placing the concrete from the bottom up using a concrete pump to work against concrete head will exert extra live load on a column form and must be used with care. A precast column can also be used in the place of a cast-in-place column. In such cases the precast columns must have an oversize recess for ease of placing panel and matching panel with column (Waddell, 1974; PCA, Tilt-up

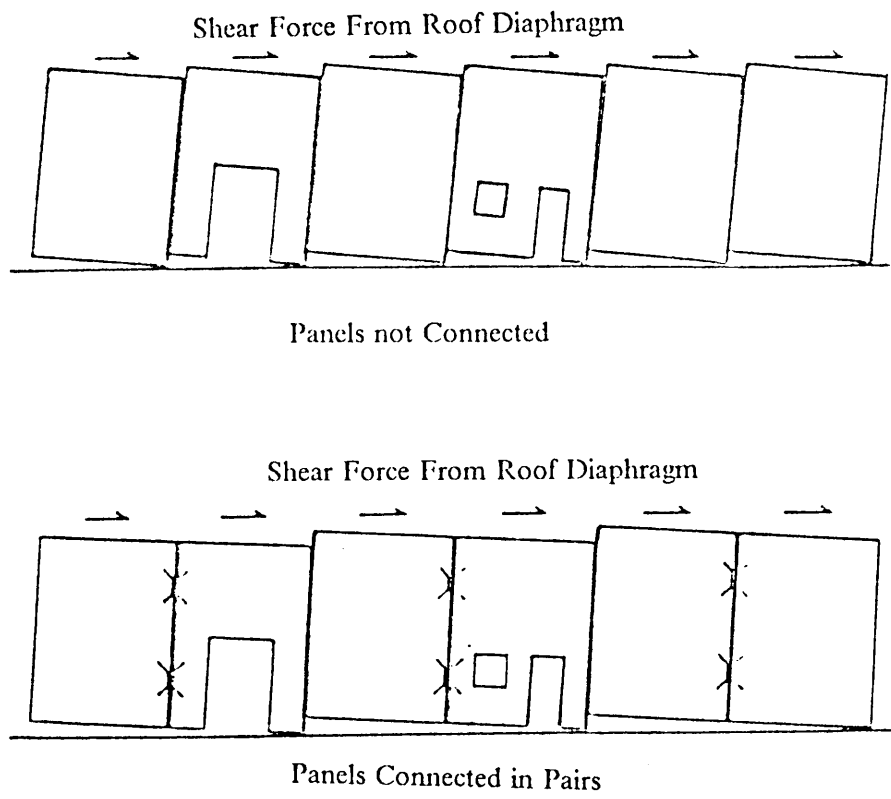


Figure 25. Panels connected in groups resist overturning forces.

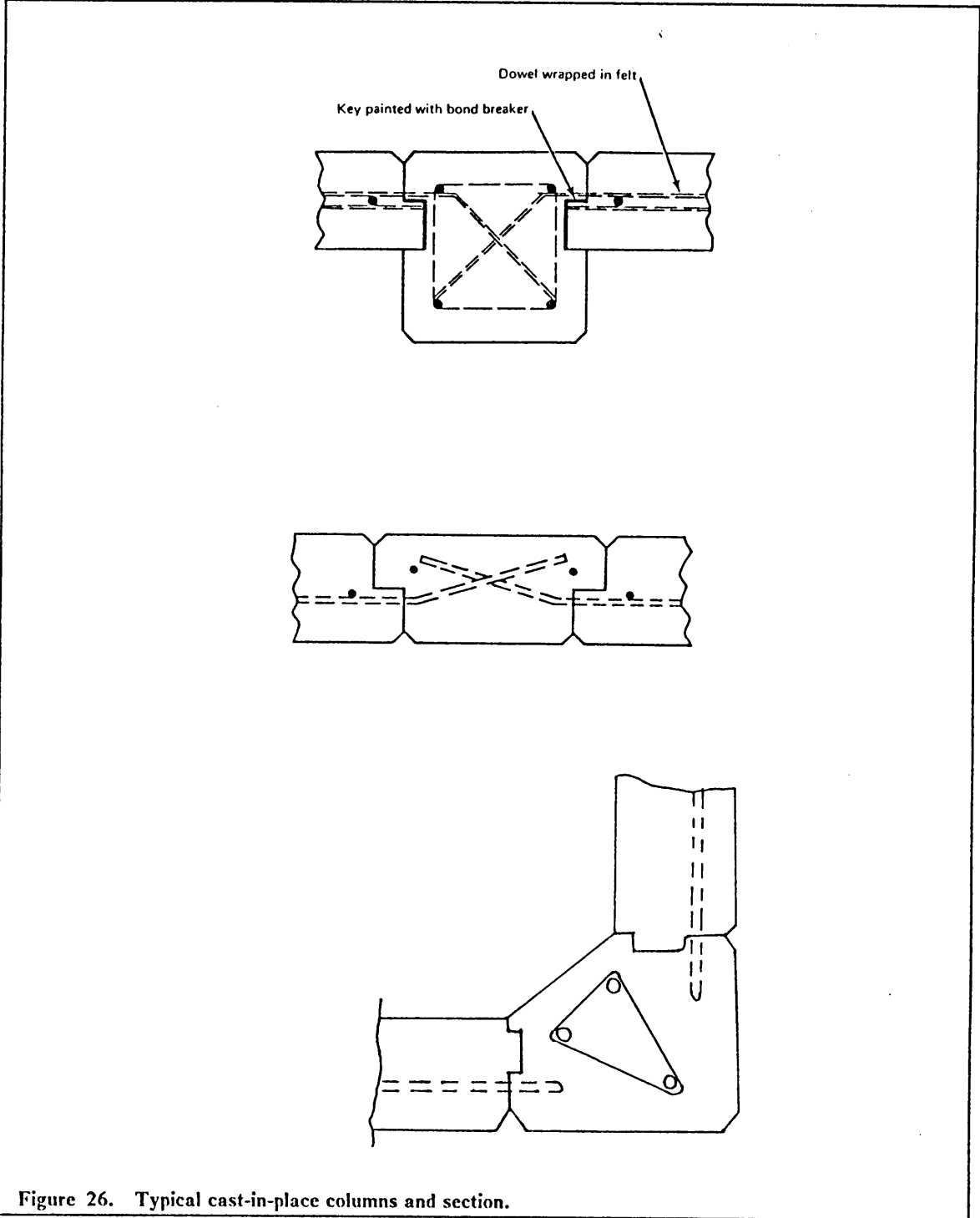


Figure 26. Typical cast-in-place columns and section.

Concrete Walls, 1970). The cast-in-place column/pilaster can easily be formed with pilaster clamp and taper tie (tie) assembly from hardware suppliers. Two typical sets of these assemblies are shown in Figure 27 on page 58 (Dayton Superior, 1983).

Sometimes weld plates are cast into the panel to allow a welding connection to be made between a panel and a concrete column, panel and steel column, or directly between two panels. The weld plates in welded connections are usually arranged at a distance of four feet apart. (See Figure 28a, b, and c). In the case of the connection between concrete column and panel as shown in Figure 28a, the weld pockets on the sides of the column are dry-packed after the welding is completed and then the vertical void between panels are grouted. The weld plate connections are easier to form. However, they do not have any fire rating since there is no concrete protection over the connections. Besides, the weld plate connections tend to have poor cyclic loading characteristics (Waddell, 1974; Weiler, 1986).

5.4 Panel to Foundation Connection

Tilt-up wall panels are commonly supported by continuous or isolated footings, although some other forms of foundation such as continuous beam on grade or pile cap are possible. It is essential that a method of aligning and leveling the wall panels such as using shimpaks or grout pads be provided.

In a connection of a wall panel to its foundation, it is important to keep them in relative position to each other, especially in a seismic zone. This also prevents the panels from being displaced in case of an accidental equipment impact. Earthquake forces are usually transmitted from the ground to the foundation and floor slab before they reach the roof and other floor levels where most of the seismic energy is absorbed. Displacement between panel and footing may occur if transverse restraint in the form of dowel, weld plate, or continuous longitudinal slot in the footing is not provided.

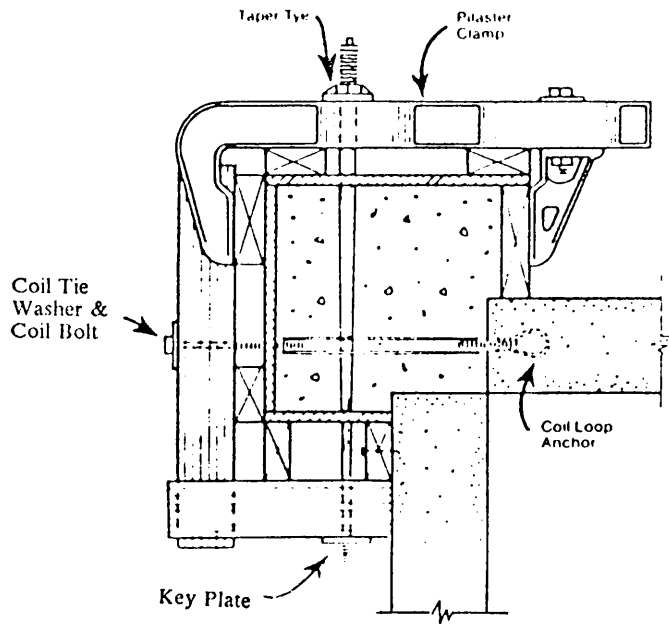
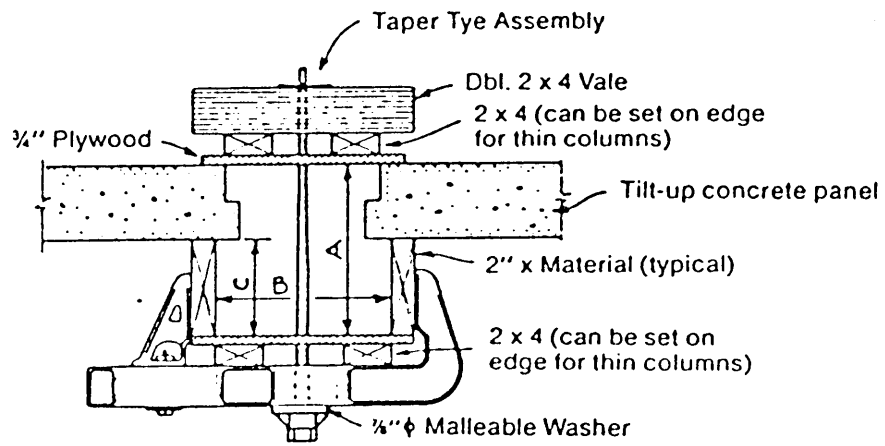
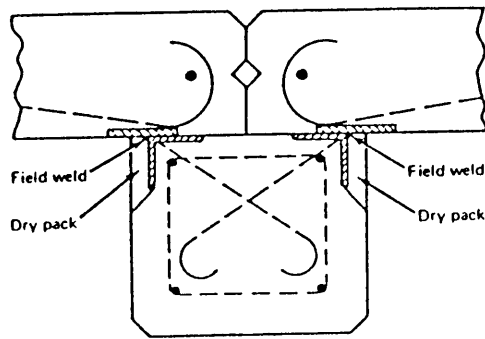
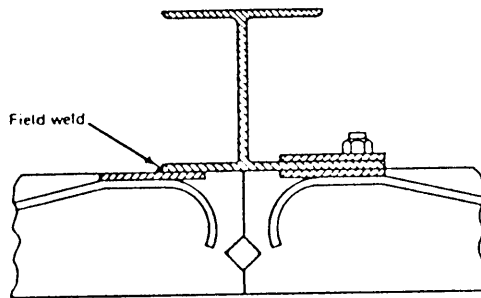


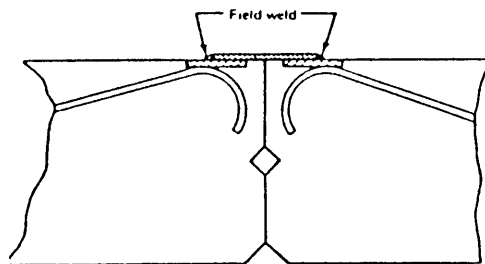
Figure 27. Pilaster clamp and taper tie (tie) assembly for forming column (Dayton Superior, 1983).



a



b



c

Figure 28. Some welded connections for joining precast panels (Waddell, 1974).

Some typical wall panel to foundation connections are shown in Figure 29 on page 61. Figure 29a and 29b shows the use of angle sections which can be welded to anchor plates or fastened by bolts cast/embedded in the concrete element. Shimpaks can be used to level and align the panel in this case. In order to transfer the design loads, the space between foundation and panel has to be dry-packed with non-shrink grout to achieve uniform bearing along the length of the panel. In Figure 29c, a connection between foundation and wall panel is made with dowels. A conduit or blockout is first made in the foundation to receive the dowel from the panel above. The dowel hole or blockout is filled with non-shrink grout immediately before the panel is placed. Leveling and aligning the panel and dry-packing the space between panel and footing should be done in the same way as the connections shown in Figure 29a and 29b described above. Stirrups can be wrapped around the conduit whenever a greater pull out strength is desired (PCI Manual, 1977). For non shear wall panels, the connection can be made with a continuous shear key filled with non-shrink grout as shown in Figure 29d. (PCA Special Report #14, 1972). If the panel is to rest on an isolated footing, dowels can be installed in the footing to provide lateral restraint, and a mortar pad is placed between footing and panel to achieve uniform bearing. See Figure 29e (PCA, 1973, 1982).

Usually the floor slab of the building is held back a few feet from the perimeter of the building to allow excavation for the footing and installation of any mechanical or plumbing lines. After the panels are positioned and connected to the footing, the area between the floor slab and wall panel can be filled and compacted, and the floor slab can then be extended to join the wall panel. It is a common practice to provide some form of connection between the wall panel and floor slab for additional integrity. The connection may take the form of a dowel projecting from a panel face or by welding to an anchor plate embedded in the panel. See Figure 29f (Weiler, 1986). For those connections which occur below the highest water table anticipated, it is important to make the connection watertight.

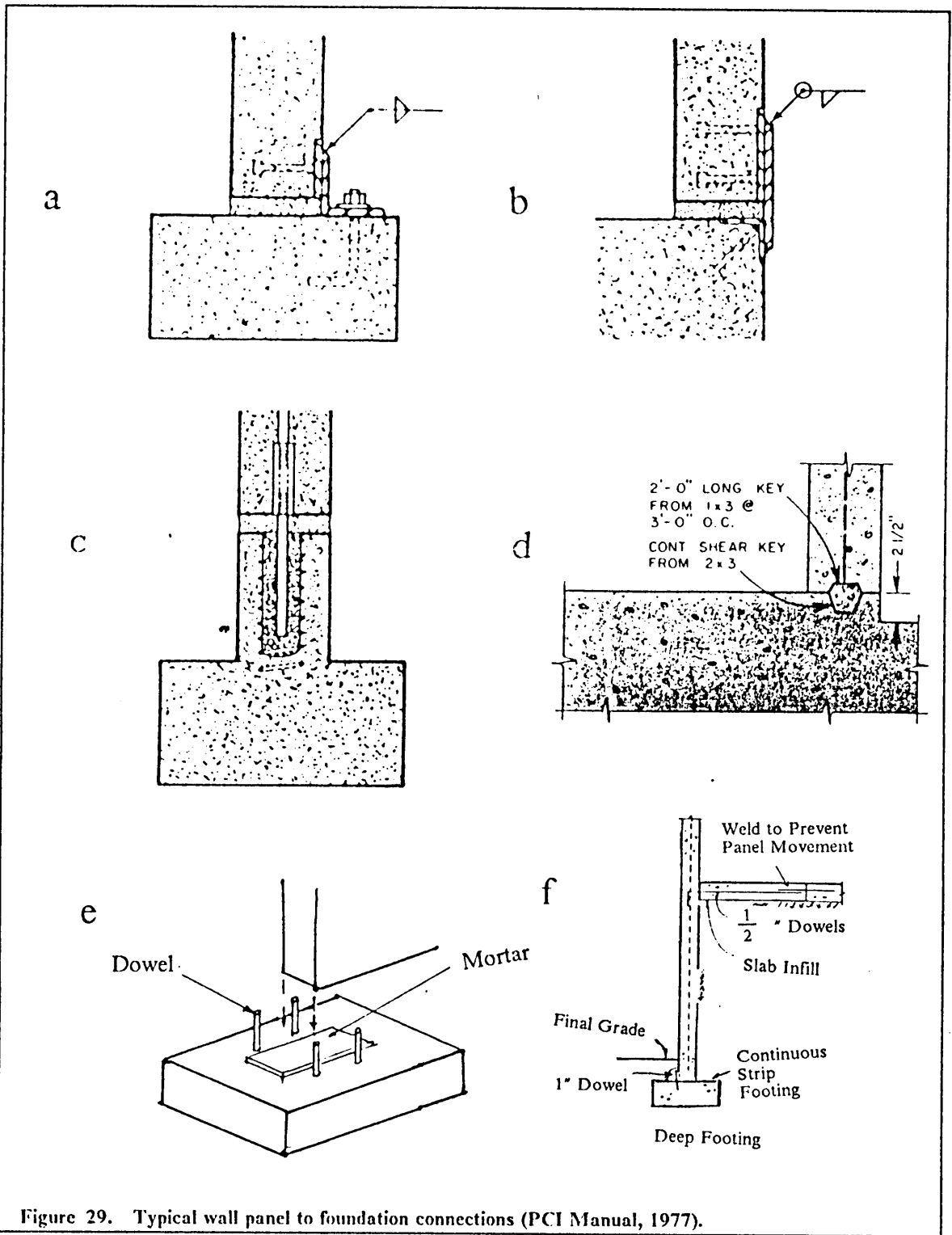
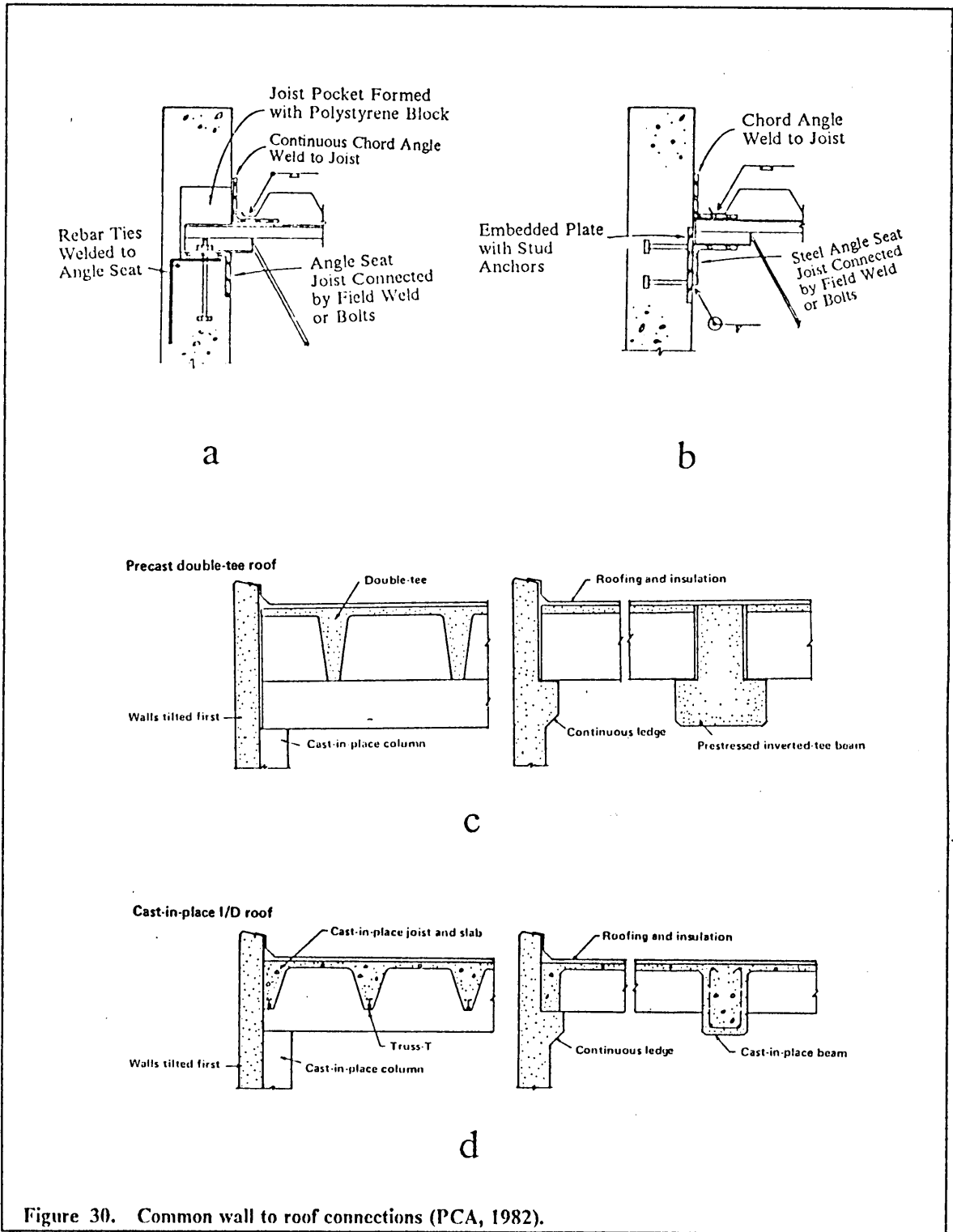


Figure 29. Typical wall panel to foundation connections (PCI Manual, 1977).

5.5 *Wall Panel to Roof Connection*

To connect tilt-up wall panels to the roof of a structure, usually a pocket is recessed in the wall panel with an angle seat cast in the pocket. Steel joist can then be bolted or field welded to the seat. The pocket in the wall panel can be formed by embedding a polystyrene block in the concrete (see Figure 30a). Alternatively, an angle seat can be formed by welding to a flat steel plate anchored by embedded studs (Figure 30b). There are no projections above the panel surface in the connections mentioned above for ease of screeding and stack casting. If a beam instead of a joist is to be connected to the angle seat in a recessed pocket, the reaction must be less than 12,000 lb since there is not sufficient concrete depth to install confining ties (Weiler, 1986). Large loads from beams are usually supported by a corbel or full height pilaster (see Figure 30c and 30d). The roof of the structure can be of either cast-in-place joist and slab, concrete shell, or precast double tees (PCA, 1973). It is customary to have dowels protruding from the surface of a wall panel or dowels threaded into inserts in the wall so that they can be embedded by the cast-in-place concrete topping to provide structural integrity. Generally, precast girders and double tee beams are rested on neoprene pads to allow for some rotational movement.

If the structure has a wooden roof, the timber joists are usually supported on a wood ledger connected by bolts cast/drilled into wall panel (Weiler, 1986). It is also customary to encase the outer one-third ends of the top horizontal steel reinforcing chords in the wall panel for this type of roof and floor system. The steel chords are spliced and field welded to each other and then the space between wall panels can be filled with compressible material and sealed. This allows the panels to expand and contract freely. See Figure 31a and 31b. The use of grouted keys have also been used successfully for connecting wall panels to a roof. Figure 31c and 31d shows the typical details of the grouted key connection employed successfully by Challenge Development Inc, California in constructing a seven story apartment building in the San Francisco Bay Area in 1972.



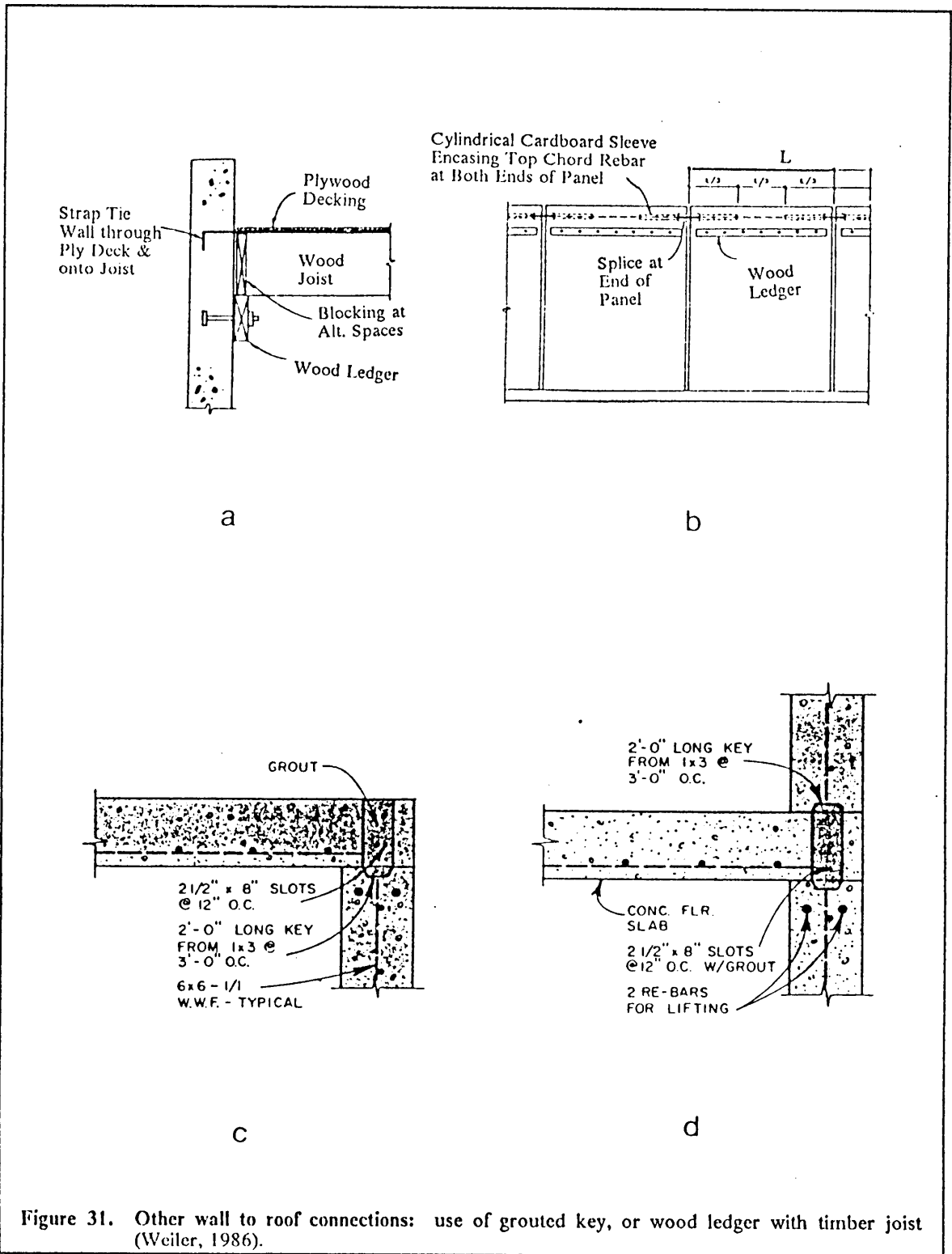


Figure 31. Other wall to roof connections: use of grouted key, or wood ledger with timber joist (Weiler, 1986).

5.6 *Weatherproofing of Joints and Connections*

Panel joints and connections must be weatherproofed in order for the structure to be serviceable. Portland cement grout can be used to seal the base of the wall panel to the foundation. However, it is not recommended for the joint between wall panels because relative movement due to temperature changes may cause the grout to crack. Caulking compounds or sealants are commonly used to weatherproof the joint between the wall panels. The most widely used sealants include polysulphides, acrylics, and polyurethane polymers. The concrete surface must be cleaned before the sealant is applied. The sealing compound is injected through a nozzle of proper size with a power activated gun. A backup filler strip, which may be made of foamed polyurethane, polystyrene, or polyethylene is used to control the depth of sealant. Sometimes, the sealant is tooled slightly concave to improve its appearance.

The sealant and the backup strip should have the elastomeric ability to expand and contract with the movement of the panels without cracking. Some sealants are of one-compound type which can be used as it is. The two-compound type requires the addition of an accelerator and has greater bonding capacity (Waddell, 1974; Huntington, 1981).

For wall panels poured from a concrete mix of high water-to-cement ratio, water-repellents such as silicones, or epoxies can be applied over the surface to improve the water proofing property. Epoxies are more durable than silicones but they are also more expensive.

6.0 ERECTION STRESS ANALYSIS

During the design of a tilt-up wall panel, it is crucial to make sure that the tensile strength of the concrete is not exceeded. The analysis is usually based on the uncracked section of panel. In addition to its self weight and dynamic load, mechanical bondage and suction caused by a vacuum between panel and casting surface must also be considered during the initial stage of separating a panel from the casting bed. Suction is particularly significant if the edges of the panel are wet because it prevents penetration of air into the interface. Therefore it is not advisable to lift panels following a rain. Mechanical bonds can be broken/reduced by sliding the panel a little with jacks to break the bond before lifting. Alternatively, wedges can be hammered into the interface to break the bond. To minimize stresses set up, wedges must be inserted in line with lifting inserts (Burke Co., 1984). Estimates of the additional load due to suction and bond varies from negligible to 20 psf, depending on panel size, interface texture, and moisture that may be present between the panel face and slab. However, maximum flexure stress usually occurs when the panel is between 20 and 50 degrees with the horizontal (Dayton Superior, 1983).

As a panel is tilted from a horizontal to a vertical position, the bending moments and stresses are constantly changing. This is because, as the cable attached to the lifting plate is changing its angle during rotation, the force components acting on the lifting plate and hence the insert also changes accordingly. This will cause the tension load on one insert to increase while the tension

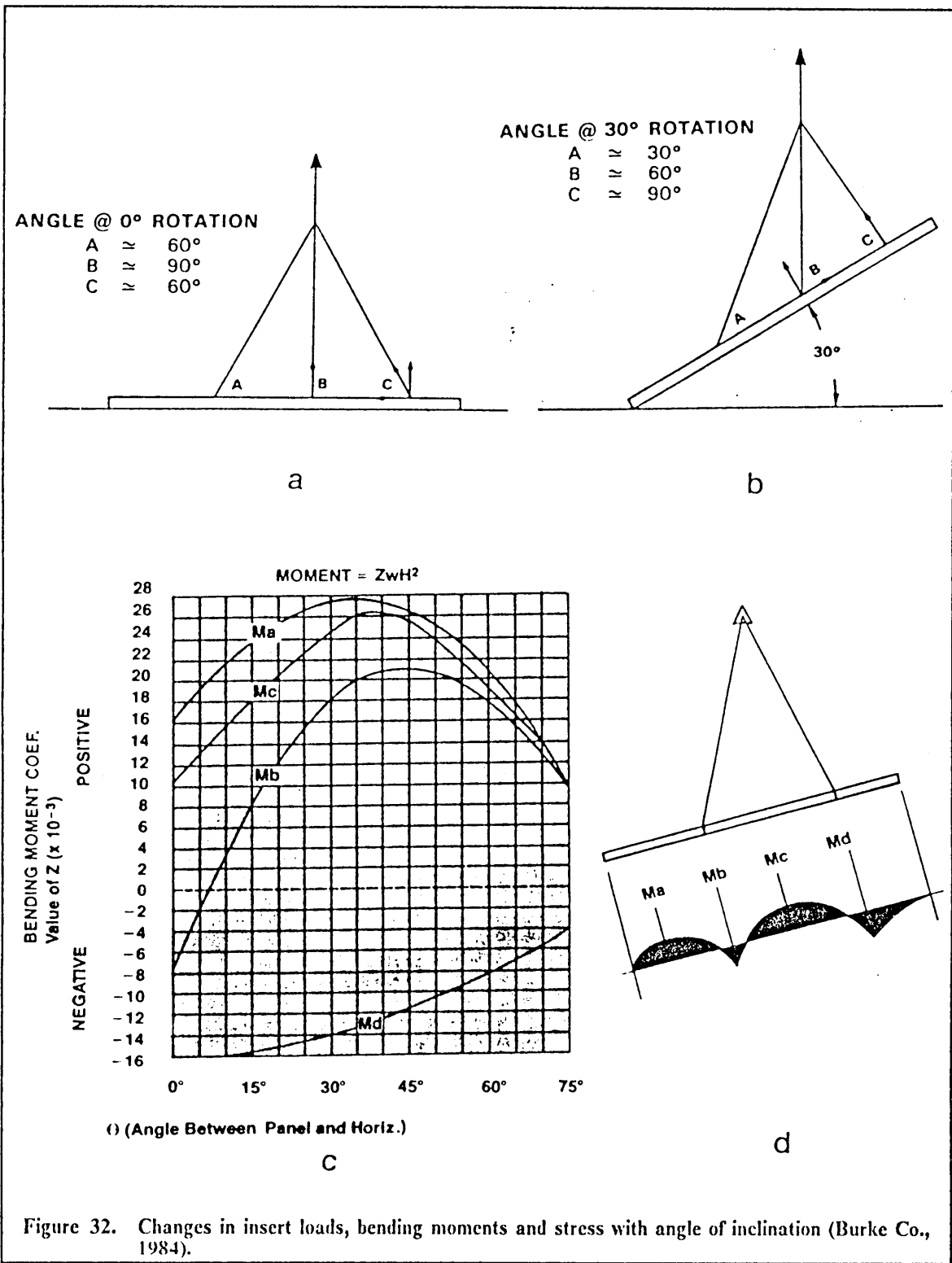


Figure 32. Changes in insert loads, bending moments and stress with angle of inclination (Burke Co., 1984).

load on another may decrease. For example, in Figure 32a the load at B is 100% tension and the load at C is about 85% tension. When the panel is rotated to 30 degree in Figure 32b, the tension at C has increased to 100% while the tension on B has decreased to 80%. As with the tension load, shear load on inserts will also varies with the angle of rotation.

As mentioned before, the most critical bending moment and stress in a panel usually occurs between 20 and 50 degrees rotation depending on panel geometric shape, number of inserts required and the type of rigging. In Figure 32c and 32d, a two-high rigging and its corresponding variation of bending moment with angle of rotation are shown qualitatively (Burke Co., 1984).

The determination of bending moments and stresses at various angle of rotation can be very complex if the rigging type employed is complicated. Openings in a panel will further complicate the task of erection flexure stress analysis. Most suppliers now have technical services which use computers to analyse the panel for a trial insert layout. The analysis is carried out for various angles of rotation between 0 and 90 degrees. For a nominal charge, a contractor is able to obtain detail drawings showing lifting and bracing insert locations, crane rigging and cable lengths, and any extra steel or strongback which may be required (Courtois, 1980).

6.1 Manual Computation of Flexure Stress During Erection

Manual calculation of flexure stresses is usually limited to solid rectangular panels without an opening lifted with simple rigging. The analysis is carried out twice: once in the height direction, and once in the width direction.

The following example illustrates the analysis procedures of a panel in the height direction when it is in the horizontal position.

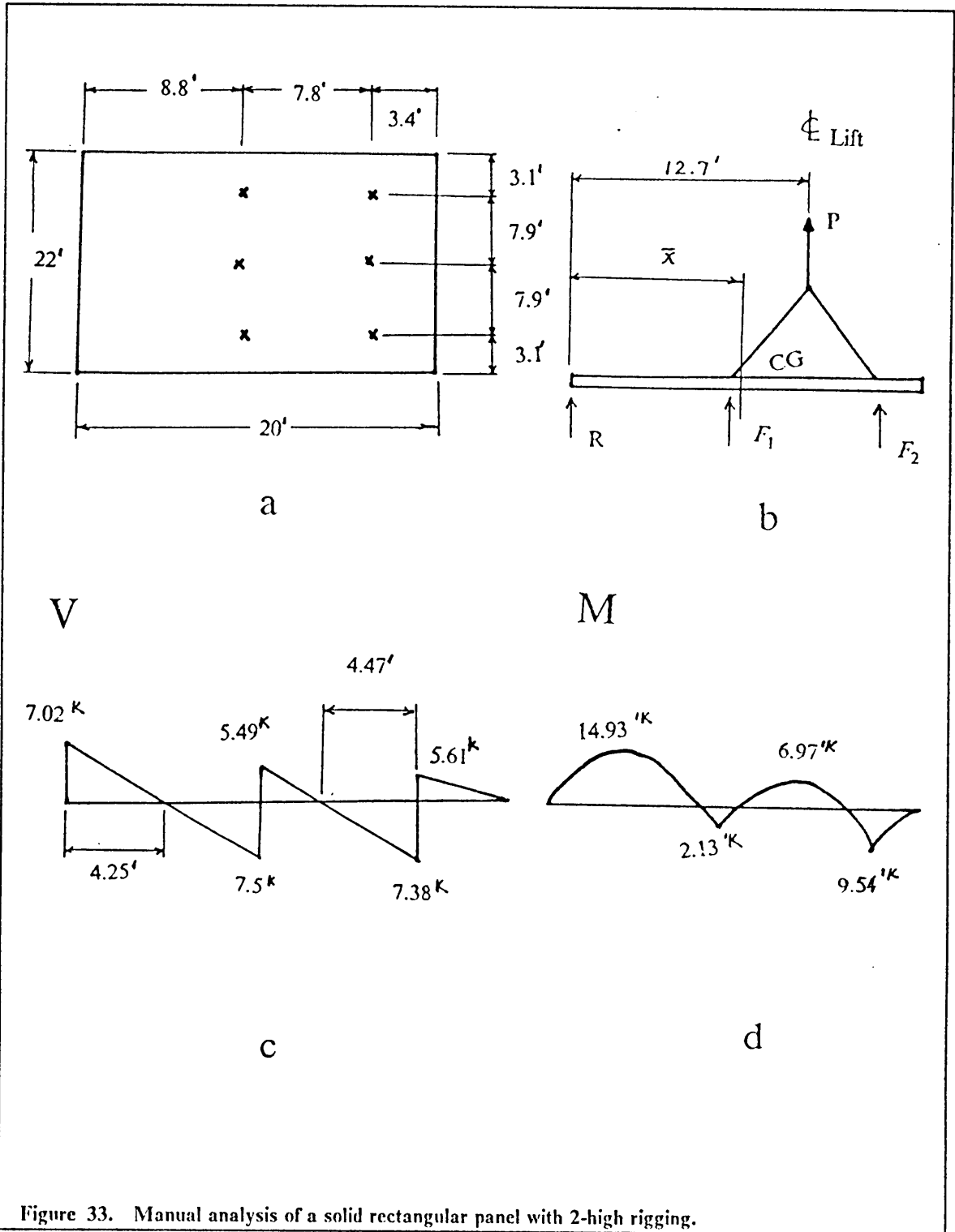


Figure 33. Manual analysis of a solid rectangular panel with 2-high rigging.

Example. Assume that suction and bonding force are negligible for the wall panel shown in Figure 33, determine the maximum flexure stress occurring in the panel in the horizontal position.

From Figure 33a and 33b we can see that

$$\text{Height of panel } H = 20 \text{ ft}$$

$$\text{Width of panel } W = 22 \text{ ft}$$

$$\text{Panel thickness} = 6 \text{ in.}$$

$$f_c \text{ at lifting} = 2500 \text{ psi.}$$

$$\bar{x} = 0.5H = 10 \text{ ft}$$

$$\text{Panel weight } W_t = \frac{20 \times 22 \times 6 \times 150}{12 \times 1000} = 33 \text{ k}$$

$$\text{Center line of lift} = 8.8 + \frac{7.8}{2} = 12.7 \text{ ft.}$$

$$P = \frac{33 \times 10}{12.7} = 25.98 \text{ k}$$

$$R = W_t - P = 33 - 25.98 = 7.02 \text{ k}$$

$$F_1 + F_2 = P = 25.98 \text{ k}$$

But $F_1 = F_2$ when the panel is horizontal.

$$\text{Therefore } F_1 = F_2 = 12.99 \text{ k}$$

The shear and moment diagrams are as shown in Figure 33c and 33d.

$$\text{Maximum moment} = 14.93 \text{ k ft}$$

$$I = \frac{22 \times 0.5^3}{12} = 0.2292 \text{ ft}^4$$

$$\begin{aligned} \text{Therefore } f &= \frac{M \times c}{I} \\ &= \frac{14.93 \times 0.25}{0.2292} \\ &= 16.28 \text{ ksf} \\ &= 113 \text{ psi} \end{aligned}$$

which is smaller than 375 psi, the modulus of rupture. A similar analysis can be carried out in the width direction.

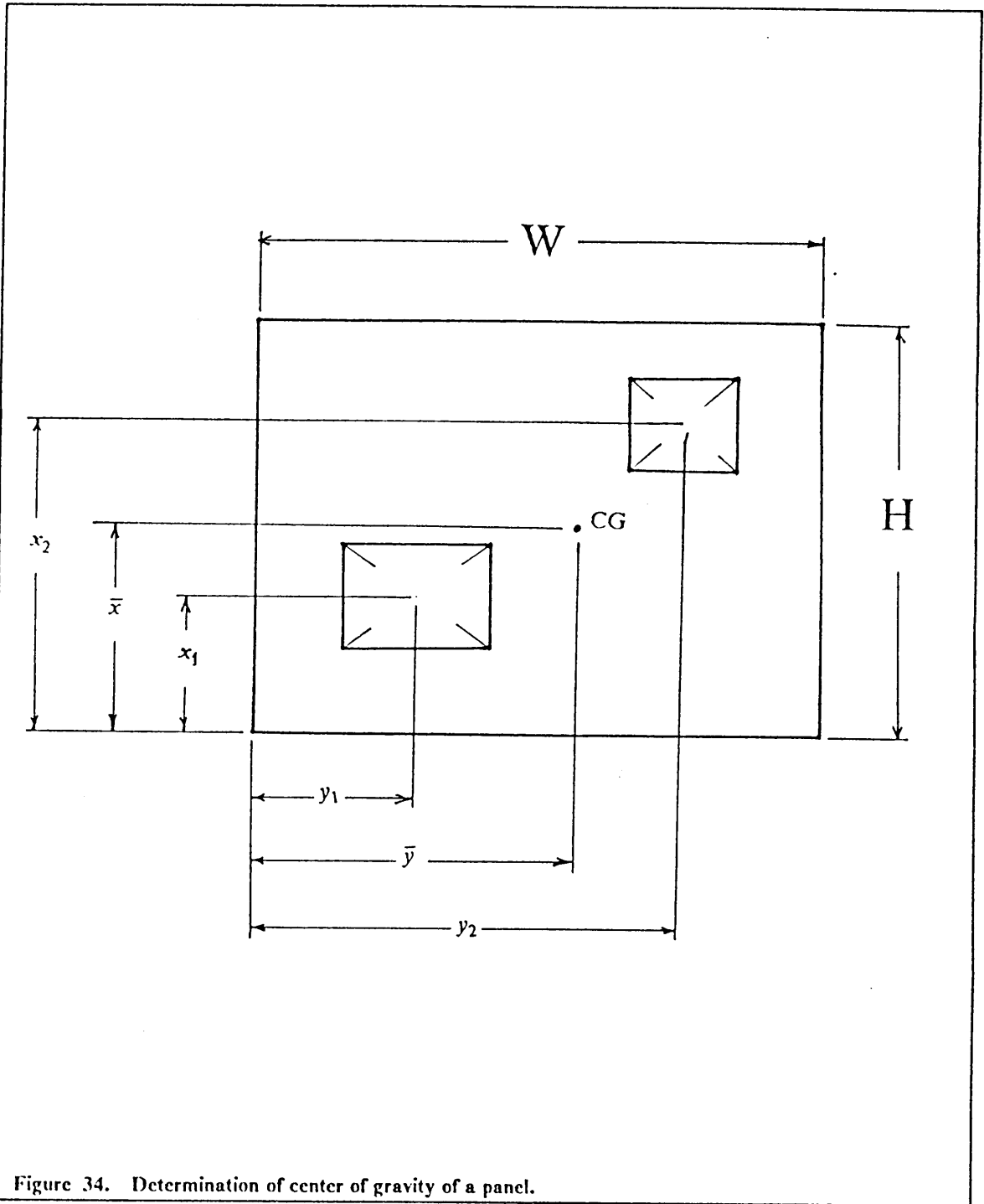
If the panel is inclined at an angle θ with the horizontal, the cable geometry is determined first and then the tension in the cable is found. The steps are given in the beam method of computer analysis for two-high rigging which is discussed in the next section. The force on inserts and the moment diagrams can then be obtained as before.

6.2 Computer Analysis of Tilt-up Wall Panel by Beam

Method

This method is able to handle more complicated cable geometry and a panel with openings. As in the case of manual computation, the center of gravity of the panel must be determined first. From the panel in Figure 34 on page 72, the centroidal distances from the lower left hand corner of the panel are:

$$\bar{x} = \frac{\frac{WH^2}{2} - A_1x_1 - A_2x_2}{WH - A_1 - A_2}$$



$$\bar{y} = \frac{\frac{W^2 H}{2} - A_1 y_1 - A_2 y_2}{WH - A_1 - A_2}$$

where A_1 and A_2 are the areas of the openings. Similar expressions can be obtained for panels having different number of openings. In order to find the loading on inserts, the cable geometry and cable tension for an angle of panel inclination θ must be determined first. The loadings on inserts are then calculated and input as concentrated force applied on the panel (Payne, 1977).

6.2.1 Two-high Rigging

A two high rigging is shown in Figure 35 on page 74. The values of distances A, B, C, and cable length L must be specified. From the figure, the total length of cable = $L = L_1 + L_2$. The horizontal force equilibrium at the pulley requires the cable to make an equal angle α with each side of the center of lift. The rigging geometry will be fully known if we can find the angles α , β_1 , and β_2 . Again from the figure we can see that

$$b = L_1 \sin(\alpha)$$

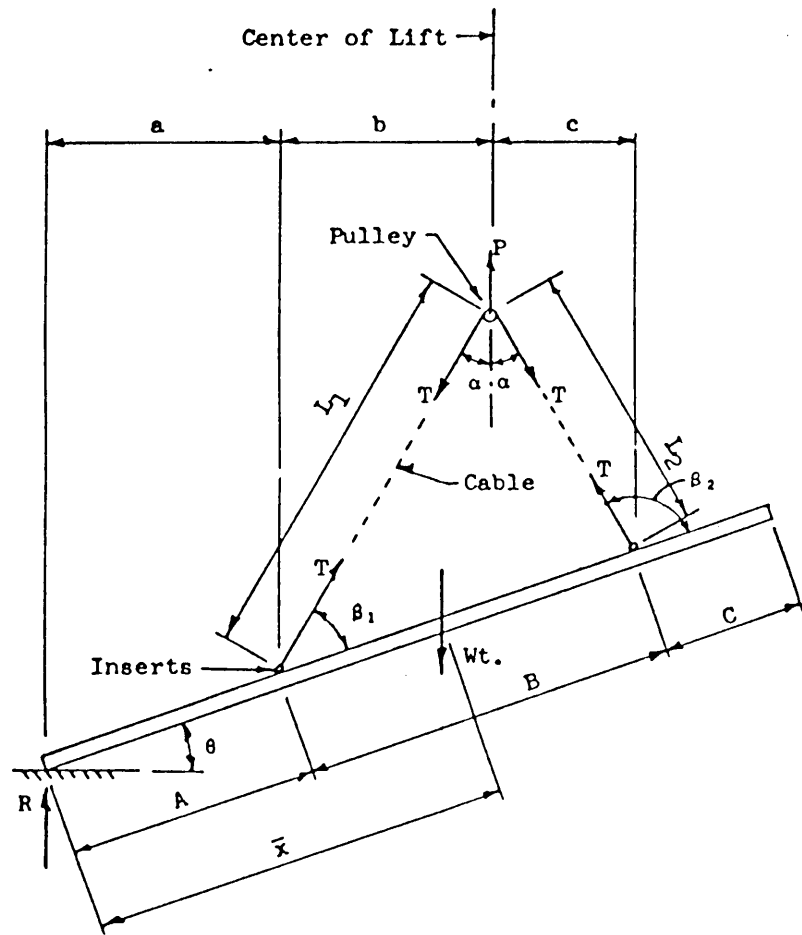
$$c = L_2 \sin(\alpha)$$

$$b + c = B \cos(\theta)$$

$$\text{Therefore } (L_1 + L_2) \sin(\alpha) = B \cos(\theta)$$

$$\text{or } L \sin(\alpha) = B \cos(\theta)$$

$$\text{Hence } \sin(\alpha) = \frac{B}{L} \cos(\theta)$$



$$L = \text{total length of cable} = L_1 + L_2$$

Figure 35. Cable geometry of a two-high rigging (Payne, 1977).

$$\alpha = \sin^{-1} \frac{B}{L} \cos(\theta) \quad (1)$$

$$\text{Also } \beta_1 = 90^\circ - (\alpha + \theta) \quad (2)$$

$$\text{and } \beta_2 = 2\alpha + \beta_1 \quad (3)$$

From Equations (1) to (3), the values of α , β_1 , and β_2 can be determined. To find the cable force T , consider the rotational equilibrium about the reaction R :

$$(W_r)\bar{x} \cos(\theta) = P(a + b)$$

$$\text{or } P = \frac{W_r \bar{x} \cos(\theta)}{(a + b)}$$

Also, vertical force equilibrium at the pulley gives:

$$P = 2T \cos(\alpha)$$

$$\text{Hence } 2T \cos(\alpha) = \frac{W_r \bar{x} \cos(\theta)}{a + b}$$

$$T = \frac{W_r \bar{x} \cos(\theta)}{2(a + b) \cos(\alpha)} \quad (4)$$

where $a = A \cos(\theta)$ and $b = L_1 \sin(\alpha)$. By the Law of Sines,

$$\frac{L_1}{\sin(180^\circ - \beta_2)} = \frac{B}{\sin(2\alpha)}$$

or

$$\frac{L_1}{\sin(\beta_2)} = \frac{B}{\sin 2\alpha}$$

$$L_1 = \frac{B \sin \beta_2}{\sin 2\alpha}$$

We note that $b = L_1 \sin \alpha$. Therefore

$$\begin{aligned} b &= \left[\frac{B \sin \beta_2 \sin \alpha}{\sin 2\alpha} \right] \\ &= \left[\frac{B \sin \beta_2}{2 \cos \alpha} \right] \end{aligned}$$

It is essential to make sure that the inserts are located such that the center of lift is beyond the center of gravity of the panel, otherwise the panel will flip over. Therefore

$$a + b > \bar{x} \cos \theta$$

Substituting expressions of a and b:

$$A \cos \theta + \frac{B \sin \beta_2}{2 \cos \alpha} > \bar{x} \cos \theta$$

$$\text{or } A + \frac{B \sin \beta_2}{2 \cos \alpha \cos \theta} > \bar{x}$$

must be satisfied. After the cable force T is known (using Equation (4)), the tension and shear force at each insert can be determined. The reaction R is:

$$\begin{aligned} R &= W_t - P \\ &= W_t - \frac{W_t \bar{x} \cos \theta}{a + b} \end{aligned}$$

where a and b are as defined before.

6.2.2 Three-high Rigging

A three-high (222) rigging is shown in Figure 36. The values of the distances A, B, C, and the total length of the cable L are specified. The values of the angles $\alpha_1, \alpha_2, \alpha_3$ and the cable lengths L_1, L_2 , and L_3 are to be determined. From the figure, horizontal force equilibrium at the center of the lift requires that

$$2T \sin \alpha_1 + 2T \sin \alpha_2 = 2T \sin \alpha_3$$

$$\text{or } \sin \alpha_2 = \sin \alpha_3 - \sin \alpha_1 \quad (5)$$

Also, from Figure 36

$$b = k + d$$

$$\text{or } L_1 \sin \alpha_1 = B \cos \theta + L_2 \sin \alpha_2$$

$$\sin \alpha_1 = \frac{B}{L_1} \cos \theta + \frac{L_2}{L_1} \sin \alpha_2 \quad (6)$$

and

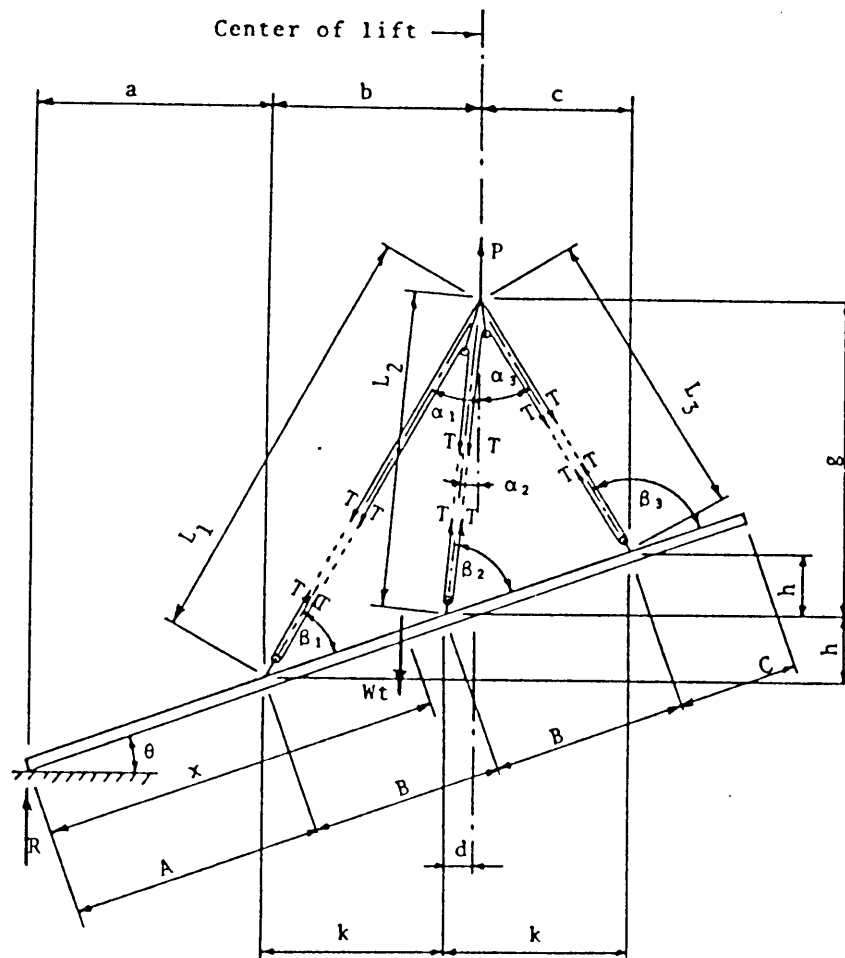
$$c = k - d$$

$$L_3 \sin \alpha_3 = B \cos \theta - L_2 \sin \alpha_2$$

$$\sin \alpha_3 = \frac{B}{L_3} \cos \theta - \frac{L_2}{L_3} \sin \alpha_2 \quad (7)$$

Referring to Figure 36,

$$\begin{aligned} L_1^2 &= (k + d)^2 + (g + h)^2 \\ &= [B \cos \theta + L_2 \sin \alpha_2]^2 + [L_2 \cos \alpha_2 + B \sin \theta]^2 \end{aligned}$$



α_2 is shown in the positive sense

$$L = \text{total length of cable} = 2L_1 + 2L_2 + 2L_3$$

Figure 36. Cable geometry for a three-high (222) rigging (Payne, 1977).

which yields

$$L_1 = \sqrt{B^2 + L_2^2 + 2BL_2 \sin(\alpha_2 + \theta)} \quad (8)$$

Referring to Figure 36 again,

$$\begin{aligned} (L_3)^2 &= (k - d)^2 + (g - h)^2 \\ &= [B \cos \theta - L_2 \sin \alpha_2]^2 + [L_2 \cos \alpha_2 - B \sin \theta]^2 \end{aligned}$$

which yields

$$L_3 = \sqrt{B^2 + L_2^2 - 2BL_2 \sin(\alpha_2 + \theta)} \quad (9)$$

Also, from the figure we can see that

$$L_2 = \frac{L}{2} - L_1 - L_3 \quad (10)$$

Now we substitute Equation (6) and (7) into (5):

$$\begin{aligned} \sin \alpha_2 &= B \cos \theta \left[\frac{1}{L_3} - \frac{1}{L_1} \right] - L_2 \sin \alpha_2 \left[\frac{1}{L_3} + \frac{1}{L_1} \right] \\ &= B \cos \theta \left[\frac{L_1 - L_3}{L_1 L_3} \right] - L_2 \sin \alpha_2 \left[\frac{L_1 + L_3}{L_1 L_3} \right] \end{aligned}$$

$$\text{or } \sin \alpha_2 \left[1 + \frac{L_2(L_1 + L_3)}{L_1 L_3} \right] = B \cos \theta \left[\frac{L_1 - L_3}{L_1 L_3} \right]$$

$$\sin \alpha_2 \left[\frac{L_1 L_3 + L_1 L_2 + L_2 L_3}{L_1 L_3} \right] = B \cos \theta \left[\frac{L_1 - L_3}{L_1 L_3} \right]$$

$$\sin \alpha_2 = \frac{B \cos \theta (L_1 - L_3)}{L_1 L_3 + L_1 L_2 + L_2 L_3}$$

which will give the following after substituting L_2 from Equation (10):

$$\sin \alpha_2 = \frac{2B(L_1 - L_3) \cos \theta}{L(L_1 + L_3) - 2(L_1^2 + L_1L_3 + L_3^2)} \quad (11)$$

Also, by substituting Equation (10) into (8) we get

$$L_1 = \sqrt{B^2 + \left(\frac{L}{2} - L_1 - L_3\right)^2 + 2B\left(\frac{L}{2} - L_1 - L_3\right) \sin(\alpha_2 + \theta)} \quad (12)$$

where L_1 must be a positive number.

Now we substitute Equation (10) into (9) to get:

$$L_3 = \sqrt{B^2 + \left(\frac{L}{2} - L_1 - L_3\right)^2 - 2B\left(\frac{L}{2} - L_1 - L_3\right) \sin(\alpha_2 + \theta)} \quad (13)$$

Similarly, expressions equivalent to Equations (11) to (13) can be derived for three-high (211), three-high (121), and three-high (112) riggings (Payne, 1977).

We note that Equations (11) to (13) represent a set of three simultaneous equations with three unknowns α_2 , L_1 , and L_3 . They can be solved using numerical methods. One of the possible methods (Payne, 1977, 1980) is to guess the initial values of the unknowns for a given angle of inclination θ , and then calculate the new values of the same unknowns. This process is repeated until a desired degree of accuracy is achieved. It has been found that convergence of the variables are often very rapid (Payne, 1977). A good guess of the initial values can be obtained by assuming that the panel is horizontal, then

$$\theta = 0$$

$$\alpha_2 = 0$$

$$L_3 = L_1$$

Therefore $L_1^2 = L_2^2 + B^2$ (for a right angle triangle).

For a three-high (222) rigging, $L_2 = \frac{L}{2} - L_1 - L_3$. Hence we get:

$$L_1^2 = \left(\frac{L}{2} - L_1 - L_3\right)^2 + B^2$$

$$= \left(\frac{L}{2} - 2L_1\right)^2 + B^2$$

$$\text{or } 3L_1^2 - 2LL_1 + \left(\frac{L^2}{4} + B^2\right) = 0$$

Since both L and B are known quantities, the equation above represents a quadratic equation in L_1 which can be solved readily. L_3 and α_2 are then obtained from the two remaining equations. (Note that when $\theta = 0$, $\alpha_2 = 0$ and $L_3 = L_1$). This set of values for $\theta = 0$ is used as an initial guess for the case when θ is greater than zero (Payne, 1977).

After the values of the three unknowns as mentioned above are known, L_2 , α_1 , and α_3 can be found from Equation (10), (6), and (7) respectively. The angles which the cables make with the panel β_1 , β_2 , and β_3 are then determined as follows:

$$\beta_1 = 90^\circ - \alpha_1 - \theta$$

$$\beta_2 = \beta_1 + \alpha_1 - \alpha_2$$

$$\beta_3 = \beta_1 + \alpha_1 + \alpha_3$$

To find the cable force T, consider the rotational equilibrium about the reaction R:

$$W_r \bar{x} \cos \theta = P(a + b)$$

$$\text{or } P = \frac{W_r \bar{x} \cos \theta}{a + b}$$

However, the vertical force equilibrium at the top pulley requires that

$$P = 2T[\cos \alpha_1 + \cos \alpha_2 + \cos \alpha_3]$$

$$\text{Therefore } T = \frac{W_r \bar{x} \cos \theta}{2(a + b)(\cos \alpha_1 + \cos \alpha_2 + \cos \alpha_3)} \quad (14)$$

The tension and shear loading on each insert can be found once the tension in the cable is known.

The equations that govern the cable geometry of a four-high rigging are more complicated and more elaborate to derive (Payne, 1977). However, the approach is essentially the same as the derivation of three-high rigging.

The foregoing derivation of cable geometry equations for two-high and three-high rigging was based on the assumption that the panel bottom edge will continuously rest on the ground when the panel is tilted into vertical position. When the loadings on the inserts are input as concentrated forces acting on the panel, it is assumed that there is no dragging of the panel on the ground; otherwise there will be a horizontal force acting along the bottom edge, increasing the tensile stresses there.

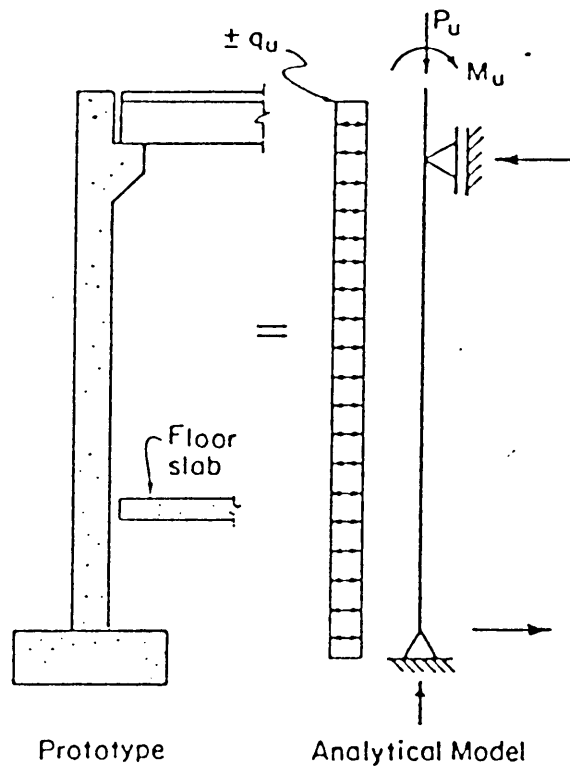
When using a computer program which is based on the stiffness method to analyse the panel (as a beam), it may be necessary to make sure that the structure is stable. Therefore instead of applying all the insert forces as loads, one of the inserts is assumed as a support. The reaction at this support will then be equal to the cable force acting on the insert (and can possibly be used as a check).

The flexure stresses in the panel are calculated using the formula $\frac{Mc}{I}$. For a panel with an opening, the position of its center of gravity will be different from a solid panel. For a section through the opening, the moment of inertia I will be reduced and hence the flexure stress will be increased accordingly.

A more sophisticated and precise way of analyzing the flexure stresses in the panel is the finite element method. Because of the time and cost involved, some designers feel that the method is only justified for complicated panel due to its size, shape (geometry), or extraordinary rigging configuration. However, the picture is changing now, as more and more design engineers and practitioners are exposed to different finite element computer programs of varying levels of sophistication (Saia, 1985).

7.0 DESIGN OF TILT-UP WALL PANELS

ACI 318-83 has provisions on wall design in Chapters 10 and 14. However, due to several reasons the design equations provided are not suitable for application to tilt-up walls. The empirical equation in Section 14.5 (Equation (14-1)) was developed mainly for short walls under reasonably concentric loads which are not usually met by tilt-up walls (Kripanarayanan, 1980). The moment magnification method of compression member design as given in Section 10.11 can only be applied when $\frac{kl_u}{r}$ is less than 100. However, most tilt-up bearing walls usually have $\frac{kl_u}{r}$ greater than 100 and require the analysis defined in Section 10.10.1 which takes into consideration the influence of axial loads and variable moment of inertia on member stiffness and fixed end moments, effects of deflections on moments and forces, and the effects of duration of loads. It was toward these ends that some of the design charts and tables were developed to facilitate the design of tilt-up walls.



(a) Wall on continuous footing

Figure 37. Analytical model of tilt-up wall panel (PCA, 1979).

7.1 *PCA Tilt-up Load Bearing Walls Design Aid*

A set of design aids was developed by PCA in 1974 and updated in 1979. It is the most extensively used set of design aids in the tilt-up construction industry.

7.1.1 Design Parameters and Assumptions in PCA Design Aid Development

In the development of the design aid, the wall panel is considered as hinged along its loaded edges and free along its vertical edges (see Figure 37). In terms of stability, this is a rather conservative assumption since the wall should have been modeled as a plate simply supported on four edges, its in-plane buckling load will be several folds higher. The wall panel is also assumed to be resting on a continuous footing, initially straight, and laterally restrained at the top. Tilt-up walls are usually reasonably stiff in their own plane, and because they are connected together by roof and floor slabs, lateral sway is usually negligible. The design aid ignores the tensile strength of concrete under in-place loading and assumes a unit weight of 150 pcf for the concrete, $f'_c \leq 4000$ psi, and a yield strength of 60 ksi for the reinforcement. The parameters used in the design aid for determining the load carrying capacity of the wall are shown in Table 3 (Kripanarayanan, 1980).

7.1.2 Effects of Isolated Footing on Load Carrying Capacity

For a wall panel resting on isolated footings instead of a continuous footing, an approximate reduction factor has to be applied to its load carrying capacity. Referring to Figure 38:

$$\eta = \text{reduction factor} = \frac{P_{crl}}{P_{crc}}$$

Table 3. PCA tilt-up wall panel design aid parameters (Kripanarayanan, 1979).

Parameter	Tables (Appendix A)
Unit weight of concrete (w), pcf	150
Compressive strength of concrete (f'_c), psi	≤ 4000
Yield strength of reinforcement (f_y), ksi	60
Capacity reduction factor (ϕ)	1.0
Panel thicknesses (h), in.	5½, 6½, 7½, 9½
Reinforcement ratios (ρ), %	0.15, 0.25, 0.50, 0.75
Transverse loads (q_u), psf	0, 15, 30, 45, 60, 75, 90, 105
End eccentricities (e), in.	1.0, $h/2$, $h/2 + 3\frac{1}{2}$
Slenderness ratios ($k\ell_u/h$)	20, 30, 40, 50

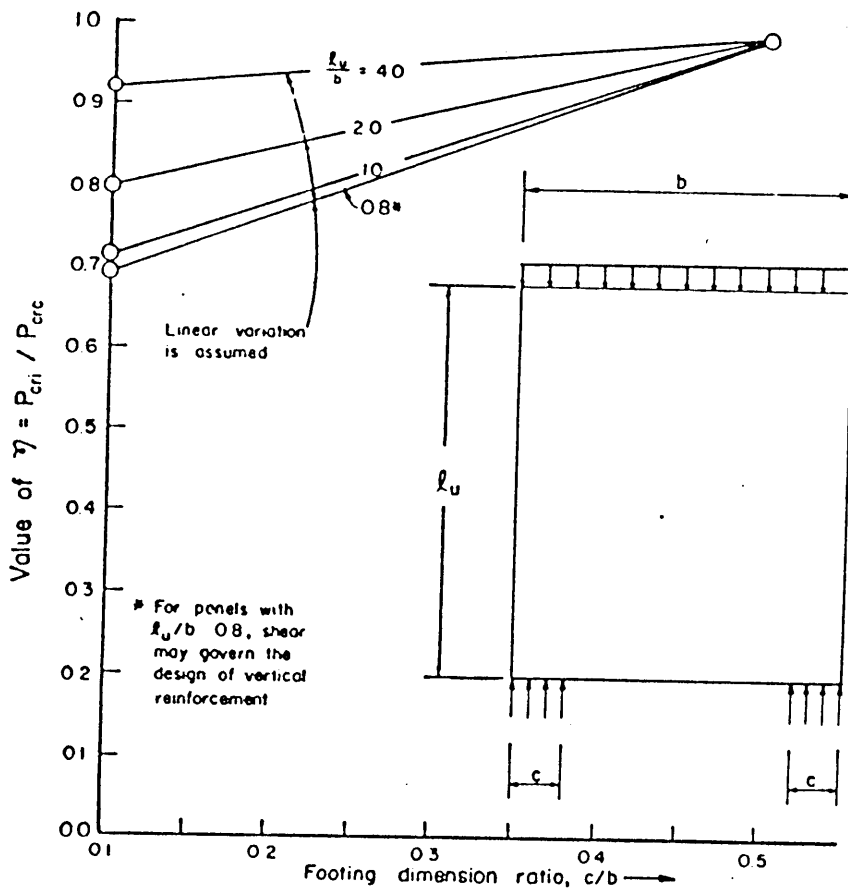


Figure 38. Capacity reduction factor for panels on isolated footing (PCA, 1979).

where

P_{crl} = buckling load of a strip of element (unit width)
of a panel on isolated footings of specific dimensions.

P_{crc} = buckling load of a strip of element (unit width)
of a panel on continuous footing.

For a panel resting on isolated footings, its vertical reinforcement must also satisfy shear requirements based on deep beam action. ACI 318-83 Section 11.8 (Equations (11-29) and (11-31)) provides for vertical reinforcement for fulfilling the shear strength requirement.

7.1.3 Effects of Sustained Loads

Sustained service loads on tilt-up walls varies from 500 to 1000 lb per linear foot (PCA, 1979). To account for the additional eccentricity caused by creep, PCA has adopted the following approximate expression:

$$e_a = \frac{\beta C_t h}{20}$$

where

e_a = additional end eccentricities due to creep, in inches,

β = ratio of sustained load to total design load,

C_t = creep coefficient, ratio of creep strain to
initial strain (varies between 1 and 2),

h = overall panel thickness in inches.

The equation above is an approximate means of incorporating the effects of sustained loads and does not include slenderness ratio in the expression (PCA, 1979).

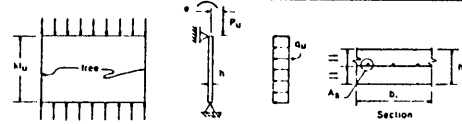
7.1.4 Additional Design Considerations

For a wall panel with openings, loads are transferred to the foundation via the element framing the openings. A rational and conservative approach is to distribute the loads that originally exist over the widths of the openings to the remaining portion of the panel, proportionally. Therefore, the remaining width of the panel has to be designed to carry this additional load. The wall beam above the opening must also be checked for flexure, shear, and torsional stresses.

When an interior beam is resting on top of a wall panel, a concentrated force is acting on it. The bearing stress on the concrete must be checked, and the design bearing strength should not exceed $\phi(0.85f_cA_1)$ as per ACI 318-83 Section 10.15 where A_1 is the loaded area and ϕ is the strength reduction factor (taken as 0.70 for bearing stress). The concentrated load is assumed to be distributed over a specific effective width of the panel. According to ACI 318-83 Section 14.2.4, effective width to be considered is the smaller of (1) center-to-center distance between loads and (2) width of bearing plus four times the wall thickness.

Examples of PCA design tables are shown in Figure 39 on page 90. From the figure, we can see that the load carrying capacities of the wall panel are practically independent of the amount of reinforcement at low eccentricities, as the capacities approach the Euler loads. For high eccentricities, there is a very substantial increase in capacity with the amount of reinforcement.

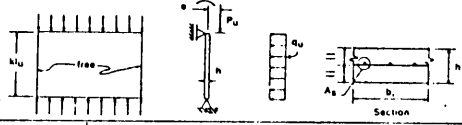
Table A5. Load Capacity Coefficients of Tilt-up Concrete Walls* ($h = 6\frac{1}{2}$ " and $q_u/\phi = 0$ or 15 psf)



$w = 150 \text{ pcf}$ $f_c \leq 4000 \text{ psi}$	$\rho = \frac{A_s \times 100}{b \times h}$	End eccentricity, e, in.	$q_u/\phi = 0 \text{ psf}$ Slenderness ratio, $k\ell_u/h =$				$k\ell_u/h @$ coeff. † ≤ 0.001	$q_u/\phi = 15 \text{ psf}$ Slenderness ratio, $k\ell_u/h =$				$k\ell_u/h @$ coeff. † ≤ 0.001
			20	30	40	50		20	30	40	50	
			$P_u/\phi = (\text{coeff.}) b, h'f_c$									
	0.15	1.00	0.498	0.347	0.227	0.155	**	0.468	0.331	0.191	0.085	**
		3.25	0.094	0.042	0.018	0.013	**	0.087	0.021	0.005	—	49
		6.75	0.018	0.014	0.005	0.003	**	0.017	0.009	0.003	—	49
	0.25	1.00	0.498	0.347	0.227	0.155	**	0.468	0.331	0.191	0.090	**
		3.25	0.110	0.050	0.026	0.018	**	0.105	0.037	0.011	0.003	**
		6.75	0.029	0.022	0.010	0.006	**	0.025	0.015	0.006	0.002	**
	0.50	1.00	0.498	0.347	0.227	0.155	**	0.483	0.331	0.191	0.100	**
		3.25	0.128	0.066	0.034	0.022	**	0.124	0.055	0.023	0.011	**
		6.75	0.049	0.034	0.020	0.012	**	0.045	0.029	0.016	0.009	**
0.75	1.00	0.498	0.347	0.227	0.155	**	0.498	0.331	0.191	0.110	**	
	3.25	0.146	0.082	0.042	0.026	**	0.142	0.073	0.035	0.019	**	
	6.75	0.069	0.046	0.030	0.018	**	0.065	0.044	0.026	0.016	**	

*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, $k\ell_u/h$, greater than 50 are not recommended.
 † This column gives the values of the slenderness ratios above which the walls have negligible load-carrying capacity.

Table A6. Load Capacity Coefficients of Tilt-up Concrete Walls* ($h = 6\frac{1}{2}$ " and $q_u/\phi = 30$ or 45 psf)



$w = 150 \text{ pcf}$ $f_c \leq 4000 \text{ psi}$	$\rho = \frac{A_s \times 100}{b \times h}$	End eccentricity, e, in.	$q_u/\phi = 30 \text{ psf}$ Slenderness ratio, $k\ell_u/h =$				$k\ell_u/h @$ coeff. † ≤ 0.001	$q_u/\phi = 45 \text{ psf}$ Slenderness ratio, $k\ell_u/h =$				$k\ell_u/h @$ coeff. † ≤ 0.001
			20	30	40	50		20	30	40	50	
			$P_u/\phi = (\text{coeff.}) b, h'f_c$									
	0.15	1.00	0.468	0.316	0.035	—	49	0.438	0.110	—	—	39
		3.25	0.079	0.011	—	—	39	0.067	—	—	—	29
		6.75	0.016	0.005	—	—	39	0.014	—	—	—	29
	0.25	1.00	0.468	0.316	0.151	0.030	**	0.438	0.301	0.065	—	49
		3.25	0.101	0.026	0.006	—	49	0.092	0.016	—	—	39
		6.75	0.024	0.014	0.004	—	49	0.023	0.009	—	—	39
	0.50	1.00	0.483	0.316	0.151	0.040	**	0.453	0.301	0.070	0.010	**
		3.25	0.121	0.046	0.016	0.004	**	0.114	0.036	0.010	0.003	**
		6.75	0.042	0.028	0.013	0.003	**	0.040	0.024	0.009	0.002	**
0.75	1.00	0.498	0.316	0.151	0.050	**	0.468	0.301	0.070	0.020	**	
	3.25	0.141	0.068	0.026	0.009	**	0.137	0.056	0.021	0.006	**	
	6.75	0.061	0.042	0.023	0.007	**	0.059	0.039	0.020	0.005	**	

*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, $k\ell_u/h$, greater than 50 are not recommended.
 † This column gives the values of the slenderness ratios above which the walls have negligible load-carrying capacity.

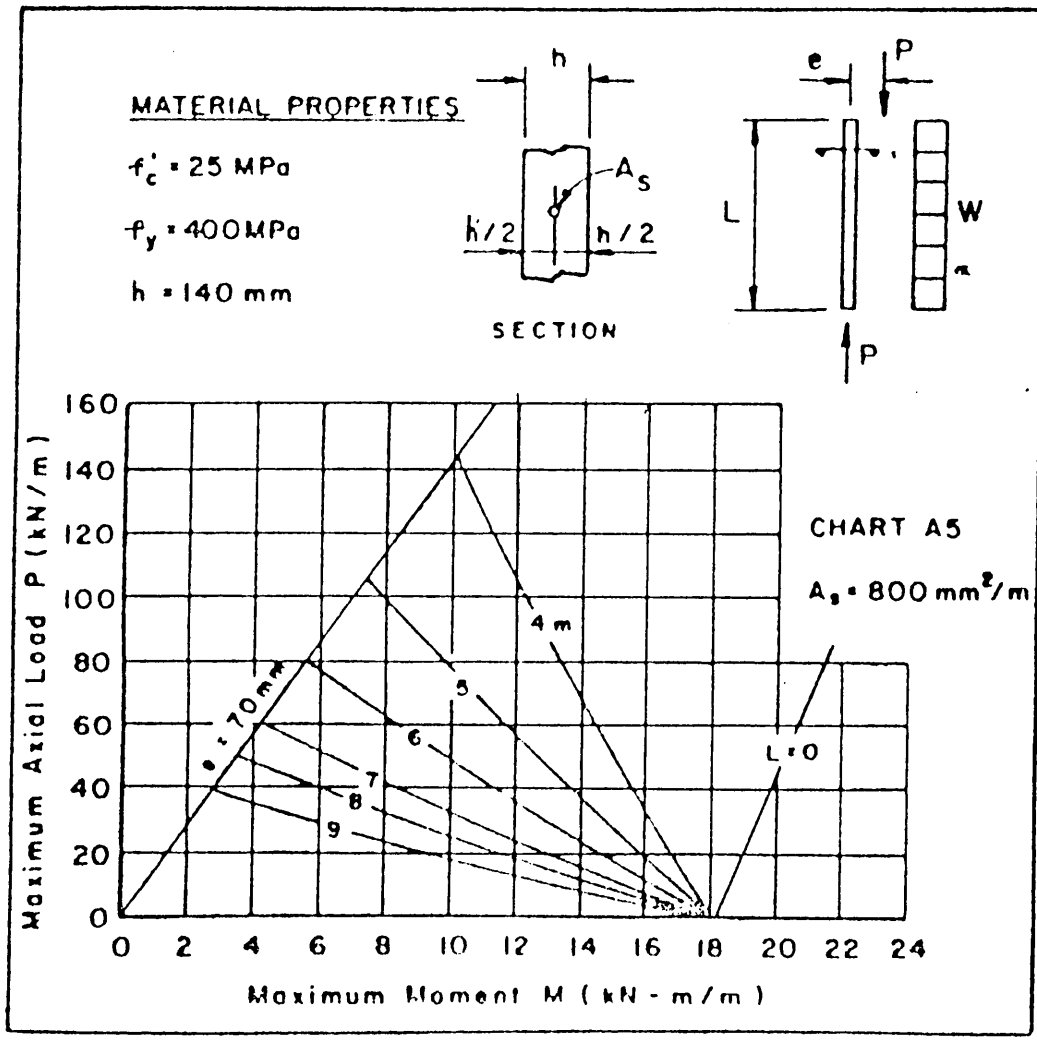
Figure 39. Examples of PCA tilt-up concrete walls design tables.

7.2 *Alternative Tilt-up Wall Panel Design Aid*

In addition to the PCA design aids discussed earlier, there is another similar set of design aids on tilt-up wall panels prepared at the University of British Columbia and distributed by the Canadian Portland Cement Association.

This set of design charts covers three wall thicknesses: 140 mm (5.5 in.), 165 mm (6.5 in.), and 190 mm (7.5 in.). The area of reinforcement ranges from 300 mm^2/m (0.14 in^2/ft) to 1400 mm^2/m (0.66 in^2/ft) which can either be placed in a single layer at the center or in two layers with 20 mm (0.8 in) clear cover. The concrete's 28-day strength and the reinforcement's yield strength are assumed at 25 MPa (3.5 ksi) and 400 MPa (58 ksi) respectively. Other assumptions regarding the end conditions of the wall panel and stress-strain curve of concrete and reinforcement are essentially the same as the first set of design aids. Panel with openings are also treated in the same manner. Concentrated loads are distributed over an allowed maximum length of loaded area plus 12 times (rather high) panel thickness. To take into account the effects of creep and initial out-of-straightness of a panel, a small deflection in the order of 0.8 in. is added at the mid-height when calculating the primary bending moment.

An example of such design charts developed is shown in Figure 4. In using the design charts, a designer first computes the maximum factored axial load P_u , and the maximum factored primary moment M_u which may be due to eccentricity or lateral load. The designer then enters a design chart corresponding to the panel thickness and amount of reinforcement chosen. If the point defined by P_u versus M_u falls below the interaction curve for the specified unsupported panel height, then the section is considered adequate.



Typical design parameter chart in common

Figure 40. One of the tilt-up wall design charts developed at the University of British Columbia (Weiler and Nathan, 1980).

7.3 Design Examples with PCA Design Aid

There are three load cases which have to be considered in designing a tilt-up wall panel (PCA, 1979):

1. D + L
2. D + L + Q
3. D + Q

where D = dead load L = live load Q = wind load.

Example 1: For the wall panel shown in Figure 41, check if it is adequate to support a dead load of 600 lb/ft, a live load of 700 lb/ft and a wind load of 20 psf.

Solution: To be conservative, a strength reduction factor ϕ of 0.70 is assumed.

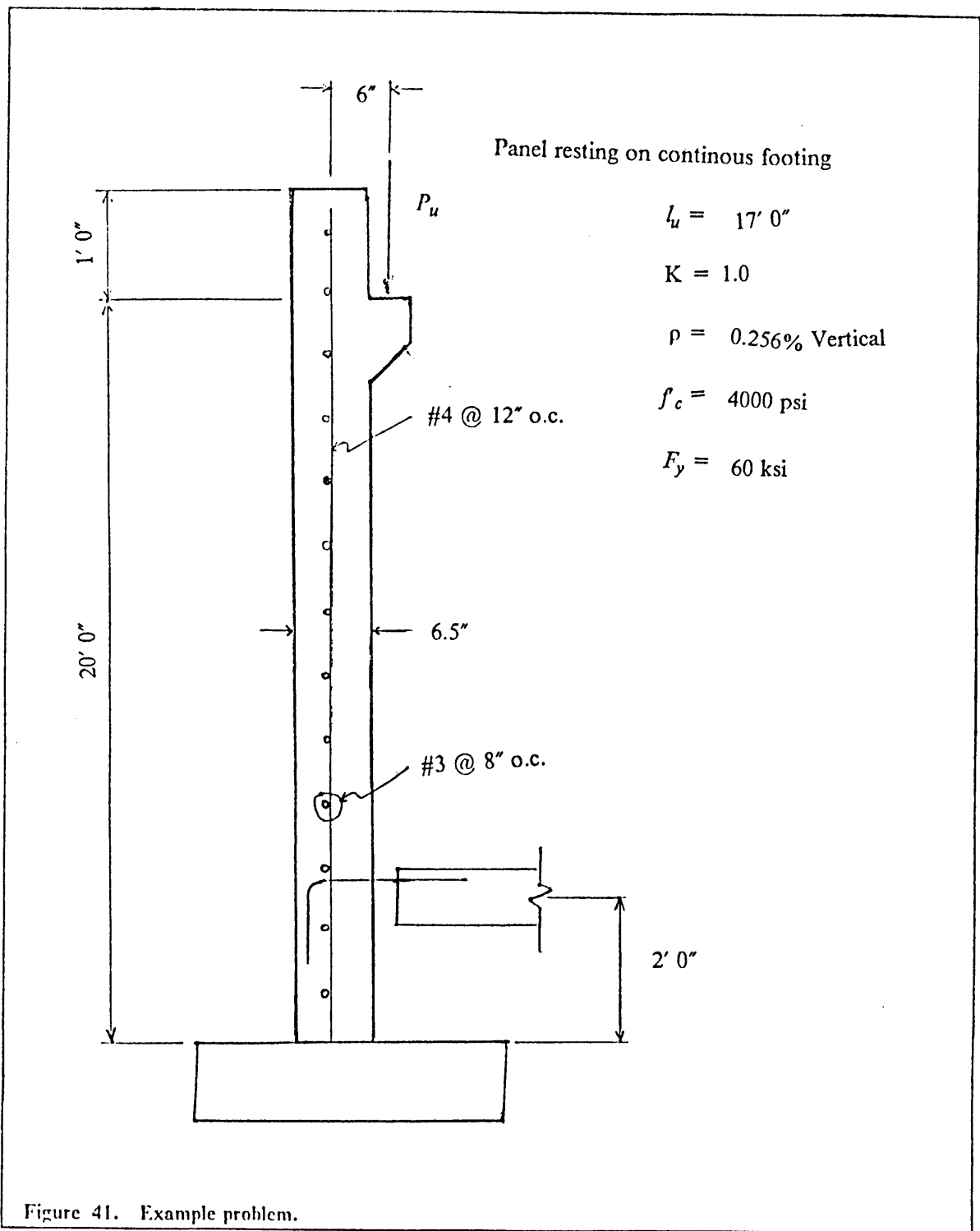
Case 1:

$$\begin{aligned}P_u &= 1.4D + 1.7L \\ &= 1.4(600) + 1.7(700) \\ &= 2030 \text{ lb/ft}\end{aligned}$$

$$\begin{aligned}\frac{P_u}{\phi} &= 2030/0.7 \\ &= 2900 \text{ lb/ft}\end{aligned}$$

$$\frac{kl_u}{h} = \frac{1.0 \times 17.0 \times 12}{6.5} = 31.4$$

From Table 3 on page 86 with $e = 6$ in., the interpolated coefficient is 0.0260.



$$\begin{aligned}\text{Therefore, furnished capacity} &= 0.0260 \times 12 \times 6.5 \times 4000 \\ &= 8110 \text{ lb/ft} > 2900 \text{ lb/ft O.K.}\end{aligned}$$

Case 2:

$$P_u = 0.75(1.4D + 1.7L)$$

$$\begin{aligned}P_u &= 0.75(1.4 \times 600 + 1.7 \times 700) \\ &= 1520 \text{ lb/ft}\end{aligned}$$

$$\frac{P_u}{\phi} = 1520/0.7 = 2180 \text{ lb/ft}$$

$$\frac{q_u}{\phi} = \frac{0.75 \times 1.7 \times 20}{0.7} = 36.4 \text{ psf}$$

From Table 3, the interpolated coefficient is 0.012.

$$\begin{aligned}\text{Furnished capacity} &= 0.012 \times 12 \times 6.5 \times 4000 \\ &= 3744 \text{ lb/ft} > 2180 \text{ lb/ft O.K.}\end{aligned}$$

Case 3:

$$P_u = 0.9D + 1.3Q$$

To be consistent with the assumptions during the development of the design aids as discussed earlier (see Figure 37), the wind is assumed to produce no axial load on the panel and is treated separately from the live and dead load.

$$P_u = 0.9 \times 600 = 540 \text{ lb/ft}$$

$$\frac{P_u}{\phi} = 540/0.7 = 771 \text{ lb/ft}$$

$$\frac{q_u}{\phi} = \frac{1.3 \times 20}{0.7} = 37.1 \text{ psf}$$

The interpolated coefficient is 0.012.

$$\begin{aligned} \text{Furnished capacity} &= 0.012 \times 12 \times 6.5 \times 4000 \\ &= 3740 \text{ lb/ft} > 771 \text{ lb/ft O.K.} \end{aligned}$$

The design is considered to be satisfactory.

Example 2: Check the adequacy of the wall panel in Example 1 if it has a width of 20 ft and is resting on isolated footings with a c/b ratio of 0.25.

Solution:

$$\frac{l_u}{b} = \frac{17.0}{20} = 0.85$$

$$\frac{c}{b} = 0.25$$

From Figure 38 on page 87, reduction factor = $\eta = 0.80$ must be applied to the capacity coefficient.

Case 1:

$$\text{Interpolated coefficient} = 0.80 \times 0.0260 = 0.0208$$

$$\begin{aligned} \text{Furnished capacity} &= 0.0208 \times 12 \times 6.5 \times 4000 \\ &= 6490 \text{ lb/ft} > 2900 \text{ lb/ft O.K.} \end{aligned}$$

Case 2:

$$\text{interpolated coefficient} = 0.80 \times 0.012 = 0.0096$$

$$\begin{aligned}\text{Furnished capacity} &= 0.0096 \times 12 \times 6.5 \times 4000 \\ &= 3000 \text{ lb/ft} > 2180 \text{ lb/ft O.K.}\end{aligned}$$

Case 3:

$$\text{Interpolated coefficient} = 0.80 \times 0.012 = 0.0096$$

$$\begin{aligned}\text{Furnished capacity} &= 0.0096 \times 12 \times 6.5 \times 4000 \\ &= 3000 \text{ lb/ft} > 771 \text{ lb/ft O.K.}\end{aligned}$$

The design is considered satisfactory. However, the reinforcements must also satisfy the deep beam requirements.

Example 3: Check the adequacy of the wall panel in Example 1 if the width of the panel is 20 feet and it has to support concentrated loads from two interior beams of 12 in. width. The beams are 10 feet apart. For each beam dead load = 5000 lb, live load = 6000 lb. The wind load is 20 psf.

Solution: Bearing strength is adequate in this case.

$$\begin{aligned}\text{Effective wall length} &= 12 + 6.5 \times 4 \\ &= 38 \text{ in.}\end{aligned}$$

Case 1:

$$P_u = (1.4 \times 5000 + 1.7 \times 6000) \left(\frac{12}{38} \right) = 5431 \text{ lb/ft}$$

$$\frac{P_u}{\phi} = 5431/0.7 = 7760 \text{ lb/ft}$$

From Example 1,

$$\text{Furnished capacity} = 8110 \text{ lb/ft} > 7760 \text{ lb/ft O.K.}$$

Case 2:

$$P_u = 0.75(1.4 \times 5000 + 1.7 \times 6000)\left(\frac{12}{38}\right) = 4073 \text{ lb/ft}$$

$$\frac{P_u}{\phi} = 4073/0.7 = 5820 \text{ lb/ft}$$

$$\frac{q_u}{\phi} = (0.75 \times 1.7 \times 20)/0.7 = 36.4 \text{ psf}$$

From Example 1,

Furnished capacity = 3744 lb/ft < 5820 lb/ft No Good

Increase ρ to 0.50. With $e = 6$ in. and $\frac{kl_u}{h} = 31.4$, the interpolated coefficient is 0.027.

$$\begin{aligned} \text{Furnished capacity} &= 0.027 \times 12 \times 6.5 \times 4000 \\ &= 8424 \text{ lb/ft} > 5820 \text{ lb/ft O.K.} \end{aligned}$$

Case 3:

$$P_u = 0.90 \times 5000 \times \frac{12}{38} = 1421 \text{ lb/ft}$$

$$\frac{P_u}{\phi} = 1421/0.7 = 2030 \text{ lb/ft}$$

$$\frac{q_u}{\phi} = (1.3 \times 20)/0.7 = 37.1 \text{ psf}$$

For $\rho = 0.50$, furnished capacity = 8424 lb/ft > 2030 lb/ft O.K.

Therefore, using number 4 rebars at 6 in. on center for the vertical reinforcement will be adequate for this case.

7.4 Approximate Costs of Tilt-up Construction

The cost of installing a typical 20'x20' tilt-up wall panel of 6" in 1986 was approximately \$5.45 per square foot, and \$5.95 per square foot for a 8" thick panel. The figures above reflect the in-place cost and include the allowance for general conditioning and overhead as well as 10% profit for the contractor.

During the same period, it cost approximately \$6.43 per square foot for the construction of a 6" cast-in-place concrete wall (Gang Formed), and \$6.82 per square foot for a 8" panel, even before the inclusion of overhead and profit (Richardson Engineering Service, 1986). However, it should be noted that the comparison above are not meant to be exact. The difference in the unit price may be greater or smaller depending on the size of the project, size of the individual panel, the type of surface finish desired, the type of connection employed, and the geographic location of the project.

8.0 SUMMARY AND CONCLUDING REMARKS

In this paper, it has been shown that a tilt-up project typically consists of precasting members on site and involves the following stages:

1. Examination of building site and the nature of the structure to determine its feasibility.
2. Preparation of subgrade and casting of floor slab which usually serves as the casting surface for the wall panels and the working ground of the crane during panel erection.
3. Precasting of the panels on site.
4. Erecting the panels and put them in their proper positions.
5. Bracing the panels to keep them in their place before the connections are made.
6. Development of structural integrity through connections between panels themselves, between panels and floors or roof, and between panels and foundation.

A few concluding remarks on the use of tilt-up construction are in order. Although tilt-up started in America and is practiced to a considerable extent only in this part of the world, it has started to affect the construction industry overseas in recent years. In 1982, Warrington and Runcorn Development Corporation arranged a visit to the United States to examine the feasibility of implementing tilt-up in Britain. The result was the successful development of a prototype advance industrial unit with a cost advantage of 12% in building construction, 30% of cost in use over metal-clad steel frame alternative. There were at least two other tilt-up structures of significant scale that have been completed in Britain in recent years.

The main problem of implementing tilt-up in other countries is the rainy climate that exists over the other parts of the world. It has been suggested that good programming and reduction of the number of concrete pours may reduce this problem since the risk of disruption due to rain will be considerably reduced.

Tilt-up construction derives its advantages by using local labor and materials in addition to the advantages of time, cost, and the choices of architectural treatment available. However, some architectural finishes such as handplacing of flat rocks may forgo the cost advantage inherent in tilt-up construction. The cost of handplacing flat rocks to create a 'prestigious' appearance for corporate buildings varies from \$5 to \$20 extra per square foot of panel surface.

It is more economical to use the floor slab as the casting surface of the wall panels. Therefore the owner as well as the designer must be aware of the extra cost which will incur if temporary casting slab has to be constructed and removed later. In order to avoid stack casting which may pose some problems at times, the building must have a minimum area of about 25,000 square feet. However, stack casting may sometimes become inevitable, especially for taller structures. The possibility of precasting the floor panels on site should also be explored. In some cases, the floor and the wall panels can be connected to form a horizontal and vertical diaphragm thereby eliminating the need for columns, beams, or separate lateral bracing.

The importance of proper and adequate analysis of stresses in concrete during panel erection and designing of lifting insert layout can not be overemphasized. Should a panel fail during erection, it not only endangers the life of the workmen but also incurs extra cost and delays the completion

of the project. The present practice is to leave this task to the responsibility of the contractor and/or the supplier; and the consulting engineer is only concern with the in-place loading design. It will be better if the consulting engineer takes a more active role during this important stage of the building design. Whenever a stress analysis shows that the concrete will be over stressed, the panel should be redesigned by either changing the insert locations, by increasing the number of inserts, by adding extra reinforcing steel, or by using strongbacks. Changing insert locations or increasing the number of inserts should be preferred over using extra reinforcement or a strongback, since a strongback may cause cracking and spalling of concrete especially at the point of connection, and the additional reinforcing steel will not contribute much to the flexure strength until the section becomes cracked.

In the case when it is necessary to carry a large reaction from an interior beam or girder, a column may be the best choice. With the availability of a pilaster clamp and a taper tie assembly, cast-in-place columns can be formed easily and cheaply.

For the in-place loading design of tilt-up wall panels, there are design aids available from various sources. One of the most commonly used set of design aids for this purpose was developed by the Portland Cement Association in the United States in 1974 and updated in 1979. Although this design aid had been very satisfactory in the past, it only covers the in-place loading of the wall panel.

Tilt-up construction has grown out of its original image of one story industrial warehouses and factories and has been used successfully in constructing taller structures up to five stories high. It is now the time to pay more attention to other important aspects of tilt-up such as the design of connections and proper modeling and analysis of tall slenderwalls which may be as high as 60 feet. It is hoped that ACI Committee 551, Tilt-up Concrete Construction, will consolidate efforts and pool resources in developing a standards or specifications in tilt-up construction. For the designing engineers tilt-up is a method worthy to be considered seriously whenever feasible because of its speed, economy, and efficiency.

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