

Design and Construction of Falsework

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DESIGN AND CONSTRUCTION OF FALSEWORK
A GUIDE TO GOOD PRACTICE

1- Introduction

Falsework consists of temporary structures used in construction to support a permanent structure until its construction is sufficiently advanced to support itself.

The British Standards of practice for falsework, BS 5975:2008, defines falsework as "Any temporary structure used to support a permanent structure while it is not self-supporting."

Proprietary falsework systems are used on almost every construction site worldwide. Workers erecting these systems may have little or no formal training and engineers are often reliant on technical advice from suppliers for their design and safe use. The failure of any falsework system will have serious consequences, and it is therefore essential we have confidence in the technical advice given by suppliers and the falsework systems we use on our projects. Within USA, there are more than 10 falsework systems in common use - but within Guangdong province, there are over 100 manufacturers and suppliers for just one of these systems!

The safe working loads and capacities for falsework systems are normally specified by the supplier – but it is not uncommon to have different load capacities for similar systems, from different suppliers!

This course note is written to provide recommended loading capacities for the common types of falsework systems available worldwide, as well as good practice notes for their design, erection and dismantling.

2- Falsework Risks

Falsework supports elevated horizontal concrete elements such as building slabs, bridges, and beams. For years, owners and engineers have recognized falsework as being a temporary process to construction that needs to be engineered by a licensed, professional engineer. For years, vertical construction was not in this category, but in recent years, even vertical formwork has fallen under the scrutiny of the owner and his engineer. No owner wants any sort of incident associated with the project, no matter how large or small. Figure 1, for example, shows falsework over a river using pipe post falsework. The risks of this type of work are illustrated from fall risks, equipment risks, and environmental risks at a minimum.



Figure 1- Pipe Post Falsework

The intent of this course is not to prepare the structural engineer for designing and building formwork and falsework to construct the new San Francisco/Oakland Bay Bridge, the Tappan Zee Bridge in New York, or a 100-story downtown office building. The intent here is to educate the engineer for designing and building safe systems that are economically efficient and within the project's budget. The larger projects will be experienced eventually when the student builds confidence and is surrounded by a team of experts in their field.

2.1 Falsework Accidents

It seems that a year never goes by in the United States without a report of a falsework accident. These accidents are not just the result of design problems but are often caused by human error or bad decisions made by field supervisors from superintendents to foremen and below. Even though methods and procedures are just as important, if not more important than design factors in temporary structures, the focus of this book will remain on design. In any case, whatever their cause, accidents are always unfortunate; and the only good that comes from them is the inherent lesson to be learned.

Who is responsible of charge for design, construction and maintenance of falsework?

Mostly, the contractor is responsible for the design, construction, and maintenance of falsework according to contract requirements. The contractor determines the type of falsework to be used and the erection and removal methods to be employed, subject to compliance with the design criteria, safety, and the conditions of use found in the specifications.

Case Study of A Falsework Accident

Project Overview

Soon after the turn of the 21st century, a highway project was constructed in northern California by a joint venture company for the State of California (owner agency). The project involved road widening, overhead bridge ramps, and accompanying drainage on Highway 149 in Butte County. Figure 2 shows typical frames (bents) and stringers (beams) in place on this project for the typical pipe falsework system.



FIGURE 2- Typical falsework for this project

The completed project would eliminate the need for surface intersections with stop signs or signals. On July 31, 2007, an overpass that was under construction collapsed.

Fortunately, no one was killed as a direct result of the collapse. However, one fatality resulted from traffic buildup, and one construction worker and one person driving underneath the collapse were injured. This relatively fortunate outcome was not the result of good decision making or good planning but was pure luck. The contractor and the State of California dodged a deadly and costly bullet.

Forensic studies were performed, as one would expect, by Cal OSHA. However, the cause of the accident was not ultimately decided by a judge. No one served any time in prison. The injured persons settled with the contractor and insurance companies; and by 2011, one would have thought nothing had ever happened. So what actually caused the collapse of the falsework?

The Cause

A falsework bent consisting of pipes and beams collapsed onto a passing truck, nearly crushing the driver and causing a construction worker to fall, when a guying cable was removed. The frames and beams were placed during a previous night shift with the highway closed in both directions with traffic detoured. During this shift, the frames were braced with wire rope guys to temporary deadmen (counterweights). Before the wire rope guys could be transferred to the adjacent bent for a more permanent attachment, the shift had ended and the highway was reopened to traffic. The following morning shift began, and the crew and owner agency realized the guying was not complete. Instead of setting up another road closure the following evening and correcting the situation without live traffic, the decision was made by the contractor and owner agency to relocate the guys that morning.

When the guy was released, so much tension had built up that the frame moved enough to generate momentum that toppled the frame and stringers it supported. The falsework bent failure was caused by human error. Had the plan been followed, the accident would probably have been avoided. However, had a better bracing system been designed and installed initially, this could have prevented the accident.

Corrective Measures

The project was delayed for several weeks. Once the investigation allowed the contractor to continue, a new bracing system was put into place. Figure 3 shows the new pipe bracing system in place.

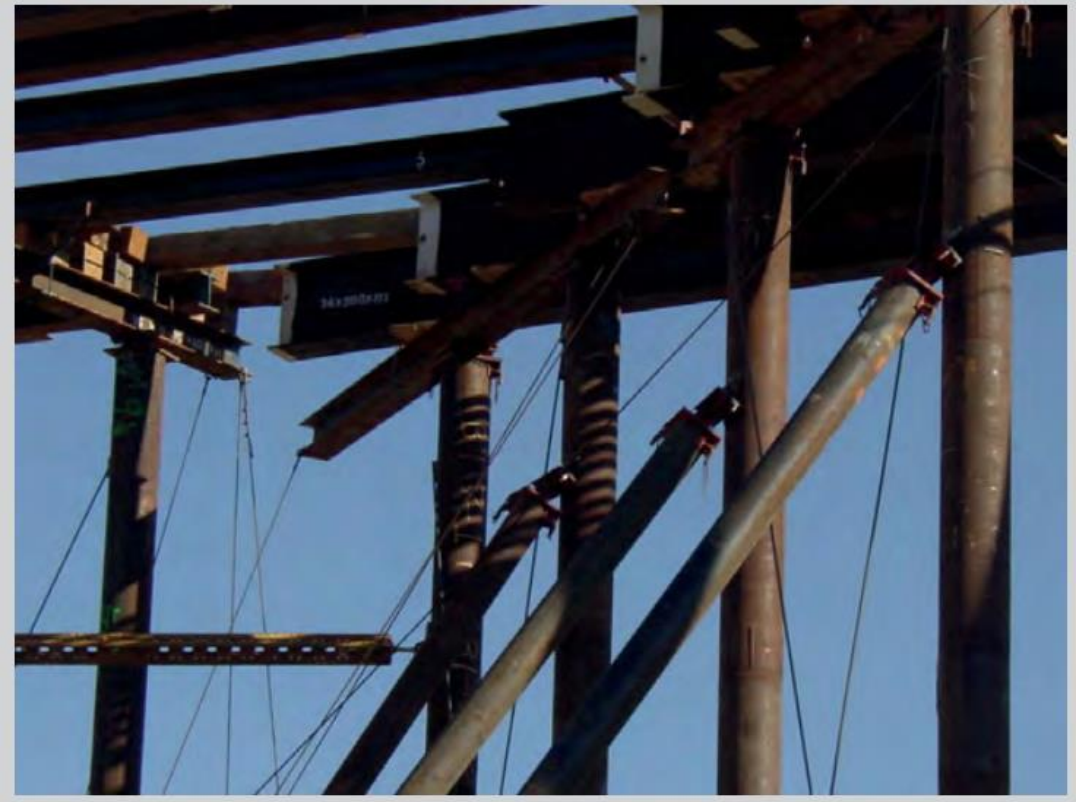


FIGURE 3- Push–pull braces added to falsework.

This bracing was added to the new falsework bents that were erected after the accident. They went to a push–pull system to take the wire rope tension out of the equation and to allow bracing on only one side of the bent since traffic was on the other side.

The parties involved spent several years, after the completion of the project, in settlement negotiations. The settlements were never made public.

The next section will discuss the measures that are taken by falsework contractors and owner/engineers to assure that accidents due to either design or to human error or incorrect methods are avoided.

3- Falsework Review Process

Two types of falsework review procedures are common in the construction industry. These review procedures are engineering checks and balances and they are:-

(1) in-house reviews and

(2) owner/engineer reviews.

In-House Reviews:

In-house reviews occur within the company performing the falsework erection and removal. This part of the process can, and frequently does, involve consulting engineers when the contractor does not have an engineer on his payroll or its engineering staff is too busy. Regardless of the engineering component, this group represents the contractor.

Some contractors develop written procedures that are followed during the design process, prior to submitting it to the owner/engineer. These written procedures contain some of the following information:

- ✓ Type of falsework risk
- ✓ Eligibility of persons involved
- ✓ Redundancy
- ✓ Risk avoidance

The contractor must categorize the risk somehow. An example would be to rate the falsework type as low, medium, or high risk. Once the levels have been decided, each risk level would represent a specific type of falsework. In other words, the lower risk falsework would be used for the most straightforward, easier systems; higher risk falsework would be used for projects spanning sensitive waters, busy highways, or in very tall systems. Table 1 has been prepared as one of many options for a review policy matrix. The dimensions used are recommendations or estimates. Each company should consult its legal department before preparing a similar policy.

Each company should develop a policy, communicate that policy to the employees, and manage the policy. Obviously, some terms would still have to be carefully defined in the document. For instance, the word “minimal” could mean different things to different people. The policies cannot have vague wording or contradictions.

TABLE 1 Example of Falsework Risk Categories

Risk Category	1. Designer and 2. Check Engineer	Falsework Type
Low	1. In-house engineer, consultant engineer 2. In-house engineer, consulting engineer (not the same)	Elevation less than or equal to 20 ft in height, beam spans less than or equal to 30 ft in length. No risk to public; no risk to environment.
Medium	1. Consultant engineer 2. In-house engineer, consulting engineer (not the same)	Elevation less than or equal to 30 ft in height, beam spans less than or equal to 40 ft in length, minimal risk to public, some minor roadways, minimal risk to environment.
High	1. Consultant engineer 2. Additional consultant engineer	Elevation greater than 30 ft in height, beam spans less than 40 ft in length, high probability there is risk to public, such as highways, railroads, and private property, high probability there is risk to environment, both which would mean substantial financial company losses.

Persons who are eligible to design and check the design should be clearly specified as well. In no case should these two responsibilities be performed by the same person. Also, in no case should these two responsibilities be performed in-house. In some cases (high risk), the company may not want any in-house involvement. The risk, in this case, would be 100% subcontracted to a consulting firm. Each company should evaluate its own risk and write a specific policy that addresses the most critical management concerns.

Owner/Engineer Reviews:

On the other side, that of the owner/engineer, similar concerns are shared for possibly different reasons. No owner/engineer wants any problem of any kind to occur during the construction process. Since falsework comes with additional risks from normal construction activities, it is scrutinized to a higher degree. Falsework approvals are, in fact, so important to CalTrans that the CalTrans Standard Specifications manual contains a separate specification section for falsework. Their specifications cover anything that has or could possibly go wrong on a highway project.

Owner/engineer reviews can contain a great deal of requirements. The following list is just some items that might be required in a falsework submittal:

- ✓ Material compliances and specifications
- ✓ Letters of certification
- ✓ Welding certifications for previously welded splices
- ✓ Calculations for all elements
- ✓ Shop drawings, separate submittals for each bridge and each frame
- ✓ Erection and removal procedures, with sequencing
- ✓ Material grades and species
- ✓ Anticipated settlements
- ✓ Required experience of designer when high risk (minimum 3 years)

4- Falsework Design Criteria

The basic design checks that apply to both formwork and falsework are:

Bending stress for plywood and dimensional lumber, $l = 10.95\sqrt{F_b S/w}$
Shear stress for plywood, $L = F_v(Ib/Q)/0.6w$
Shear stress for dimensional lumber, $L = [F'_v(b)(d)/0.9w] + 2d/12$
Deflection for plywood and dimensional lumber, $l = 1.69\ 3\sqrt[3]{EI/w}$, l is clear span in inches, and L is clear span in feet.

If a form designer is using plywood and dimensional lumber for all falsework components, these formulas will be instrumental. When other materials are used, as will be the case later in this chapter, other formulas and charts will be necessary.

Falsework loads, even though originating from material weight and gravity, are actually fairly light compared to hydrostatic pressure. Often, the loads from hydrostatic pressure were in the range of 800–1200 psf. Many falsework loads, except for large public works and transportation projects, are in the range of 200–500 psf. The Falsework loads include dead loads and live loads and self-weight of stringers. However, even with these additional forces, most falsework loads are still lighter than formwork loads. As contractors, we need to be continuously reminded of the risk in construction of elevated horizontal concrete works and always perform the checks and balances that go along with this type of work that were detailed earlier in this chapter.

This course will show three examples of different types of falsework in construction. The first two will be standard wood falsework supporting elevated reinforced concrete slabs. One will use the conventional method for calculating loads, and the other will use the formwork charts.

As known, that the elevated slab loads are a combination of the concrete and material dead load (DL) and the live load (LL) from workers, equipment, tools, and the weight of the falsework beams. The wood and plywood weight are included with the DL. The formula given was *P*max = DL (concrete, rebar and forms) + LL (workers, equipment, tools and steel beams).

DL = Slab thickness (feet) X 160 pcf (160 = concrete unit weight including rebar and forms)

LL = Depends on the type of construction (ranges from 25 to 75 psf)

Light duty = 25 psf (painters and cleaning)

Medium duty = 50 psf (carpentry)

Heavy duty = 75 psf (concrete, masonry, demolition, and steel)

4.1- Load Paths for Falsework Design

The following paths and component terms will be used in this course, depending on the type of construction. Following the example types, the load paths described will be flat slab with wood posts; flat slab with dropped beams using aluminum joists, beams, and frames; and bridge falsework using wood and steel beams.

1) Elevated slab support with all wood components—plywood, joist, cap beam, post, ground, or lower slab

2) Elevated slab support with aluminum components—plywood, aluminum joist, aluminum cap beam, aluminum 15-k frames, ground or lower slab (Figure 4)

3) Bridge falsework with a combination of wood and steel beams—plywood, wood joists, steel stringers, steel cap beams, posts, lower cap beams, sandjacks/ wedges/corbels, pads, or ground slab



FIGURE 4 Water tank falsework using aluminum system

Example 1 - Flat Slab Falsework
Design falsework to support a 25″ slab with heavy-duty live load. In order to save time, deflection will be ignored in these first two examples. Use Douglas fir–Larch, 4 × 4 joists, and 4 × 8 cap beams.

Step 1: Determine the loading from the DL + LL (see Figure 5).

DL = 25″⁄12″⁄ft X 160 pcf = 333 psf
LL workers⁄equipment = 75 psf
Total = 408 psf (use 410 psf)

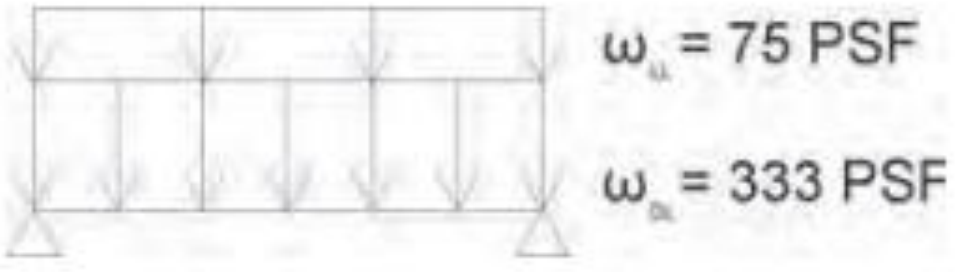


FIGURE 5 Load diagram of example 10.1

Step 2: Select the plywood and determine the joist spacing.

Since the plywood will be placed by hand and the loading is relatively light compared to wall forming, use $\frac{5}{8}''$ BB-OES in the strong direction. The plywood has an allowable bending stress value of 1600 psi and allowable rolling shear value of 60 psi.

$$W = 410 \text{ psf} \times 1 \text{ ft} = 410 \text{ plf}$$

Bending

$$l = 10.95\sqrt{(F_b S/w)}$$

where l = clear span (in)
F_b = allowable bending stress of material (1600 psi)
S = section modulus (0.339 in^3)
W = uniform load on span (410 plf)
10.95 = constant

$$l = 10.95\sqrt{(1600)(0.339)/410)} = 12.59 \text{ in}$$

Shear-Plywood

$$L = \frac{F_v(I_b/Q)}{0.6\,w}$$

where L = clear span (ft) divide by 12 to compare in inches
F_v = allowable shear stress of material (60 psi)
Ib/Q = rolling shear constant (5.824 in^2)
W = uniform load (410 plf)
0.6 = constant

$$L = \frac{(60)(5.824)}{0.6(410)} \times 12''/\text{ft} = 17.04 \text{ in}$$

Deflection is ignored for this example because it only controls long spans with light uniform loads.

$$l = 1.69\sqrt[3]{(EI/w)}$$

where l = clear span (in)
I = moment of inertia (in^4)
W = uniform load on span (plf)
E = modulus of elasticity
1.69 = constant

Ignoring deflection, the joist spacing can't exceed 12.6 in of clear span, which is governed by *bending*. The on-center spacing of the joists will be greater, and the amount depends on the width of the joist. If a Douglas fir–Larch (DFL) 4 × 4 (b = 3.5″) joist is used, then the on-center spacing would be 12.6 + 3.5″ = 16.1″, say 16″ OC. Figure 6 shows this spacing between joists.

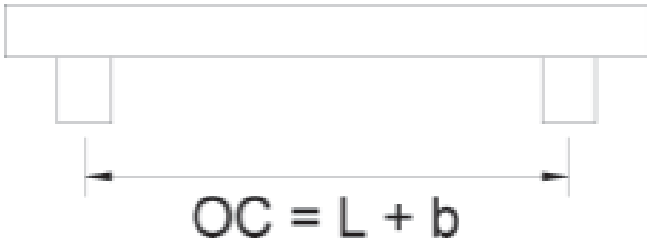


FIGURE 6 Joist spacing for example1.

Step 3: Determine the joist size and cap beam spacing.

In step 2, the joist size was selected to be a DFL 4 × 4. This joist size will be used to determine the cap beam spacing. First the uniform load on one joist must be recalculated.

$$W = 410 \text{ psf} \times (16''/12''/\text{ft}) = 546 \text{ plf}$$

along each joist. With this uniform load and the properties of the DFL 4 × 4, determine the cap beam spacing. Perform the two checks, ignoring deflection.

Bending

$$l = 10.95\sqrt{(1000)(7.15)/546} = 39.63 \text{ in}$$

Shear—Dimensional Lumber

As a reminder, the dimensional lumber calculation for shear is different than plywood:

$$L = \left(\frac{F_v' bd}{0.9w}\right) + \left(\frac{2d}{12}\right)$$

where L = clear span (ft), divide by 12 to compare in inches
F_v' = twice the allowable shear stress (2 × 95 psi = 190 psi)
b = base dimension of wood section (3.5 in)
d = depth dimension of wood section (3.5 in)
w = uniform load (546 plf)
0.9 = constant

$$L = \frac{190 \times 3.5 \times 3.5}{0.9(546)} + \frac{2 \times 3.5}{12} = 5.32 \text{ ft}$$

$$l = 5.32 \times 12''/\text{ft} = 63.8 \text{ in}$$

Deflection is ignored for this example because it only controls long spans with light uniform loads.

Ignoring deflection, the cap beam spacing can't exceed 39.6 in of spacing, which is governed by *bending*. Space the cap beams at 39″ OC because 39″ is 3.25 ft, and

this will normally work out better for the overall layout of the falsework. Figure 7 shows the cap beam spacing.

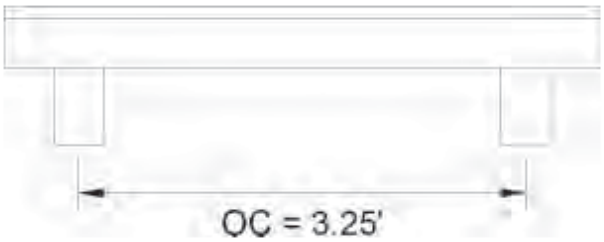


FIGURE 7 Cap beam spacing for example1

Step 4: Determine cap beam size and post spacing.

In step 3, the cap beam size was selected to be a DFL 4 × 8. This cap beam size will be used to determine the post spacing. Once again, the uniform load on one cap beam must be recalculated.

$$W = 410 \text{ psf} \times (39''/12''/\text{ft}) = 1333 \text{ plf}$$

along each cap beam. With this uniform load and the properties of the DFL 4 × 8, determine the post spacing. Perform the two checks, again ignoring deflection. Be sure not to double the cap beam properties as was done in formwork using double walers. Cap beams are single beams, so the properties are left single.

Bending

$$l = 10.95\sqrt{(1000)(30.66)/1{,}333} = 52.51 \text{ in}$$

Shear

$$L = \frac{190 \times 3.5 \times 7.25}{0.9(1333)} + \frac{2 \times 7.25}{12} = 5.23 \text{ ft}$$

$$l = 5.23 \times 12''/\text{ft} = 62.8 \text{ in}$$

Deflection is ignored once again.

Ignoring deflection, the post spacing can't exceed 52.5 in of spacing, which is governed by *bending*. Space the cap beams at 52″ OC. Figure 8 shows the cap beam spacing.

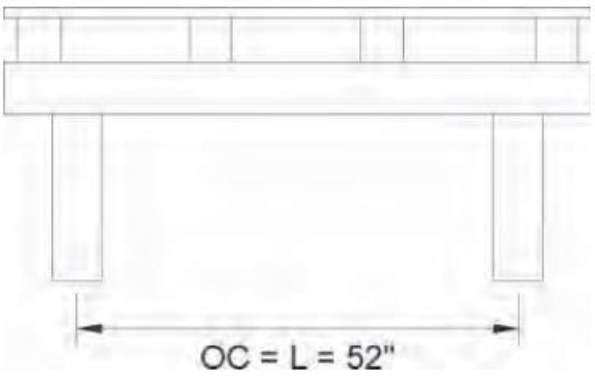


FIGURE 8 Post spacing for Example 1

Step 5: Determine post load and sizing.

The post load can be determined by distributing the P_{max} load over the tributary area of one post.

$$P_{post} = P_{max} \times (\text{cap beam spacing} \times \text{post spacing})$$
$$P = 410 \text{ psf} \times (39''/12''/\text{ft}) \times (52''/12''/\text{ft}) = 5{,}774 \text{ lb}$$

The tributary area per post is 14.08 $(39'' \times 52''/144)$ ft^2. A post must be designed that can support 5774 lb. For this example, it is assumed that the post height (from ground to bottom of cap beam) is 13 ft. This represents L_e = effective post length. With this information, a 4 × 4 DFL post can be checked. If this post does not work, a larger size should be checked or the post spacing can be decreased to lower the load per post. Typically, L_e can't be changed because the elevated slab must remain at the same elevation per the contract plans. However, L_e can be reduced if lateral bracing is added. For this example, it is assumed that lateral bracing will not be used, and the L_e will remain at 13 ft.

To check the post size, perform the following check:

$$\frac{P}{A} > \frac{0.3E}{(L_e/d)^2}$$

where P/A = normal axial stress on the post
E = modulus of elasticity of DFL
L_e = effective length of the post in inches
d = least dimension in a cross section of the post

Normal stress on the post would be

$$\frac{P}{A} = \frac{5774 \text{ lb}}{(3.5)(3.5)} = 471 \text{ psi}$$

This is the normal stress at the end of a 4 × 4. The allowable compression stress parallel to the grain for DFL is 1600 psi, which is well below the crushing limits. Buckling is the main issue when a 4 × 4 is required to support a load as a 13-ft-long post. To do this, use the formula above to determine the allowable buckling stress and ultimately the allowable load the post can support:

$$\frac{P}{A} > \frac{0.3(1{,}500{,}000)}{(13' \times 12''/\text{ft})/(3.5'')^2} = 227 \text{ psi}$$

The allowable buckling stress for this condition is much less than the normal stress; therefore, the buckling stress dictates the design of the post. To determine the allowable post load, plug the buckling stress back into $f_{bs} = P/A$, solve for P, and compare this to 5774 lb.

$$P_{all} = 227 \text{ psi} \times (3.5 \times 3.5) = 2781 \text{ lb} < 5{,}774 \text{ lb} \quad \text{(does not work)}$$

Since the allowable load due to buckling is not equal to or greater than the actual load of 5774 lb, a larger post must be considered. Before going to a 6× (which is very

expensive), the designer should try a 4×6, 4×8, and so forth. The largest 4× that should be considered is the size just below the cross-section area of a $6 \times 6 = 36$ in^2 (which would be the first 6× to be tested). A 4×8 is 32 in^2 and a 4×10 is 40 in^2; therefore, if a 4×8 does not work, then the 6×6 should be considered. Since the d dimension of both a 4×6 and 4×8 are 3.5″, the allowable buckling stress can be multiplied by the cross-sectional area to quickly calculate the allowable load on each. In other words, there is no need to recalculate the allowable buckling stress, as it will not change as long as the least dimension, d, does not change:

$$P_{all} \text{ for } 4 \times 6 = 227 \text{ psi} \times (3.5'' \times 5.5'') = 4370 \text{ lb} < 5774 \text{ lb} \quad \text{(does not work)}$$

$$P_{all} \text{ for } 4 \times 8 = 227 \text{ psi} \times (3.5'' \times 7.25'') = 5760 \text{ lb} < 5774 \text{ lb} \quad \text{(does not work, but very close)}$$

By observation, it appears a 4×10 would work, but for economy purposes, it was pointed out that a 6×6 should be used if a 4×8 did not work. Unfortunately, now, the buckling stress must be recalculated due to the increased d dimension changing from 3.5″(4×) to 5.5″(6×).

$$F_{bs} = 0.3(1{,}500{,}000 \text{ psi})/(13 \times 12''/\text{ft}/5.5)^2 = 559 \text{ psi}$$

$$P_{all} = 559 \text{ psi} \times (5.5 \times 5.5) = 16{,}910 \text{ lb} > 5774 \text{ lb} \quad \text{(OK)}$$

Usually when the post goes up to the next minimum dimension, the allowable capacity doubles or triples. Also, if the designer really wanted to use the 4×4's, he could go back to the beginning, then add lateral bracing and decrease the L_e. Another way to make the 4×8 work would be to slightly decrease the spacing between either the posts or cap beams, thus reducing the tributary area per post and eventually the post load. At this point, a cost comparison should be done between the 6×6 and the labor and material cost for the lateral bracing (lacing). Finally, the savvy student has also recognized that since we used a 6×6, the b dimension used in step 4 should be increased to 5.5 in, which changes the post load. Technically, this slight change can be ignored because, by using the 6×6, we built in an additional factor of safety = 16,910/5774 = 2.93 FOS.

Bearing Capacity

The crushing stress at the end of the 4×4 was discussed, but the crushing between the 4×8 cap beam and the 6×6 post has not been checked. This is more critical because this crushing is considered perpendicular to the grain of the 4×8 cap beam, which is more sensitive than the end of the post (parallel to the grain of the post).

Figure 9 illustrates the bearing area in question

$$f_{c\perp} = P/A(\text{rea})$$

$$f_{c\perp} = 5774 \text{ lb}/19.25 \text{ in}^2 = 300 \text{ psi}$$

The allowable crushing stress perpendicular to the grain for DFL is 625 psi > 300 psi (OK).

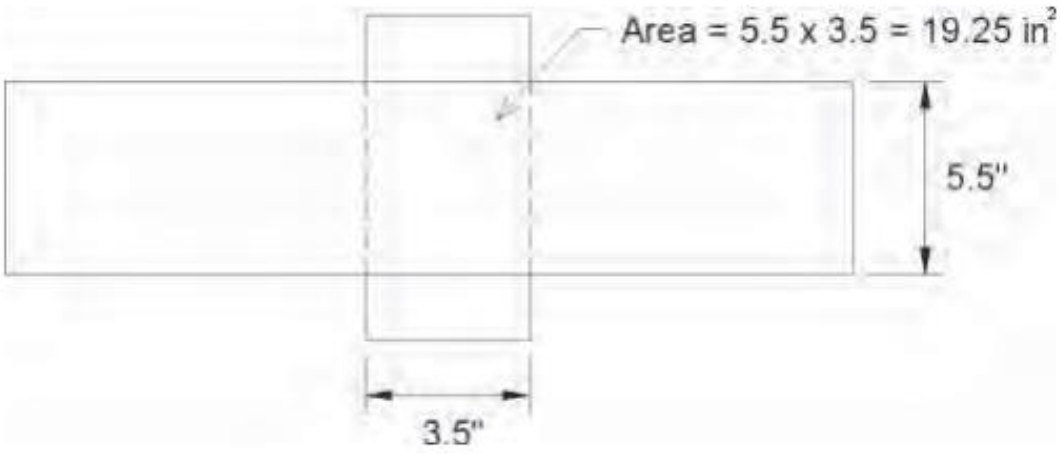


FIGURE 9 Bearing area between the post and cap beam

Final Design:

5/8″ BB-OES plywood used in the strong direction
DFL 4 × 4 joists spaced at 15″ OC
DFL 4 × 8 cap beams spaced at 42″ OC
DFL 6 × 6 × 13 ft long
No lacing for posts, crushing in both directions (OK)

4.2- Falsework Design Using Formwork Charts

The next example will utilize the formwork charts. The charts selected in this course are from the *Williams Form Catalog*. These particular charts use fairly liberal shear values, hence all spacings are governed by either bending or deflection.

Example 2 – Flat Slab Falsework Using Formwork Charts

Using the formwork tables in Appendix 1, design falsework to support a 21″ slab with medium-duty live load. Deflection will still be ignored.
Design a falsework system using the charts in Appendix 1 and following these parameters:

DL = 21″⁄ 12″⁄ft X 160 pcf = 280 psf
LL workers⁄equipment = 50 psf
Total = 330 psf (Figure 10)

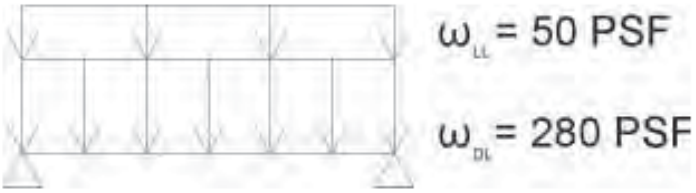


FIGURE 10 Slab load diagram

Step 1: Plywood selection and joist spacing.
For this example, 3⁄4″ BB-OES plywood will be used. Based on this plywood and the charts in Appendix 1, determine the joist spacing. This appendix is using material that has the following characteristics:

Max deflection L/360, not to exceed 1/16″
F_v = 72 psi

Fb= 1930 psi
E = 1,500,000 psi
From the previous calculation, *w* = 330 psf

Based on the plywood being used with the face grain parallel to the span (left side of chart) and *w* = 330 psf, the joist spacing should not exceed 17″ OC. In order to make the fabrication and erection most productive, the joists will be spaced at an even 16″ OC. Like the charts in the previous chapter, the values given are the center-to-center spaces.

Step 2: Joist selection and cap beam spacing.
Uniform joist load: *w* = 330 psf × (16″⁄12″⁄ft) = 440 plf

The next chart in Appendix 1 that is used is intended for all single-beam (studs, joist, and others) members. Also, this chart represents beams that are supported by three or more supports ($M = wl^2/10$), not simply supported ($M = wl^2/8$.).

In this example, a 2 × 6 joist will be used. So with a new uniform load on the joist of 440 pfl, determine the maximum spacing of the cap beams, which is the maximum span of the 2 × 6.

Some interpolation has to be used on the charts when the uniform loads are in between the values shown. Always go to the more conservative number. Taking this into account, cap beams should not be spaced farther than 56″ OC. If the designer wanted to make the layout of this system go without the potential of error, 54″ could be selected as an even 4′6″ OC.

Step 3: Select a cap beam size and determine the safe spacing between the posts.
A 4 × 8 has been selected for a cap beam. The chart selected for this step is the same as the single-beam chart because a cap beam is a single beam.
The new uniform load on a single 4 × 8 cap beam is determined as follows:

$$W = 330 \text{ psf} \times 4'\text{-}6'' = 1481 \text{ plf}$$

The single-beam chart indicates that a 4 × 8 cap beam can span 60 in between posts. Therefore, post spacing should not exceed 60″ (5′0″).

Step 4: Determine the post load on a single interior post (post design will be skipped in this example, so refer to the Example1).

$$P_{all} = \frac{P_{max} \times (\text{cap beam spacing} \times \text{post spacing})}{P_{max}}$$
$$= 330 \text{ psf} \times (4.5' \times 5') = 7425 \text{ lb}$$

***Final Design*:**
3 /4″ BB-OES plywood used in the strong direction
2 × 6 joists spaced at 16″ OC
4 × 8 cap beams spaced at 4′6″ OC
Space posts at 5′ 0″ OC (post size not determined in this example)
Post load = 7425 lb

Example 3 – Elevated Slab with Dropped Beams

Design an elevated slab and dropped beam falsework using an aluminum beam and shoring frame system.

This example will utilize standard form plywood, aluminum joists, and cap beams supported by 15 k per leg capacity aluminum frames. All components except the plywood are pre-manufactured metal products. Appendix 1 will be used for plywood properties and Appendix 2 will be used for the aluminum beam properties.

The sketch in Figure 11 shows a typical section through the slab and dropped beams. This type of falsework is more complicated because the dropped beams and the slab are at different elevations. Therefore, two levels of decking are required.

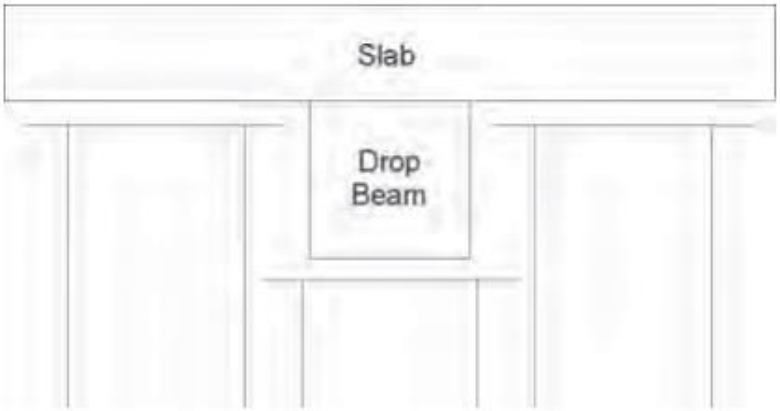


FIGURE 11 Slab and dropped beam

The design load including live load and material will be given as the following:
Elevated slab section = 220 psf
Dropped beam = 480 psf

As mentioned earlier, the aluminum beam charts from the previous chapter will be sufficient for the joists and cap beams; and each post of the modular frames can support 15 k, which already includes a 2.2 : 1.0 factor of safety. The frames come in a variety of widths and heights. Figure 12 shows an arrangement of shoring frames for an elevated slab and beam system.



FIGURE 12 Slab and dropped beam

Step 1: Determine the joist spacing for both the slab and dropped beams using the same 3/4″ plywood as Example 2. Use Appendix 1 for aluminum beam spacing.

Elevated slab = 220 psf, maximum joist spacing = 19″ OC
Dropped beams = 480 psf, maximum joist spacing = 15″ OC

Stage 1: Elevated Slab

Step 2A: Determine the maximum spacing of the slab joists using 19″ spacing and the chart for aluminum beams in Appendix 2

$$W = 220 \text{ psf} \times (19''/12''/\text{ft}) = 348 \text{ plf}$$

The aluminum beam chart in Appendix 2 determines the maximum span of the aluminum joist supporting the elevated slab. It will be assumed that these joists will span three cap beams (two spans). Therefore, the two-span section of the chart will be used. In the two-span column, go down to 348 pfl and the beam span will be indicated on the left, y axis.

Using the 1/4″ maximum deflection (started values in Appendix 2) the beam can span 10.5 ft. This is a very long span, but at the same time, the load is very light. At this point a decision has to be made on whether to space the joists at 10.5 ft or less to change the cap beam load. To make this decision, list the different cap beam loads that apply to some various joist spans.

Table 2 shows span options for the aluminum beams and cap beams

TABLE 2 Aluminum Beam Spans with Cap Beam Spacing from Appendix 2

Joist Span (ft)	Cap Beam Uniform Load (plf)	Spans (ea)	Cap Beam Spacing (ft)
10.5	2310	2	4.0
9	1980	2	4.5
8	1760	2	5.5
6	1320	2	7.0
5	1100	2	7.5

Another consideration is the size of the slab sections in plan view. If the slab lengths are 15 ft, for instance, then half of the section would be 7.5 ft. The joists can't span the whole 15 ft; therefore, it would make sense to split the distance in half and span the joists only 7.5 ft. In addition to the geometry of the concrete segments, the shoring frames come in certain sizes that range from 4-ft dimensions to 7-ft dimensions as shown in Figure 13. With this said, it would not make sense for either span to be more than 7 ft.

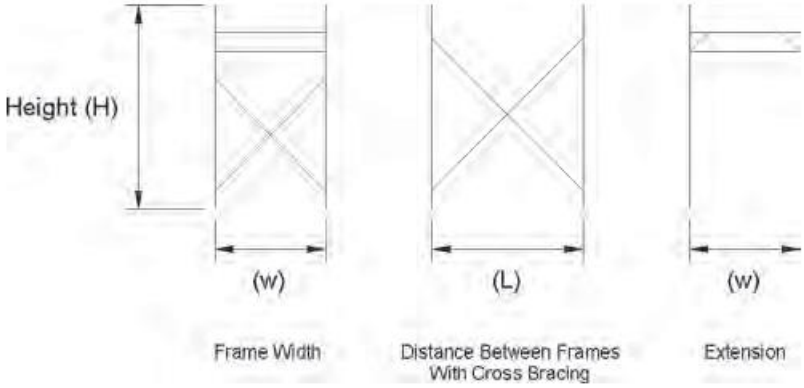


FIGURE 13 Shoring frame options.

Looking at Table 2, many options are available; but if we look at the shoring frame geometry, the spacing between the frame legs can dictate the beam spacing. Sometimes in temporary structure design, the beams allow greater spacing than is practical for the rest of the design. This happens frequently with aluminum components because aluminum is so efficient that the rest of the supports can't match its strength; therefore, the aluminum gets underutilized to compensate for the system. This does not mean that it is not economical. Even underutilized, aluminum systems have great value.

Table 3 shows some examples of the available frame sizes that can be purchased or rented.

TABLE 3 Some Available Shoring Frame Sizes

Frame Options	(*W*) Width of Frame (ft)	(*L*) Length with Cross Brace (ft)	(*H*) Height of Frame
1	4	8	6 ft
2	4	10	6 ft
3	2	8	5 ft
4	2	10	5 ft
1 Extension	4	8 or 10	5′4″
2 Extension	2	8 or 10	4′4″

In this example, we will space the aluminum beams to match an efficient aluminum frame system shown in the charts above. The 4-ft-wide frames are the most common and should be used whenever space allows. These frames will also maximize the system by using fewer frames.

Figure 14 shows the plan view layout of the shoring frames. The dropped beams are shown with double lines and the flat slabs are in between.

Stage 2: Drop Beam

Step 2B: Determine the maximum spacing of the drop beam joists using 15″ spacing and the chart for aluminum beams.

$$W = 480 \text{ psf} \times (15''/12''/\text{ft}) = 600 \text{ plf}$$

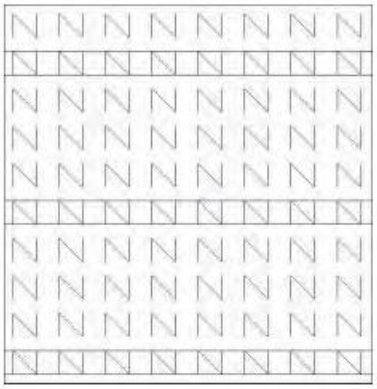


FIGURE 14 Plan view layout of shoring frames

The aluminum beam chart in Appendix 2 determines the maximum span of the aluminum joist supporting the drop beam. It will be assumed that these joists will span three cap beams (two spans). Therefore, the two-span section of the chart will be used. In the two-span column, go down to 600 pfl and the beam span will be indicated on the left, *y* axis. As before, the span is very long and the geometry of the structure may ultimately govern the actual beam spans. The chart in Appendix 2 indicates a maximum span of 9 ft using the 1/4″ max deflection. However, like the slab, this spacing may not meet the geometry of the structure. The beams are less than 2 ft wide so only one row of frames per beam will be necessary. If a 4-ft frame is used, the beam will load a single row of frames. Figure 15 shows a single row of frames under a single cap beam.

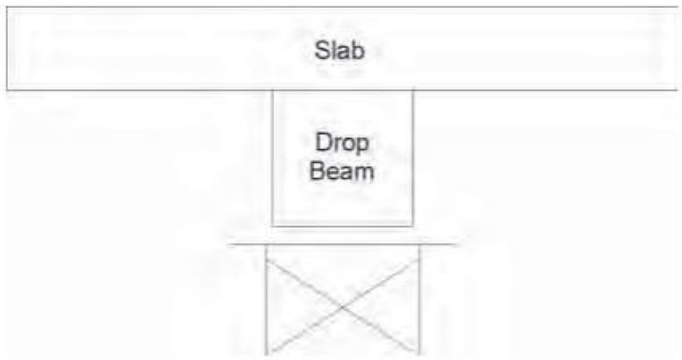


FIGURE 15 Beam shoring

Stage 3: Shoring Frames

Step 3: Shoring tower capacities are rated based on what one leg can support with a 1.5–2.5 : 1.0 FOS. The design of the shoring towers involves calculating how much load (tributary area) is supported by one individual leg. Flat slab reinforced concrete decks are easily calculated for each shoring leg because the unit weight per square foot is the same throughout the system. This dropped beam scenario, however, is more complicated because of the two different thicknesses of the top slab (220 psf) and the dropped beam (480 psf).

If the top slab shoring creates a 4-ft × 7-ft pattern, the load per shoring leg is as follows:

$$220 \text{ psf} \times 7 \text{ ft} \times 4 \text{ ft} = 6160 \text{ lb} < 15{,}000 \text{ lb (OK)}$$

Using these same frames under the dropped beam as in Figure 15 would result in the following shoring leg load:

$$480 \text{ psf} \times 2 \text{ ft wide beam} = 960 \text{ plf of beam}$$

If the same pattern is used (4′ × 7′), then the load for one leg would be as follows.

$$960 \text{ plf} / 2 \text{ sides} \times 7 \text{ ft} = 3360 \text{ lb}$$

Or

$$480 \text{ psf} \times 7 \text{ ft} = 3360 \text{ lb}$$

Most aluminum systems are rated for 10–15 k per leg, which includes the applicable FOS. These shoring towers would be more than acceptable for this example; and one might even consider a lighter weight steel system (10–11 k).

The case study to follow is an example of how a building contractor decides to form multiple-story slab decks and the methods used. This case study is purposely not similar to the chapter examples so diversity can be illustrated.

5- CASE STUDY: BUILDING FALSEWORK AND FORMWORK—CHARLES PANKOW BUILDERS, LTD.

Background

Selecting optimal formwork systems for a concrete framed building can be a daunting task. For the past 50 years, Charles Pankow Builders, Ltd. has been a leader in the building industry, specifically in buildings that utilize concrete as the building structure. This case study will provide insights on why Pankow selected the forming systems they did on two different projects.

Criteria

The following criteria are reviewed by Pankow's preconstruction group, superintendents, and foremen, when selecting form work systems. One of the first and foremost criteria is selecting forming systems that reduce on-site labor, the riskiest part of the construction process. Other criteria include:

- Labor productivity
- Familiarity of forming system
- Form reuse
- Prefabrication and rental costs
- Schedule
- Site constraints

In general, Pankow does not own its own formwork and typically rents from formwork suppliers, which allows the company to be flexible in selecting the right formwork for the job.

Case Study 1

20-Story Apartment Building, Los Angeles

Statistics

- 18 typical residential floors + 2 garage levels
- 8″ thick mild steel and post-tensioned slabs
- 9′8″ floor-to-floor height
- 154,000 ft2 per floor, one slab pour per floor
- Rectangular concrete core
- 2 shear walls outside of core

• 15 concrete gravity columns, regularly spaced and in-line
• Hoisting via tower crane (30,000-lb capacity)

For this case study, we will concentrate on the typical floor forming systems shown in Figures 16 and 17.



FIGURE 16 Twenty-story apartment building, Los Angeles, California (courtesy of Charles Pankow, Ltd and Doka Forms)

CASE STUDY: BUILDING FALSEWORK AND FORMWORK—CHARLES PANKOW BUILDERS, LTD.
(Continued)

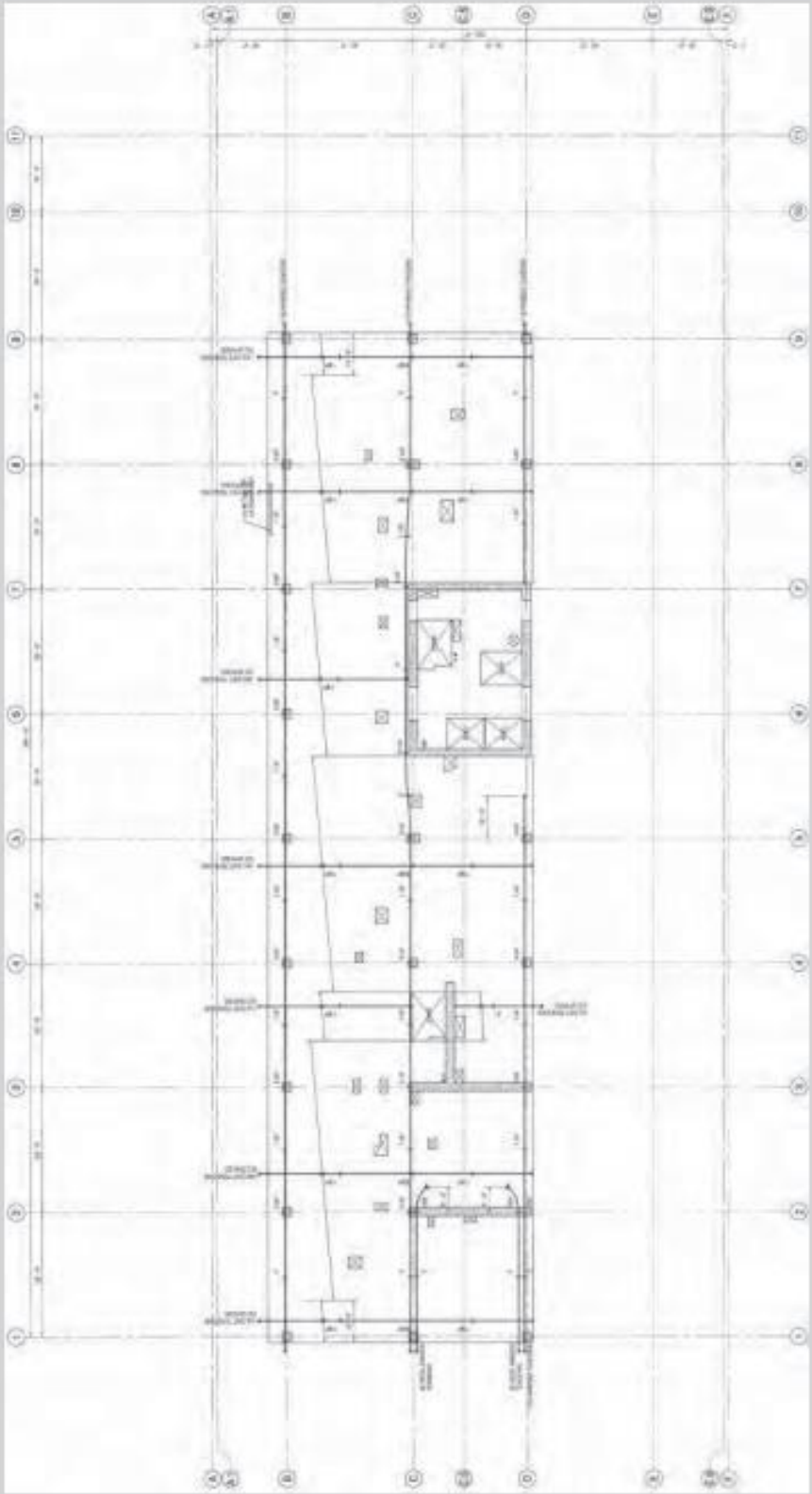


FIGURE 17 Typical floor plan (courtesy of Charles Pankow, Ltd and Doka Forms).

Floor Slab Forming System

Given the size of the floor plate, the regularly spaced and aligned columns, and the number of reuses of the forms, a column-hung, large fly table system (rented from Atlas Construction Supply, Inc.) was selected, a system that is Pankow's first choice for a project such as this. A fly table slab-forming system (Figure 18) provides Pankow good labor productivity, is fast setting, is a system Pankow's crews are familiar with, and reduces on-site labor when compared to other systems, for example, handset. The fabrication costs for these fly form systems are much higher than handset systems; therefore, a high reuse of fly forms is imperative for these systems to be economical for a project. A rule of thumb in the Pankow forming "handbook" is that fly tables will make economic sense when they are regular and can be reused at least 14 times.

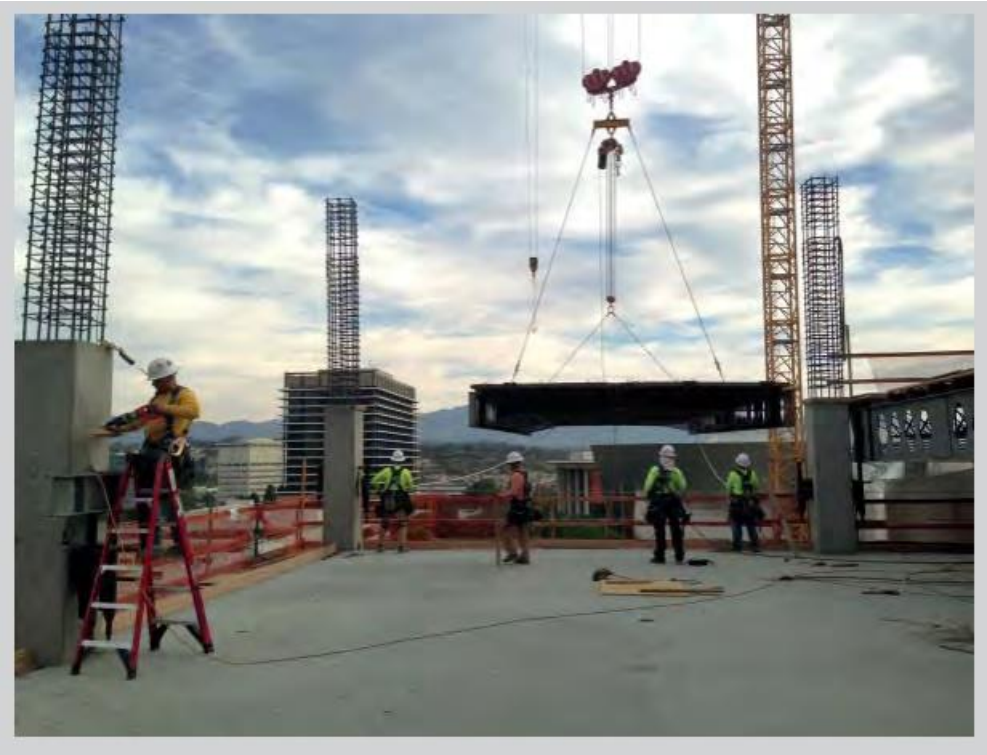


FIGURE 18 Fly table (courtesy of Charles Pankow, Ltd and Doka Forms)

When comparing fly table systems, one aspect to consider is column-hung supports or truss/shored supports. The advantage of the column-hung systems over truss/shored systems is they do not require reshores. For truss/shored fly table systems, reshores are required to be installed three floors below the floor being placed in order for the wet concrete load to be transferred safely to the floors below. These shores get in the way of trades working below the concrete deck. The column-hung fly table system does not require reshores and allows the drywall and MEP trades to begin layout and installation immediately without reshores being in the way. This helps the overall construction schedule and keeps work flow closely behind the concrete operations. Figure 19 shows a fly table being installed.

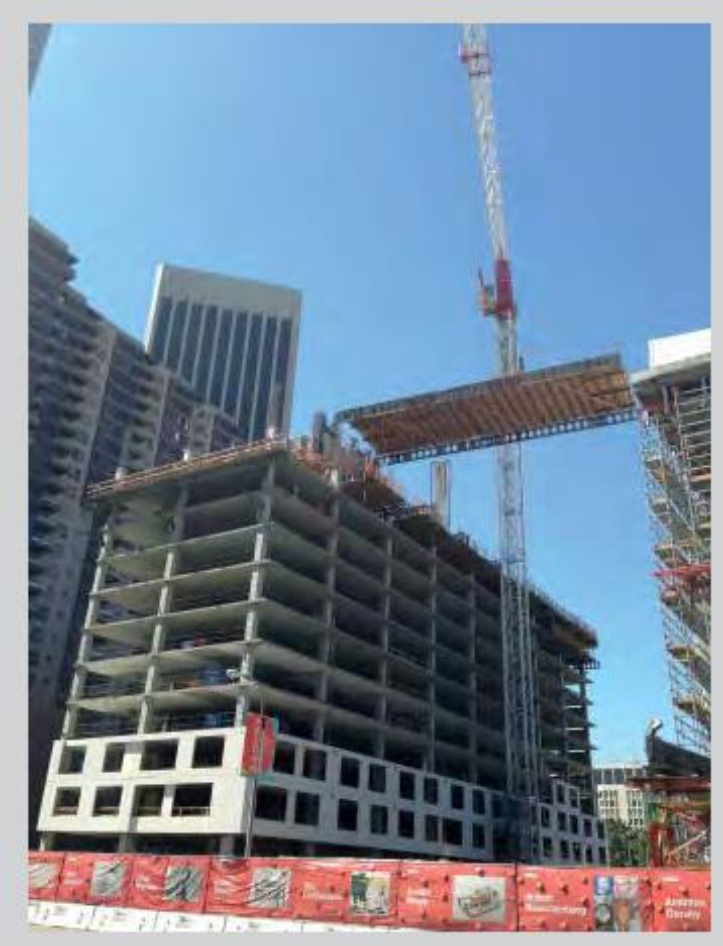


FIGURE 19 Fly table installation (courtesy of Charles Pankow, Ltd and Doka Forms).

Concrete Core Forming System

A crane set, jump form system (Figure 20) was selected to form the concrete core. Because the core walls formed a rectangular box, casting these walls ahead of the slab was very feasible. Like the flying form systems, jump form wall systems cast ahead of the slabs have high first costs when compared to other systems; however, the savings in labor and time makes up for this higher initial cost.

One convincing reason to cast the core walls ahead of the decks was schedule. This system allows the core walls and decks to be formed simultaneously, thereby reducing the floor-to-floor schedule by at least 3 days, when compared to casting the walls and decks concurrently. When extended over the entire duration of the project, this equates to nearly 2 months of construction time and provides the owner the building early for 2 months of additional rent.



FIGURE 20 Jump form system (courtesy of Charles Pankow, Ltd and Doka Forms).

A self-climbing, jump form system was also evaluated for this project; however, the project team felt the added cost for the hydraulic system associated with a "self-climber" was not worth it. As long as there is available tower crane time for raising a crane-set jump form system, a self-climber is not needed. In its spare time, the crane can also service the reinforcing steel subcontractor.

Also, if the jump form system is no more than three floors above the working deck, a personnel hoist does not need to be installed. The jump form system is accessed via ladders from a platform set inside the core.

Concrete Gravity Columns and Shear Walls

The column and wall forms (Figure 21) were set off of the previously cast slab. In order to meet the schedule, a full floor of column and wall forms were fabricated, and they had to be set and concrete placed in a single day. All of these forms were stripped the following day. The column forms selected utilized hinged forms that allowed the forms to be easily set and stripped by the carpenter crew. All the column and wall forms were flown down to the ground,
requiring adequate storage space. The project site was tight; however, the plaza level at the ground floor provided enough space for these forms to be stored during the deck-forming operation.



FIGURE 21 Concrete gravity columns and shear walls (courtesy of Charles Pankow, Ltd and Doka Forms).

Typical Floor-Floor Schedule (5-day + Sat):

• Day 1: Finish stripping fly tables from floor below, set on column-hung brackets at floor above (slab was stressed previously).
• Day 2: Install slab rebar and Post Tensioning (PT), set MEP blockouts, set edgeforms.
• Day 3: Install slab rebar and Post Tensioning (PT), set MEP blockouts; place concrete core concrete.
• Day 4: Place slab concrete, set column rebar cages, and crane-up column forms at end of day.
• Day 5: Set column and wall forms, place column, and wall form concrete.
• Day 6: Strip column and wall forms and crane down to ground floor.

NOTE: A minimum of 2 h of overtime (OT) is worked every day to meet this schedule.

Case Study2:

11-Story Apartment Building, San Francisco

Consider the Mission Street apartment building (Figure 22)



FIGURE22 - 1321 Mission Street apartment building (Courtesy of Charles Pankow, Ltd and Doka Forms).

Statistics

• 11 typical floors above grade, one garage level
• 6.5″ thick mild steel and post tensioned slabs
• 9′6″ floor–floor
• 9000 ft3 per floor, one slab pour per floor
• Nonrectangular (J-shaped) concrete core
• 2 straight shear walls outside of core
• 31 concrete gravity columns, regularly spaced and in-line
• Hoisting via tower crane (13,000-lb capacity)

The typical floor plan is shown in Figure 23

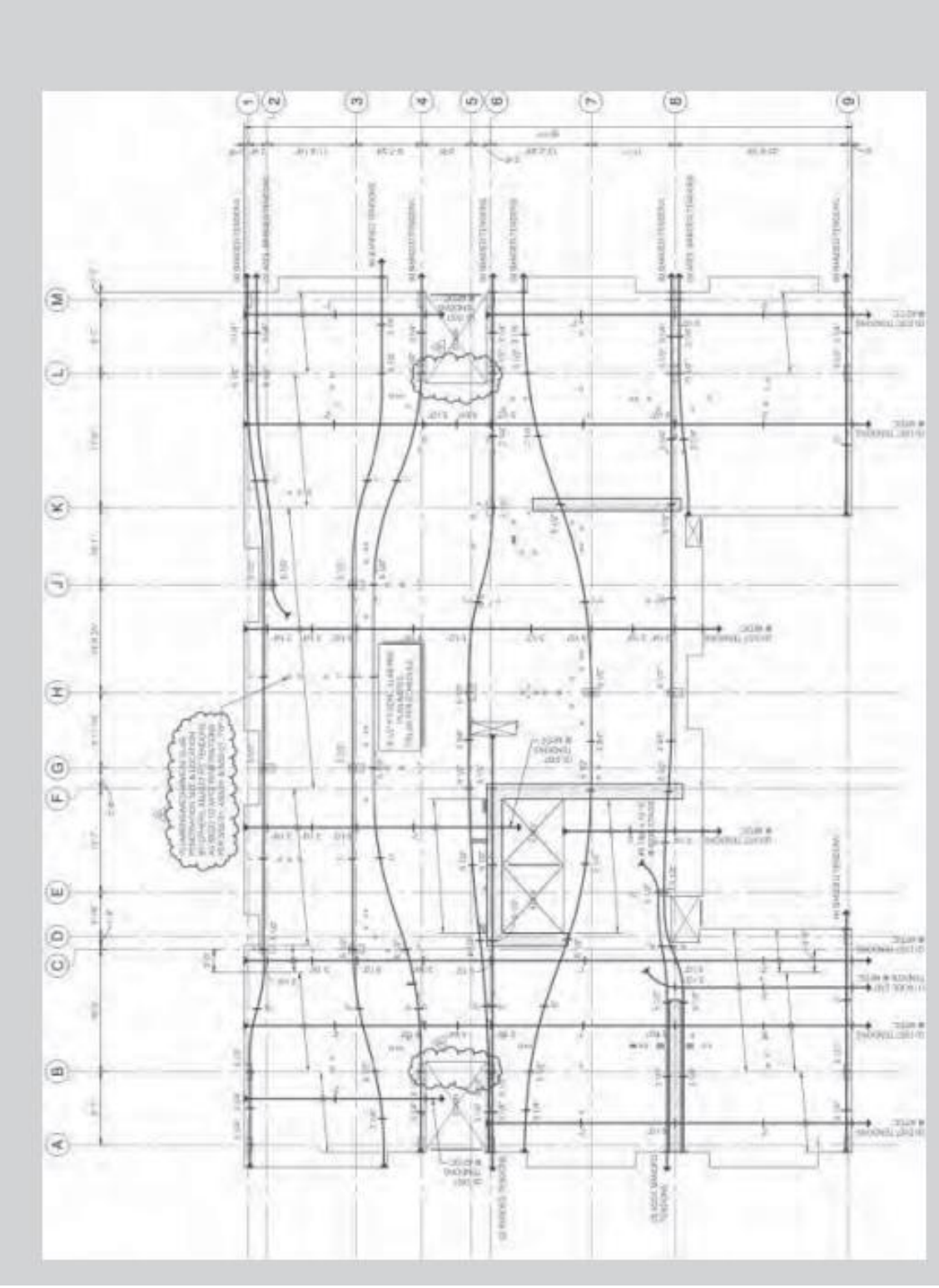


FIGURE 23 - Typical floor plan. (courtesy of Charles Pankow, Ltd and Doka Forms).

Floor Slab Forming System:

A large fly table system is Pankow's first choice to form these types of slabs; however, for this project, they did not make economic sense. First of all, the height of the building only allowed 10 reuses. Second, the shape of the building and irregular column spacing did not lend itself to large fly tables. And lastly, high-voltage electrical lines for the Muni bus line on Mission Street did not allow for the flying of large tables at the lower floors. In this case, Pankow elected to utilize a "handset" system that resembled "Ellis Shore" style forms but with a modern twist—Doka's Doka Flex System (Figure24).



FIGURE 24- Doka form system (courtesy of Charles Pankow, Ltd and Doka Forms)

Pankow did evaluate a "mini" table system for this project. An advantage of these mini tables is the preinstalled perimeter safety rail on the tables. Similar to the larger fly tables, the perimeter rail is preinstalled at the perimeter and, therefore, reduces leading edge safety risks. Although the tables have to be transported to the perimeter for the fly out, this is done very effectively via an electric transporter, affectionately called the "turtle." However, in the end, Pankow elected to go to a handset system because:

The handset system could be utilized from the ground floor slab through the roof and not require the crews to switch systems to form level 3 slab. The mini tables could not be utilized

to form the ground floor and level 2 slab, a handset system would need to be used anyway. Switching systems would add another learning curve for the crews.

The handset system is not as dependent on crane time as the mini tables. The mini tables would require use of the crane for the entire day they are set and stripped. The handset system is broken down into small bundles that can be flown up to the floor above in a short amount of time.

The irregular column layout and shape of the building lent itself to a more flexible handset system.

Many of the crews in the Bay Area are familiar with the Ellis Shore system, and the Doka Flex System is similar to Ellis Shore, but is quicker and easier to install. For this reason, the crew's learning curve with the system would be shortened.

This system requires reshores 3 floors below the formed slab. However, since the building is only 11 stories tall, the overall impact of reshores to the schedule is minimal.

Concrete Core Forming System

Unlike the Los Angeles project case study, the concrete core for the 1321 Mission project does not lend itself to be cast ahead of the slab. One reason is this project's core wall is J shaped and fully enclosed, that is, rectangular. A fully enclosed jump form "works" better than one that is not fully enclosed. Second, because there are so many columns (31 each) on the floor and because there are two additional shear walls outside the main core wall, the time saved by a jump form is reduced. Third, the added cost of a jump form as compared to a gang form set off the slab is difficult to amortize over just 11 floors. And lastly, even if time can be saved with a jump form (say 1 day per floor), that equates to about 2 weeks on the overall schedule, which is not worth the added cost (Figures 25 to 27).

The decision Pankow made was to utilize a crane set, gang form system that is cast concurrently after the slab is cast. This system requires the forms to be flown to the ground while the decks are being formed, and on this site, the storage area was very limited. In order to decrease the required storage space, the forms on the inside of the elevator were set on a platform covering the elevator shaft and did not need to be flown to ground.

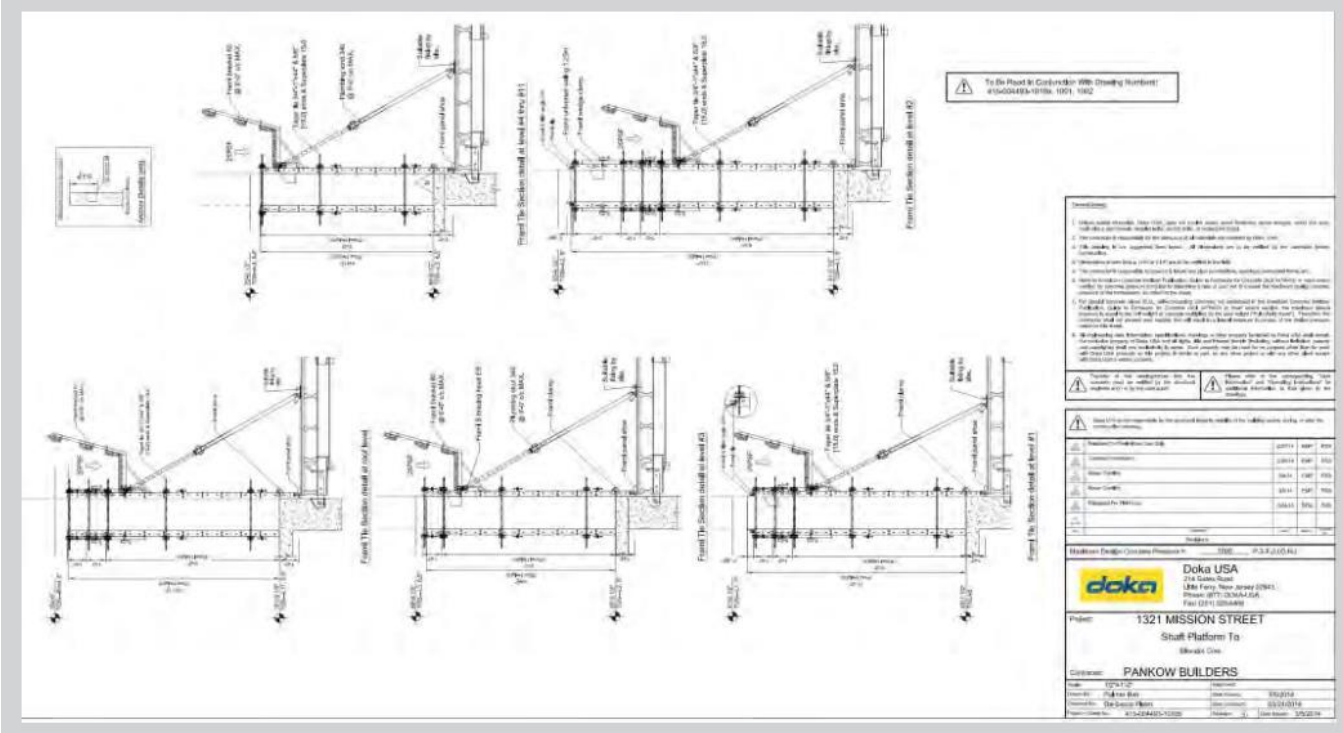


FIGURE 25- Column forms (courtesy of Charles Pankow, Ltd and Doka Forms)

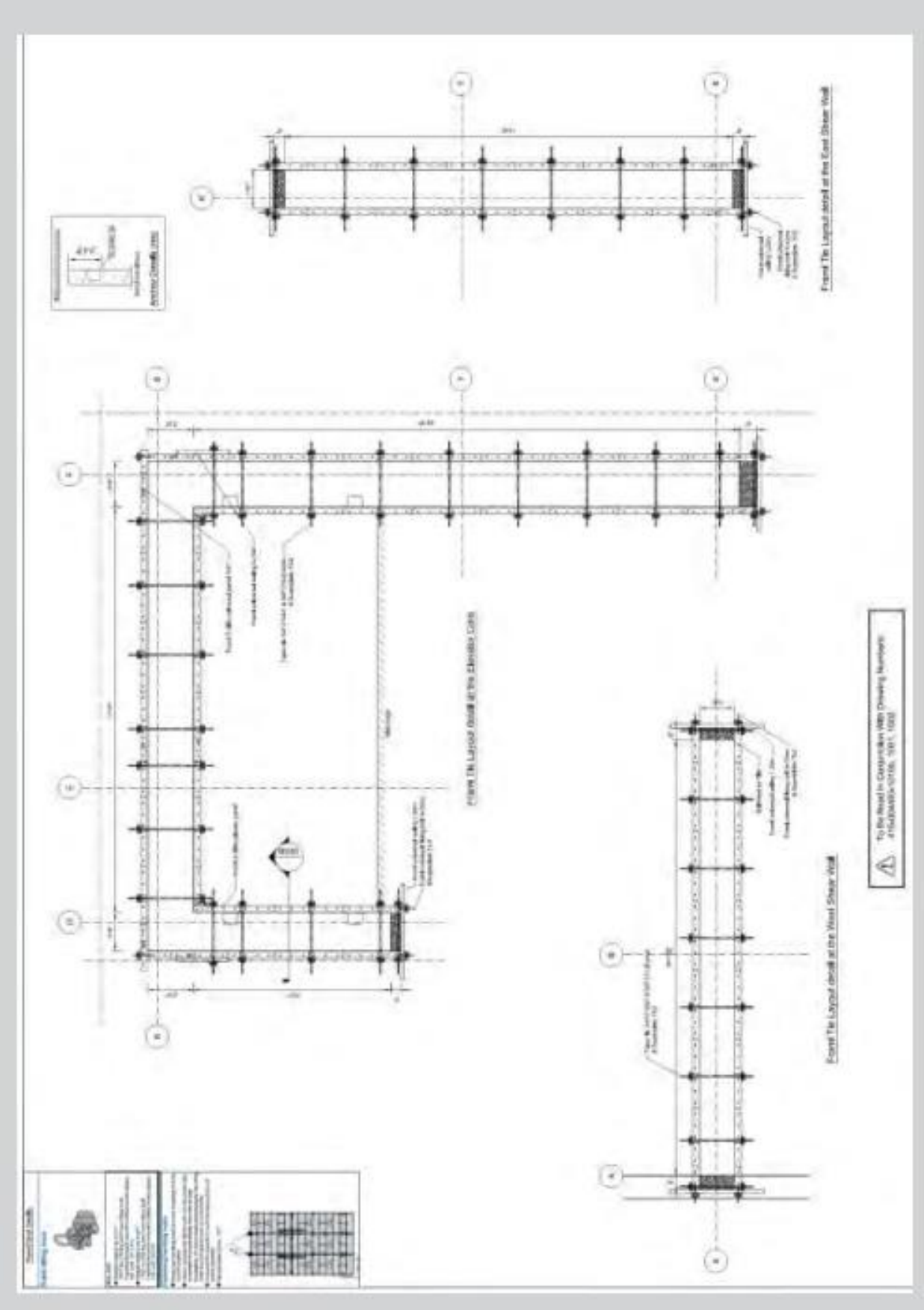


FIGURE 26- J wall (courtesy of Charles Pankow, Ltd and Doka Forms).



FIGURE 27- Column forms (courtesy of Charles Pankow, Ltd and Doka Forms).

Concrete Gravity Columns and Shear Walls

Pankow selected similar forming systems for these elements as the Los Angeles project. Hinged column forms and gang wall forms (Figure 28) were used. Due to the large number of columns and walls, a 3-day schedule was established for the forming and casting of these elements.



FIGURE 28- Wall forms (courtesy of Charles Pankow, Ltd and Doka Forms)

Typical Floor–Floor Schedule (8-day with Sat)

• Day 1: Strip deck forming system from floor below (slab was stressed previously), set on slab above.
• Day 2: Finish setting deck forming system, install slab rebar and PT, set MEP blockouts, set edge forms.
• Day 3: Install slab rebar and PT, set MEP blockouts.
• Day 4: Install slab rebar and PT, set MEP blockouts, set column and wall rebar cages, raise core wall platform.
• Day 5: Place slab concrete, set column and wall rebar cages, crane-up column forms at end of day.
• Day 6: Set column and wall forms on previously placed slabs and strip edge form (reduced crew size).
• Day 7: Pour 1/2 of columns, pour 1/ 2 of shear walls, set wall forms, Stress slab PT.
• Day 8: Strip and set columns, pour second 1/2 of columns, pour 1/ 2 of shear walls.
• Day 9: Strip columns and walls, hoist the forms to the ground.

6- Bridge Project

Box-Girder Falsework Design :A box-girder bridge is a very common bridge design used by various county, state, and federal agencies and is predominantly reinforced concrete. The foundations are the same as any other type of bridge using driven or drilled piles, concrete footings, and concrete piers. The cap beams spanning the piers are typically incorporated with the superstructure. In other words, the cap beams are within the depths of the superstructure and not between the superstructure and the top of the piers. The superstructure is a series of open boxes consisting of a top slab (driving surface), vertical stem walls, and a bottom slab. The number of boxes depends on the width of the bridge and the height of the box depends on the bridge span lengths. The longer the spans, the deeper the bridge is (more moment of inertia). The stem walls contain post-tensioning ducts with high-strength stands that are tensioned after the top deck is placed and cured.

The design of a box-girder falsework system is usually very complicated. However, the design can be simplified if all the conditions are reduced to basic scenarios such as:

- Simple box-girder geometry
- Not spanning over water, live traffic or environmentally sensitive areas
- Heights less than 30 ft

If these conditions are favorable, then a fairly simple step-by-step list can be developed in order to produce an economical falsework system. Figure 29 shows a very common, but simple box-girder bridge cross section.

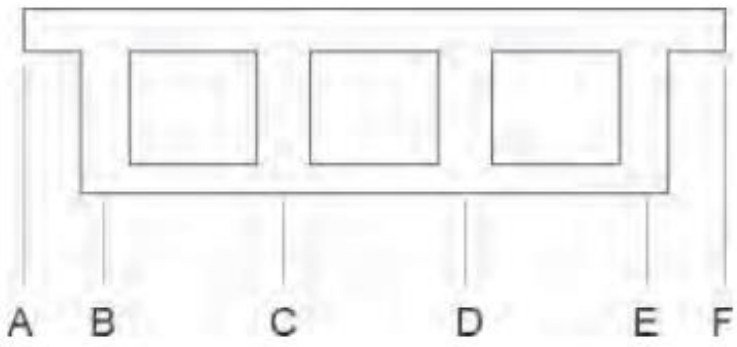


FIGURE 29- Box-girder bridge cross section

Step 1: Stringer loads
Assume a stringer will be at each location A through F line (6 stringers). The two side stringers (A and F) are under the edge of deck (EOD) and the rest of the stringers are under the centerline of each outside and inside stem wall (B, C, D, and E). Determine the associated concrete area of each line. For example, stringer B will have to support half the overhang to

line A and half the box to line C. Therefore, line B will take a shape like Figure 30.

Draw each section A–F and calculate the concrete volume (CF) of that section. Convert the volume into a weight using 160 pcf. Consider this section as a 1 ft-long bridge section and multiply by 1 ft.

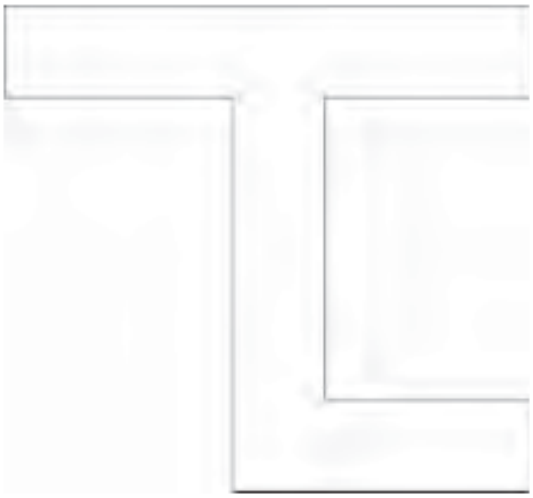


FIGURE 30- Stringer line B tributary area

Weight = Area × 160 pcf × 1 ft

Add a live load of 50 psf over the shadow area of the section.
Add 100–150 plf of bridge for the self weight of the stringer, which is not known yet.

Step 2: Soffit Design
Find the widest box width (stem wall to stem wall) and calculate the linear load from the bottom slab. Assume plywood and a joist has to span this distance and support the bottom slab load. Figure 31 shows this section.



FIGURE 31- Bottom slab spanning stem to stem

W = (thickness⁄12″⁄ft) × 160 pcf × 1 ft

Add live load of 50 psf for live load.
Use the charts in Appendix 7 to determine the best plywood and joist size based on this load.

Step 3: Stringer Design
Determine the longest span of the bridge. Using the uniform loads per linear foot of bridge for each stringer line, determine the minimum size of each stringer A–F. For example, if line C and D weigh 3000 plf of bridge and the longest span is 34 ft, the minimum stringer moment and section modulus would be as follows:

M = (3000 lb)(34′)2⁄8 = 433,500 ft-lb
S_x = (433,500 ft-lb)(12″⁄ft)⁄21,600 psi = 240.8 in^3

Perform this calculation for all lines that are different. Notice that A and F are the same, B and E are the same, and C and D are the same. Therefore only three of these calculations should be run. A table like Table 4 should be used in order to keep the work organized. Only C and D are filled in using the w = 3000 plf and L = 34 ft.

From this information in the table, determine how many braces each stringer would need and decide which of the three stringers are best for each line with a 34-ft span.

TABLE 4 -Stringer Design

	Lines A and F	Lines B and E	Lines C and D
Linear load (W = plf)	?	?	3,000
Length (ft)	34	34	34
M_{max} (ft-lb)	?	?	433,500
Minimum section (S_x)	?	?	240.8
Beam option 1	?	?	W18 × 130
Beam option 2	?	?	W21 × 111
Beam option 3	?	?	W24 × 103
Lateral braces required (#)	?	?	lb = $20{,}000/F_y(d/A_f)$

Step 4: Top Cap Design

Determine the minimum size of the top cap beam supporting the stringers. Select how many posts will be needed. In the bridge section used in this example, three to four posts would be adequate. Using the linear loads from A–F and the 34-ft span, apply each total to the top cap beam and set three to four posts as supports. Figure 32 shows this FBD.

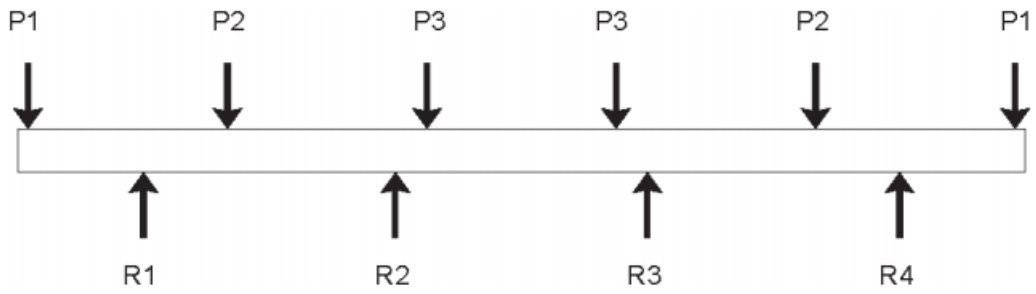


FIGURE 32 Stringers loading top cap beam

This is an indeterminate structure with four posts (supports). Using a beam program would be the most accurate and less complicated. Whichever method is used, determine the lightest top cap beam that will support this loading condition. The loads are set by the stringer layout and cannot be changed. However, the posts can be moved left and right to establish the lowest moment in the beam, thus, economizing the design. Finally check for web yielding over each post and determine if stiffeners are required.

Step 5: Post Design

Determine the load on the heaviest loaded post using that line's longest span and heaviest linear load. Assume that lines C and D have the heaviest linear load and the span of 34 ft is the longest.

$$P = W \times L$$

$$P = 3000 \text{ pfl} \times 34 \text{ ft} = 102{,}000 \text{ lb } (102 \text{ k})$$

Now determine the longest post length. In order to do this, elevations from the drawings are used and the elevation from the top of bridge to the ground needs to be known. The post length then becomes as follows:

Bridge top deck elevation (elev) - Gound elevation (elev) = gross dimension (ft) - decking material (ft) - tallest stringer and top cap (ft) - pads, corbels,wedges, and sandjacks and bottom cap (ft) = post length (ft)

With the maximum post load (P) and the maximum post length (L) calculate the minimum (square) post required:

$$P = \frac{(0.3)(E)}{(l/d)^2} \times \text{area} \leq P_{\text{act}}$$

Step 6: Lateral bracing load of the heaviest loaded falsework bent.
Determine the total bridge weight at the heaviest loaded bent. This would be the $P1$, $P2$ and $P3$ loads all added up from step 4.

$$F = (2 \times P1) + (2 \times P2) + (2 \times P3)$$

Apply 2% (minimum load) laterally to the top cap beam of the bent as shown in Figure 33.

Step 7: Stem wall formwork design.
Design the wall formwork for one of the internal stem walls (lines C or D). Assume full liquid head pressure for P, 5/8 ″ plywood, all 2 × 4 studs and walers, and snap ties with max capacity of 3500 lb. The wall should be formed from the top of the bottom slab to the bottom of the top slab. In other words, the wall height is equal to the bridge thickness minus the top and bottom slab.

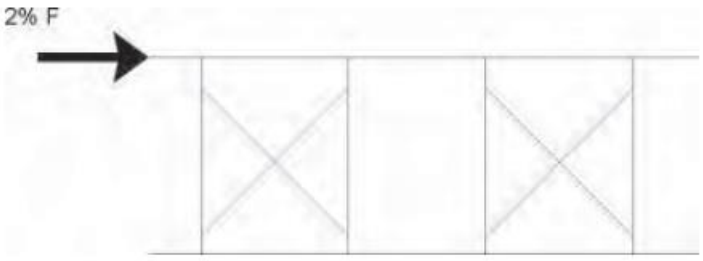


FIGURE 33- Lateral force

Step 8: Top Deck Formwork Design
Design the formwork for the top deck slab from stem wall B to C or C to D, whichever length is greater. By choosing the widest box, the worst-case scenario is used. Use 5/ 8 ″ plywood, 2 × joists, 4 × cap beams, and 4 × 4 posts.

7. Falsework Design Codes & Guides

	Reference	Issued By
Falsework	BS5975.2019	BSI UK
Falsework	Falsework-Prevention of Collapse	HKCOP
Formwork	Formwork A Guide to Good Practice v3	Concrete Society UK
Access Scaffolding	BS5973.1985 (Withdrawn)	BSI UK
Access Scaffolding	Metal Scaffolding Safety	HKCOP
Temporary Works Control	TW Procedures	TW Forum HK
Lifelines	Safety Belts & Anchorage Systems	HKCOP
Lifelines	Lifelines & Anchors	TW Forum HK
Edge Protection	Metal Scaffolding Safety	HKCOP
Edge Protection	A Guide to the Selection & Use of Temporary Edge Protection Systems	EPF UK
Edge Protection	BS13374.2013	BSI UK

8. Falsework Good Practice Notes

8.1 - Impact Protection (Figure 34)

- Provide suitable warning signs / speed restrictions / protection to prevent vehicles and plant impacting and collapsing the falsework.
- Falsework erected adjacent to public areas and highways should comply with relevant HK codes (Temporary Works Forum (HK-TWf)) and legislation.
- Provide suitable warning signs and barriers to prevent the public and workers from entering or climbing through falsework systems.

• Provide suitable warning signs / catch fans / barriers / covered walkways to protect workers and the public from falling debris and dust.



Figure 34

8.2- Debris Netting (Figure 35)

• Debris netting should be fixed to the full external face of falsework systems to ensure dust and debris are retained within the system.
• Debris netting should be fixed as each level of the falsework is erected, and not after erecting the full falsework system.
• When working adjacent to public areas and highways, additional layers of debris netting should be provided.
• Debris netting will increase the typhoon wind loads on the falsework system.
The falsework designer must consider debris netting within their calculations; construction drawings must clearly show any additional works or actions required before typhoon signals are raised.



Figure 35

8.3- Stability of Unloaded Falsework & Loading Platforms

- Construction drawings should clearly indicate **all additional works** that are necessary to maintain the stability of unloaded falsework / slender towers in typhoon winds.
- When slender falsework towers are fixed together for stability, the designer should consider the stability of the whole system at every stage of construction. Where additional works are required to maintain the stability of slender falsework towers during erection / dismantling / typhoon winds, these must be highlighted on the construction drawings.
- Whenever practicable, **loading platforms** should be braced across the full face of the falsework system. The designer should confirm the system will safely support the dynamic vertical and horizontal loads, and that the unloaded system will not overturn in typhoon winds. Do not connect loading towers to adjacent scaffold systems unless they have been designed to support the additional lateral loads.

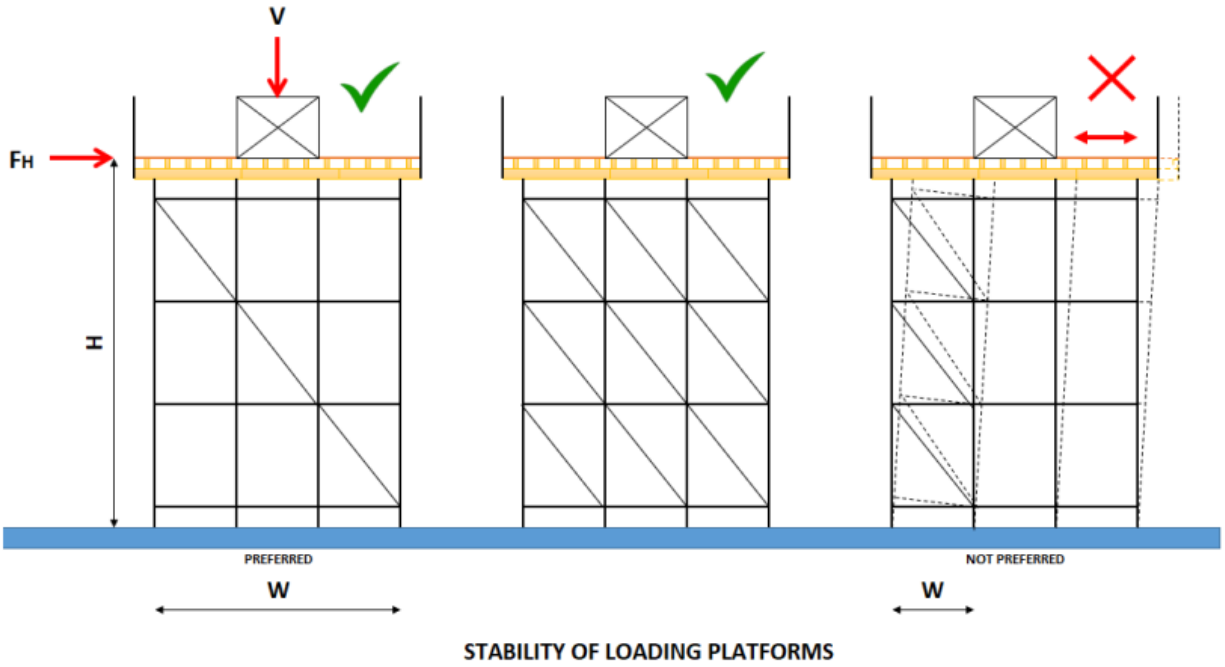


Figure 36

8.4- Mixing Falsework Systems & Components

- Avoid using similar scaffold systems and components within falsework structures / projects.
- The load capacity for similar scaffolding systems can vary significantly, and it may not be possible to distinguish between systems or components when erected within a falsework structure.

8.5- Jack Extension / Eccentric Loads / Base Jacks (Figure 37)

- Whenever practicable, top and bottom jack extensions should be < 300mm to minimise any requirement for jack bracing.
- Maximum permissible jack extensions and jack bracing details should be noted on construction drawings.
- Calculations for unbraced jacks should include eccentric and lateral loads.
- Top or bottom jacks should be loaded concentrically; U-head jacks should be twisted on plan to center bearers over the jack to minimize eccentricity.
- Where base jacks are not supported on permanent works – it will normally be necessary to provide timber / steel / concrete load spreaders below the base jacks. In every case, the contractors engineer must confirm that the permanent works / ground will safely support the imposed load.

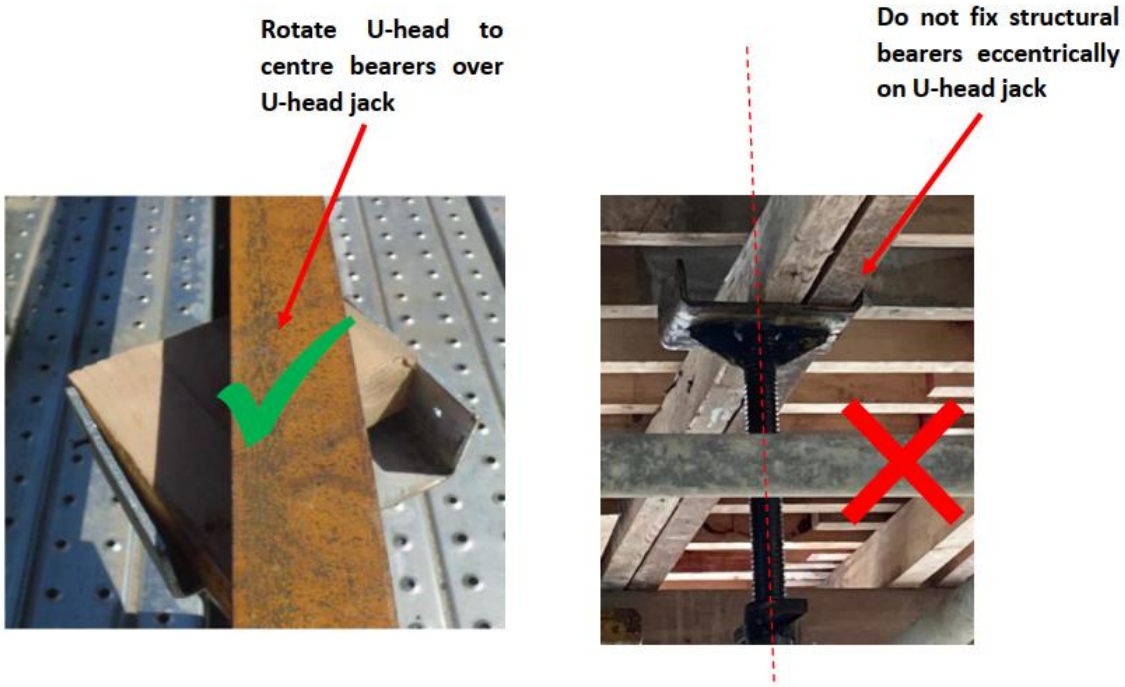


Figure 37

8.6 Safe Access / Walkways (Figures 38 & 39)

- Plan for safe (and controlled) access through large falsework structures. Safe access may be required on multiple levels, and to suit each stage of construction.
- Walkways should provide safe and controlled access for:

a) Workers erecting and dismantling large falsework / formwork structures.
b) Supervisors to monitor and control works / ensure works are carried out safely.
c) Engineers to inspect large / complex structures and ensure they are in accordance with construction drawings / are safe to load / dismantle.
d) Timely treatment or recovery of injured workers.

- Whenever possible, walkways (and stairs) should be > 1.0m wide to allow for stretcher access. Lighting and direction signboards may also be required within larger / basement structures.
- Suitable **protection must be provided above and around walkways** and stairs to prevent falling debris injuring workers.
- DO NOT permit access to walkways / within any falsework structure when concreting.



Figure 38

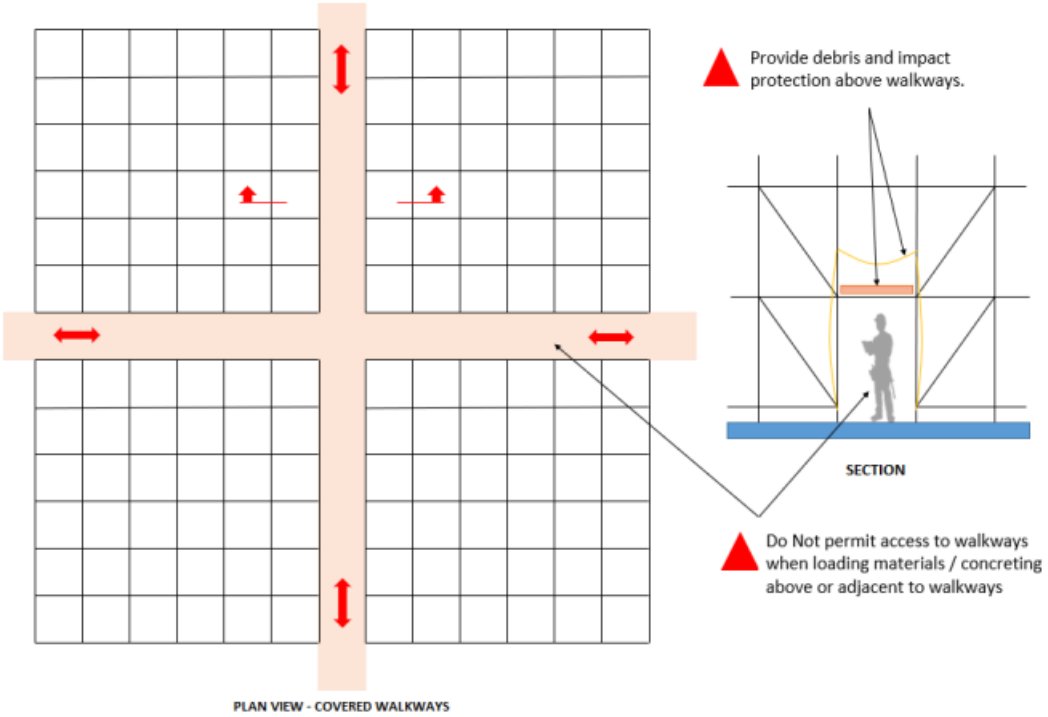


Figure 39

Common Forming Lumber Properties

Properties of Structural Lumber

Nominal Size (in) b x h	American Standard Size (in) b x h S4S* 19% Maximum Moisture	Area of Section (in^2) $A = bh$		Moment of Inertia (in) $I = (bh^3) / 12$		Section Modulus (in) $S = (bh^2) / 6$		Board Feet (per lineal ft of piece)	Approx. Weight (lbs per lineal ft)**
		Rough	S4S	Rough	S4S	Rough	S4S		
4 x 2	3-1/2 x 1-1/2	5.89	5.25	1.30	0.98	1.60	1.31	2/3	1.5
6 x 2	5-1/2 x 1-1/2	9.14	8.25	2.01	1.55	2.48	2.06	1	2.3
8 x 2	7-1/4 x 1-1/2	11.98	10.87	2.64	2.04	3.25	2.72	1-1/3	3.0
10 x 2	9-1/4 x 1-1/2	15.23	13.87	3.35	2.60	4.13	3.47	1-2/3	3.9
12 x 2	11-1/4 x 1-1/2	18.48	16.87	4.07	3.16	5.01	4.21	2	4.7
2 x 4	1-1/2 x 3-1/2	5.89	5.25	6.45	5.36	3.56	3.06	2/3	1.5
2 x 6	1-1/2 x 5-1/2	9.14	8.25	24.10	20.80	8.57	7.56	1	2.3
2 x 8	1-1/2 x 7-1/4	11.98	10.87	54.32	47.63	14.73	13.14	1-1/3	3.0
2 x 10	1-1/2 x 9-1/4	15.23	13.87	111.58	98.93	23.80	21.39	1-2/3	3.9
2 x 12	1-1/2 x 11-1/4	18.48	16.87	199.31	177.97	35.40	31.64	2	4.7
3 x 4	2-1/2 x 3-1/2	9.52	8.75	10.42	8.93	5.57	5.10	1	2.4
3 x 6	2-1/2 x 5-1/2	14.77	13.75	38.93	34.66	13.84	12.60	1-1/2	3.8
3 x 8	2-1/2 x 7-1/4	19.36	18.12	87.74	79.39	23.80	21.90	2	5.0
3 x 10	2-1/2 x 7-1/4	24.61	23.12	180.24	164.89	38.45	35.65	2-1/2	6.4
3 x 12	2-1/2 x 11-1/4	29.86	28.12	321.96	296.63	56.61	52.73	3	7.8
4 x 4	3-1/2 x 3-1/2	13.14	12.25	14.39	12.5	7.94	7.15	1-1/3	3.4
4 x 6	3-1/2 x 5-1/2	20.39	19.25	53.76	48.53	19.12	17.65	2	5.3
4 x 8	3-1/2 x 7-1/4	26.73	25.38	121.17	111.15	32.86	30.66	2-2/3	7.0
4 x 10	3-1/2 x 9-1/4	33.98	32.38	248.91	230.84	53.10	49.91	3-1/3	9.0

* Rough dry sizes are 1/8" larger, both dimensions.

** Based on a unit weight value of 40 lb. per cu. ft. Actual weights vary depending on species and moisture content.

Data supplied by the National Forest Products Association

*Form Loading in Pounds/ Sq. Foot for Incremental Slab Thickness**

Concrete Weight (lbs per sq ft)	Slab Thickness							
	2 in.	4 in.	6 in.	8 in.	10 in.	12 in.	14 in.	16 in.
100	67	84	100	117	134	150	167	184
115	70	89	108	127	146	165	185	204
125	71	92	113	134	155	175	196	217
135	73	95	118	140	163	185	208	230
150	75	100	125	150	175	200	225	250

* Values above include 50 psf live load for construction loads. Formwork dead load is not included.

Safe Spacing (ℓ) in inches of supports for plywood sheathing continuous over four or more supports. Table based on APA rated plywood class 1.

$\Delta max = \ell / 360$, but not to exceed 1/16"
$F_s = 72$ psi
$F_b = 1930$ psi
$E_a = 1{,}500{,}000$ psi
$E = 1{,}650{,}000$ psi

Safe Spacing of Supports for Plywood Sheathing

Pressure or Load of Concrete (lbs per sq ft)	Sanded Thickness, Face Grain Parallel to Span				Sanded Thickness, Face Grain Perpendicular to Span			
	1/2 in.	5/8 in.	3/4 in.	1 in.	1/2 in.	5/8 in.	3/4 in.	1 in.
75	21	24	2	32	14	16	21	28
100	19	22	25	30	12	14	19	26
125	18	21	23	28	12	13	17	24
150	17	20	22	27	11	12	16	23
175	16	19	21	26	10	11	15	22
200	15	18	20	25	8	11	15	21
300	13	16	18	22	7	9	12	18
400	12	14	16	20	7	8	11	16
500	11	13	15	19	6	7	10	14
600	11	12	14	17	6	7	9	13
700	10	12	13	16	6	6	9	12
800	10	11	13	16	5	6	8	11
900	9	11	12	15	5	6	8	11
1000	9	10	12	14	5	5	7	10

Safe Spacing of Supports

Safe Spacing (ℓ) in inches of supports for joists, studs, etc. single span.

$\Delta max = \ell / 360$, but not to exceed 1/4"

E = 1,600,000 psi

Safe Spacing of Supports - Simple Span Single-Ply Wales

Uniform Load (lbs per lineal ft)	Nominal Size of S4S Lumber	2x4	2x6	2x8	2x10	2x12	3x6	4x4	4x8
	Fb (psi) =	1500	1250	1200	1050	975	1250	1500	1200
100		60	94	115	138	160	106	80	142
200		48	75	97	116	135	89	64	120
300		42	61	79	94	11	78	55	108
400		37	53	68	82	96	68	50	101
500		33	47	61	73	86	61	47	93
600		30	43	56	67	78	56	44	85
700		28	40	51	62	72	51	42	79
800		26	37	48	58	68	48	40	74
900		24	35	45	54	64	45	37	70
1000		23	33	43	51	60	43	35	66
1100		22	32	41	49	58	41	34	63
1200		21	30	39	47	55	39	32	60
1300		20	29	38	45	53	38	31	58
1400		19	28	36	43	51	36	30	56
1500		19	27	35	42	49	35	29	54
1600		18	26	34	41	48	34	28	52
1700		17	25	33	39	46	33	27	50
1800		17	25	32	38	45	32	26	49
1900		17	24	31	37	44	31	26	48
2000		16	23	30	36	43	30	25	46

Safe Spacing (ℓ) in inches of supports for joists, studs, etc.
continuous over three or more supports.

$\Delta max = \ell / 360$, but not to exceed 1/4"
E = 1,600,000 psi

Safe Spacing of Supports - Multi-Span Single-Ply Wales

Uniform Load (lbs per lineal ft)	Nominal Size of S4S Lumber	2x4	2x6	2x8	2x10	2x12	3x6	4x4	4x8
	Fb (psi) =	1500	1250	1200	1050	975	1250	1500	1200
100		75	110	135	162	188	125	97	167
200		58	84	108	129	152	105	79	140
300		47	68	88	105	124	88	69	127
400		41	59	76	91	107	76	62	117
500		37	53	68	82	96	68	56	105
600		33	48	62	74	87	62	51	95
700		31	44	58	69	81	58	47	88
800		29	42	54	64	76	54	44	83
900		27	39	51	61	71	51	42	78
1000		26	37	48	58	68	48	40	74
1100		25	35	46	55	64	46	38	70
1200		23	34	44	52	62	44	36	67
1300		23	33	42	50	59	42	35	65
1400		22	31	41	49	57	41	33	62
1500		21	30	39	47	55	39	32	60
1600		20	29	38	45	53	38	31	58
1700		20	28	37	44	52	37	30	56
1800		19	28	36	43	50	36	29	55
1900		19	27	35	42	49	35	29	53
2000		18	26	34	41	48	34	28	52

All values below bold line indicate failure in bending.

Values are based on NDS 2001 for S.Y.P. #2

Safe Spacing of Supports

Safe Spacing (ℓ) in inches of supports for double wales single span.

$\Delta max = \ell / 360$, but not to exceed 1/4"

E = 1,600,000 psi

Safe Spacing of Supports - Simple Span Double-Ply Wales

Uniform Load (lbs per lineal ft)	Nominal Size of S4S Lumber	2x4	2x6	2x8	2x10	2x12	3x6	4x4	4x8
	Fb (psi) =	1500	1250	1200	1050	975	1250	1500	1200
100		76	111	137	164	190	126	98	169
200		60	94	115	138	160	106	80	142
300		53	83	104	125	145	96	70	129
400		48	75	97	116	135	89	64	120
500		44	67	86	103	121	83	59	113
600		42	61	79	94	111	78	55	108
700		39	56	73	87	102	73	53	104
800		37	53	68	82	96	68	50	101
900		34	50	64	77	90	64	48	98
1000		33	47	61	73	86	61	47	93
1100		31	45	58	70	82	58	45	89
1200		30	43	56	67	78	56	44	85
1300		29	41	53	6	75	53	43	82
1400		28	40	51	62	72	51	42	79
1500		27	38	50	59	70	50	41	76
1600		26	3	48	58	68	48	40	74
1700		25	36	47	56	65	47	38	72
1800		24	35	45	54	64	45	37	70
1900		24	34	44	53	62	44	36	68
2000		23	33	43	51	60	43	35	66

Safe Spacing (ℓ) in inches of supports for double wales continuous over three or more supports.

$\Delta max = \ell / 360$, but not to exceed 1/4"
E = 1,600,000 psi

Safe Spacing of Supports - Multi-Span Double-Ply Wales

Uniform Load (lbs per lineal ft)	Nominal Size of S4S Lumber	2x4	2x6	2x8	2x10	2x12	3x6	4x4	4x8
	Fb (psi) =	1500	1250	1200	1050	975	1250	1500	1200
100		93	130	161	193	223	148	115	198
200		75	110	135	162	188	425	97	167
300		65	97	122	146	170	113	86	151
400		58	84	108	129	152	105	79	140
500		52	75	97	116	136	97	73	133
600		47	68	88	105	124	88	69	127
700		44	63	82	98	114	82	65	122
800		41	59	76	91	107	76	62	117
900		39	56	72	86	101	72	59	110
1000		37	53	68	82	96	68	56	105
1100		35	50	65	78	91	65	54	100
1200		33	48	62	74	87	62	51	95
1300		32	46	60	71	84	60	49	92
1400		31	45	58	69	81	58	47	88
1500		30	43	56	67	78	56	46	85
1600		29	42	54	64	76	54	44	83
1700		28	40	52	62	72	52	43	80
1800		27	39	51	61	71	51	42	78
1900		26	38	49	59	69	49	41	76
2000		26	37	48	58	68	48	40	74

All values below bold line indicate failure in bending.

Values are based on NDS 2001 for S.Y.P. #2

APPENDIX 2

ALUMINUM BEAMS (ALUMA)



Aluma Strongback Load Chart
Imperial*

Span (ft)	Allowable Deflection L/360 (in)	1 Span (lbs/ft)		2 Span (lbs/ft)		3 Span (lbs/ft)	
4.00	0.13	10006 M		7603 R		8640 R	
4.50	0.15	7906 M		6758 R		7680 R	
5.00	0.17	6404 M		6082 R		6912 R	
5.50	0.18	5101 D		5292 M		6283 R	
6.00	0.20	3929 D		4447 M		5559 M	
6.50	0.22	3090 D		3789 M		4737 M	
7.00	0.23	2474 D		3267 M		4084 M	
7.50	0.25	2012 D	*2012	2846 M		3558 M	
8.00	0.27	1657 D	*1554	2502 M		3127 M	*2932
8.50	0.28	1382 D	*1219	2216 M		2608 D	*2301
9.00	0.30	1164 D	*970	1977 M		2197 D	*1831
9.50	0.32	990 D	*781	1774 M		1868 D	*1475
10.00	0.33	849 D	*636	1601 M	*1533	1601 D	*1201
10.50	0.35	733 D	*524	1452 M	*1261	1383 D	*988
11.00	0.37	638 D	*435	1323 M	*1047	1203 D	*820
11.50	0.38	558 D	*364	1211 M	*877	1053 D	*687
12.00	0.40	491 D	*307	1112 M	*739	927 D	*579

Aluma Beam Load Chart
Imperial*

Span (ft)	Allowable Deflection L/360 (in)	1 Span (lbs/ft)		2 Span (lbs/ft)		3 Span (lbs/ft)	
4.00	0.13	3151 M**		2471 R		2808 R	
4.50	0.15	2490 M		2196 R		2496 R	
5.00	0.17	2017 M		1977 R		2246 R	
5.50	0.18	1537 D		1797 R		2042 R	
6.00	0.20	1184 D		1402 M		1753 M	
6.50	0.22	931 D		1193 M		1728 R	
7.00	0.23	745 D		1030 M		1288 M	
7.50	0.25	606 D	*606	896 M		1144 D	*1144
8.00	0.27	499 D	*468	788 M		942 D	*883
8.50	0.28	416 D	*367	676 M		786 D	*693
9.00	0.30	351 D	*292	622 M		662 D	*551
9.50	0.32	298 D	*235	558 M		563 D	*444
10.00	0.33	256 D	*192	509 M	*462	482 D	*362



This information is proprietary to Aluma Systems and is subject to change. It is intended to be used by technically skilled designers knowledgeable in the field and is to be used with other data.

References:

1) ACI 318-19 Building Code Requirements for Structural Concrete and Commentary, ACI , Farmington Hills, Michigan, USA.

2) ASCE, 2022, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-22, American Society of Civil Engineers, Structural Engineering Institute, Reston, Virginia.

3) BS 5975:2008+A1:2011, Code of practice for temporary works procedures and the permissible stress design of falsework

4) Falsework — Prevention of Collapse, This Guidance Notes is prepared by the Occupational Safety and Health Branch Labour Department, 1998

5) Control and Management of Temporary Works: A guide to good practice, Published by HK Temporary Works Forum (HK-TWf), 2018

Design and Construction of Falsework - Quiz

Updated: 1/1/2025

1. Falsework consists of temporary structures used in construction to support a permanent structure until its construction is sufficiently advanced to support itself. Is this statement correct?

   a. Yes

   b. No

2. The failure of any falsework system will have serious consequences and it is therefore essential we have confidence in the technical advice given by _____,

   a. Structural engineer

   b. Mechanical engineer

   c. Owner

   d. supplier

3. Who is responsible of charge for design, construction and maintenance of falsework?

   a. contractor
   b. owner.
   c. consultant
   d. mechanical engineer .

4. The falsework review procedures are common in the construction industry, they are:_____.

   a. in-house reviews only

   b. owner/engineer reviews only

   c. in-house reviews and owner/engineer reviews

   d. None of the above

5. _____ develop written procedures that are followed during the falsework design process, prior to submitting it to the owner.

   a. Consultant
   b. Mechanical engineers
   c. Some contractors
   d. Professional design engineers

6. The lower risk falsework would be used for _____.

   a. The complicated systems
   b. the most easier systems
   c. High rise buildings
   d. Tunnels

7. The _____ risk falsework would be used for projects spanning sensitive waters, busy highways, or in very tall systems

   a. higher
   b. medium
   c. lower
   d. none of the above

8. Falsework loads, even though originating from material weight and gravity, are _____ compared to hydrostatic pressure

   a. fairly light
   b. fairly heavy
   c. fairly dense
   d. medium weight

9. Many falsework loads, except for large public works and transportation projects, are in the range of _____.

   a.200–500 psf.

   b.200-400 psf.

   c.200-300 psf.

   d.100-500 psf.

10. Concrete Core Forming System allows the core walls and decks to be formed simultaneously, thereby reducing the floor-to-floor schedule by at least _____, when compared to casting the walls and decks concurrently.

    a. 2 days

    b. 5 days

    c. 4 days

    d. 3 days

11. A box-girder bridge is a very common bridge design used by various county, state, and federal agencies and is predominantly_____.

    a. reinforced concrete

    b. masonry

    c. timber

    d. steel

12. _____ must consider debris netting within their calculations

    a. The falsework designer

    b. Owner

    c. Consultant

    d. Mechanical engineer

13. _____ should clearly indicate all additional works that are necessary to maintain the stability of unloaded falsework / slender towers in typhoon winds.

    a. Construction drawings
    b. Contract
    c. Bids
    d. None of the above

14. When slender falsework towers are _____ for stability, the designer should consider the stability of the whole system at every stage of construction

    a. Fixed together
    b. Hinged together
    c. Rolled together
    d. Close together

15. _____ loading towers to adjacent scaffold systems unless they have been designed to support the additional lateral loads.

    a. Connect
    b. Do not connect
    c. Fixed
    d. Tie up with ropes

16. _____ using similar scaffold systems and components within falsework structures / projects

    a. Avoid
    b. must be
    c. Should be
    d. None of the above

17. _____ or similar scaffolding systems can vary significantly, and it may not be possible to distinguish between systems or components when erected within a falsework structure

    a. The toughness
    b. The cost
    c. The load capacity
    d. The weight and density

18. Whenever practicable, top and bottom jack extensions should be _____ to minimize any requirement for jack bracing.

    a. < 200mm
    b. < 400mm
    c. < 100mm
    d. < 300mm

19. _____ permissible jack extensions and jack bracing details should be noted on construction drawings.

    a. Maximum
    b. Minimum
    c. Average
    d. Absolute value of

20. Calculations for unbraced jacks _____ include eccentric and lateral loads

    a. should
    b. should not
    c. Is not necessary but prefer
    d. none of the above