Formwork Design

By Richard C. Ringwald[1]

Abstract: The American Concrete Institute's methodology for designing formwork adequate for its loading is condensed, organized, and simplified into a two-page, step-by-step format of equations and instructions that cover most formwork design situations. Plotted design curves are shown and their limitations discussed.

Introduction

The Office of Safety and Health Administration (OSHA) and the magnitude of court awards to the injured have compelled some contractors to reconsider continuing with informal concrete formwork design methods. The need has developed to determine by some rational engineering approach that a form system is both safe and economical. The American Concrete Institute's Committee 347 addressed this problem and developed standards for formal design contained in the "ACI Formwork Standard" enlarged upon and explained in "Formwork for Concrete" written and edited by M. K. Hurd (1). The ACI method is the only method referred to in design specifications created by manufacturers of patented form systems. Pragmatically speaking, the contractor who adopts the methodology of this book and can prove his design was thus based, can defend himself against criticism by OSHA representatives. Yet, the method is state of the art and will probably turn out to be more than sufficiently conservative for most practitioners in the future.

This article discusses only wood formwork design. The writer does not disparage other types of formwork but is merely recognizing that wood formwork in whole or part is, and probably always will be, used in many formwork systems and the need exists for its engineered design. Also, design of many metal and wood-metal composite systems is based on concepts similar to those used for wood formwork.

Basics

There are certain assumptions implicit in the use of the ACI method and, if the built-in conservatism is to hold true, the following must be kept in mind by the ethical contractor:

1. The weight of concrete and reinforcement is 150 pcf. A small increase in this (<10% perhaps) will not seriously impact the safety of the method. The design pressure equations can be easily modified for heavier concrete.
2. The number of reuses of panelized units is not excessive. Ob-

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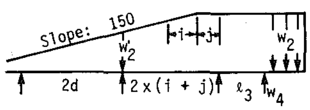
Note.—Discussion open until May 1, 1986. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on September 11, 1985. This paper is part of the *Journal of Construction Engineering and Management*, Vol. 111, No. 4, December, 1985. ©ASCE, ISSN 0733-9364/85/0004-0391/$01.00. Paper No. 20192.

J. Constr. Eng. Manage., 1985, 111(4): 391-403

FORMWORK DESIGN FORMAT — GIVEN → FIND

STEP 1	DESIGN PRESSURES: p in psf or w in p.lf	
		R < 7'/hr: $p = 50 + 9{,}000\ R/t$; MAX: 2,000 or 150 H_1 (LESSER) R = 7'-10'/hr: $p = 150 + 43{,}000/t + 2{,}800\ R/t$; DITTO MAX. R > 10'/hr: $p = 150\ H_1$
	SLAB	Plain: $p = (y/12) \times 150 + 50$ for L. L. (+25 using motorized carts) One way or Waffle: [5-11] or calc. per sf horizontal projection
	LAT'L LOADS	Lateral w at top edge slab: [5-6] [TEXT TABLE NO.] Lateral w at top edge of wall: [5-7]
	COL'M	$p = 150 + 9{,}000\ R/t$; MAX: 3,000 or 150 H_1 (LESSER)
Steps 2,3,4,:		Formulae for $\geq$ 4 supports (for < 4, see "Columns", fig. 2). Pick least ℓ & adjust for convenience or least section.
STEP 2	$w_2 = p \times 1$: *Plyform Th ↔ Stud, Joist, Stiffener* ℓ_2 *(in.)*	
	Th → ℓ_2	$\ell_B = \sqrt{120\ fs/w_2}$ — ℓ_2 → Th — $S = w_2\ell_2^2/(120\ f)$
		$\ell_s = 20v\ (Ib/Q) \div w_2$ <If $w_2 > 1360$ ALSO CHECK> $Ib/Q = 0.05\ w_2\ell_2'/v$
		$\ell_D^* = 1.69\ \sqrt[3]{EI/w_2}$ <IF $w_2 < 300$ ALSO CHECK> $I^* = w_2\ell_2^3/(4.83E)$
	BEARING CHECK:	$w_2\ell_2/(144\ b_3)$ < LEAST C$\perp$ (UNNEEDED IF C$\perp$ > 76)
STEP 3	$w_3 = w_2\ell_2/12$: STUD, JOIST b_3h_3 ↔ WALE, STRINGER OR YOKE ℓ_3	
	$b_3h_3 \rightarrow \ell_3$	$\ell_b = \sqrt{120\ fs/w_3}$ — $\ell_3 \rightarrow b_3h_3$ — $S = w_3\ell_3^2/(120f)$
		$\ell_s = 13.3hb_3h_3/w_3 + 2h_3$ — SHEAR CHECK: $(13.3hb_3h_3/w_3 + 2h_3) > \ell_3$
		$\ell_D^* = 1.69\ \sqrt[3]{EI/w_3}$ *<ALSO CHECK IF* $w_3 < 1100$*>* $I^* = w_3\ell_3^3/(4.83E)$
	POSITIONING WALES & YOKES IN THE P-ENVELOPE	If $j < (\ell_3/2)$; $i = 0.08\ (w_2 - \sqrt{w_2^2 - 150\ w_4})$ $d = 0.08\ (w_2' - \sqrt{(w_2')^2 - 150\ w_4})$ Quick: $\ell_3' = \ell_3 \times w_2/w_2'$
	BEARING CHECK:	$w_2\ell_3/(12b_3b_4)$ < C$\perp$
STEP 4	$w_4 = w_2\ell_3/12$: WALE, STRINGER b_4h_4 ↔ TIE, SHORE ℓ_4	
	$b_4h_4 \rightarrow \ell_4$	$\ell_b = \sqrt{120\ fS/w}$ — $\ell_4 \rightarrow b_4h_4$ — $S = w_4\ell_4^2/(120f)$
		$\ell_s = 13.3hb_4h_4/w_4 + 2hy$ — SHEAR CHECK: $13.3hb_4h_4/w_4 + 2h_4 > \ell_4$
	NOW, FOR T, ROD TIE CAP"Y OR B, RQD SHORE CAP"Y:	T OR B = $w_4\ell_4/12$ lbs
	BEARING:	WALE-TIE: T/HDW. AREA < C$\perp$: STRINGER -SHORE: $B/(b_4b_5)$ < C$\perp$



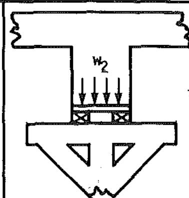
*DEFLECTION BASIS: ΔMAX = ℓ/360; FOR ΔMAX = 1/8": $\ell_D = \sqrt[4]{217\ EI/w_3}$

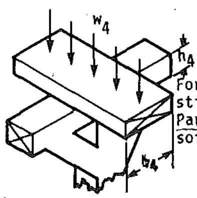
FIG. 1.—Formwork Design Format, Steps 1–4

viously, each reuse is an exposure to failure, and each reuse increases the probability that the condition of plyform plus studs, joists, and stiffeners are such that panel integrity is lost. There is no way that this requirement can be quantified to cover every situation. The appropriate number of reuses must remain a judgment call which every constructor must be prepared to defend based on the care that the form handlers exert in dismantling and erecting formwork.

Figs. 1 and 2 comprise a format containing design steps condensed from the Hurd text (1). Steps 1–4 deal with the design of members not subject to stiffness problems. Step 5, dealing with design of shores and

FORMWORK DESIGN FORMAT

<table>
<tr><td rowspan="4">STEP 5</td><td>From B (step 4) and shore L(ft.), Find shore b_5h_5:</td><td>From $W_5 = (W_{1at} \times H_1) \div (H_2 \cos A)$ and brace L (ft.), find brace b_5h_5 and ℓ_5;</td></tr>
<tr><td colspan="2">Note: L is <u>unsupported length</u> of shore or brace</td></tr>
<tr><td colspan="2">$b_5 = 12L/50$ rounded up to standard.
Calculate. $12L/b_5$ and $K = 0.671\ E/C$
check, if $L < 3'$ for $12L/b_5 < 11$ calc. $c' = c_{\parallel}$ or   check, if $L < 6'$ for $12L/b_5 > 11 < K$: $c' = c_{\parallel}\,[1-1/3(\frac{12L}{b_5K})^4]$ or   most used: $12L/b_5 > K$: $c' = 0.3E/(12L/b_5)^2$</td></tr>
<tr><td>Calc. $h = B/C'b_5$
$h_5 = h$ rounded up to greater of next std. or b_5
sill bearing: $B/b_5h_5 < C_{\perp}$</td><td>Try $h_5 = b_5$
Then $\ell_5 = (C'b_5h_5/W_5) \times 12$
If ℓ_5 is too small for convenience, raise h_5 as needed, or set ℓ_5 and solve for h_5.
Bearing: $W_5\ell_5/(12 \times \text{contact area}) < C_{\perp}$</td></tr>
<tr><td>BEAMS</td><td colspan="2">

(No Stringers)
Include beam wgt. in W_2, beam side FW, <u>if</u> formal design RQD, designed like <u>wall</u> FW. Soffit FW: use step 2 for plyform with W_2 in PLF <u>transverse</u> to soffit. W/O stringers, step 3 omitted. In this case, $W_4 = W_2 \times$ soffit width. Step 4 for shore spacing vs. multiple stiffeners. (S and I are additive.)


For lumber soffit sheathing W/O stiffeners, use step 4, w_4 in PLF <u>Parallel</u> to soffit, b_4 and h_4 are for soffit board.
(No Joists <u>or</u> Stringers)</td></tr>
<tr><td>COLUMNS</td><td colspan="2">Sheathing on < 4 supports: $\ell_B = \sqrt{96fS/w_2}$ or $S = w_2\ell_B^2/96f$
Steps 2, 3: (Δ = 1/16"): $\ell_D = 3.23\sqrt[4]{EI/w_2}$ or $I = w\ell_3^4/109E$
Other than above changes, design as per steps 1, 2, 3 (no step 4).
Note: All col. stiffener ℓ's are considered clear. Thus, for calculating w_3, ℓ_2 must be converted to on-center values.</td></tr>
</table>

FIG. 2.—Step 5 of Formwork Design Format and Special Instructions on Beams and Columns

braces subject to stiffness limitations, appears in Fig. 2 along with special instructions on using the format to design formwork for beams and columns.

Design Format

The following step-by-step explanation of the format uses the Hurd text symbols with some necessary changes. Refer to Appendix III for complete notation.

The SI unit equations corresponding to those in the format are shown in Appendix I. They were not shown immediately following their equivalents as is the custom, for fear that doing so would have cluttered to the point of confusion and defeated the desire to present all procedures in two pages.

Evaluation of Loadings (Step 1).—As shown in Fig. 1, Step 1 provides for the calculation of design loads to be used in other design steps. Concrete density of 150 pcf is assumed in this step. Wall and column equations involving R take into account that if the concrete is not poured too fast, part of the concrete will get an initial set after which its fluid pressure on the forms is reduced. These equations and the maximums given for each (except for 150 H_1) are largely empirical in origin.

The slab dead load calculation is obvious and the 50 psf live load normally added is based on a fairly conservative estimate of likely field situations where only hand-pushed concrete carts are used.

The lateral loads, obtained from tables in Ref. 1, are based on wind pressures. In the case of very small wind loads, a minimum design load is provided by these tables.

Plyform Thickness versus Support Spacing (Step 2).—Step 2 equations allow the designer to assume a plywood thickness, then find a spacing or vice versa. Since it is convenient to design for a strip of plyform 12 in. wide, $w_2 = p \times 1$. If, as in many cases, w_2 is between 300 plf and 1,360 plf only the bending equation need be used, since said pressures represent the maximum and minimum governing values for deflection and shear, respectively, as determined by solving $l_D = l_B$ and $l_S = l_B$ for w_2. Note, however, that these limits apply only to 5/8 in. and larger plyforms, and assume face grain parallel to span ("strongway").

The shear formula is based on v, the allowable stress in what the American Plywood Association APA calls rolling shear in the plane of the plies. It should be noted that the l_S value calculated from the formula is for the clear distance between supports and not the on-centers spacing resulting from most other equations in the format. No attempt will be made here to show the APA's derivation of the equation (suffice it to say that table values of Ib/Q for various values of plyform thickness are available from both the APA and Hurd's book).

Readers will note that in the $l_2 \rightarrow Th$ mode, equations for I and S are the equivalent of the l_D and l_B equations to their left. Also, it should be recognized that in this mode, it is necessary to look at tables of S and I values for various b and h figures then select the most advantageous b, h.

Bearing is not a problem for a supporting member against standard #2 plyform (for which $C\perp = 210$ psi). The proof of the $C\perp < 76$ limitation (not given in Hurd's book) is derived from a worst case scenario where a 2 in. nominal board (b = 1-1/2 in.) bears against plyform carrying the maximum p-value ($= w_2$) of 3,000 psf. For this w_2, l_2 (based on shear) would have to be 4 in. clear + 1.5 in. = 5.5 in. on-centers. Thus, $C\perp = w_2 l_2/144 b_3 = (3{,}000 \times 5.5)/(144 \times 1.5) = 76$. Therefore, only a wood species with a very low $C\perp$ would provide a bearing problem for Step 2.

Selecting Studs, Joists, Wales and Stringers (Steps 3 and 4).—Steps 3 and 4 are similar except that Step 3 involves spacing in what is called the pressure envelope and Step 4 ignores deflection which is not a concern for wales or stringers.

The equations used in Steps 3 and 4 are the same as for Step 2 except for the shear equation. With lumber, shear resistance parallel to grain (horizontal shear) is less than perpendicular to grain and, thus, controls.

Remember that if shear governs in Step 2, it will be necessary to convert l_S (clear) into l_2, an on-center length, using an assumed value for b before calculating w_3.

For most wood species, the horizon where deflection stops governing and bending starts is from 150 plf to 500 plf. Thus for many loadings, the deflection step may be omitted. These horizons are obtained from $l_D = l_B$ for the various woods.

For walls and columns, the plastic concrete (considered fluid) pressure increases from the top of the formwork down, from zero to a maximum, p, as defined in Step 1. The distance from the top of the pour to that point, in feet, is $p/150$. This reflects the increase in concrete pressure per square foot by 150 lbs for each foot. Plotting these pressures against distance and rotating 90° produces what is called the pressure envelope, as shown in Step 3. In the sloping portion of the pressure envelope, wales (or yokes in the instance of column forms) can be spaced farther apart than the calculated l_3 which applies in the lower part of the envelopes. The method for determining spacing involves deciding what length into the upper envelope from the next lower wale/yoke is required to generate $w_4/2$, the half load to be borne by that wale/yoke on its left. This figure is then doubled to arrive at the spacing to the next member. The expressions for these spacings, solved by the quadratic

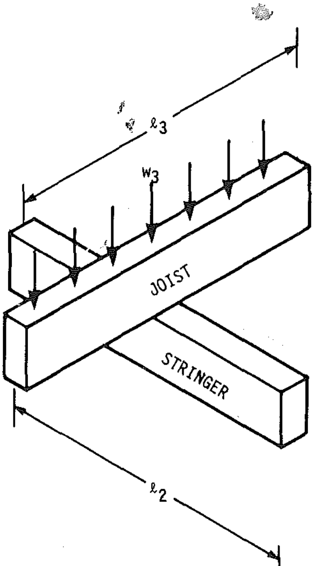


FIG. 3.—w_4 Loading on Shore or Tie

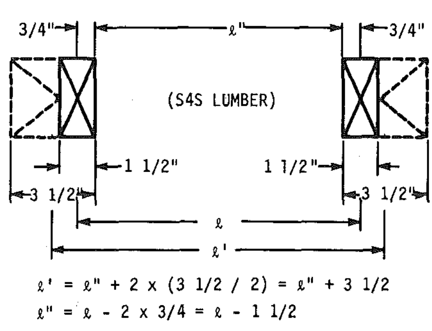


FIG. 4.—4-in. Nominal versus 2-in. Nominal Anomaly

formula, produce the equations for i and d. Since the right half-load for an upper wale/yoke using this technique is $<w_4/2$, the method is conservative. This is not precisely the method used by the Hurd book but it is equivalent.

An even more conservative, but faster, means of calculating these upper spacings is by using the plf load directly over the lower wale/yoke (w_2') and determining $l_3' = l_3 \times w_2/w_2'$.

The reader may wonder why the pressure envelope does not show a plot of w_3 over these lengths since that figure represents the actual load restrained by wales and yokes. The area over each wale/yoke represents the total load per foot over the wale/yoke, which would not be shown in a w_3 plot. Thus, analysis is more easily performed.

The expression for w_4 is probably worth an explanation. One assumption of the Hurd book (and the American Forest Product Association) is that all member loads, even if virtually point-applied, are considered to be resolved into uniform loads for design purposes.

Thus as seen in Fig. 3, the total load at the small area of contact between the joist and stringer shown is $w_3 l_3/12$. As Hurd points out, however, if spacing of point loads exceeds 1/3 to 1/2 of the span between supports, fiber stress and deflection should be checked for the worst loading condition.

This, however, is resolved into a new uniform loading, w_4, over the distance $l_2/12$ (ft) so that

$$w_4 = \frac{\dfrac{w_3 l_3}{12}}{\dfrac{l_2}{12}} = \frac{\left(\dfrac{w_2 l_2}{12}\right)\dfrac{l_3}{12}}{\dfrac{l_2}{12}} = \frac{w_2 l_3}{12}$$

The formulas for tie and shore capacity are self evident.

The Step 4 bearing check determines that the wood will not fail in bearing if the total tie load, T, is divided over a large enough tie hardware contact area.

Design for Other Deflecting Constraints.—Under certain circumstances, some practitioners prefer deflection to be maximized at 1/8 in. rather than $l/360$. The equation for l_D to use in Steps 2 and 3 under this preference is footnoted under Step 4. A formula for maximum $\Delta = 1/16$ in. is also available and is usually the preferred expression in column sheathing design. See the column notes in Fig. 2.

Spacing Anomaly.—A special anomalous situation exists regarding spacing of a 2-in. nominal versus a 4-in. nominal under a supported member. In determining any value of l for benefit of the supported member, the Hurd text assumes that all support is point support, a conservative assumption. Thus, l_2 is independent of b_3 and l_3 of b_4. However, it is observed that in reality if a 2-in. support is satisfactory under a supported member at spacing l (O.C.) then a 4-in. support at spacing $l + 2$ in. (O.C.) will give the same clear span (see proof in Fig. 4). This might be useful if, for example, a practitioner prefers to use nine 4 × 4-in. nominal studs (eight spaces at 12 in. O.C.) along an 8-ft plyform sheet rather than a greater number of 2 × 4-in. nominal at 10 in. (or 8 in.). No saving in board-feet is thus achieved, but there is a saving in carpentry labor footage.

Shores and Braces (Step 5).—The equation for w_5 translates the plf of lateral force acting in effect at the top at the formwork (elevation H_1) to the required reactive force at the elevation H_2 at the upper end of the brace acting through an angle A with the horizontal.

For either a brace or shore, buckling is as likely to govern the design as compressive strength. Thus, the slenderness ratio of length ($12L$) to smallest cross-section dimension b, is required to be less than 50. The stiffness factor, K, also impacts the design in its relationship to the slenderness ratio. Except for perhaps very short wall braces carrying lateral loads to a nearby bank, the slenderness ratio will be less than K. For that reason, the right-most equation, independent of $c_{\parallel}$, usually dictates the design c'.

With c' and b_5 determined, it is then possible to determine h_5 such that: (1) The total load of B or $w_5 l_5$ divided by the cross-sectional area $b_5 \times h_5$ is $<c'$; and (2) $h_5 \geqq b_5$ (in order that the slenderness ratio remain <50). Bearing should be checked.

For most species, the allowable lumber tensile stress is less than $c_{\parallel}$. Thus, it might seem that a wall form brace, which can be in tension, ought to be governed thereby. "Formwork for Concrete" states that if designed in accordance with Step 5, tension will not be a problem for braces (1).

For the cases where the slenderness ratio is less than 11, the c' value becomes sufficiently lower than $c_{\parallel}$ that it becomes less than the allowable tensile stress. In those unusual instances where $c' = c_{\parallel}$, e.g., in which $L = 3$ ft and $b = 3.5$ in. (4 × 4-in. nominal) no comfortable theoretical answer could be found. However, it should be noted that this full tension will exist only for the time before the pour that a wall form is erected on one side, only, unconnected to the other side by a spreader board, with wind direction normal to the braced side. During this usually brief period, the factor of safety built into the allowable stresses is sufficient to handle this rather abnormal tension event.

Beam and Column Formwork.—Beam soffit support under plyform sheathing is usually two or three stiffeners laid flat which are in turn held up by T-head shores. Step 2 provides for plyform Th and stiffener spacing (which will often be governed by soffit width rather than stress limitations). Since there are no stringers needed to carry the load to the shores, Step 3 is omitted and in this case $w_4 = w_2 \times$ soffit width. Step 4 gives shore spacing and B value.

In the event of lumber soffit sheathing, without stiffeners, the correct method to use is Step 4, with w_3 determined as the load per foot of soffit length.

Column forms are designed like wall forms except that stiffeners (usually nailed wide side against sheathing) have l_2's that are, less conservatively, considered clear. This is allowed because the ratio of stiffener contact space to clear space is quite large compared to the stud spacing on walls. For example, a two by four nominal on 12-in. spacing for a wall have a contact/clear ratio of $1.5/10.5 = 0.14$. 4 × 2-in. nominal column stiffeners on a 5-in. clear spacing have a contact/clear ratio of 3.5/

5 = 0.7. (For determining w_3, clear space must be changed to on-center value.)

Also, it is often true with columns (which are narrow compared to wall lengths) that only two or three stiffeners are used which requires use of a different l_B formula.

Computer Solutions

Two former formwork students, John Hayes and William Stotts, used the preceding format to develop working computer programs which are now utilized in formwork courses taught in the Construction Engineering curriculum at Iowa State. One program, which is extremely user-friendly, asks the user to name the species, stipulate whether design is for wall or slab, insert w_2, and then the program very quickly outputs sizes and spacings with a minimum of operator decision making. It is, however, confined to about four or five species. The other program, less user-friendly, is extremely flexible in that the operator specifies values for f, E, H, etc., for any species and the program then outputs size-spacing combinations. Using either method, design time is in minutes, not hours.

Design Curves

The second shortcut attempt, involving a set of curves developed from the writer's format, allows an engineer to quickly read size-spacing combinations starting from w_2 alone and with minimal calculations. This method is shown in Figs. 5–7. Because it assumes stress-graded construction-grade Douglas Fir-Southern Pine species with somewhat low f, c, $c_{\|}$, H, and E values, it is on the conservative side.

However, conservatively designed forms may last longer, i.e., have more uses before it is necessary to scrap them; therefore, they may achieve a payoff even though initially costing more. Also, using Figs. 5 and 6 as a pattern, an engineer could develop his own design curves using the format and design stress values appropriate for lumber species available in his working area. Thus equipped, he could design quickly (under bidding pressures, for example).

The method of plotting Fig. 6 is self evident. Horizon loading pressures (indicated by small circles) were obtained by solving, e.g., $l_D = l_B$ for w.

There is a wide band of loadings for which l_B governs. Also, l_2 is calculated from w_2; w_3 from w_2 and l_2; l_3 from w_3; w_4 from l_3; and l_4 from w_4. Thus, it might seem that a set of l_3 and l_4 curves could be plotted against w_2 so that only one chart would be needed. The reason the writer did not attempt such a construction is because designers usually adjust a theoretical value to a more convenient value (13.1 in. to 12 in., for example), rendering such a plot inaccurate.

Paul Sommers of Algernon Blair, Inc., has made an interesting set of interacting plots based on the assumption of exact l values being used unadjusted in the field (2,3). His graph of R versus p for various values of t could be incorporated with the Step 2 curves suggested here.

The stairstep Fig. 5 plot may be preferred by designers who make it

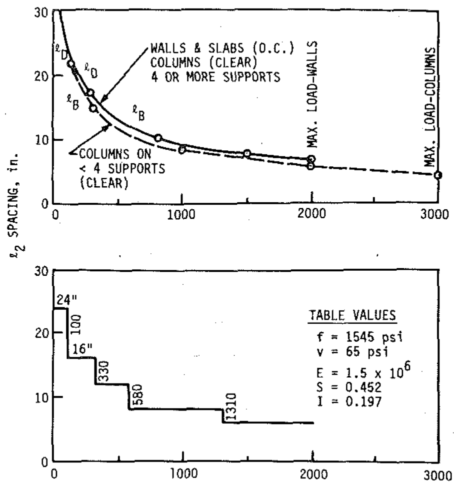


FIG. 5.—Step 2 Curves for 3/4-in. Plyform

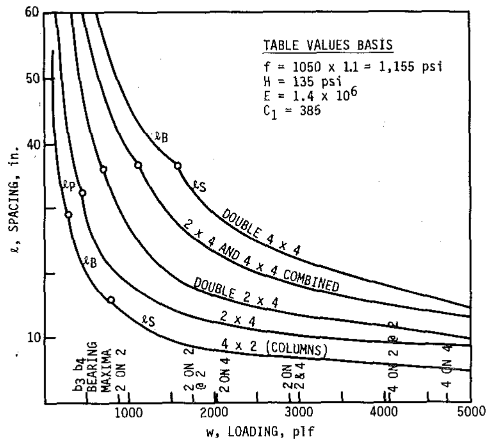


FIG. 6.—Step 3 and Step 4 Curves. Lumber Sizes are for Supported Members. I **and** S **are Figured Conventionally for S4S Lumber**

a practice to divide studs uniformly along the 96 in. length of a plyform sheet. Where this is an absolute practice, it would be possible to superimpose a curve for w_3 with a right-hand vertical axis for w_3. This curve has been omitted from Fig. 5 to avoid cluttering.

Fig. 6 illustrates spacing versus loading per foot for various lumber sizes.

Brace Design Using Curves.—Fig. 7 illustrates a few Step 5 lateral loading situations for various brace unsupported lengths. Additional coverage is available by extrapolation. From $12L/b = 50$, the absolute maximum unbraced length of a 2 × 4-in. nominal is 6.25 ft and a 4 ×

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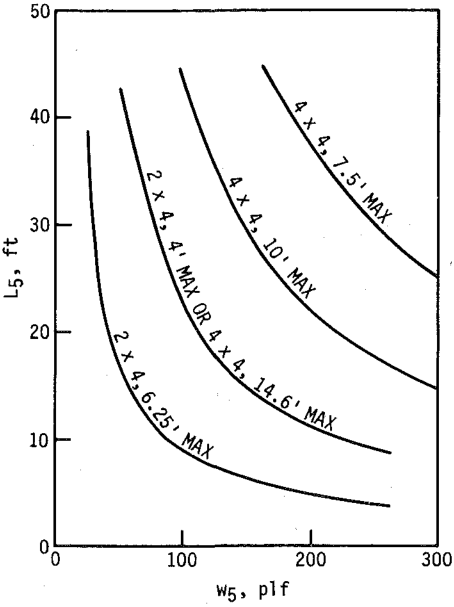
4-in. nominal is 14.6 ft. Three other brace lengths were selected to give a reasonable range. It was found that the $0.3E/(12L/b_5)^2$ equation for c' governed throughout the range (in the case of the 7.5-ft long 4 × 4-in. nominal, however, there is a horizon with the $c_{\parallel}[1 - 1/3(12L/b5K)^4]$ expression).

Curves were constructed by using $E = 1.4 \times 10^6$ and $c_{\parallel} = 925$, from which $K = 0.671\sqrt{E/c_{\parallel}} = 26.1$ which is in all cases exceeded by $12L/b_5$. Then $c' = 0.3E/(12L/b_5)^2$. From L_5 (ft) $= l_5/12 = (c'b_5h_5/w_5) \times 12/12 = c'b_5h_5/w_5$ for which the trial b_5h_5 values become plottable L_5 versus w_5 curves.

The Fig. 7 curves do not apply to shore calculations. Step 5 for shores involves determining only shore size (b_5h_5) from B, required shore capacity, and L, unsupported shore length; there is no w_5 in shore equations. Actually, if the investigation is confined to the universally popular 2 × 4 nominal and 4 × 4 nominal, a simple procedure can be arrived at that modifies Steps 4 and 5.



ALL CURVES REPRESENT UPPER LIMITS ONLY, E.G. A 6.25' 2 x 4 BRACE AT 4' SPACING IS OK FOR W_5 = 100.

BASIS: $E = 1.4 \times 10^6$

$$c' = \frac{0.3E}{(12L_{MAX} \div b_5)^2}$$

$$L_5 = \frac{c'b_5h_5}{W_5}$$

FIG. 7.—Step 5 Curves: Lateral Bracing

As observed above, the absolute maximum for L is 6.25 ft for a 2 × 4 nominal and 14.6 ft for a 4 × 4 nominal. These produce $c' = 0.3E/(12L/b_5)^2 = 0.3 \times 1.4 \times 10^6/50^2 = 168$ psi. Then max $B = c'b_5h_5 = 168 \times 3.5 \times 3.5 = 2{,}060$ lbs (the 2 × 4 L maximum of 6.25 ft is too short for most shore situations). This investigation has shown that the following steps are necessary:

1. Limit l_4 such that $B < 2{,}060$ lbs.
2. Reduce unsupported length, L, by introducing intermediate bracing in order to increase c' and, hence, increase maximum B permitted.
3. Construct a larger section by combining a 4 × 4-in. nominal with a 2 × 4-in. nominal or another 4 × 4-in. nominal (care must be taken in the field to join these members as continuously as possible).

Thus, some practitioners may prefer using rules 1, 2, and 3 rather than parts of Step 4 and 5 to determine l_4, B and b_5, and h_5.

Conclusions

Formwork design can be accomplished from a rational engineering design approach. For most species and design pressures, this approach can be reduced to a convenient set of curves. The trained designer might even assign part of the design work to a paraengineer working under his supervision, reserving unusual situation designs for himself as well as the right to review all designs.

Acknowledgment

The author gratefully acknowledges the support of the Engineering Research Institute at Iowa State University, and its Office of Editorial Services in particular, for assistance in preparing this manuscript.

Appendix I.—Format (Figs. 1 and 2) Equations in SI Units

Equations listed below are in same order as found in the format. Equations differing in subscript number only are not repeated. Generic equations, applicable in both unit systems, are not repeated here.

Step 1

Walls: $R_{SI} < 2$ m/h: $p_{SI} = 7.2 + 7.85R_{SI}/(t_{SI} + 17.8)$

Walls: R_{SI} 2–3 m/h: $p_{SI} = 7.2 + 1{,}156/(t_{SI} + 17.8) + 244R_{SI}/(t_{SI} + 17.8)$

Maximi for above: 95.8 Pa or $23.5h_{SI}$

Walls: $R_{SI} > 3$ m/h: $p_{SI} = 23.5h_{SI}$

Slab: $p_{SI} = 238y_{SI} + 2{,}390$ (+1,145 for motorized carts)

Step 2

$$l_{B.SI} = \sqrt{1{,}000f_{SI}S_{SI}/w_{SI}}$$

$$l_{S.SI} = 167v_{SI}(I_{SI}b_{SI}/Q_{SI})/w_{SI}$$

$$l_{D.SI} = 3.43(E_{SI} I_{SI}/w_{SI})^{1/3}$$

Bearing Check: $w_{SI} l_{SI}/(10{,}000 b_{SI}) <$ least $C_{\perp}$

Step 3

$$w_{3SI} = 0.01 w_{2SI} l_{2SI}$$

$$l_{S.SI} = 111 H_{SI} b_{SI} h_{SI}/w_{SI} + 2h_{SI}$$

$$i_{SI} = 0.0457\,(w_{2SI} - \sqrt{w_{2SI}^2 + 43.76 j_{SI} w_{2SI} - 2{,}188 w_{4SI}})$$

$$d_{SI} = 0.0457\,(w'_{2SI} - \sqrt{(w'_{2SI})^2 - 2{,}188 w_{4SI}})$$

Bearing Check: $0.01 w_{2SI} l_{3SI}/(b_{3SI} b_{4SI})$

Step 4

$$w_{4SI} = 0.01 w_{2SI} l_{3SI}$$

T_{SI} or $B_{SI} = 0.01 w_{4SI} l_{4SI}$

Step 5

$$b_{5SI} = 100 L_{SI}/50$$

For $(100 L_{SI}/b_{5SI}) > K_{SI}$ where $K_{SI} = 0.671 E_{SI}/C_{\parallel SI}$:

$$c'_{SI} = 0.3 E_{SI}/(100 L_{SI}/b_{5SI})^2$$

$$l_{5SI} = 100\,(c'_{SI} b_{5SI} h_{5SI}/w_{5SI})$$

Bearing: $w_{5SI} l_{5SI}/(100 \times \text{contact area}) < C_{\perp SI}$

Columns

For less than 4 supports: $l_{BSI} = \sqrt{800 f_{SI} S_{SI}/w_{2SI}}$

Appendix II.—References

1. Hurd, M. K., *Formwork for Concrete*, 4th ed., American Concrete Institute, Detroit, MI.
2. Sommers, Paul H., "Charts Aid in Design of Horizontal Formwork," *Concrete Construction*, July, 1984, p. 648.
3. Sommers, Paul H., "Charts Simplify Design of Vertical Formwork," *Concrete Construction*, Apr., 1984, p. 392.

Appendix III.—Notation

The following symbols are used in this paper:

B = shore load or capacity, lb (N);
b = cross section width, in. (cm);
c = allowable compressive stress, psi (kPa);
c' = c, adjusted for stiffness, psi (kPa);
d = half spacing in upper pressure envelope, in. (cm);
E = modulus of elasticity, psi/in. (N/cm);
FW = formwork,
f = allowable bending stress, psi (kPa);

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H	=	allowable horizontal shear stress, psi (kPa);
H_1	=	height of pour, ft (in.);
h	=	cross section height, in. (cm);
I	=	moment of inertia, in.^{4} (cm^4);
K	=	stiffness factor,
L	=	brace or shore unsupported length, ft (m);
LL	=	abbreviation for live load,
l	=	member spacing o.c., in. (cm);
l'	=	clear member spacing, in. (cm);
p	=	design load, psf (Pa);
R	=	rate of rise of concrete in forms, fph (m/h);
S	=	section modulus, cu in. (cm^3);
T	=	load or capacity of a tie, lb (N);
Th	=	plyform thickness, in. (cm);
t	=	temperature of concrete in the forms, °F (°C);
v	=	plyform rolling shear allowable stress, psi (kPa);
w	=	design load on a member, plf (N/m);
y	=	slab thickness, in. (cm);
Δ	=	maximum allowable deflection, in. (cm);
[#]	=	reference table number in Hurd text, and
$\rightarrow$	=	given what precedes $\rightarrow$, find what follows.

Subscripts

2, 3, 4, and 5	=	step numbers for which a member's spacing or size is sought,
B	=	bending,
D	=	deflection,
S	=	shear,
$\perp$	=	perpendicular to grain,
$\parallel$	=	parallel to grain, and
SI	=	SI units.

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