Design of Steel Deck for Concentrated and Non-Uniform Loading

MARCH 21, 2018

Presented by:

Michael Martignetti, CANAM
Mike Antici, NUCOR

Polling Question

- New requirement to earn PDH credits
- Two questions will be asked during the duration of today's presentation
- The question will appear within the polling section of your GoToWebinar Control Panel to respond

Disclaimer

The information presented herein is designed to be used by licensed professional engineers and architects who are competent to make a professional assessment of its accuracy, suitability and applicability. The information presented herein has been developed by the Steel Joist Institute and is produced in accordance with recognized engineering principles. The SJI and its committees have made a concerted effort to present accurate, reliable, and useful information on the design of steel joists and Joist Girders. The presentation of the material contained herein is not intended as a representation or warranty on the part of the Steel Joist Institute. Any person making use of this information does so at one's own risk and assumes all liability arising from such use.

Learning Objectives

- Recognize load cases that require additional analysis beyond distribution as a uniform load
- Understand the limit states for design under concentrated loads
- Examine different load paths for varying concentrated load conditions
- Review current SDI design approach for concentrated loads
- Demonstrate potential shortcuts to concentrated load design
- Present example problems for design with concentrated loads

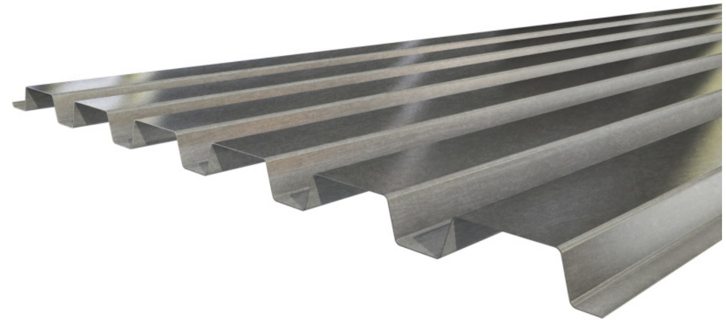
Presentation Outline

- ✓ Identify Typical Deck Types
- ✓ Introduction to Concentrated Loads Types
- ✓ Roof Deck Limit States and Design Example
- ✓ Floor Deck Limit States and Current Design Methodology
- ✓ Composite Deck Design Examples – Shortcuts for Multiple Loads
- ✓ Form Deck and Steel Fibers

Deck Types

Roof Deck

- Permanent Structural Member
- No Concrete Topping

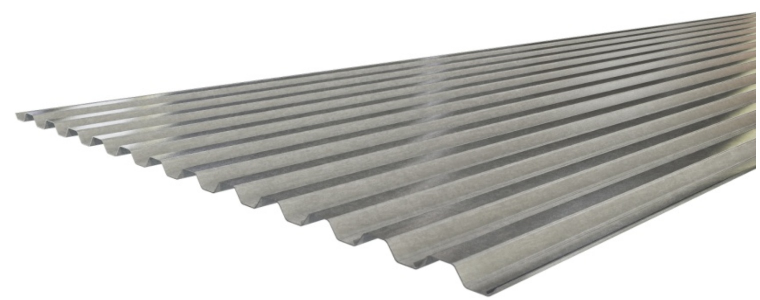


Composite Deck

- Deck and Concrete Work Together
- Embossments – Composite Action

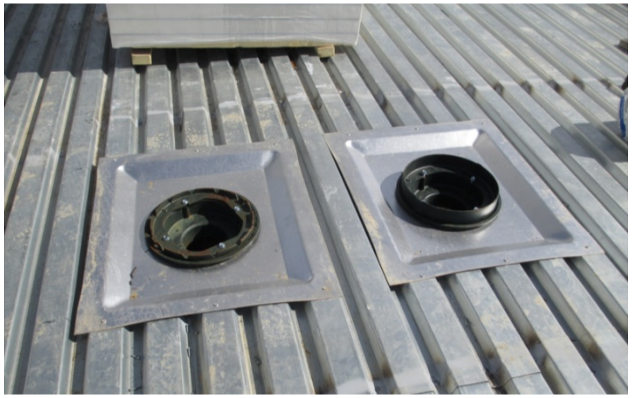
Form Deck

- Deck is Permanent Form
- Deck Often Carries Slab Weight

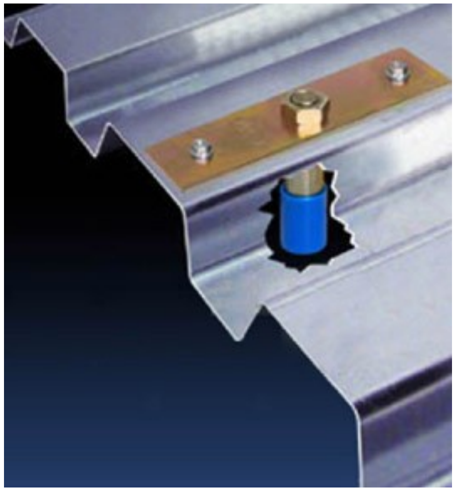


Concentrated Loads on Roof Deck

Safety Anchors



Roof Drains



Suspended Loads

Solar Panels

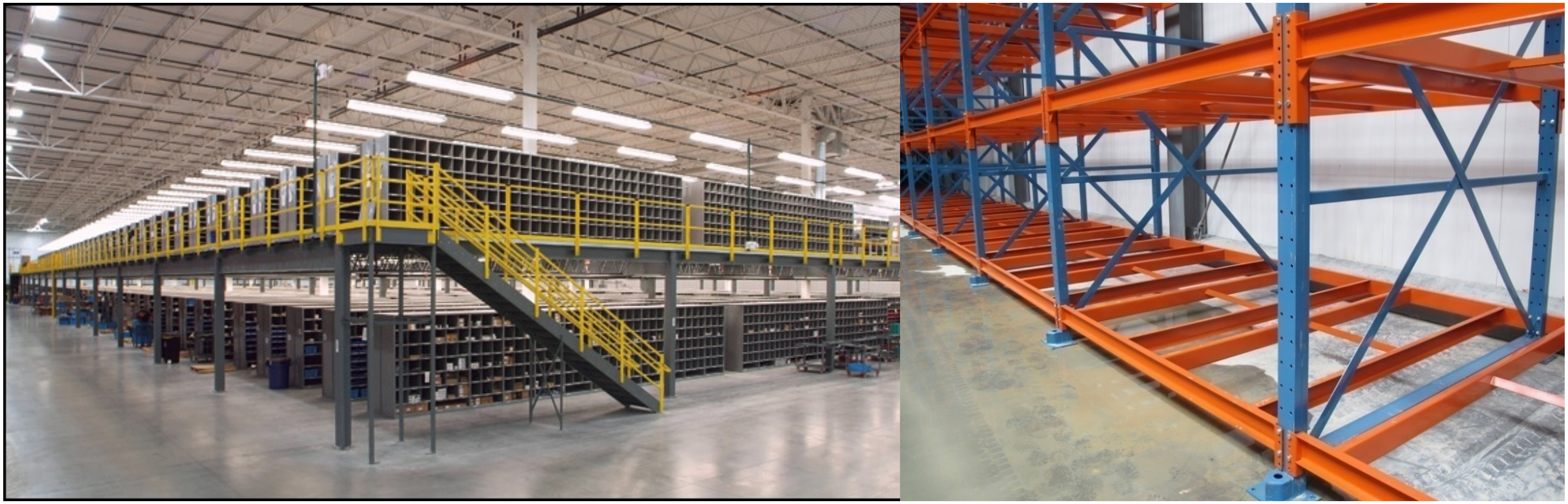
Concentrated Loads on Roof Deck

<u>Construction Loads</u>

- People
- Dollies
- Pallets
- Tool Chests
- Roofing Machinery

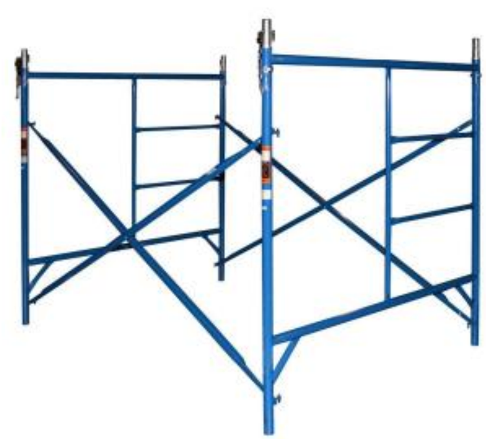
Concentrated Loads on Floor Deck

Storage Racks



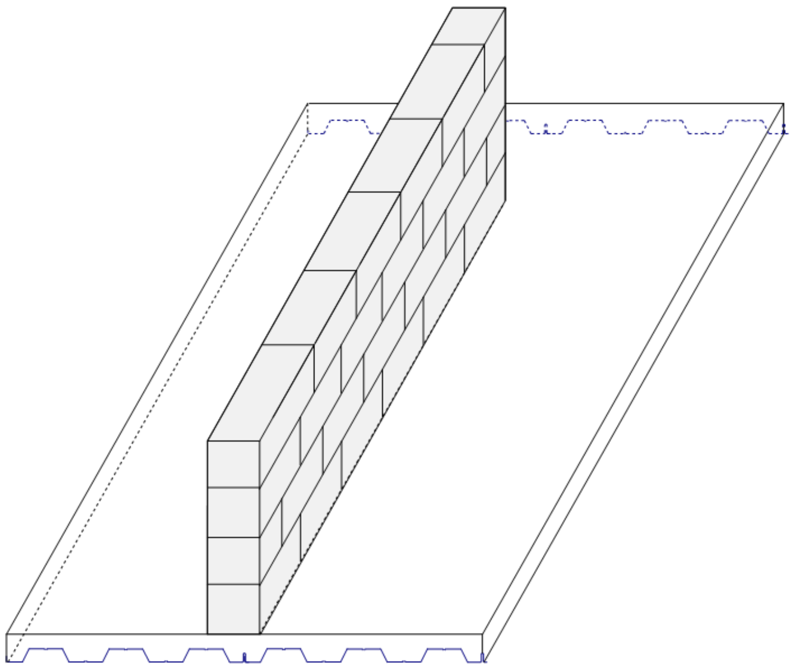
Concentrated Loads on Floor Deck

Equipment Loads

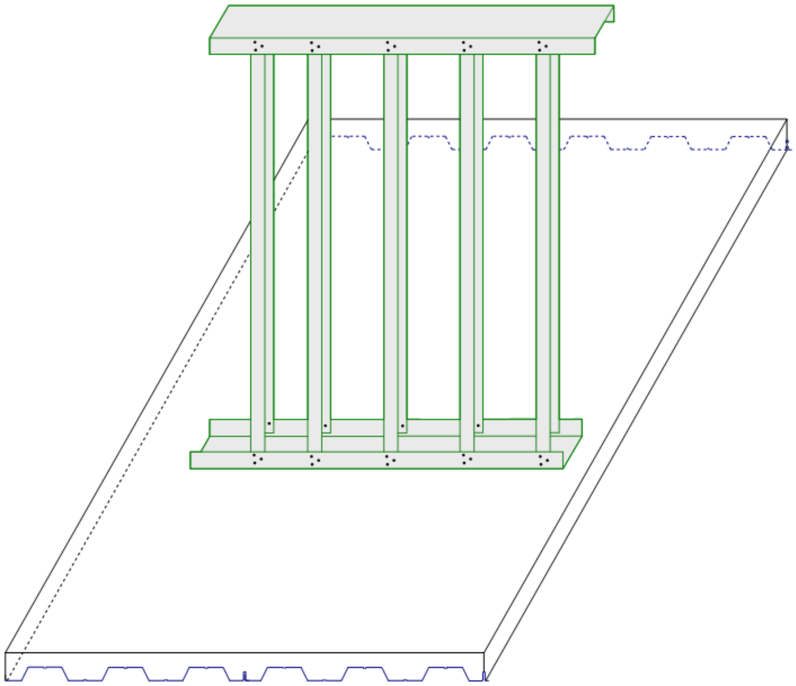


Concentrated Loads on Floor Deck

Wall Loads

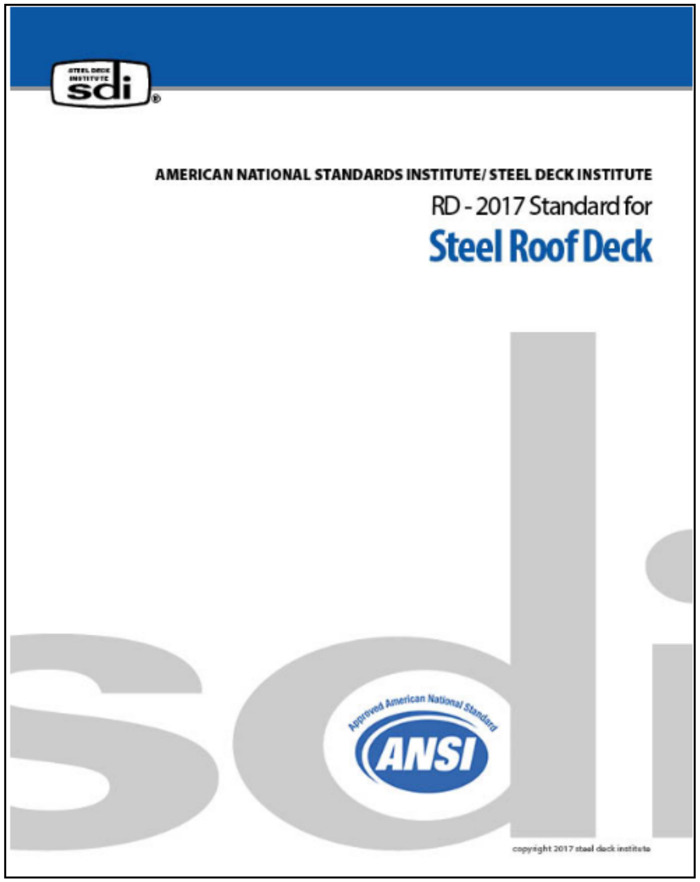


Parallel



Transverse

Roof Deck Design Standard/Manual

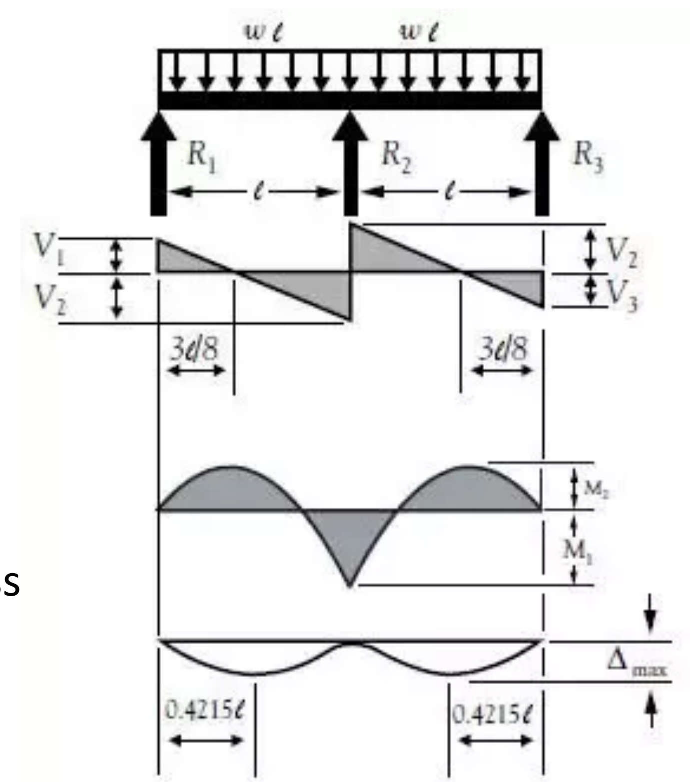


Available at www.sdi.org

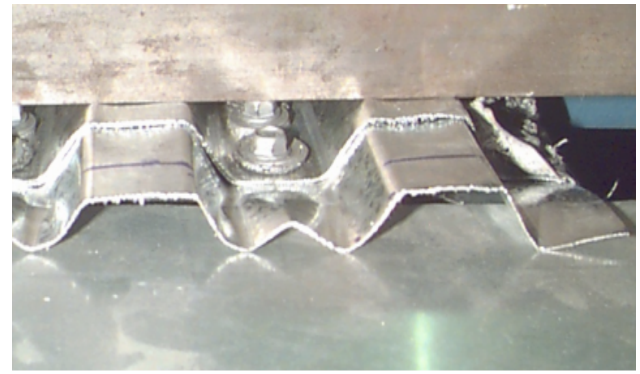
Roof Deck Design Limit States

Shear

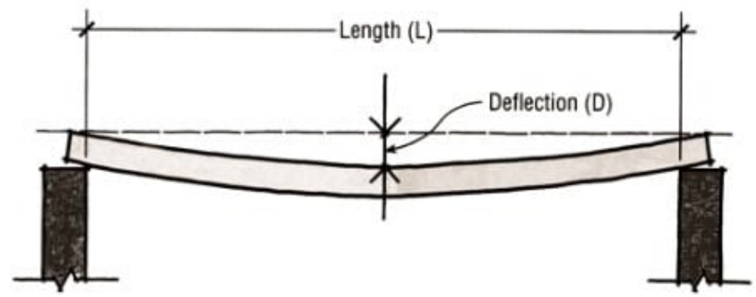
Stress



Bending/Shear Interaction



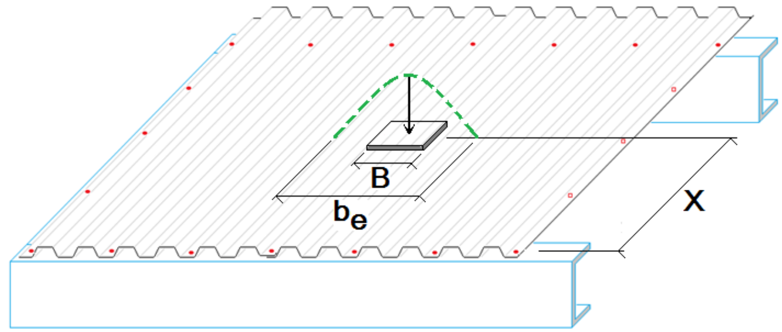
Web Crippling



Deflection

Roof Deck – Transverse Distribution

Based on 1 ½" Deck…



L = Span
X = % of Span

For $X \leq 0.25$ $\quad b_e = B + 6 > 12$

For $0.25 > X \geq 0.50$ $\quad b_e = B + 18 - \frac{3}{X} > 24 - \frac{3}{X}$

Where:

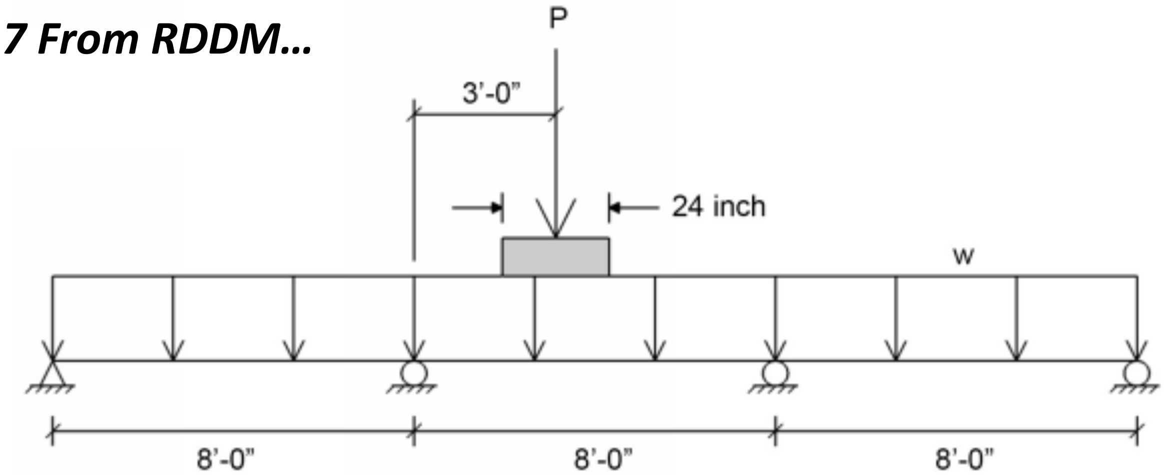
B = load footprint width transverse to the deck span. When the load centroid is not at the center of the footprint, let B equal twice the least dimension from the centroid to the baseplate edge; inches.

b_e = effective distribution width; inches

X = percentage of span, measured from the nearest support to the center of the concentrated load, ≤ 0.50

Roof Deck Design Example

Example 7 From RDDM…



<u>***Given:***</u> Select a WR deck to support the roof load condition below. Use an ASD solution. Combine loads using ASCE 7-10.

(1) Uniform Dead Load = 10 psf

(2) Uniform Live Load = 20 psf

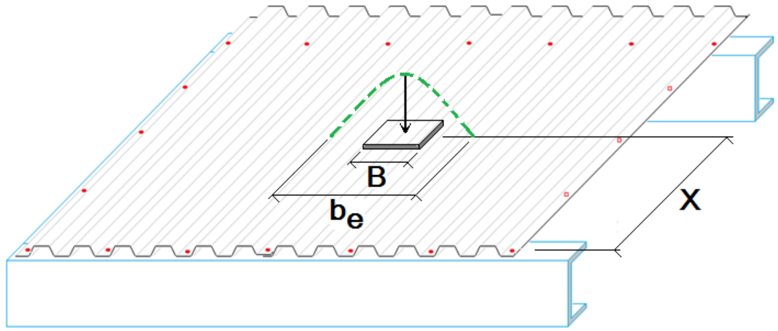
(3) Concentrated Dead Load = 700 lbs on baseplate

  (a) Baseplate size is 24 inches parallel to deck span and 30 inches perpendicular to deck span

  (b) Deck End Bearing Length = 1.5 inch

  (c) Deck Interior Bearing Length = 3 inch

Roof Deck Design Example



L = Span
X = % of Span

For $X \le 0.25$ $\quad b_e = B + 6 > 12$

For $0.25 > X \ge 0.50$ $\quad b_e = B + 18 - \dfrac{3}{X} > 24 - \dfrac{3}{X}$

Calculate the transverse distribution of the concentrated load using the procedure found in Section 2.5.

L = 8 ft $\quad$ XL = 3 ft $\quad$ X = 0.375

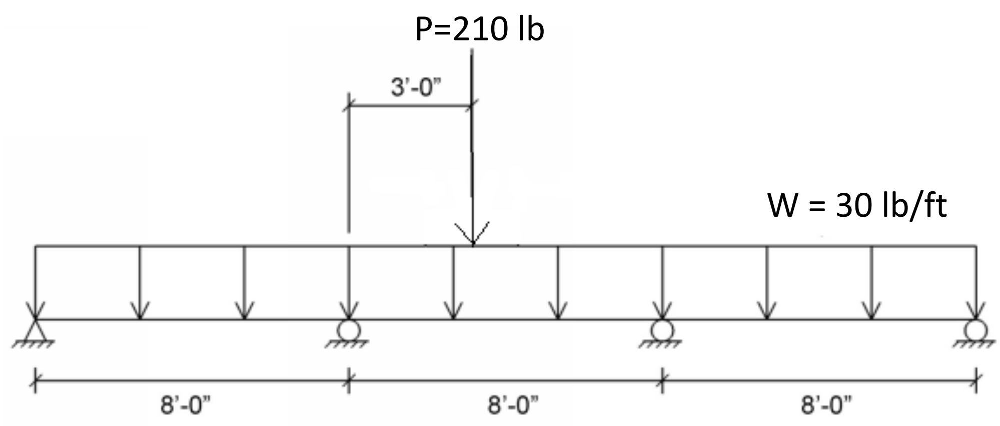
$$b_e = B + 18 - \frac{3}{X} \ge 24 - \frac{3}{X}$$

$$= 30 + 18 - \frac{3}{0.375} \ge 24 - \frac{3}{0.375}$$

= 40 inch ≥ 16 inch

Therefore the 40 inch dimension controls the transverse distribution.

Roof Deck Design Example



Concentrated Load is converted to a line load as 700 lbs × 12 / 40 = 210 plf.

From a structural analysis using w = 30 plf and P = 210 lbs, the maximum moments and shears are found in the middle span:

M_n = 3918 inch-lbs at the left support

M_p = 3632 inch-lbs under the concentrated load

V = 255 lbs at the left support

$R_{INTERIOR}$ = 416 lbs at the left support (OFI)

$R_{EXTERIOR}$ = 83 lbs at the right support of the 3rd span (OFE)

Roof Deck Design Example

Table 1 – Section Properties and Flexural Resistance

							ASD (Ω = 1.67)		LRFD (Φ = 0.90)	
Profile	Gage Number	Design Thickness (inches)	I_p ($inch^4$)	I_n ($inch^4$)	S_p ($inch^3$)	S_n ($inch^3$)	M_p/Ω inch-lbs	M_n/Ω inch-lbs	ΦM_p inch-lbs	ΦM_n inch-lbs
WR	22	0.0295	0.1473	0.1732	0.1713	0.1804	3385	3565	5088	5358
WR	20	0.0358	0.1910	0.2104	0.2122	0.2247	4193	4440	6302	6674
WR	18	0.0474	0.2741	0.2791	0.2883	0.2963	5697	5855	8563	8800
WR	16	0.0598	0.3528	0.3528	0.3695	0.3722	7301	7355	10974	11054

Table 6 – Shear and Web Crippling Strength

				Web Crippling							
		Shear (lbs)		ASD (lbs)				LRFD (lbs)			
Profile	Gage Number	ASD Ω = 1.60	LRFD Φ=0.95	Ω = 1.70 OFE	Ω = 1.75 OFI	Ω = 1.80 TFE	Ω = 1.75 TFI	Φ=0.90 OFE	Φ=0.85 OFI	Φ=0.85 TFE	Φ=0.85 TFI
NR, IR, WR	22	1325	2014	541	857	521	1057	828	1276	797	1573
NR, IR, WR	20	1588	2413	773	1248	797	1557	1183	1856	1220	2316
NR, IR, WR	18	2068	3144	1314	2171	1483	2747	2010	3229	2268	4086
NR, IR, WR	16	2523	3835	1981	3322	2374	4239	3030	4942	3632	6305
DR	22	2224	3380	366	737	321	859	559	1096	492	1278
DR	20	3123	4747	528	1067	504	1270	808	1588	771	1889
DR	18	4129	6276	910	1845	965	2247	1393	2745	1477	3342
DR	16	5115	7775	1385	2809	1574	3470	2119	4179	2409	5161

Roof Deck Design Example

Try WR20

For this condition,

$\frac{M_n}{\Omega}$ = 4440 inch-lbs (Table 1) > 3918 inch-lbs OK

$\frac{M_p}{\Omega}$ = 4193 inch-lbs (Table 1) > 3632 inch-lbs OK

V_{ALLOW} = 1588 lbs (Table 6) > 255 lbs OK

Allowable Web Crippling, (Table 6)

OFE = 773 lbs (1.5 inch min.) > 83 lbs OK

OFI = 1248 lbs (2.5 inch min.) > 416 lbs OK

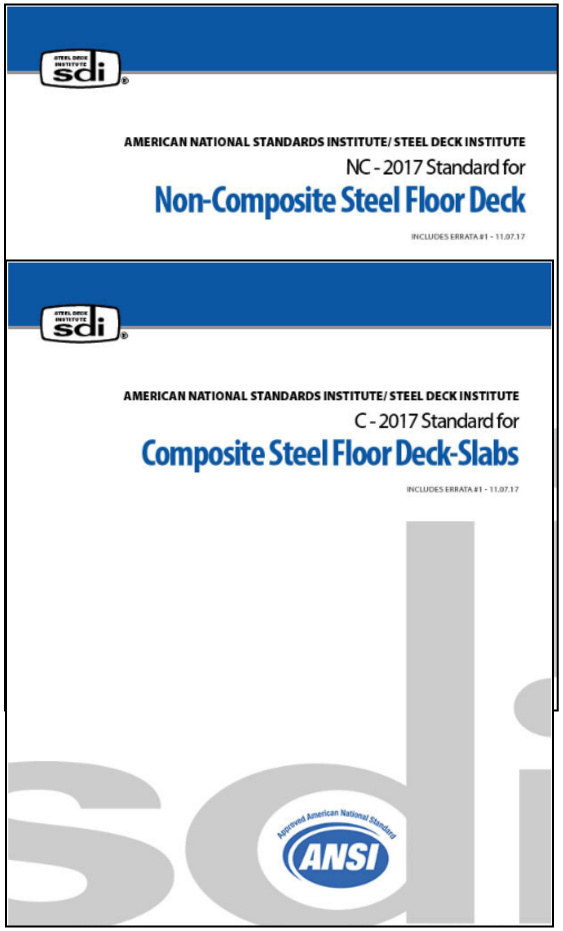
Therefore,

$$\sqrt{\left(\frac{V}{V_a}\right)^2 + \left(\frac{M}{M_a}\right)^2} = \sqrt{\left(\frac{255}{1588}\right)^2 + \left(\frac{3918}{4440}\right)^2} = 0.897 \leq 1.0 \text{ OK}$$

Result:

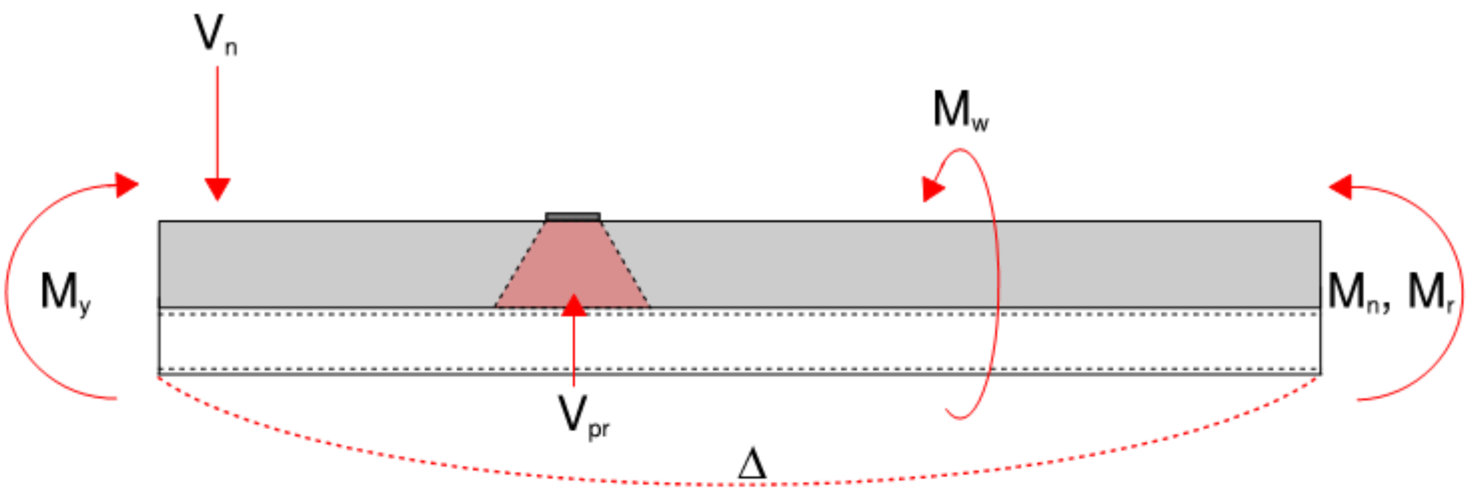
WR20 deck is acceptable for this condition.

Floor Deck Design Standards/Manual



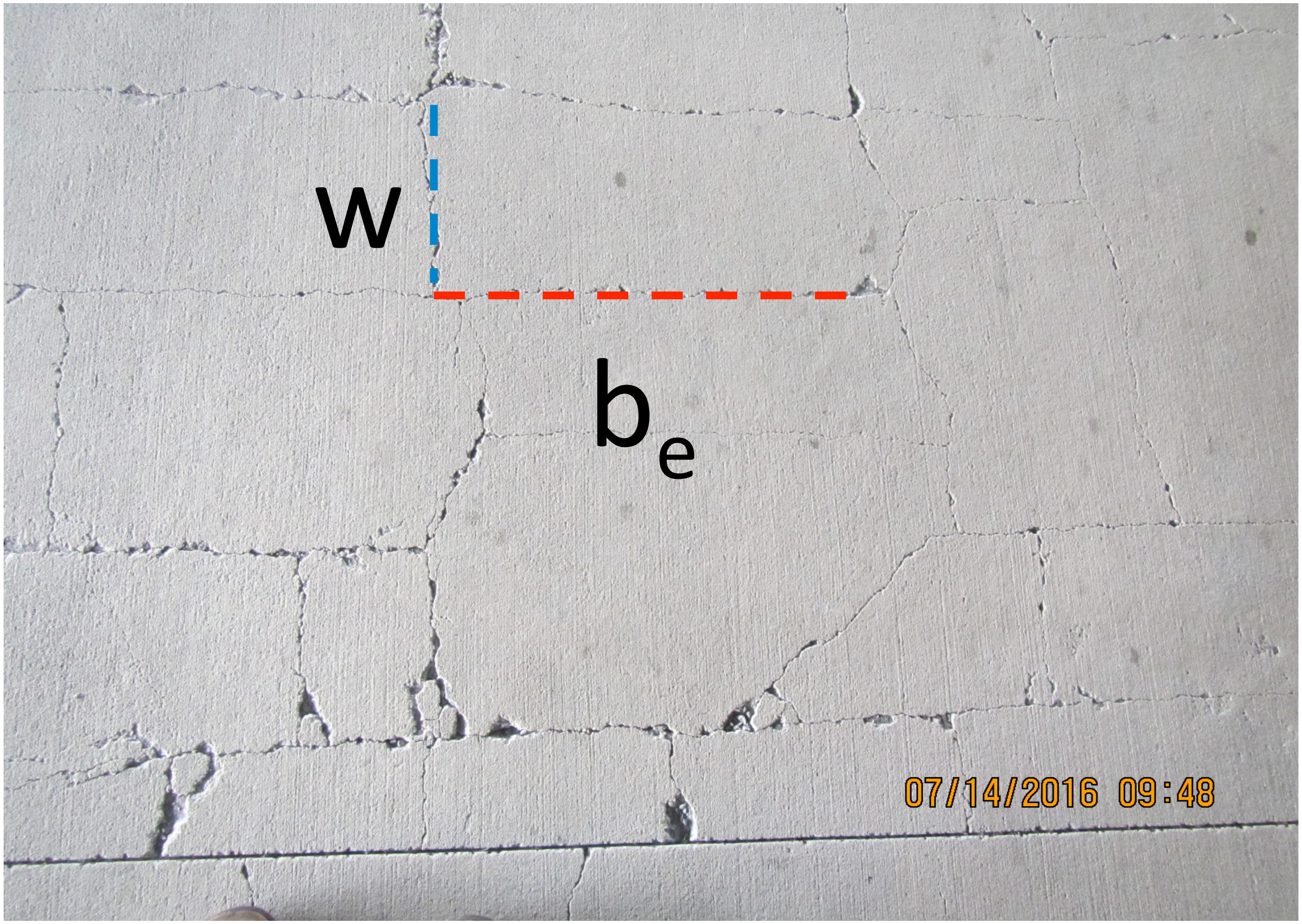
Available at www.sdi.org

Floor Deck Design Limit States



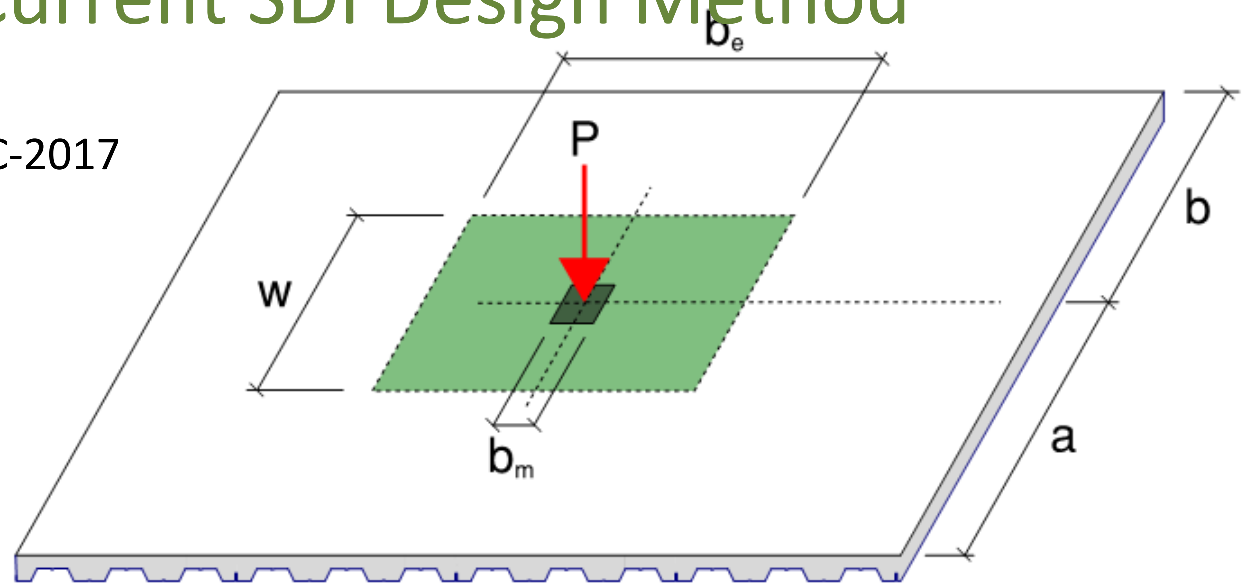
- ★ M_y Bending (+ if simple span, +/- if multiple span)
- ★ V_n One Way Beam Shear
- V_{pr} Punching Shear
- Δ Deflection
- ★★ M_w Transverse (Weak axis) Bending
- M_n, M_r Proprietary Deck-Slab Bending (no studs)

Load Distribution



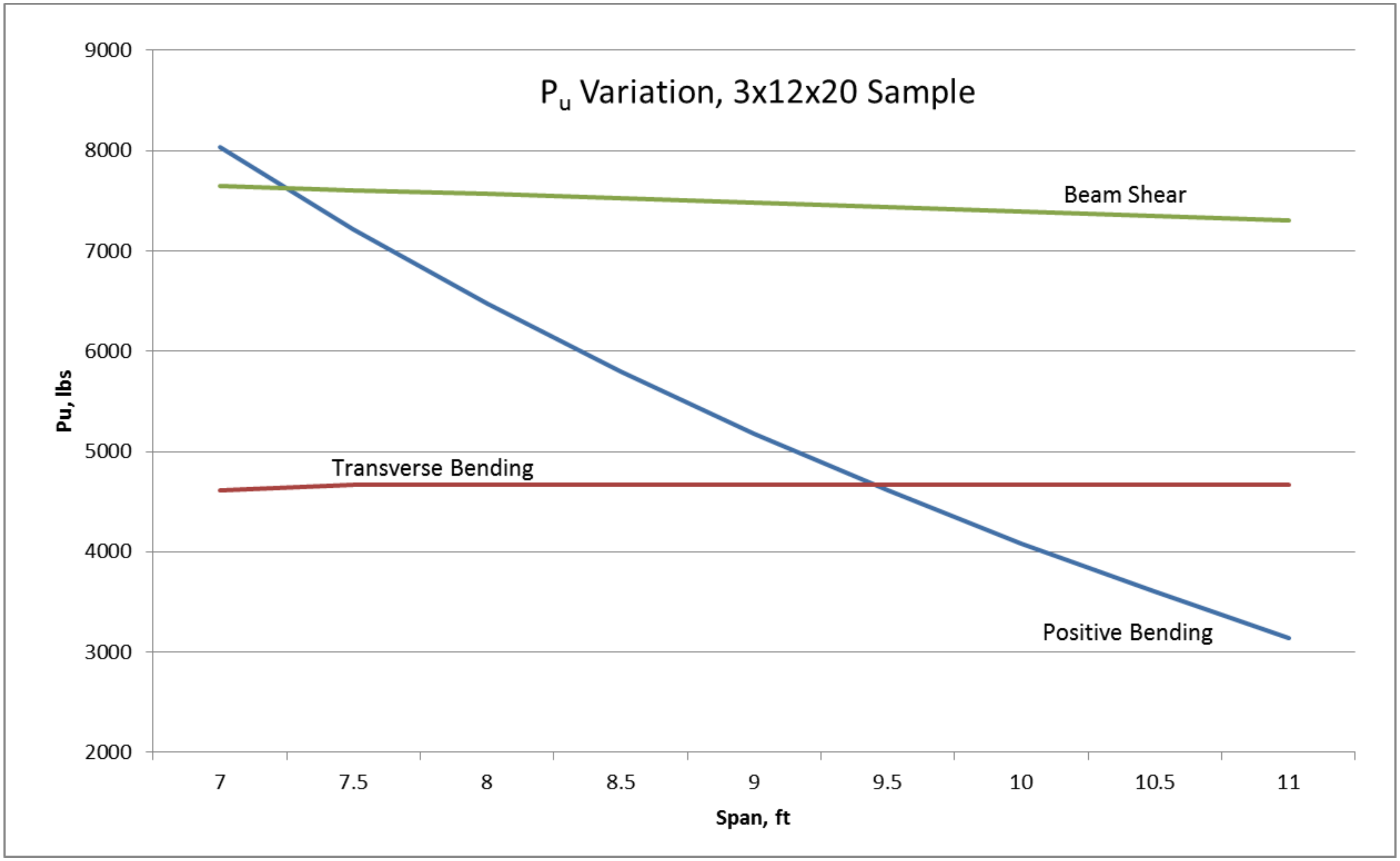
Current SDI Design Method

SDI CDDM/FDDM/C-2017



$b_m = b_2 + 2t_c + 2t_t$	Wheel/Baseplate Distribution	2.4.10
$b_e = b_m + 2\left(1 - \frac{x}{L}\right)x$	Single Span Positive Bending	2.4.11
$b_e = b_m + \frac{4}{3}\left(1 - \frac{x}{L}\right)x$	Continuous Span Positive Bending	2.4.12
$b_{ve} = b_m + \left(1 - \frac{h}{L}\right)x$	Beam Shear	2.4.13
$w = \frac{L}{2} + b_3 < L$	Transverse Bending	2.4.14

Limit States



Polling Question #1

Which Limit State is NOT Applicable for Designing Concentrated Loads on Concrete Slabs on FLOOR Deck?

a) Weak Axis Bending

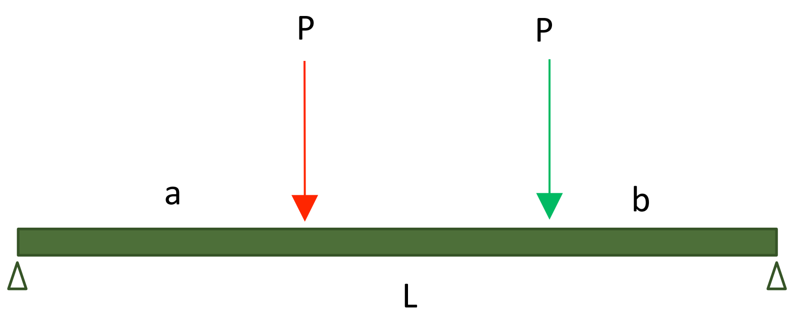
b) Web Crippling

c) Punching Shear

d) Positive Bending

e) Negative Bending

Can We Solve This Load Diagram?

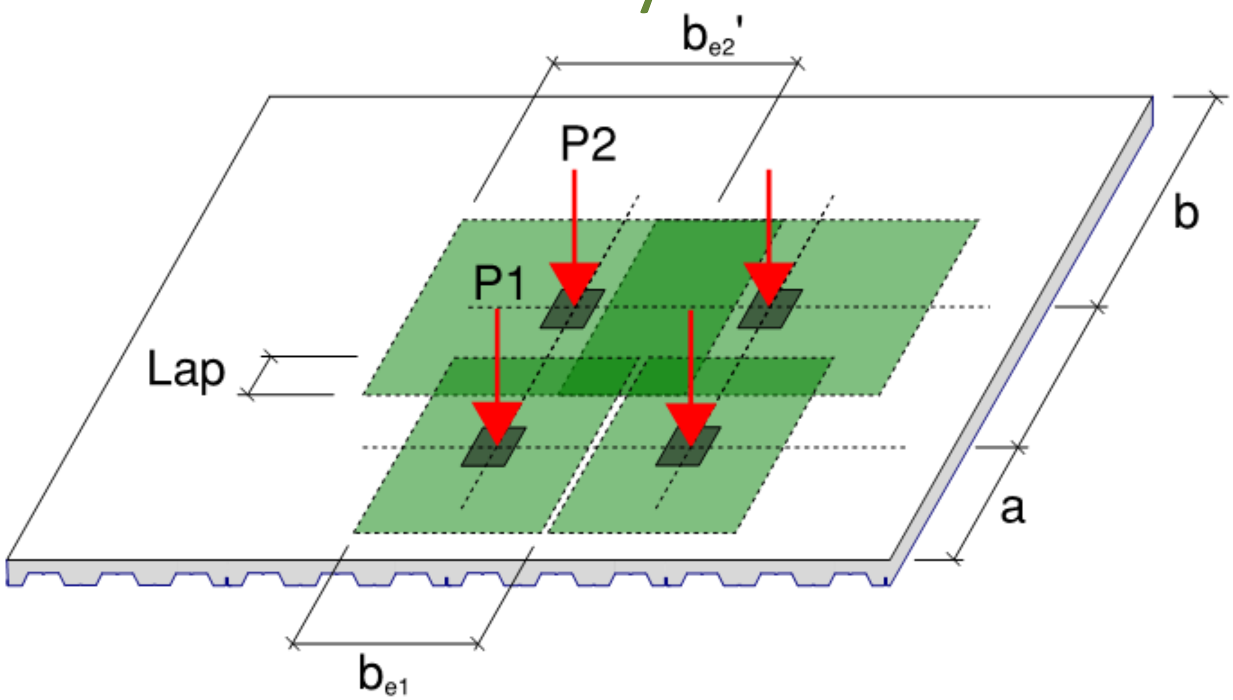


This webinar makes one assumption . . . . the webinee (that's you) can solve this simple beam for shear and bending. Additional limit states (deflection, punching) are defined in the standards, but unlikely to control. Shear and bending will be discussed in detail.

Problem solutions are shown, but intended as examples and guides for future reference. Please focus on the diagrams and techniques for load distribution, not the mathematical solution.

Shortcut Theory

NEW for this Presentation



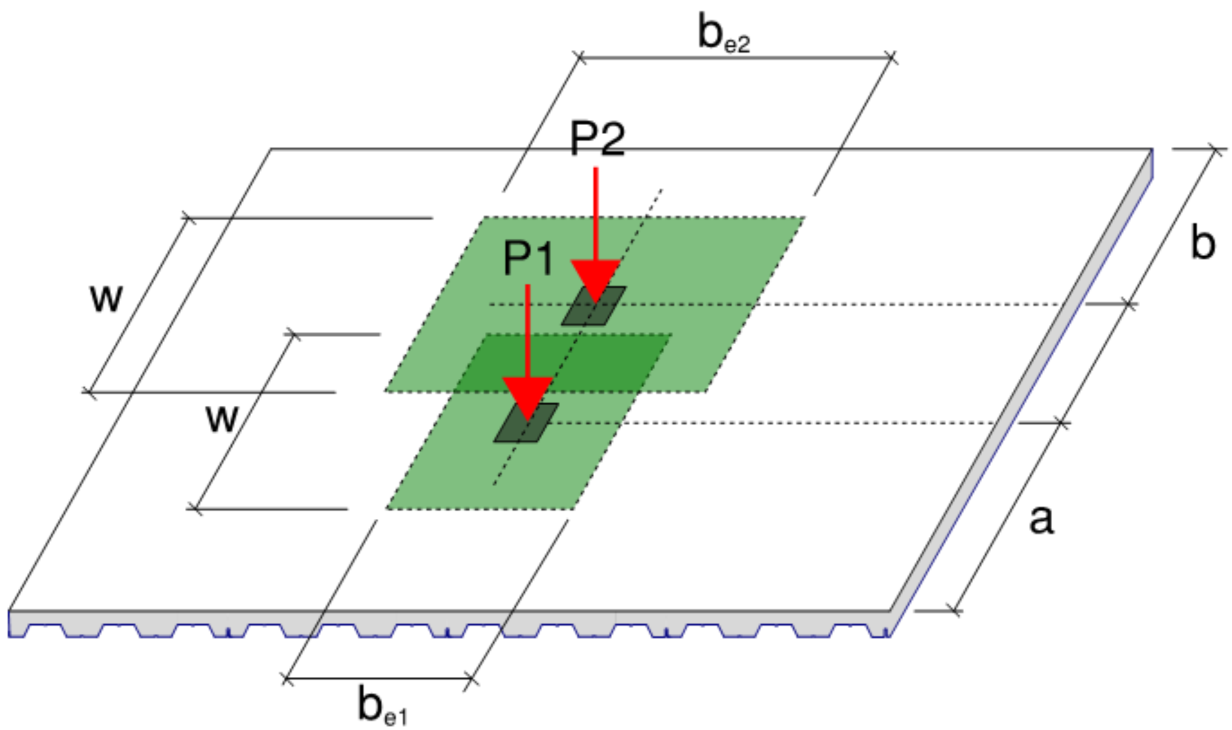
$b'_e = \dfrac{b_e + \text{Load spacing}}{2} < b_e$
$M_w = \left(\dfrac{P}{w} + \dfrac{P(Lap)}{w^2}\right)\dfrac{b_e}{15}$
$M_x = 5.5\ M_1 \left[\dfrac{x}{b_e} - \dfrac{1}{\pi}\sin\left(\dfrac{\pi x}{b_e}\right)\right]\ \text{rad}$

Shear and Positive Bending adjustment for "ADJACENT" loads.

Weak Axis Bending adjustment for "IN-LINE" loads.

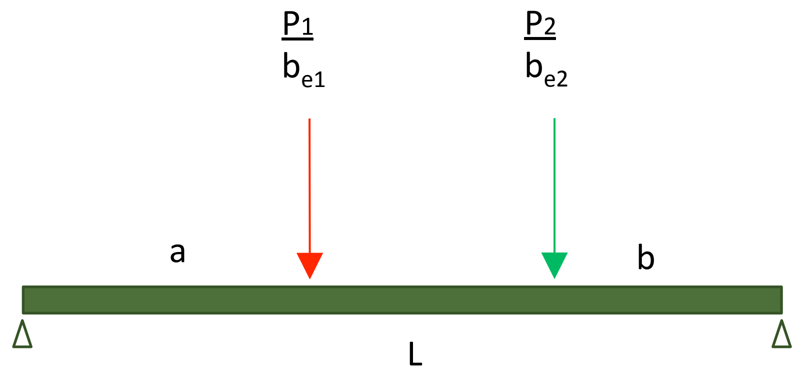
Weak Axis Bending moment envelope for "ADJACENT" loads.

2 Loads "In-Line"



Influence zones may (and usually do) overlap as illustrated. This suggests the stress in these areas is greater than the stress in non-lapped zones. The effective widths of these influence zones (b_{e1} and b_{e2}) change as loads P_1 and P_2 move along the span. In situations where load locations are fixed (storage racks, scaffolds), a simple beam diagram for shear and bending can easily be defined.

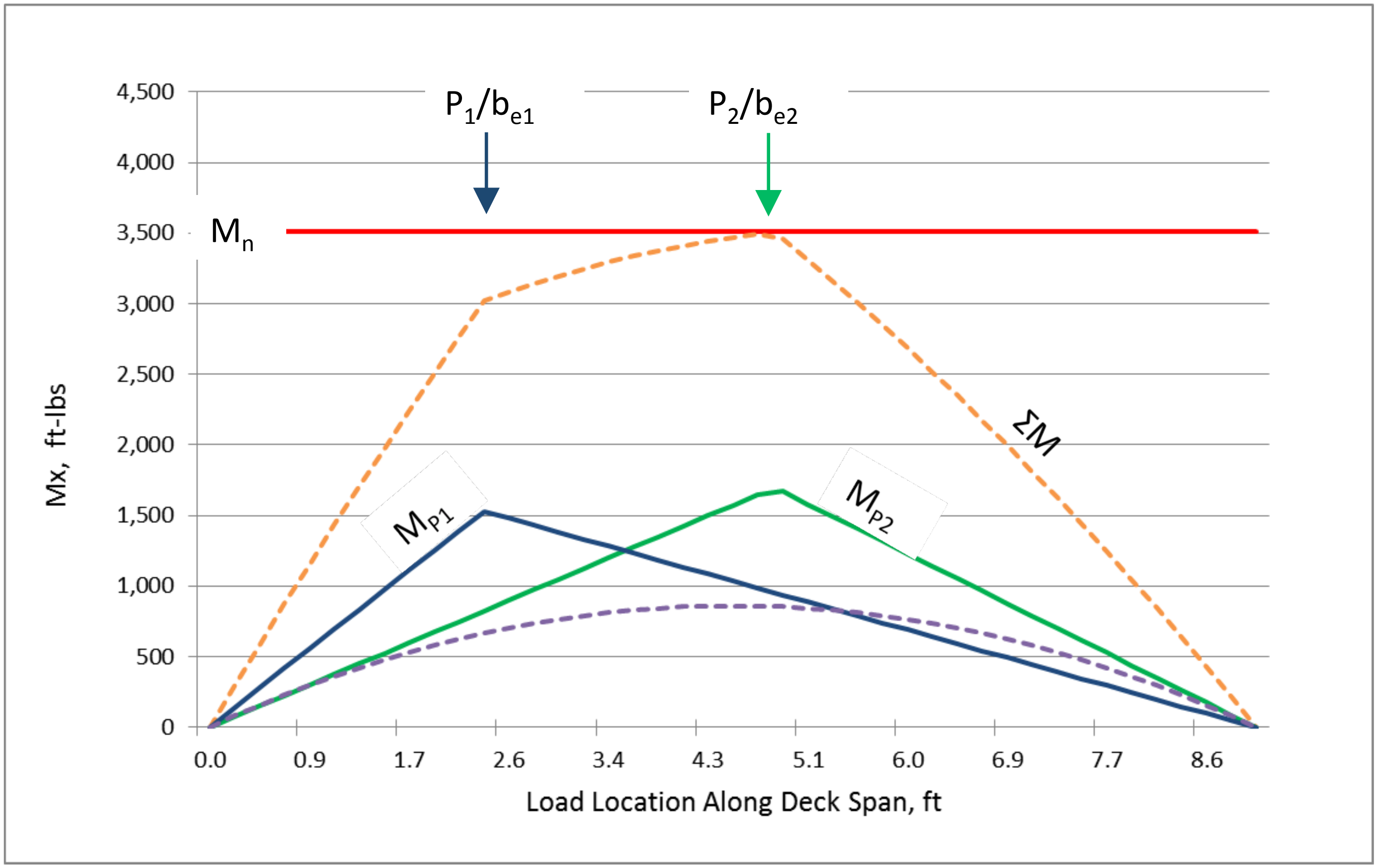
2 Loads "In-Line", M_y and V_n



For analysis purposes of M_y and V_n, two loads are on the beam and equations for shear and bending are cumbersome, but simplistic. For calculation purposes, P_1 and P_2 are typically equal loads, but distribution widths b_{e1} and b_{e2} may differ; hence, loads are illustrated as being different. Variables "L", "a" and "b" are consistent with traditional engineering load diagrams.

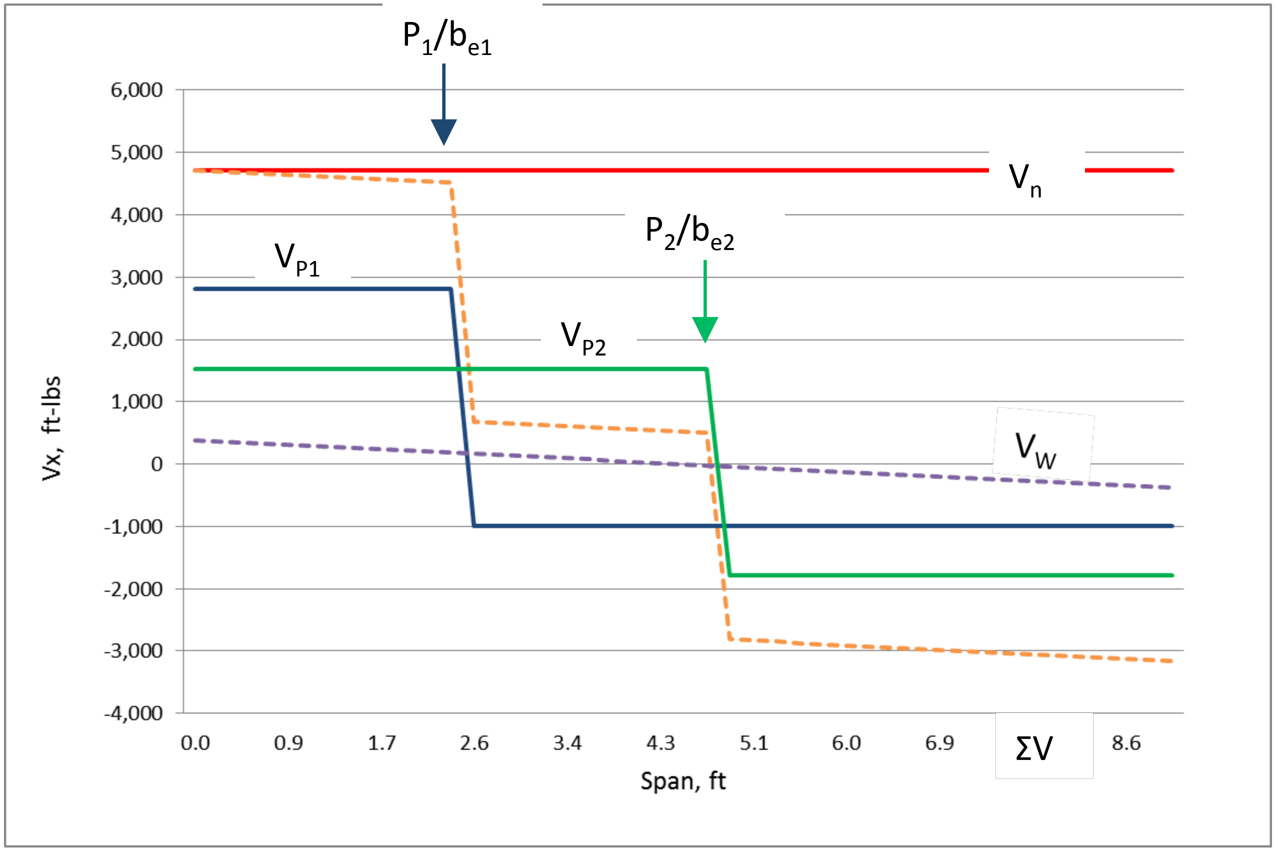
Nothing new so far, except beams are to be analyzed using distributed concentrated loads, P/b_e , in lieu of uniform loads suggested in the literature.

2 Loads "In-Line", M_y



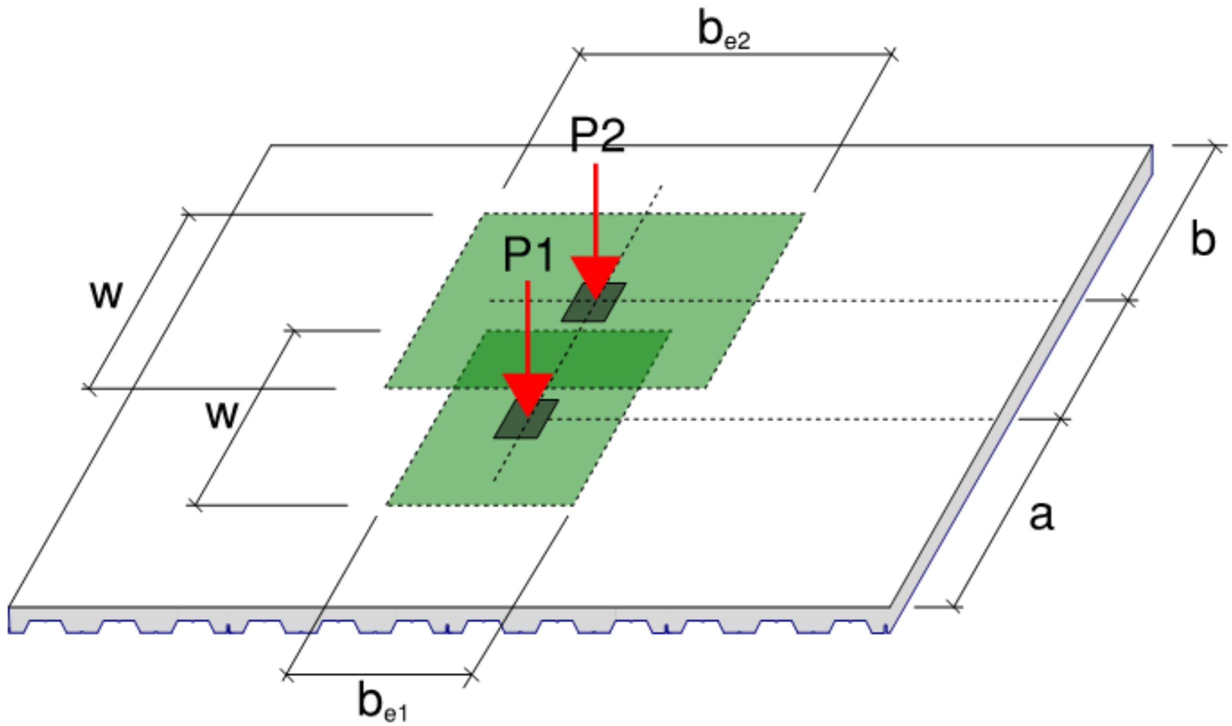
This graph illustrates bending moments for P_1/b_{e1}, P_2/b_{e2} and any uniform load along the beam. Notice that the moments are cumulative and must not exceed the allowable.

2 Loads "In-Line", V_n



A similar graph for shear. Again, P_1/b_{e1}, P_2/b_{e2} and any uniform load along the beam are cumulative and must not exceed allowable

2 Loads “In-Line”, M_w

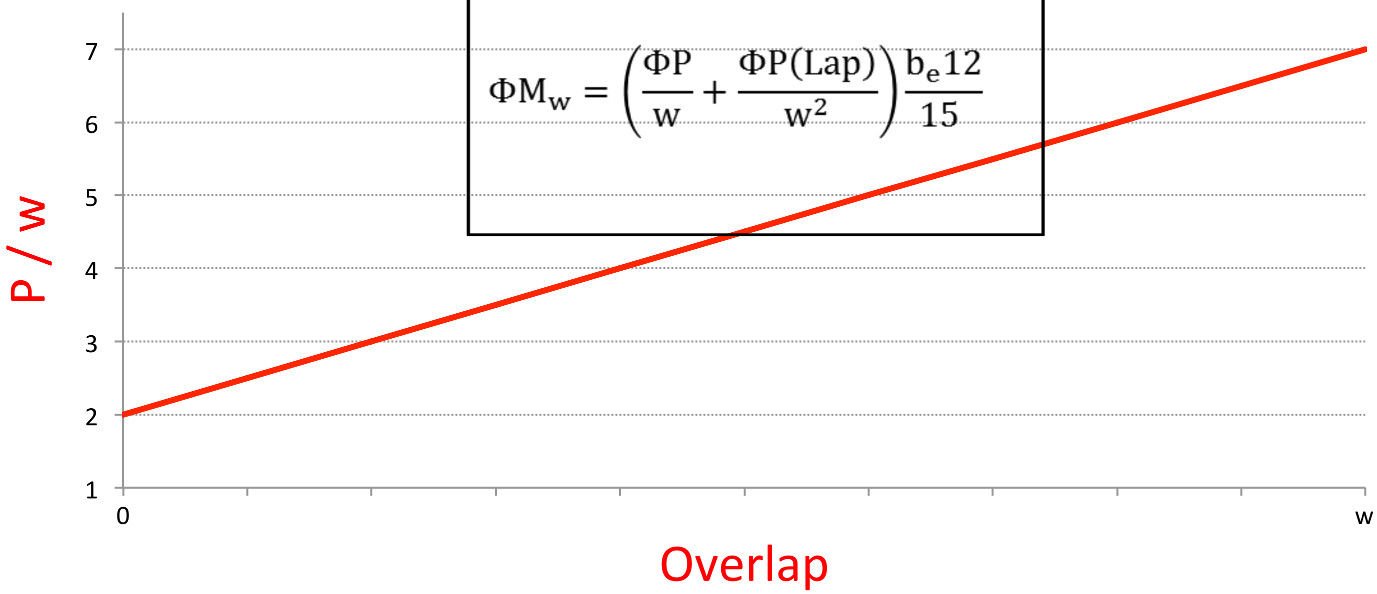


Weak axis bending for “in-line” loads will take a little more explanation. The basic premise is “Loads are uniformly distributed along the length “w”.” If influence zones overlap (*and they usually do*), the generic weak axis bending equation provided by SDI needs a slight modification.

2 Loads "In-Line", M_w

$$\Phi M_w = \frac{\Phi P\, b_e\, 12}{w\, 15}$$

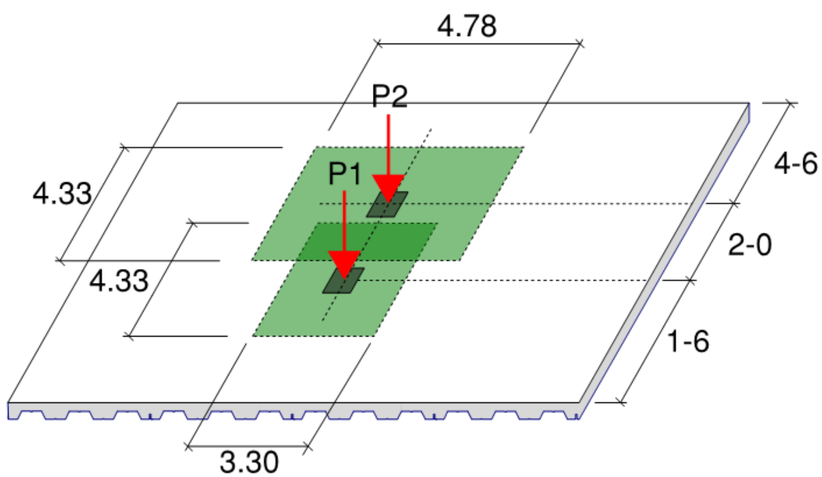
$$\Phi M_w = \frac{2\Phi P\, b_e\, 12}{w\, 15}$$



$$\Phi M_w = \left(\frac{\Phi P}{w} + \frac{\Phi P(Lap)}{w^2}\right)\frac{b_e 12}{15}$$

The new equation for multiple "in-line" loads for weak axis bending is simply a linear interpolation between a single load analysis and two loads combined. The great advantage to this equation is " ***IT WORKS EVERYWHERE*** " regardless of the overlap.

2 Loads "In-Line", Scaffold Example



- 2 x 12 x 20 ga composite deck
- 8-0 span
- 5" NW slab (t=3")
- W6xW6-W2.1xW2.1 (d=1.5")
- Scaffold post, b = 4"

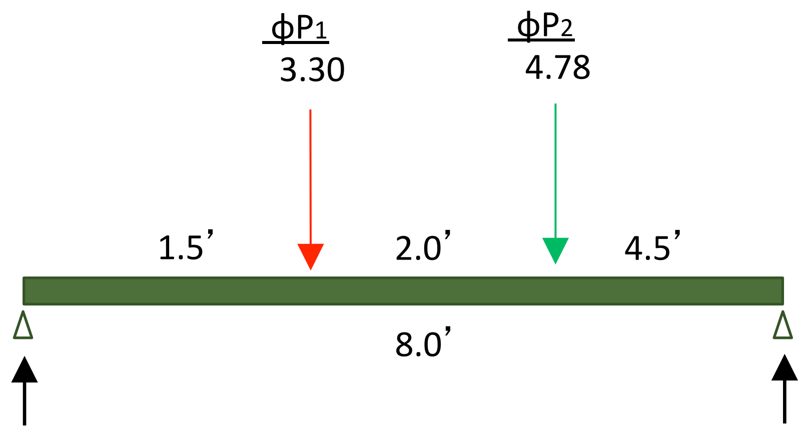
- $W_L = 0$
- W_d = (1.2) 52 psf FDDM 2C
- ϕM_y = 4140 ft-lbs/ft FDDM 4C
- ϕV_n = 5116 lb/ft FDDM 8B

ϕM_w = 2757 in-lb/ft

To demonstrate the mechanics for "in-line" loads, consider scaffolding during construction. The subcontractor has asked to use scaffolding for the brick fascia. How should you respond?

Punching shear and deflection are unlikely to limit P and will not be shown in this example.

2 Loads "In-Line", Scaffold Example, V_n



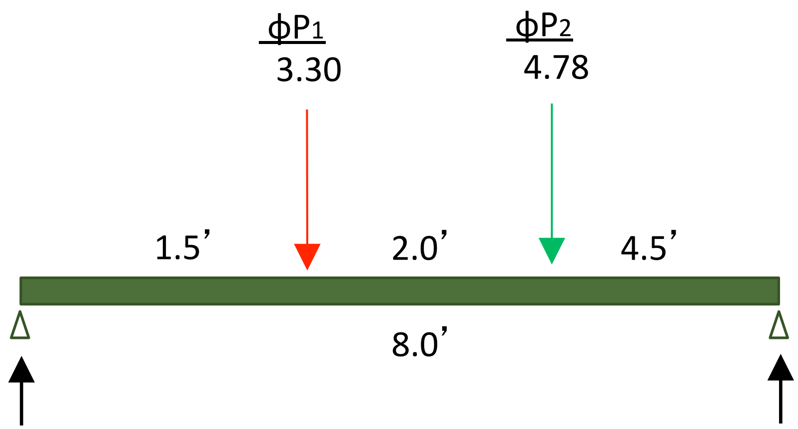
- b_{e1} = 3.30 ft
- b_{e2} = 4.78 ft
- w = 4.33 ft
- Lap = 2.33 ft (use 2.0)
- W_d = 62 psf

Shear: From FDDM 8B, ϕV_n = 5116 lbs. Distribute loads P_1 and P_2 over their effective widths, b_{e1} and b_{e2}, assume $P_1 = P_2$ and solve for P. Don't forget to add dead and applicable live loads.

$$R_R = 5116 = \frac{62(8)}{2} + \frac{\Phi P}{3.3}\left(\frac{1.5}{8}\right) + \frac{\Phi P}{4.78}\left(\frac{3.5}{8}\right) \qquad \Phi P = 33822 \text{ lbs}$$

$$R_L = 5116 = \frac{62(8)}{2} + \frac{\Phi P}{3.3}\left(\frac{6.5}{8}\right) + \frac{\Phi P}{4.78}\left(\frac{4.5}{8}\right) \qquad \Phi P = 13377 \text{ lbs}$$

2 Loads "In-Line", Scaffold Example, M_y



- b_{e1} = 3.30 ft
- b_{e2} = 4.78 ft
- w = 4.33 ft
- Lap = 2.33 ft (use 2.0)
- W_d = 62 psf

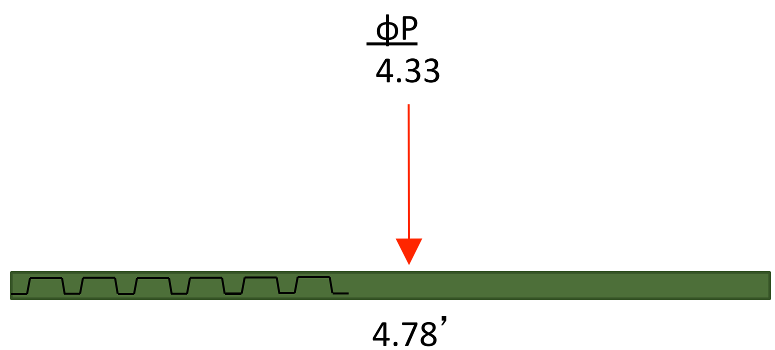
Bending: From FDDM 4C, ϕM_y = 4140 ft-lbs. Again, distribute loads P_1 and P_2 over their effective widths and solve for P.

$$M_{@P1} = 4140 = \frac{62(1.5)(6.5)}{2} + \frac{\Phi P(1.5)(6.5)}{(3.30)8} + \frac{\Phi P(4.5)(1.5)}{(4.78)8} \qquad \Phi P = 7029 \text{ lbs}$$

$$M_{@P2} = 4140 = \frac{62(3.5)(4.5)}{2} + \frac{\Phi P(1.5)(4.5)}{(3.30)8} + \frac{\Phi P(3.5)(4.5)}{(4.78)8} \qquad \Phi P = 5470 \text{ lbs}$$

2 Loads "In-Line", Scaffold Example, M_w

with NEW M_w equation



- b_{e1} = 3.30 ft
- b_{e2} = 4.78 ft
- w = 4.33 ft
- Lap = 2.33 ft (use 2.0)

Weak: This will take more explanation.

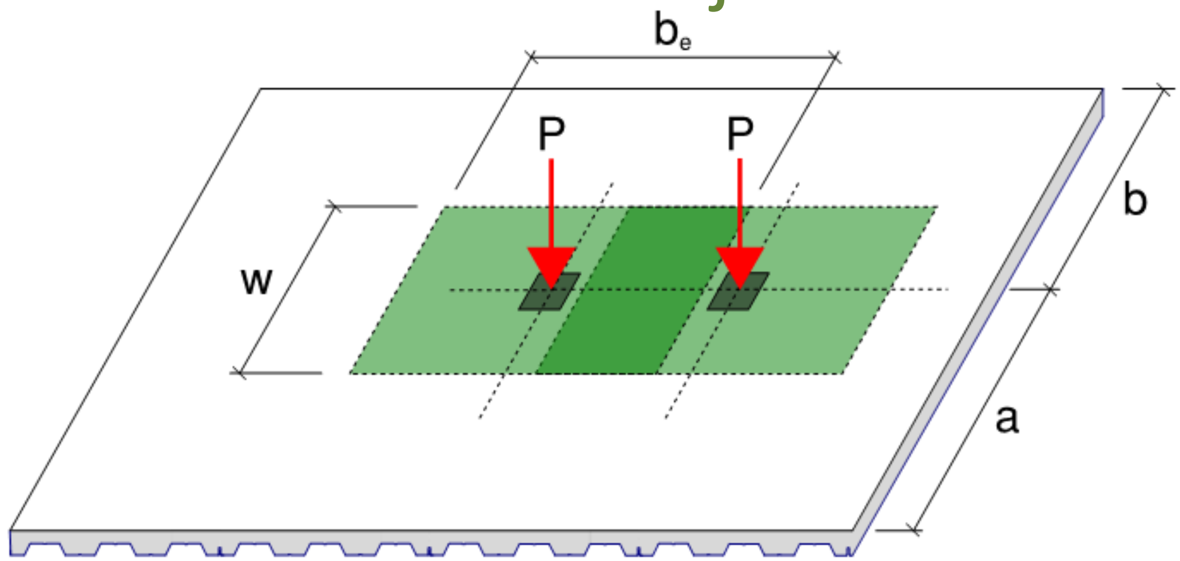
1. Notice that the load P is distributed over an effective width "w", not "b_e".
2. The weak axis beam length = b_e and will differ for P_1 and P_2.
3. b_{emax} will control.
4. With multiple "in-line" loads, use the new ϕM_w to correct for influence zone overlap.
5. Use ϕ = 0.75 and Ω = 2.0, not ACI factors.

$$\Phi M_w = \left(\frac{\Phi P}{w} + \frac{\Phi P(Lap)}{w^2}\right)\frac{b_e 12}{15}$$

$$M_{w@P1} = 2757 = \left(\frac{12\Phi P}{4.33} + \frac{12\Phi P(2.0)}{4.33^2}\right)\left(\frac{3.3}{15}\right) \qquad \Phi P = 3093\text{ lbs}$$

$$M_{w@P2} = 2757 = \left(\frac{12\Phi P}{4.33} + \frac{12\Phi P(2.0)}{4.33^2}\right)\left(\frac{4.78}{15}\right) \qquad \Phi P = 2135\text{ lbs}$$

2 Loads “Adjacent”

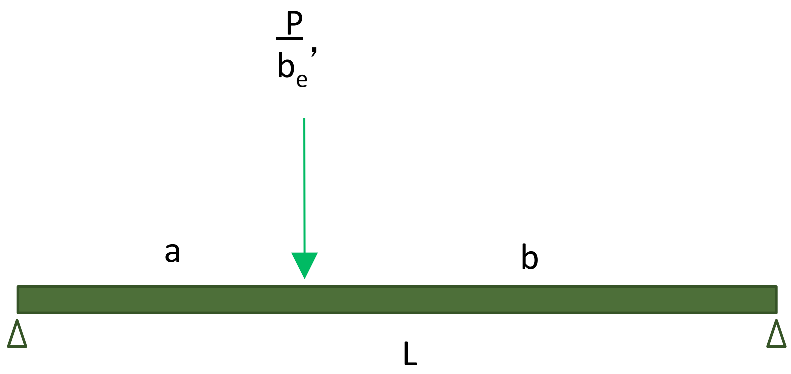


Influence zones for “adjacent” loads will overlap, but the overlap does not mean twice the stress. Intuitively, we know stresses are greatest directly under the load and dissipate along the edges. Effective width formulas for “b_e” and “w” compensate for this stress gradient.

For shear and bending, adjust b_e so concrete is not used twice. $b_e' = b_e/2$ + load spacing/2.

For weak axis bending, ΣM_w will require a more detailed discussion.

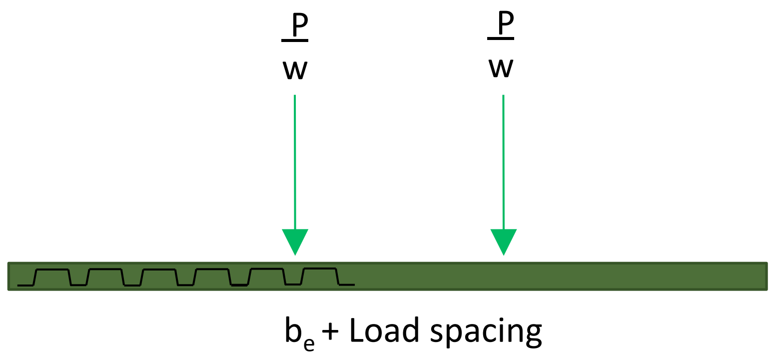
2 Loads "Adjacent", M_y and V_n



For analysis purposes of M_y and V_n, load P is distributed over b_e or b_e'. Simple.

$$b_e' = \frac{b_e + \text{Load spacing}}{2} < b_e$$

2 Loads "Adjacent", M_w

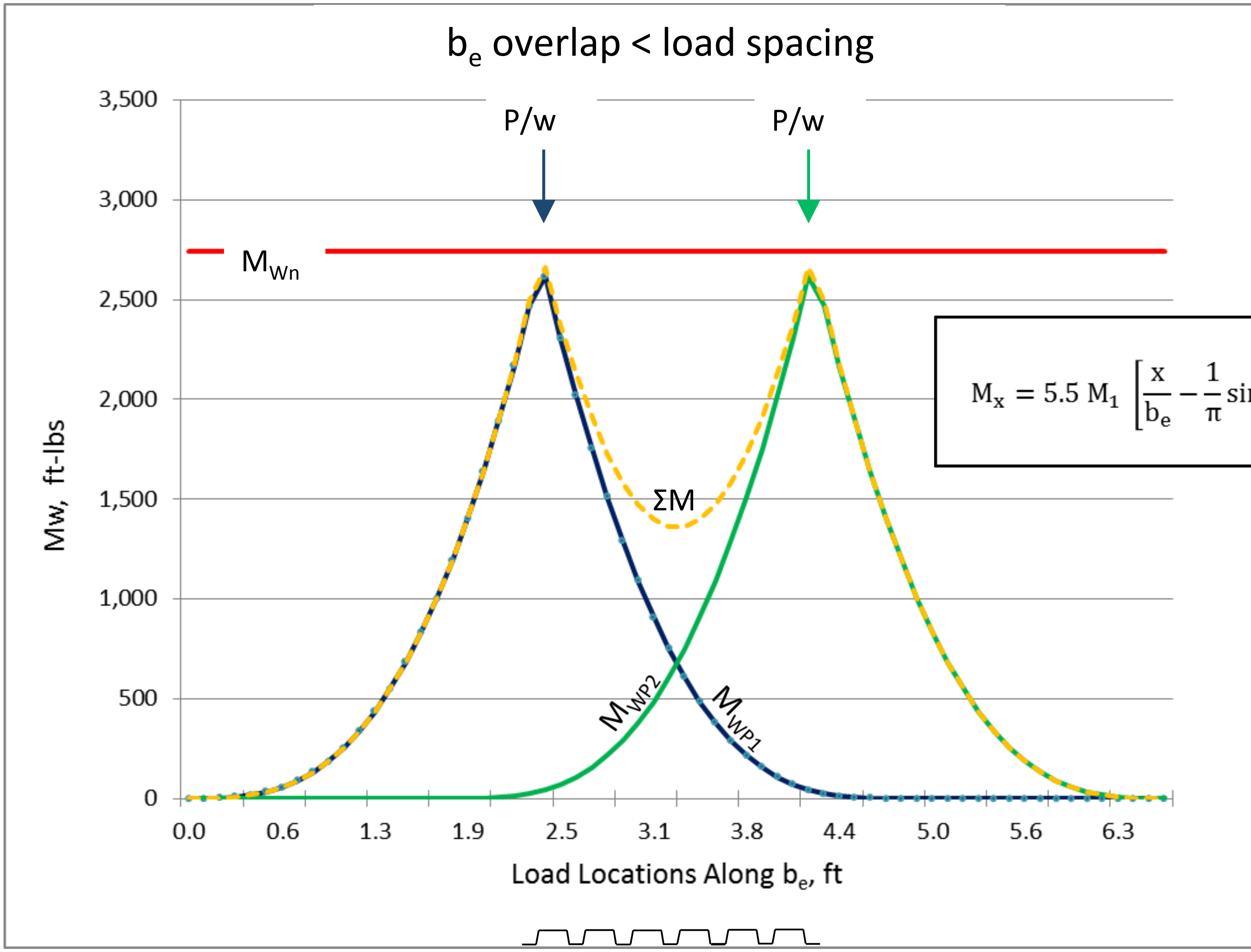


Overlapping influence zones may result in **cumulative** weak axis bending moments, and traditional engineering mechanics are not appropriate for a two-way slab problem with sinusoidal stress distribution.

Sinusoidal stress distribution? *Two way slab design?*

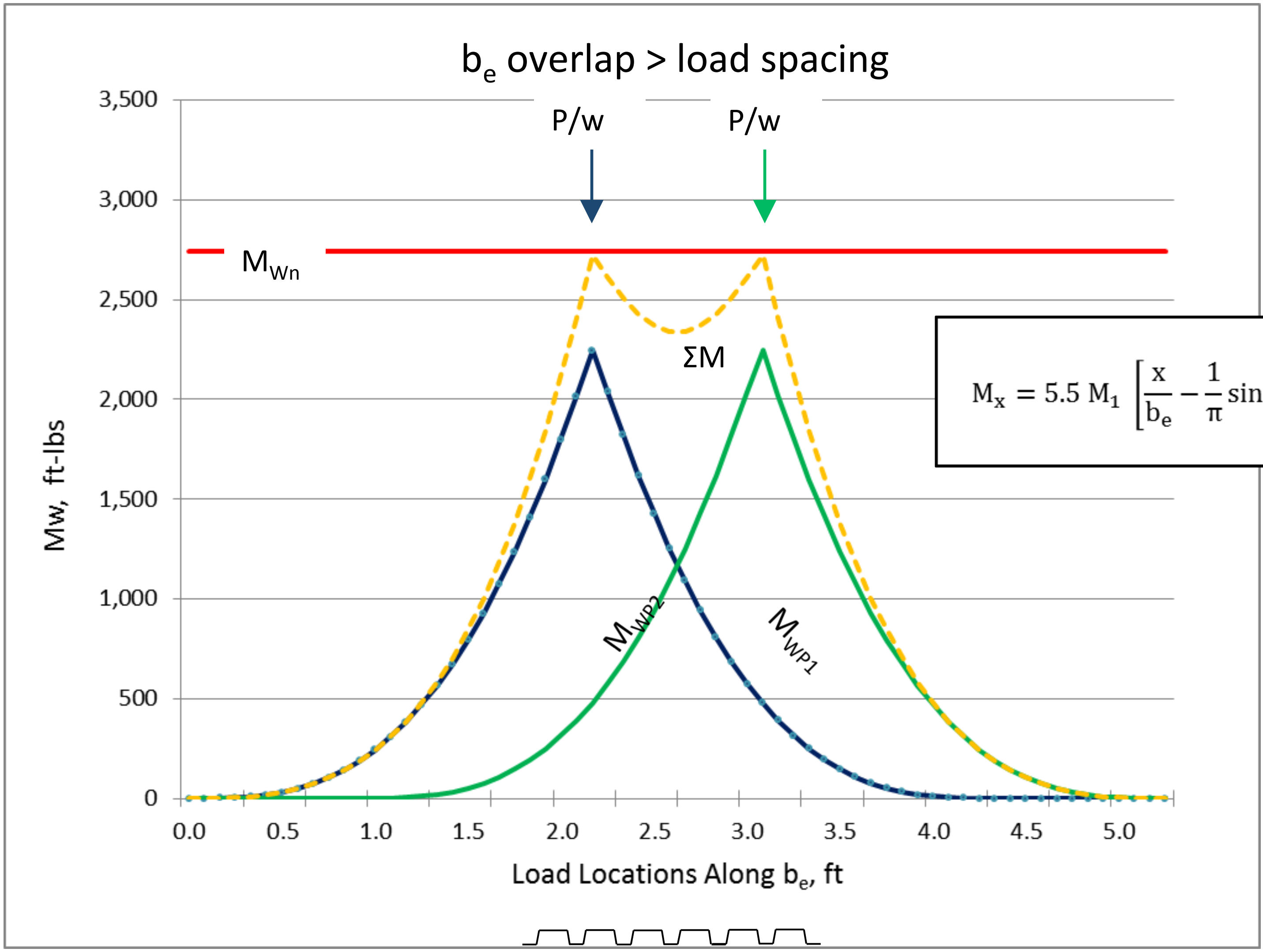
This sounds complicated, but the next few graphs and example problem makes understanding and analysis relatively easy.

2 Loads “Adjacent”, M_w



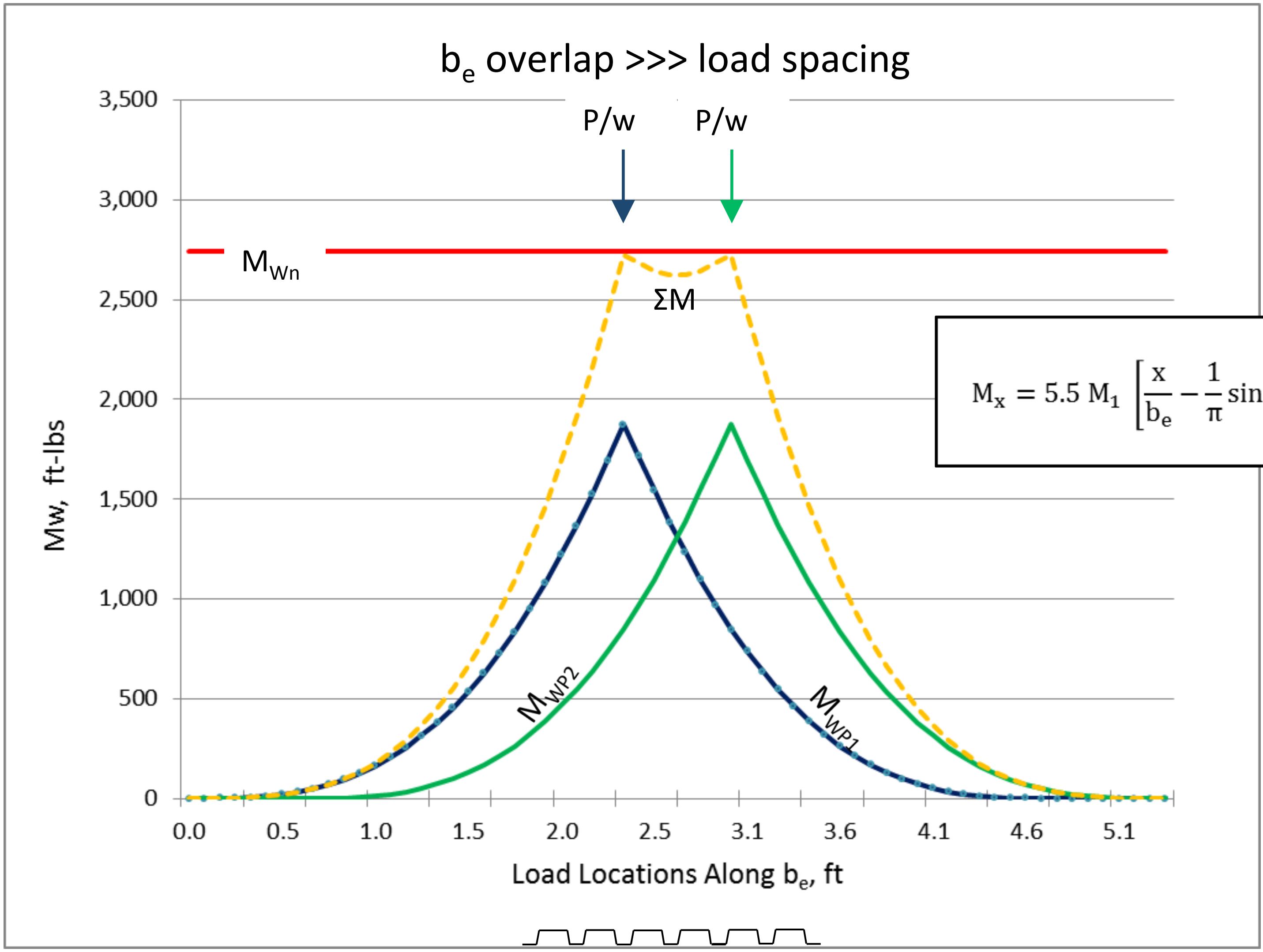
$$M_x = 5.5\, M_1 \left[\frac{x}{b_e} - \frac{1}{\pi}\sin\left(\frac{\pi x}{b_e}\right)\right] \text{ rad}$$

2 Loads "Adjacent", M_w



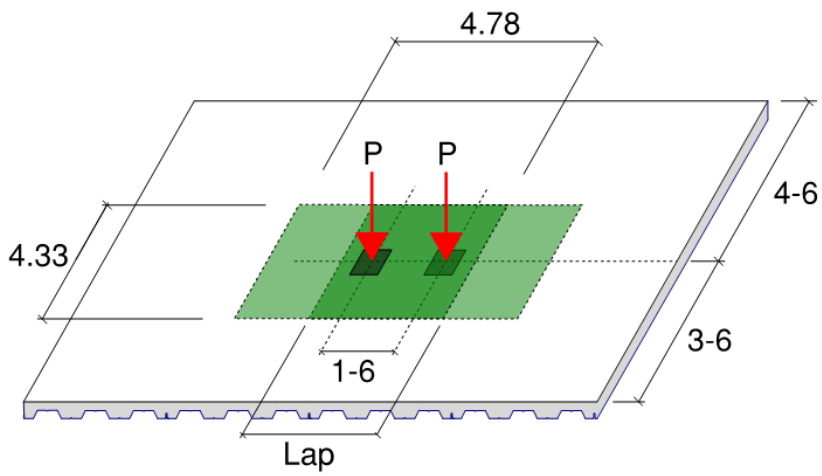
$$M_x = 5.5\ M_1 \left[\frac{x}{b_e} - \frac{1}{\pi}\sin\left(\frac{\pi x}{b_e}\right)\right]\ \text{rad}$$

2 Loads “Adjacent”, M_w



$$M_x = 5.5\, M_1 \left[\frac{x}{b_e} - \frac{1}{\pi}\sin\left(\frac{\pi x}{b_e}\right)\right] \text{ rad}$$

2 Loads "Adjacent", Scaffold Example



Same deck as "in-line" example

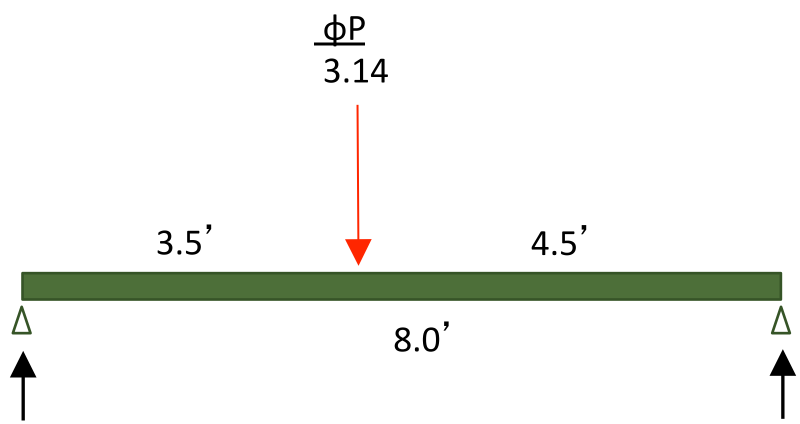
- $W_L = 0$
- $W_d = (1.2)\ 52$ psf — FDDM 2C
- $\phi M_y = 4140$ ft-lbs/ft — FDDM 4C
- $\phi V_n = 5116$ lb/ft — FDDM 8B
- $\phi M_w = 2757$ in-lb/ft

To demonstrate the mechanics for "adjacent" loads, let's rotate the scaffold from our previous example. At x = 3-6, the distribution with $b_e = 4.78$ ft, and adjacent influence zones overlap. The mechanics for M_y and V_n are similar to the previous example using a modified b_e.

$$b'_e = \left(\frac{b_e + \text{load spacing}}{2}\right) = \left(\frac{4.78 + 1.5}{2}\right) = 3.14 \text{ ft}$$

Again, punching shear and deflection are unlikely to limit P and will not be shown in this example.

2 Loads "Adjacent", Scaffold Example, M_y and V_n



- b_e = 4.78 ft
- b_e' = 3.14 ft
- W = 4.33 ft
- W_d = 62 psf

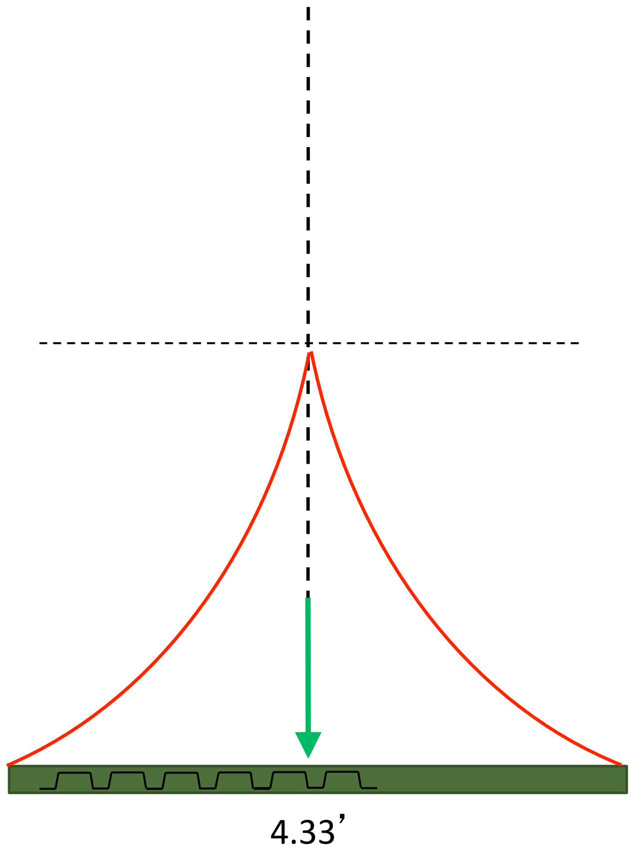
$$R_R = 5116\ \text{lbs} = \frac{62\ \text{plf}(8\ \text{ft})}{2} + \frac{\Phi P}{3.14\ \text{ft}}\left(\frac{3.5\ \text{ft}}{8\ \text{ft}}\right) \qquad \Phi P = 34938\ \text{lbs}$$

$$R_L = 5116\ \text{lbs} = \frac{62\ \text{plf}(8\ \text{ft})}{2} + \frac{\Phi P}{3.14\ \text{ft}}\left(\frac{4.5\ \text{ft}}{8\ \text{ft}}\right) \qquad \Phi P = 27174\ \text{lbs}$$

$$M_{@P} = 4140\ \frac{\text{ft}-\text{lbs}}{\text{ft}} = \frac{62\ \text{plf}\,(3.5\ \text{ft})(4.5\ \text{ft})}{2} + \frac{\Phi P(4.5\ \text{ft})(3.5\ \text{ft})}{(3.14\ \text{ft})8\ \text{ft}} \qquad \Phi P = 5824\ \text{lbs}$$

2 Loads "Adjacent", Scaffold Example, M_w

$$\Phi M_n = \frac{12\Phi P}{w}\left(\frac{b_e}{15}\right)$$

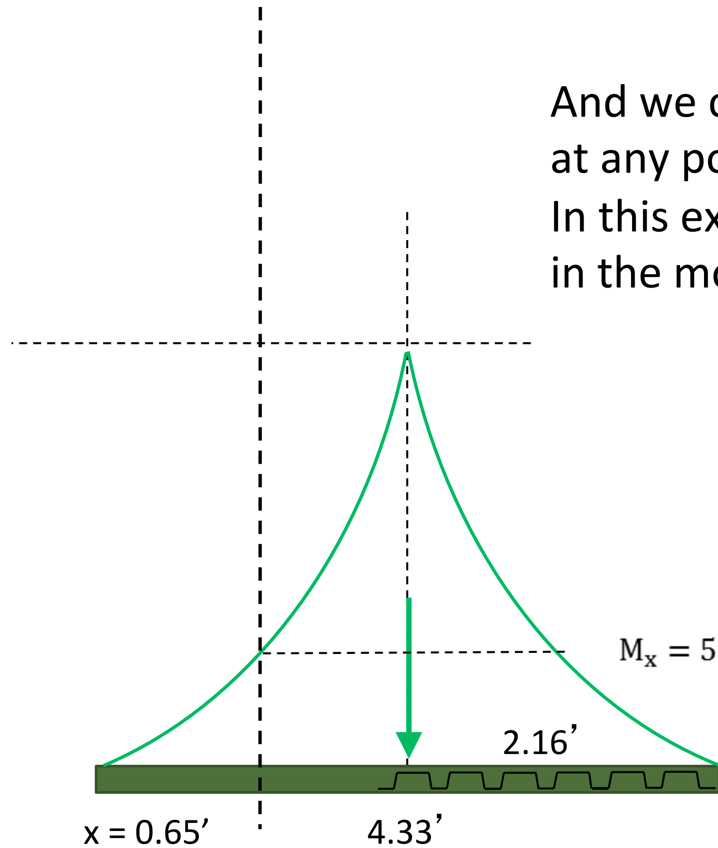


A load develops a sinusoidal moment envelope over a beam length = b_e and is resisted by the available weak axis bending moment = ϕM_w

2 Loads “Adjacent”, Scaffold Example, M_w

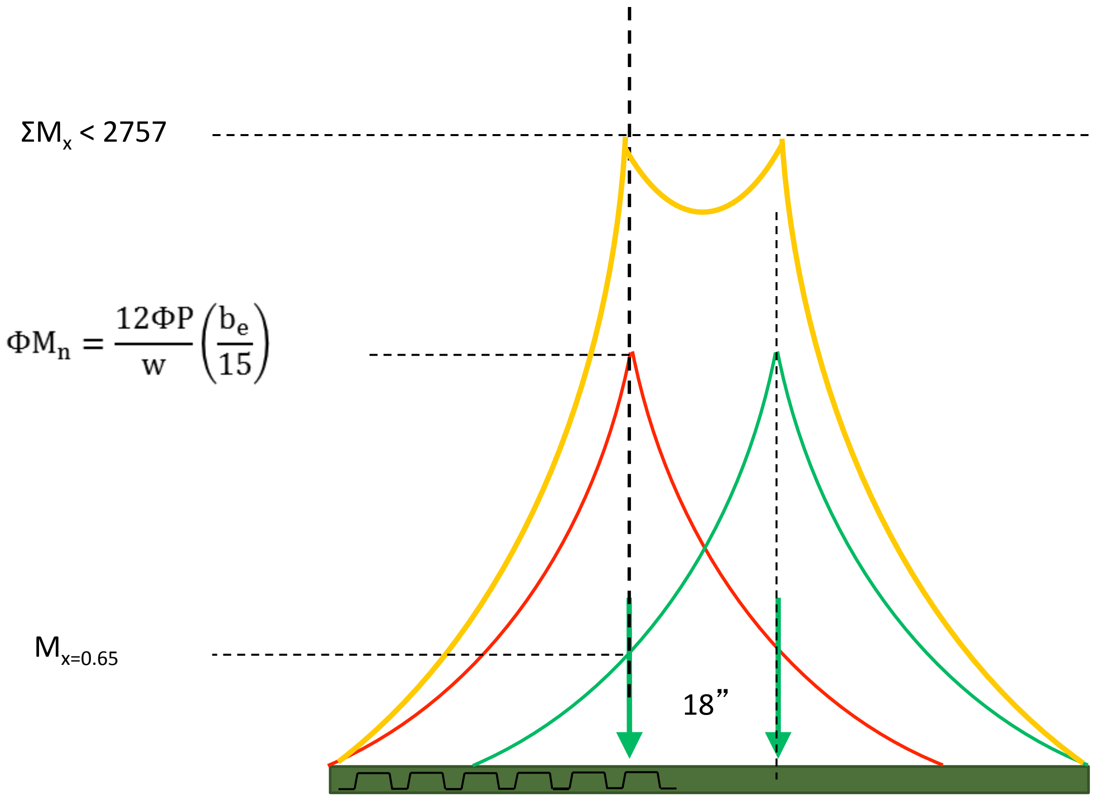
And we can calculate the moment at any point x along this curve.
In this example, we are interested in the moment at x = 0.65’.

$$\Phi M_n = \frac{12\Phi P}{w}\left(\frac{b_e}{15}\right)$$



$$M_x = 5.5\ \Phi M_n \left[\frac{x}{b_e} - \frac{1}{\pi}\sin\left(\frac{\pi x}{b_e}\right)\right]\ \text{rad}$$

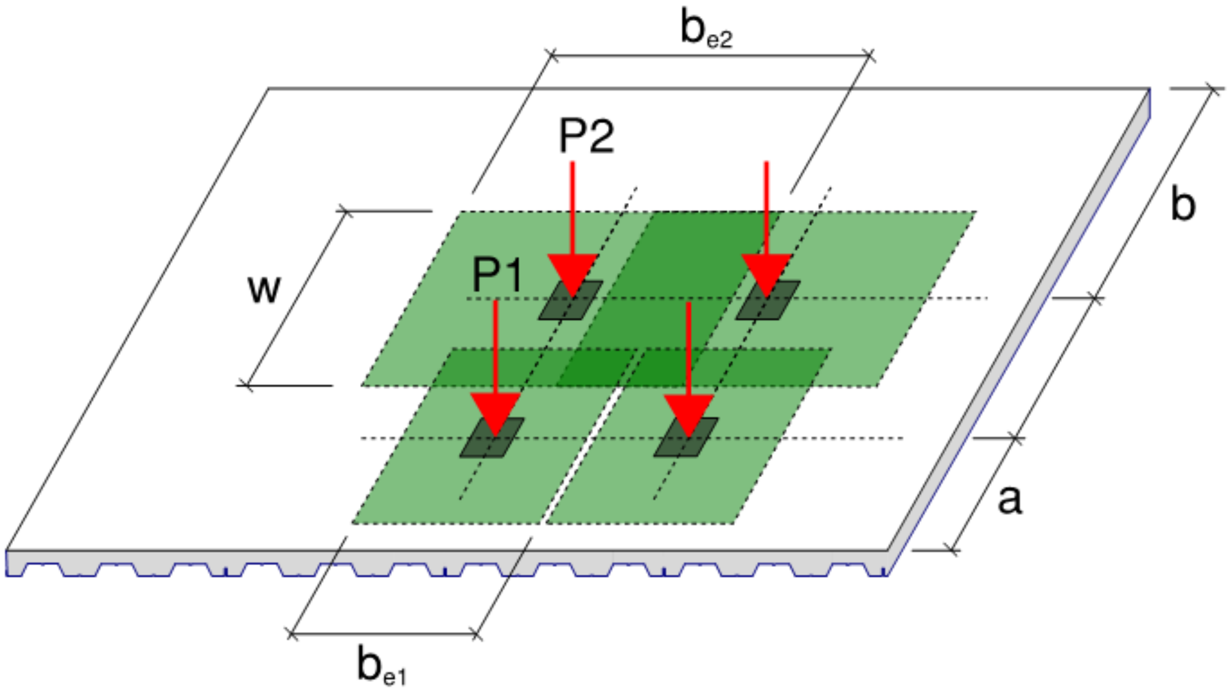
2 Loads "Adjacent", Scaffold Example, M_w



Focus on the picture, not the equation.

$$2757\frac{\text{in}-\text{lbs}}{\text{ft}} = \frac{12\Phi P}{4.33\text{ ft}}\left(\frac{4.78\text{ ft}}{15}\right) + 5.5\left[\frac{12\Phi P}{4.33\text{ ft}}\left(\frac{4.78\text{ ft}}{15}\right)\right]\left[\frac{0.65\text{ ft}}{4.78\text{ ft}} - \frac{1}{\pi}\sin\left(\frac{\pi(0.65\text{ ft})}{4.78\text{ ft}}\right)\right]\text{rad} \qquad \Phi P = 2977\text{ lbs}$$

4 Loads "In-Line" and "Adjacent"



You guessed it . . . . 4 loads . . . . "In-line" and "adjacent". If these loads are static, the calculations are tedious, but not difficult. If loads are moving, hire an intern for the summer.

For M_y and V_n, use P_1/b_{e1}' and P_2/b_{e2}' with simple shear and moment envelopes.
For M_w, use new M_w lap equation and new sinusoidal moment envelope.

Example Problem

"What size lift can this floor support?"

Slab (FDDM Example 4)

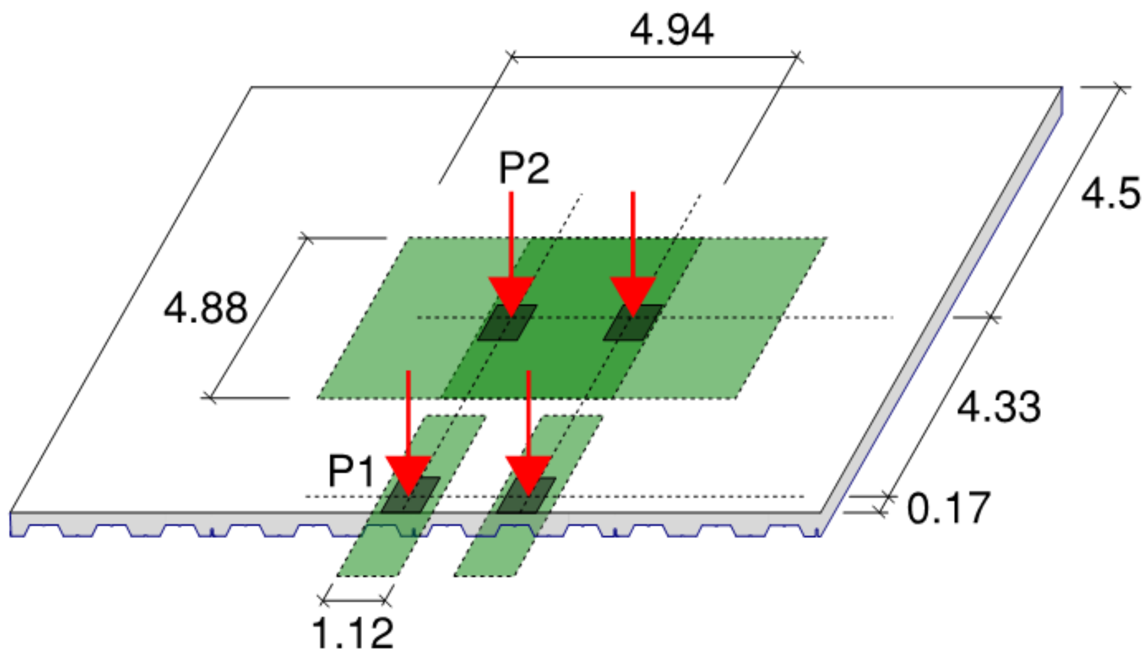
- 2 x 12 composite deck
- 20 gage
- 4 ½" total depth
- 3 ksi NW concrete
- 9-0 clear span
- 25 psf concurrent LL
- 6x6 – W2.1xW2.1 WWR
- d = 1.25"

Assumed Lift

- 52" length
- 30" width
- 12" x 4.5" tires
- 2.5 mph

Example Problem

"What size lift can this floor support?"

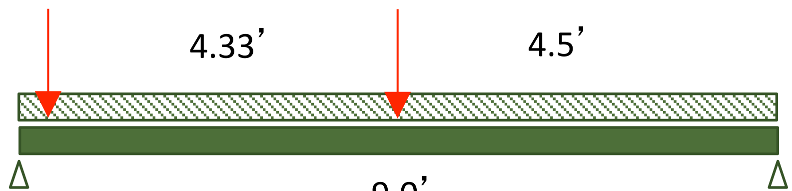


As a general rule for scissor lift <u>shear</u>, locate one tire near the support and the short axle "adjacent" creates maximum shear. If so, b_{e1} = 1.12 ft, b_{e2} = 4.94 ft, and w = 4.88 ft. For shear, P_2 adjacent influence zones overlap and b_{e2}' should be used. P_1 influence zones do not overlap, so distribution width b_{e1} needs no correction.

b_{e2}' = 4.94/2 + 2.66/2 = 3.80 ft.

Example Problem

"What size lift can this floor support?"

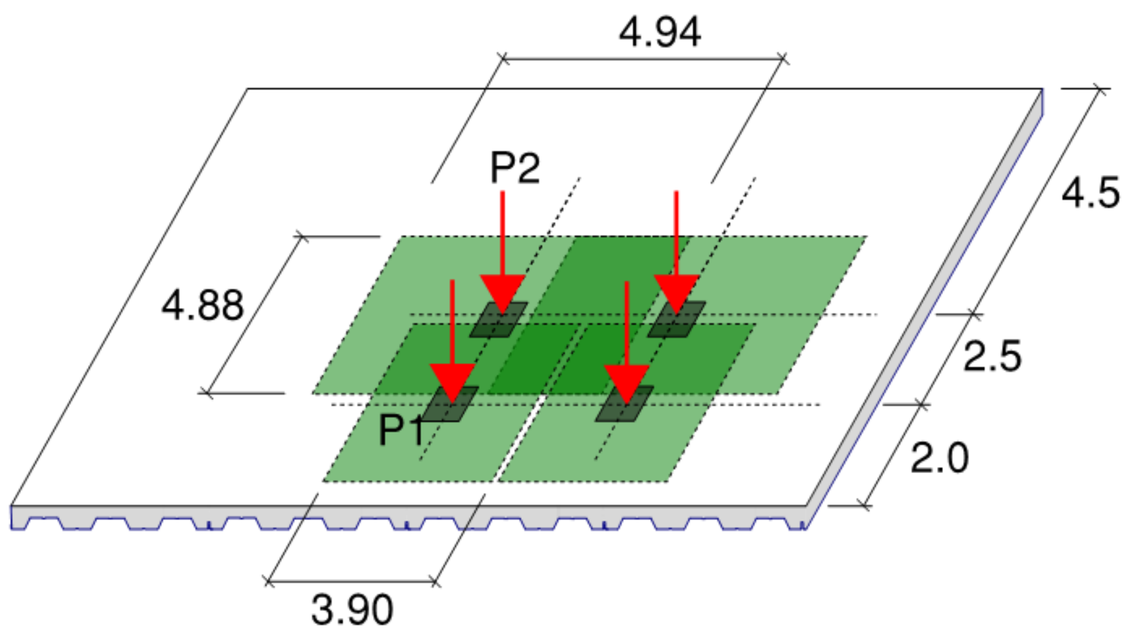


$$R_R = 4715 \text{ lbs} = \frac{53 \text{ plf}\,(9 \text{ ft})}{2} + \frac{25 \text{ plf}(1.6)(9 \text{ ft})}{2} + \frac{\Phi P}{1.12 \text{ ft}}\left(\frac{0.17 \text{ ft}}{9 \text{ ft}}\right) + \frac{\Phi P}{3.8 \text{ ft}}\left(\frac{4.5 \text{ ft}}{9 \text{ ft}}\right) \quad \Phi P = 28943 \text{ lbs}$$

$$R_L = 4715 \text{ lbs} = \frac{53 \text{ plf}\,(9 \text{ ft})}{2} + \frac{25 \text{ plf}\,(1.6)(9 \text{ ft})}{2} + \frac{\Phi P}{1.12 \text{ ft}}\left(\frac{8.83 \text{ ft}}{9 \text{ ft}}\right) + \frac{\Phi P}{3.8 \text{ ft}}\left(\frac{4.5 \text{ ft}}{9 \text{ ft}}\right) \quad \Phi P = 4264 \text{ lbs}$$

Example Problem

"What size lift can this floor support?"

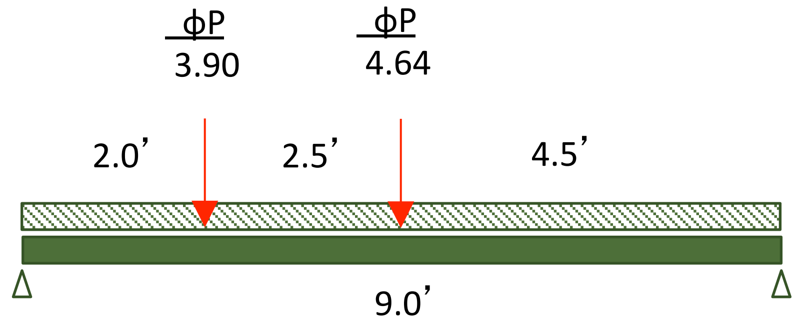


As a general rule for scissor lift <u>bending</u>, locate one tire at midspan and the short axle "in-line" creates maximum positive bending. If so, b_{e1} = 3.9 ft. b_{e2} = 4.94 ft and w = 4.88 ft. For positive bending, P_2 adjacent influence zones overlap and b_{e2}' should be used. P_1 influence zones do not overlap, so distribution width b_{e1} needs no correction.

b_{e2}' = 4.94/2 + 4.33/2 = 4.64 ft.

Example Problem

"What size lift can this floor support?"

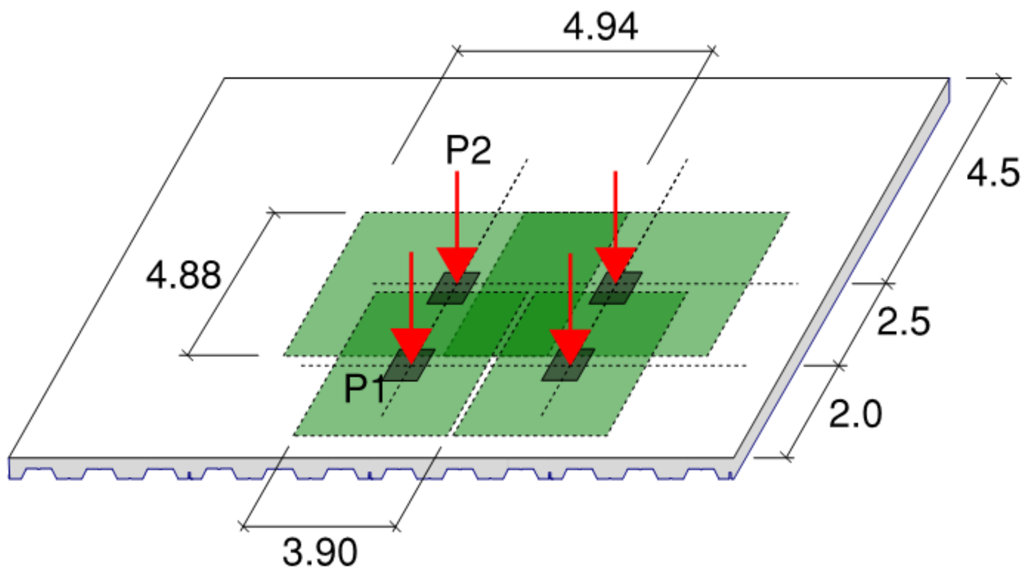


$$M_{@P1} = 3511\frac{\text{ft}-\text{lbs}}{\text{ft}} = \frac{(53\text{ plf}+25\text{ plf}(1.6))(2.0\text{ ft})(7.0\text{ ft})}{2} + \frac{\Phi P(2.0\text{ ft})(7.0\text{ ft})}{(3.9\text{ ft})9\text{ ft}} + \frac{\Phi P(4.5\text{ ft})(2.0\text{ ft})}{(4.64\text{ ft})9\text{ ft}} \qquad \Phi P = 4655\text{ lbs}$$

$$M_{@P2} = 3511\frac{\text{ft}-\text{lbs}}{\text{ft}} = \frac{(53\text{ plf}+25\text{ plf}(1.6))(4.5\text{ ft})(4.5\text{ ft})}{2} + \frac{\Phi P(2.0\text{ ft})(4.5\text{ ft})}{(3.9\text{ ft})9\text{ ft}} + \frac{\Phi P(4.5\text{ ft})(4.5\text{ ft})}{(4.64\text{ ft})9\text{ ft}} \qquad \Phi P = 3465\text{ lbs}$$

Example Problem

"What size lift can this floor support?"



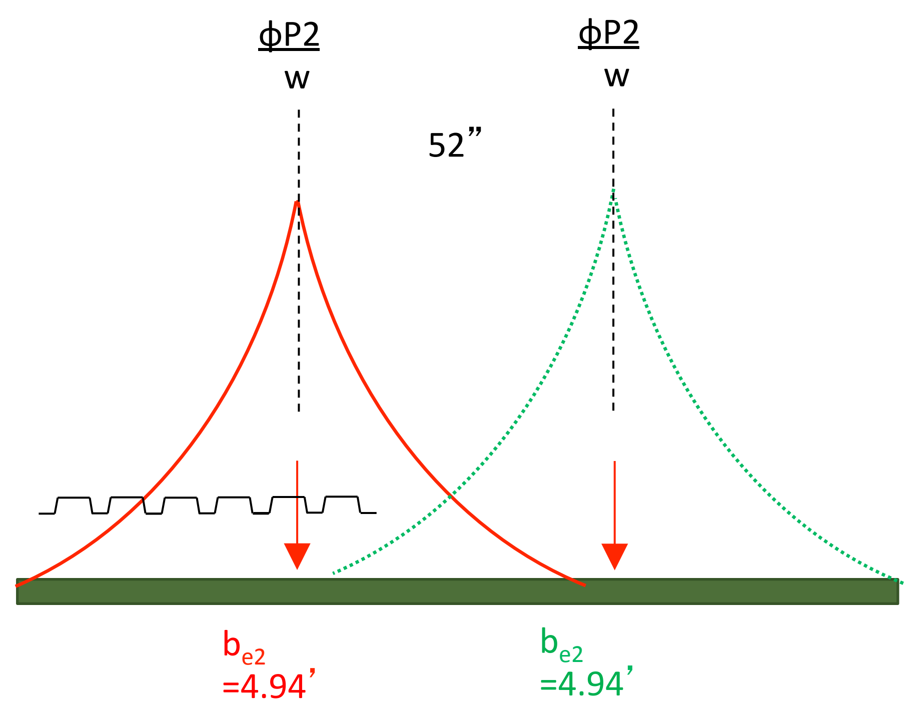
The limiting lift location for weak axis bending and positive bending are similar . . . Locate one wheel at midspan with the short axle in-line.

Notice that in-line loads P_1 and P_2 overlap and lap = 4.88' – 2.5' = **2.38'** ; therefore, in-line corrections are required.

Adjacent loads P_2 and P_2 overlap, but the overlap < wheel spacing, so no adjacent corrections are required.

Example Problem

"What size lift can this floor support?"



When comparing b_{e2} and the wheel spacing, influence lines overlap, but the overlap is less than 52". This is good news; ΣM_w calculations are not required. We only need to correct for in-line loads with the new M_w equation.

$$\Phi M_w = \left(\frac{P}{w} + \frac{P(Lap)}{w^2}\right)\frac{12b_e}{15} : \Phi M_w = 2285\frac{in-lbs}{ft}, w = 4.88\ ft, b_e = 4.94\ ft, Lap = 2.38\ ft, \mathbf{P = 2534\ lbs}$$

FDDM Scissor Lift Tables?

Slab	Gage	WWR	ϕM_w	ϕP / Span					
				5.0	6.0	7.0	8.0	9.0	10.0
4.5" (t=2.5")	22	6x6-W2.1xW2.1	2740	2648	2422	2284	2190	2272	2043
		6x6-W2.9xW2.9	3730	2881	2876	2867	2854	2657	2043
		4x4-W2.9xW2.9	5470	2881	2876	2867	2854	2657	2043
	20	6x6-W2.1xW2.1	2740	2648	2422	2284	2190	2272	2293
		6x6-W2.9xW2.9	3730	3122	3120	3112	2985	3097	2737
		4x4-W2.9xW2.9	5470	3122	3120	3112	3102	3466	2737
	18	6x6-W2.1xW2.1	2740	2648	2422	2284	2190	2272	2293
		6x6-W2.9xW2.9	3730	3570	3302	3113	2985	3097	3126
		4x4-W2.9xW2.9	5470	3570	3571	3568	3560	4535	3844
5.0" (t=3.0")	22	6x6-W2.1xW2.1	3300	3117	2861	2704	2598	2539	2657
		6x6-W2.9xW2.9	4520	3442	3438	3430	3417	3377	2657
		4x4-W2.9xW2.9	6640	3442	3438	3430	3417	3377	2657
	20	6x6-W2.1xW2.1	3300	3117	2861	2704	2598	2539	2817
		6x6-W2.9xW2.9	4520	3704	3703	3694	3549	3468	3496
		4x4-W2.9xW2.9	6640	3704	3703	3697	3686	4353	3496
	18	6x6-W2.1xW2.1	3300	3117	2861	2704	2598	2539	2817
		6x6-W2.9xW2.9	4520	4198	3908	3694	3549	3468	3848
		4x4-W2.9xW2.9	6640	4198	4202	4200	4193	5100	4885

Please consult with appropriate professional for ϕ, impact or unbalanced load factors.

30" x 52" (52" x 30") load footprint concurrent with 25 psf construction live load.

4.5" wheel

WWR d = t/2

ϕM_w

ϕV_n

ϕM_y

Example Problem

"Can my floor support this data rack(s)?"

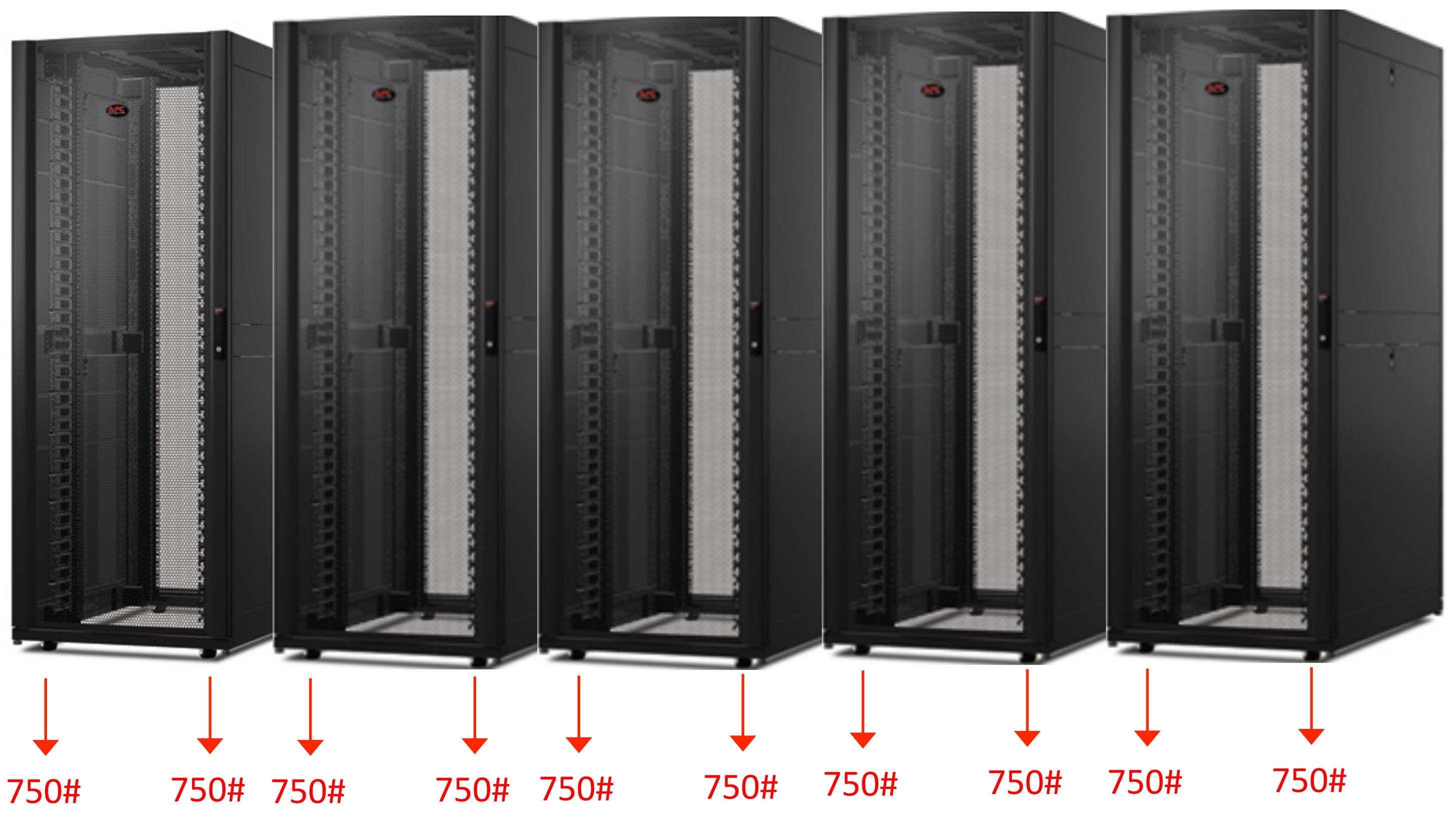
Slab
- 1.5 x 6 x 18 ga composite deck
- 5.0" Total Depth
- 3 ksi NW Concrete
- 7-0 Clear Span
- 40 psf Concurrent LL
- 6x6 – W2.9xW2.9 WWR
- d = 1.0"

Data Rack
- 42" deep
- 28" overall width
- 21" caster spacing
- 3" casters
- 3000# static capacity

First thought – 3000#/(28"x42")+40 psf = 407 psf
FDDM Table 6A = 400 psf No Good!

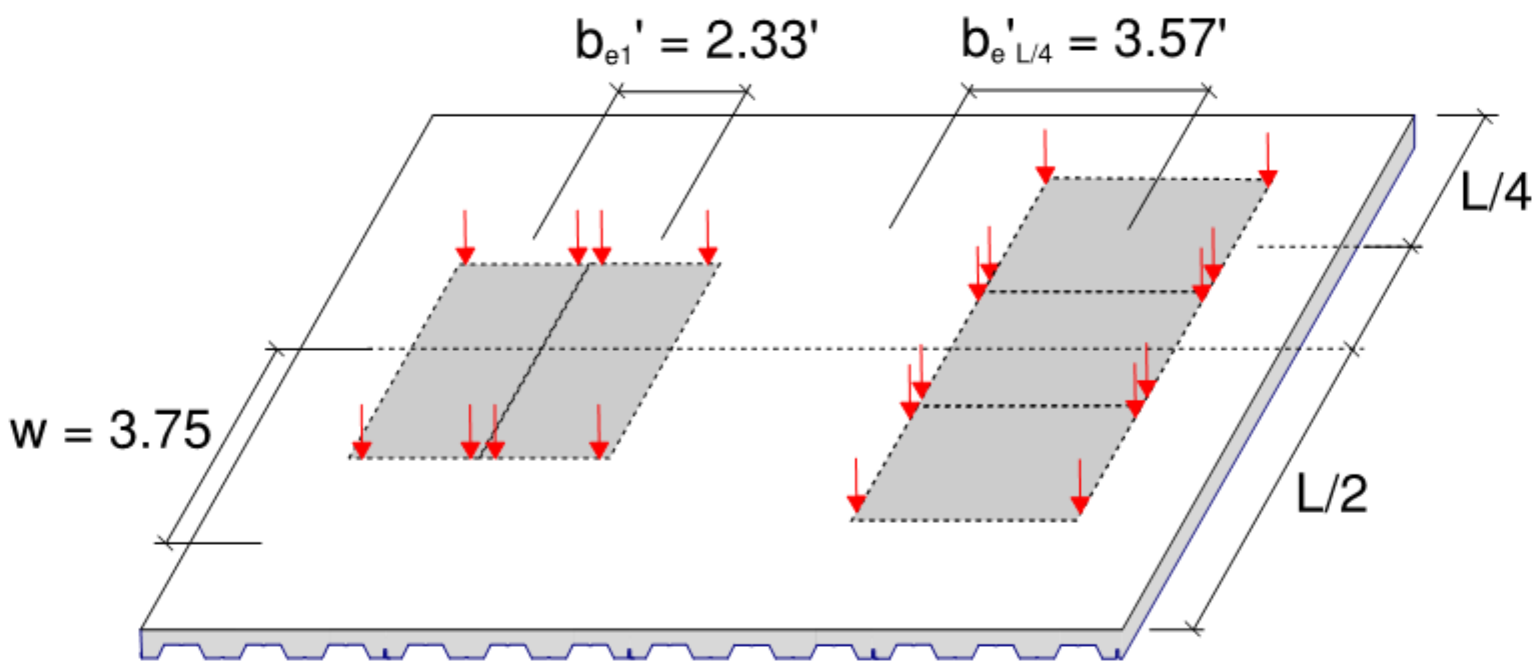
Example Problem

"Can my floor support this data rack(s)?"



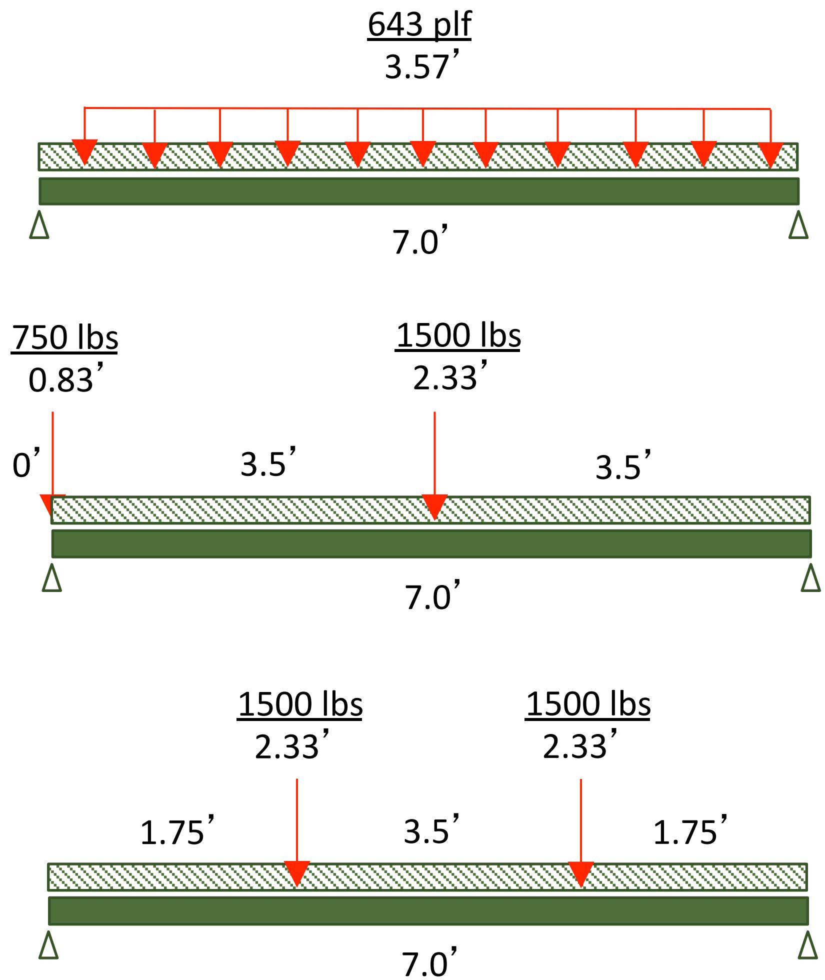
Example Problem

"Can my floor support this data rack(s)?"

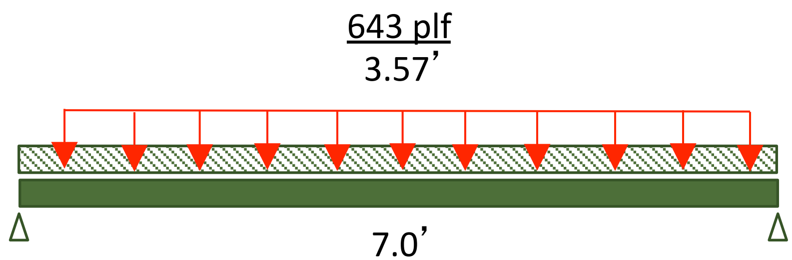


The "stacked" data rack orientation may vary. If stacked adjacent, casters may only be 14" apart, so loads would combine (1500 lbs) with a modified distributed width of = 2.33'. If stacked in-line, multiple 750 lb loads occur along the span with a modified distribution width = 3.57' width.

Data Rack – V_n, M_y



Data Rack – M_w



$$\Phi V = 1449\frac{\text{lbs}}{\text{ft}} < 3019\ \frac{\text{lbs}}{\text{ft}}$$

$$\Phi M_y = 2536\ \frac{\text{ft}-\text{lbs}}{\text{ft}} < 5980\ \frac{\text{ft}-\text{lbs}}{\text{ft}}$$

$$\Phi M_w = \left(\frac{\Phi P}{w}\right)\frac{b_e 12}{15}$$

$$P = 643\ \text{plf}(7') = 4501\ \text{lbs},$$

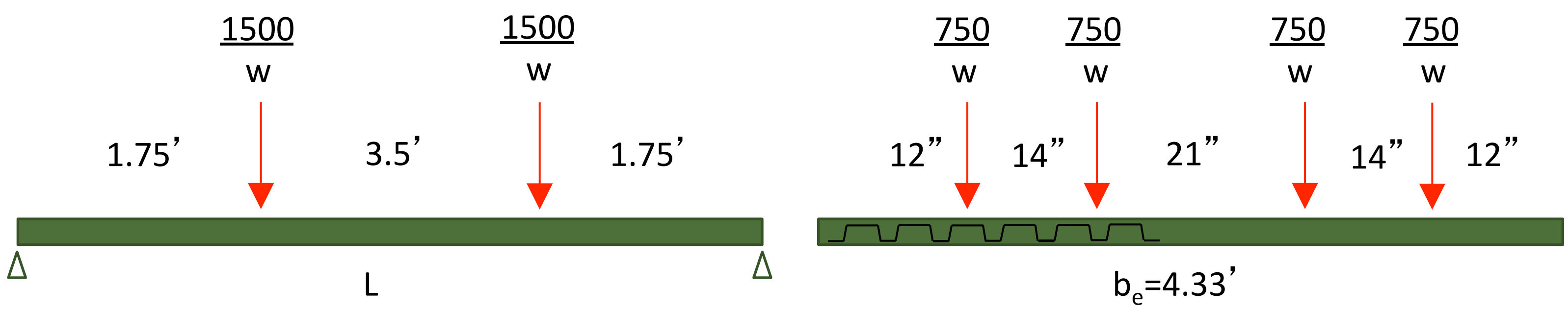
$$w = L = 7\text{ft},$$

$$b_e\ @\ 0.25L = 3.57'$$

ϕ=1.6 or 1.2?

$$\Phi M_w = \left(\frac{(1.6)4501\ \text{lbs}}{7\ \text{ft}}\right)\frac{3.57\ \text{ft}\ (12)}{15} = 2938\ \frac{\text{in}-\text{lbs}}{\text{ft}} > 2462\frac{\text{in}-\text{lbs}}{\text{ft}} \quad \text{N.G.}$$

Data Rack – M_w



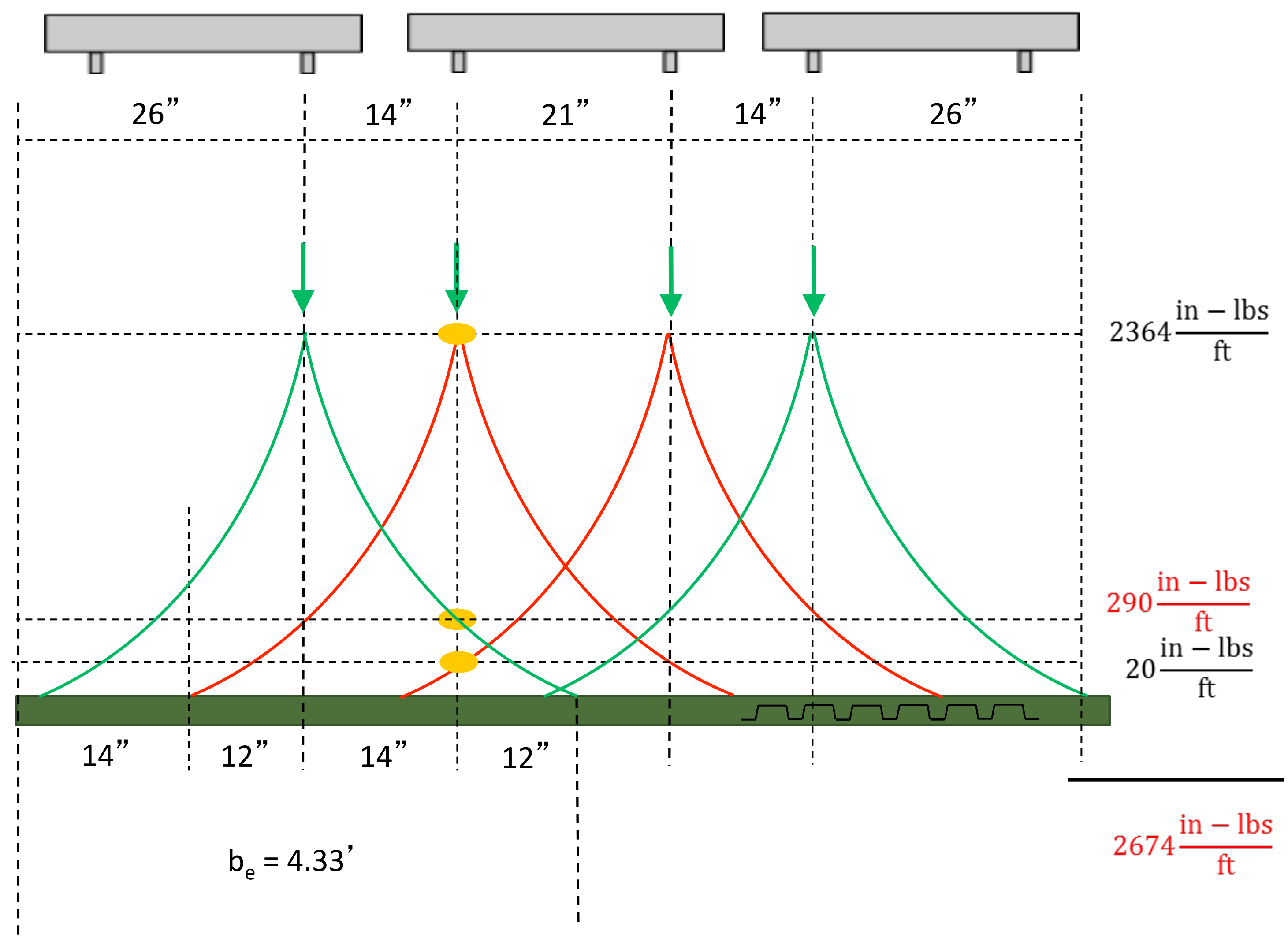
Lap = 3" so in-line correction is required.

Adjacent load spacing = 7" and 21" < $b_e/2$, so weak axis bending moments will be cumulative

$$\Phi M_w = \left(\frac{\Phi P}{w} + \frac{\Phi P(Lap)}{w^2}\right)\frac{b_e 12}{15}$$

$$\Phi M_w = 2364\,\frac{in-lbs}{ft} < 2462\,\frac{in-lbs}{ft}\ O.K$$

Data Rack – M_w - Short Axle Adjacent



Example Problem

"Can my floor support this data rack(s)?"

Regardless of data rack orientation, shear and bending capacities were more than adequate. If the data rack is considered a live load and ϕ = 1.6, weak axis bending fails. If ϕ = 1.2, weak axis bending capacity is adequate. My suggestion . . . . . drop WWR to 1.25".

Summary Page for Multiple Loads

All Cases Influence zones for data racks, lift, scaffolds will overlap.
Deflection and punching are unlikely to govern with traditional framing.
Load factors may be subjective (ϕ = 1.2, 1.4, 1.6)
FDDM tabulates ϕM_y and ϕV_n.
If slab is not restrained (no studs), consult with supplier for ϕM_n.

Beam Shear Locate one load at midspan and the short axle adjacent
Use b_e' so concrete is not used twice.
Don't forget uniform loads.

$$b'_e = \frac{b_e + \text{Load spacing}}{2} < b_e$$

Positive Bending Locate one load at midspan and the short axle in-line.
Use b_e' so concrete is not used twice.
Don't forget uniform loads.

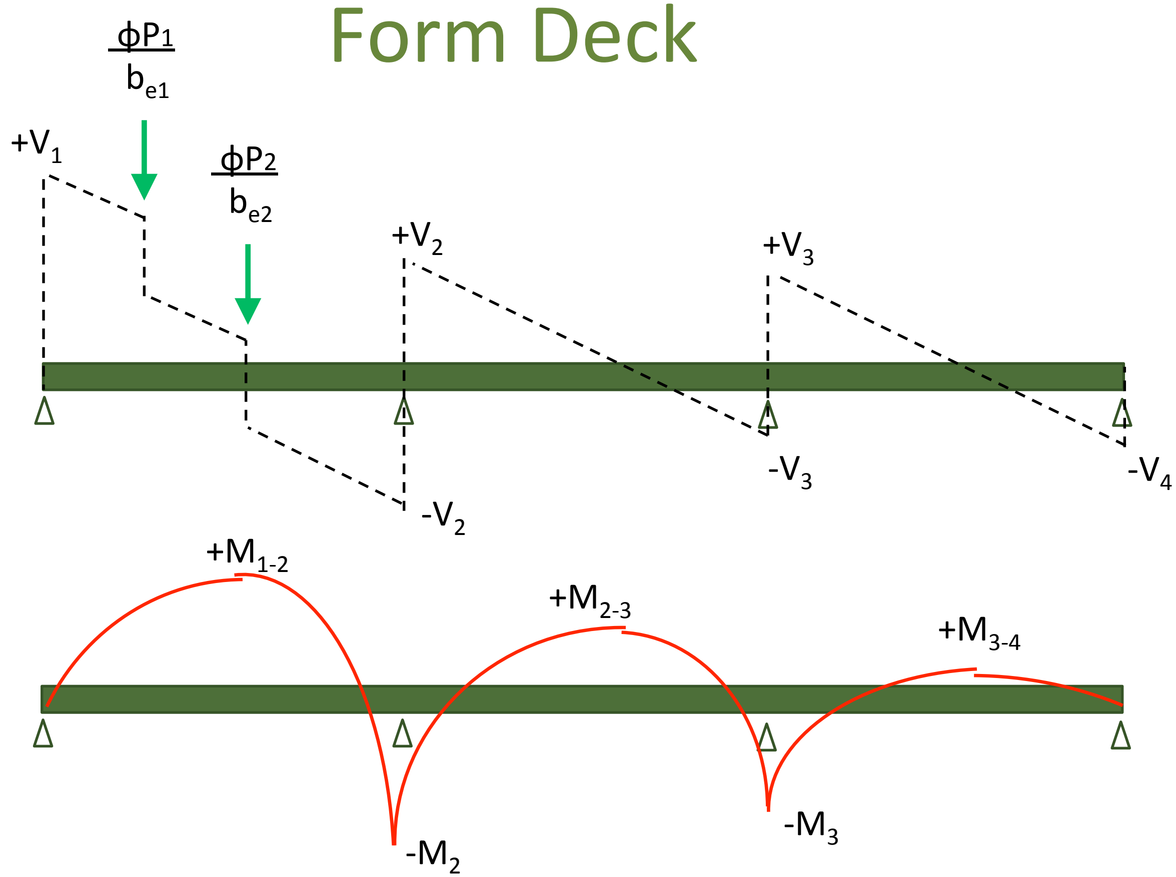
Weak Axis Bending Locate one load at midspan and the short axle in-line.

$$\Phi M_w = \left(\frac{\Phi P}{w} + \frac{\Phi P(\text{Lap})}{w^2}\right)$$

Use b_e in calculations, not b_e'
Uniform dead and live loads are supported in positive bending, so not a component of weak axis bending.
If adjacent load spacing > $b_e/2$, moments are not cumulative.
Equations compensate for "w" overlap. No other corrections are required.
If adjacent load spacing < $b_e/2$, ΣM_w using sinusoidal equation is required.

$$\Phi M_w = \left(\frac{\Phi P}{w} + \frac{\Phi P(\text{Lap})}{w^2}\right)\frac{12b_e}{15} + 5.5\,\frac{\Phi P}{w}\left(\frac{12b_e}{15}\right)\left[\frac{x}{b_{e1}} - \frac{1}{\pi}\sin\left(\frac{\pi x}{b_{e1}}\right)\right]\ \text{rad}$$

Form Deck



Prior examples were composite decks and simple spans. Form decks are typically multi-span with negative bending and interaction over the supports. Dead load (slab) is supported by the form deck, so not a variable for shear or bending; otherwise, the design approach is similar. Distribute P, compare V_{max} to V_n , $+M_{max}$ to $+M_y$ and $-M_{max}$ to $-M_y$.

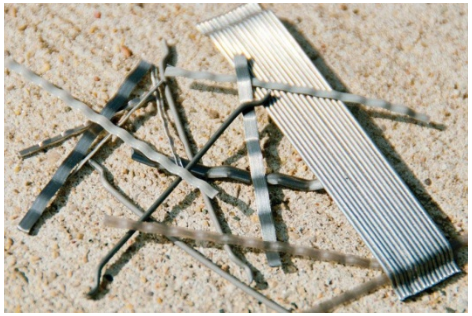
Steel Fibers

In theory, fibers are not a replacement for WWR as a tensile component, so $A_s = 0$. If so, $M_w = 0$, which suggests $P = 0$. This simply cannot be true. Load distribution with steel fibers is un-known, but old testing showed positive results. Can we *rationally* estimate load capacity with steel fibers?

•One option is ignoring the contribution of the concrete and using deck only for transverse distribution . This option reduces distribution width b_e and ϕP about 70%.

•A second option uses $b_e = 1'$. This option reduces ϕP about 75%.

A reduction in load capacity would be anticipated, but 70-75% may be conservative. Additional testing and design procedures using steel fibers is required before SDI could confidently provide guidance.



Polling Question #2

True or False… The use of shear studs on the beams will increase the allowable magnitude of concentrated loads on a slab most of the time.

a) True

b) False

Polling Question Answers

Which Limit State is NOT Applicable for Designing Concentrated Loads on Concrete Slabs on FLOOR Deck?

B) Web Crippling

True or False… The use of shear studs on the beams will increase the allowable magnitude of concentrated loads on a slab most of the time.

B) False

THANK YOU

Presented by:

Michael Martignetti, CANAM
Mike Antici, NUCOR