CANAM
Solutions + Service
Manufacturers of USD United Steel Deck products.

EXAMPLE CALCULATIONS

For this and all other example problems the following data from the Canam – United Steel Deck *Design Manual and Catalog of Steel Deck Products* will be used and are worked using LRFD methodology. A copy of the tables is provided in the Appendix and also shown on pages 42 and 43 of our *Design Manual and Catalog of Steel Deck Products.*

DECK TYPE -	2" Lok Floor Composite Floor Deck
GAGE -	20
YIELD STRESS - 40 ksi	
STEEL DECK PROPERTIES:	
I -	0.390 in^4/ft
S_p -	0.332 in^3/ft
S_n -	0.345 in^3/ft
A_s -	0.540 in^2/ft
ϕR_{be} -	800 lbs/ft
ϕR_{bi} -	1360 lbs/ft
ϕV_n -	2930 lbs/ft
w_{deck} -	1.8 psf
$\gamma_{concrete}$ -	145 pcf
$n = E_s/E_c$ -	9

ϕR_{be} is the exterior web crippling capacity based on 2.5" bearing, a phi factor of 0.9 and a 10% increase for redistribution of load. ϕR_{bi} is the interior web crippling capacity based on 5" of bearing and a phi factor of 0.85.

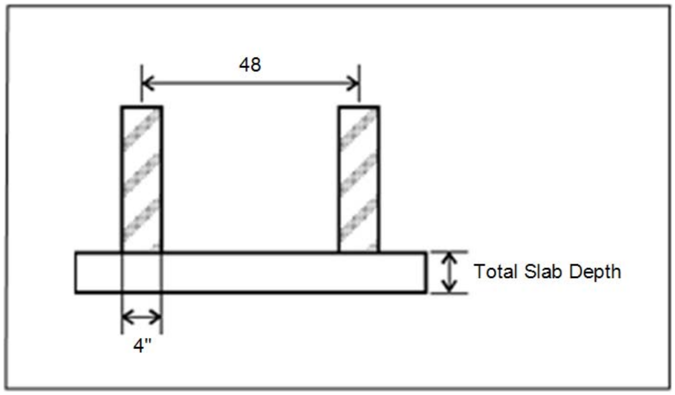
Example Problems for Concentrated Loads

Example 3: Fork Lift Truck Loading

This problem is intended to illustrate the methods of solving problems involving dynamic rolling loads. Because the very nature of these types of problems can require the designer to make individual decisions as to how some aspects of the problem will be treated, it is not intended to negate the experience of the designer. For example, the method for determining the requirement for distribution steel is newly presented in the SDI. The previous method was to simply call for distribution steel to be a percentage of the concrete area, and indeed, this experience has been retained by the SDI as a minimum requirement; however, the designer may have a method which has worked well in the past and as long as the minimums are met there is no effort by the SDI to make a change. This example problem covers different loading cases in an effort to cover most of the combinations that can occur. There may be cases that are more severe. In any case the designer must review the job's particular set of circumstances and make the judgments necessary for the design.

****NOTE – In 2010 the SDI set forth the recommendation that steel deck should be used solely as a <u>form</u> (neglecting composite action) for dynamic load applications such as forklifts. The following example is presented a historic example and final design is at the discretion of the designer.**

In an effort to prevent the concrete from developing cracks which can be “worked on” by the moving trucks, negative bending reinforcing steel is required and this part of the example is governed by the ACI rules for reinforced concrete design; the influence of the deck is simply that of the rib geometry imposed on the cross-section.



The front axle carries an 11 kip total load which includes an appropriate impact factor. The loaded back axle is negligible in this example. The deck span is 7.33 feet (88 in.) and the slab is multi-span. The tire foot print is 4" x 4" and the wheels and axles are spaced 48 in. apart. Normal weight (145 pcf) 3000 psi concrete is used.
At a glance the distribution equations in Figure 1 show that the distribution depends on the slab depth. In cases where the slab is continuous and is heavily loaded, the slab depth will be determined by negative bending. The first estimate of the slab depth is made by simply dividing the total load by the area occupied by the fork lift and loading two adjacent spans:

$$11000 / (4 \times 4) = 690 \text{ psf}; -M = wl^2 / 8 = 690(7.33)^2 / 8 = 4600 \text{ ft lbs}$$

It is a good idea to select a bar spacing that is an even division of the rib spacing so that a bar will be positioned over each rib and also so that the ribs can be used as a spacing aid. For this estimate we will assume #4 bars at 6 in. o.c. which provides an $A_s = 0.40 \text{ in}^2 / \text{ft}$. Use the historical and simple ASD formula to estimate the slab depth:

$$M = 7/8A_s(F_y \times 0.6)d = 12(4600) = 7/8(0.4)(24000)d \;;$$

and solving for d shows d ≅ 6.5 in.. With 1 in. of cover, the total slab depth would be 7.5 in., however distribution strength design may reduce the depth so 6.5 in. is selected as the first trial.

$$w_{DL} = 66 + 1.8 = 68 \text{ psf}$$

With 2" x 20 gage deck the maximum unshored span is 8.16 ft > 7.33 ft O.K.

The cover, t_c = 4.5 in., and the tire width is 4 in.:

$b_m = b_2 + 2t_c + 2t_t$
$b_m = 4 + 2(4.5) = 13$ in.

For bending distribution put the load in the center of the span:

$b_e = b_m + 4/3(1\text{-}0.5)44 = 42$ in. (for continuous slabs)

but is not to exceed $8.9(t_c/h)$ in feet $= 8.9(4.5/6.5)(12) = 74$ in.

Therefore use $b_e = 42$ in.

42/2 = 21 in. and 21 in. < 24 in. so the loaded areas do not overlap

The distributed concentrated load for bending (per foot of width) is then:

P = 5500/(42/12) = 1570 lbs

For shear distribution put the load 9 in. away from the beam centerline, which would be approximately the slab depth (6.5 in.) away from the beam edge. The distribution for shear is;

$b_e = b_m + x(1\text{-}x/l) = 13 + 9(1\text{-}9/88) = 21$ in., and the distributed concentrated load for shear (per foot of width) is then, $P_v = 5500/(21/12) = 3140$ lbs

The average bending and shear width is (42+21)/2 = 31.5 in., and the average uniform load over the span is 11000/(7.33 x 31.5/12) = 572 psf.

Loading:

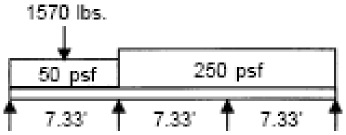
For spans where the truck is operating perpendicular to the beams include a 50 psf miscellaneous live load.

In spans where the truck is operating parallel to the beams use the uniform load based on the average of the shear and bending distribution, w = 572 psf.

In spans where the truck is not operating but has already stored material use a 250 psf storage load.

If two trucks pass parallel to each other use the uniform load of 572 psf in each span.

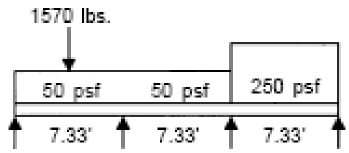
Note: Since the loading is dependent on the planned traffic pattern as well as the storage arrangement, it is apparent that the loading cases shown and the arrangement of the loads are done as examples and are not intended to address any specific design.



+M = 2320
- M = 1780
V = 1210
Δ = 0.52"

CASE I LOADING

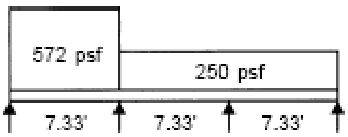
Truck ⊥ to beam at middle of exterior span. Adjacent spans fully loaded.



+M = 2590
- M = 1240
V = 1140
Δ = 0.62"

CASE II LOADING

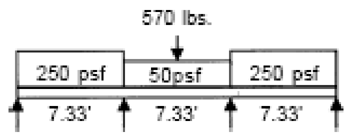
Truck ⊥ to beam at middle of exterior span. Adjacent spans loaded as shown.



+M = 2690
- M = 2500
V = 2440
Δ = 0.77"

CASE III LOADING

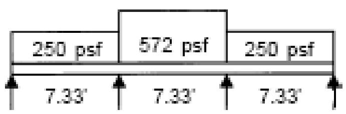
Truck | | to beam at middle of exterior span. Adjacent spans fully loaded.



+M = 1540
- M = 1670
V = 1140
Δ = 0.22"

CASE IV LOADING

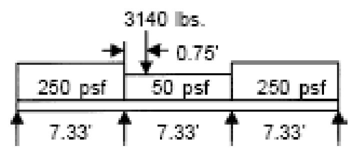
Truck ⊥ to beam at middle of interior span. Adjacent spans fully loaded.



+M = 1630
- M = 2210
V = 2100
Δ = 0.39"

CASE V LOADING

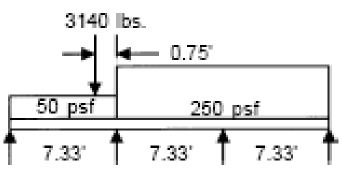
Truck | | to beam at middle of interior span. Adjacent spans fully loaded.



+M = 1150
- M = 1712
V = 3080
Δ = 0.32"

CASE VI LOADING

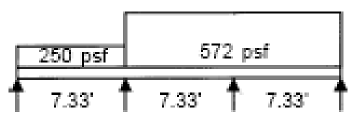
Truck ⊥ to beam, shear considered; load in interior span. Adjacent spans fully loaded.



+M = 1110
- M = 1700
V = 3230
Δ = 0.30"

CASE VII LOADING

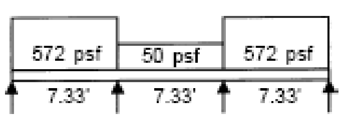
Truck ⊥ to beam, shear considered; load in exterior span. Adjacent spans fully loaded.



+M = 2320
- M = 3360
V = 2560
Δ = 0.61"

CASE VIII LOADING

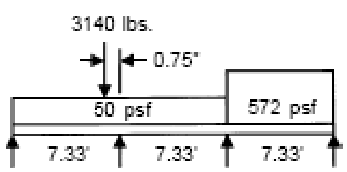
Truck | | to beam in adjacent spans. Remaining span fully loaded.



+M = 3010
- M = 1670
V = 2320
Δ = 0.93"

CASE IX LOADING

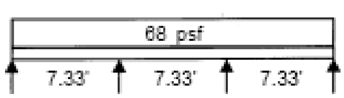
Truck | | to beam in exterior span. Interior span not loaded.



+M = 2790
- M = 1710
V = 2700
Δ = 0.84"

CASE X LOADING

Two trucks in exterior spans -- one | | to beam and one ⊥ to beam. Interior span not loaded.



+M = 293
- M = 365
V = 299

DEAD LOAD

Note: Δ is based on I = 1

Case IX shows the highest positive moment which is 3010 ft.lbs. The highest positive dead load moment is 293 ft.lbs.

Case VIII shows the highest negative moment which is 3360 ft.lbs. The highest dead load negative moment is 365 ft.lbs.

Case VII shows the worst shear to be 3230 lbs. The highest dead load shear is 299 lbs.

Check the 6.5 in. slab with 20 gage 2" x 12" deck to see if it meets the +M and V requirements:

M_{needed} = (1.6 x 3010 + 1.2 x 293)12/1000 = 62.01 in.kips

$+M_{no}$ (with no studs) = 74.31 in.kips > 62.01 in.kips **O.K.** *74.31/62.01 = 1.2*

$+M_{nf}$ (full moment capacity) = 94.50 in.kips *94.50/62.01 = 1.5*

V_{needed} = 1.6 x 3230 + 1.2 x 299 = 5530 lbs

V from the tables = 7920 lbs

7920 > 5530 **O.K.**

Design the negative reinforcing required - the φ factors and the load factors now become the ACI values:

$-M = 0.9A_sFy(d - a/2)$, where 0.9 is the φ factor. Try #4 bars at 6 in. o.c.

$A_s = 0.40$ in^2/ft

b = 6 in. (the width of the compression block, the average of the rib dimension)

$a = A_sF_y / (0.85f'_cb) = 0.40(40000) / (0.85 \times 3000 \times 6) = 1.05$ in.

$-M = 0.9(0.40)(40000)(5.5 - 1.05/2) = 71640$ in.lbs

M_{needed} = [1.6 x 3360 + 1.2 x 365]12 = 69768 in.lbs

69768 < 71640 **O.K.**

Use a 6.5 in. slab with #4 rebars at 6 in. o.c.

If the moment capacity does not exceed the needed moment try increasing the slab depth by ½ in. Check that the maximum unshored span is still acceptable and see if studs are necessary by examining the positive bending resistance.

Design the distribution steel:

The A_s (in the transverse direction) must be equal to or greater than $0.00075t_c(12)$, where t_c is the concrete cover. So, for the first try use wire mesh that is the closest to 0.045 in.^{2}. The mesh designated 6 x 6 - W2.9 x W2.9 provides A_s = 0.058 in.^{2}. This is commonly used mesh.

Bending in the weak direction = PB / (15W) where B = b_e in the bending and W = the tire dimension (b_3) plus l / 2; W = 4 + 88/2 = 48 in.. The b_e = 13 + 4/3(1 - 0.5)44 = 42 in.

M_{weak} = 5500(42) / (15 x 48) = 321 in.lbs (per inch) = 3850 in.lbs (per foot).

Assuming the center of the wire mesh is 1/2 in. above the deck, d = 4.5 - 0.5 = 4.0 in., and
a = 0.058(60000 / (0.85 x 3000 x 12)) = 0.114 in. (note the F_y of welded wire mesh is 60 ksi).

$M_{resisting}$ = 0.9 x 0.058(60000)(4.0 - 0.114/2) = 12350 in.lbs

3850 x 1.6 = 6160 in.lbs (note the dead load is carried by the deck)

12350 > 6160 so the minimum wire mesh is **O.K.**

Because the rolling load can cross over a steel beam, the composite slab would have an "anvil" below it and punching shear becomes part of the problem.

$P / [2(b_2 + b_3 + 2t_c)t_c] \leq \varphi 2(f'_c)^{1/2}$;   where φ= 0.75 (per ACI 318-08)

P = 5500 x 1.6 = 8800 lbs

$8800/[2(4 + 4 + 2(4.5))4.5] \leq \varphi 2(f'_c)$; $\varphi 2(f'_c)^{1/2}$ = 82 psi

8800 / 153 = 58 psi < 82 **O.K.**

Check deflection:

I_{av} for the 6.5 in. slab is (from table) = 18.2 in^4 (per foot of width)

$I_{cracked}$ for negative moment region:

$\rho = 0.4/(6 \times 5.5) = 0.012$ k = $\sqrt{2 \times 9 \times 0.012 + (0.012)^2} - (9 \times 0.012) = 0.357$

$$I_{cracked} = \frac{(0.357 \times 5.5)^3}{3} \times \frac{6}{9} + [(1 - 0.357) \times 5.5]^2 \times 0.4 = 6.7\,\text{in}^4$$

Because continuity is being used: Let I_{av} = (6.7 + 2 x 18.2)/3 = 14.4 in^4

The maximum Δ = 0.93 in. with I = 1 (case IX). With I = 14.4, Δ = 0.93 / 14.4 = 0.065 in..

7.33(12) / 0.065 = 1350; Δ = l / 1350 which is **O.K.**

No attempt is made to finish the re-bar design for required lengths or the number of bars to be carried continuously as this is concrete design.

The design summary is:

2" x 12" 20 gage composite floor deck with a 6.5 in. (total depth) slab.
6 x 6 - W2.9 x W2.9 is the distribution steel to be placed on the deck. Negative rebars are #4 @ 6 in. o.c. with 3/4 in. cover.

Note: A 6 ½" slab without studs is acceptable. However, if the designer considers the truck loading to be repetitive so the slab will be subjected to many cycles of loading, it is suggested that studs be required and that a thicker slab be investigated so the resistance exceeds the demand by a greater margin. Studs may be required for beam design.

****NOTE – In 2010 the SDI set forth the recommendation that steel deck should be used solely as a <u>form</u> (neglecting composite action) for dynamic load applications such as forklifts. The following example is presented a historic example and final design is at the discretion of the designer.**