Parking Structure with a Post-tensioned Deck

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The current trend toward structural steel parking structures with a pre-stress deck evokes many design questions. Some of these questions concern the following topics: 1. design loads; 2. stability; 3. deck design; 4. beam design with emphasis on long-term stresses and deflections.

All of these topics, except for long-term stresses and deflections in beams, especially those induced by prestressing, are discussed in the open literature.[2,3,9,10] Consequently, the author addresses the first three topics and refers the reader to pertinent publications. On the other hand, the last topic, rarely discussed in the open literature, is discussed comprehensively by the author.

GENERAL

Parking structures can be constructed of composite and/or non-composite sections. These sections can consist of wide-flange and/or castellated-beam sections.[8] The final beam section and mode of construction depends on architectural, zoning and economic considerations. Often zoning requirements restrict the building height and headroom allowance and mandate composite construction.

The concrete deck can be either conventional or pre-stressed. Future deck repair and maintenance costs are very significant factors in the initial deck selection. Consequently, crack control and the elimination of water penetration are essential. Therefore, a post-tensioned deck with its crack control capability is an excellent solution. However, the pre-stress forces in the deck cause elastic and creep shortening in the slabs and long-term stresses and deflections in the beams. These pre-stress effects and the means for including them in the design are discussed in the following paragraphs.

DESIGN LOADS

Gravity loads include both dead and live loads. The live loads prescribed in the governing building codes should be used; however, the National Parking Association Parking Consultants Council[7] recommends these minimum design load requirements:

1. The design live load should be the greater of 50 psf or a one-foot square, 2,000-lb. concentrated load placed at any location in the driving and parking areas.
2. Live loads to floor slabs should not be reduced.
3. Live loads to columns, girders, walls, beams and joist may by reduced in accordance with the code to a minimum of 30 psf.
4. Barrier shall be capable of resisting an ultimate horizontal load (minimum) of 10,000 pounds applied 1 ft-6 in. above the floor at any point. The barrier loading should not be reduced.
5. Snow loads should be added to the reduced live loads where vehicles park on the roof.
6. Seismic and other loads should be applied where applicable.

To establish wind loads, the code specified wind pressures should be taken over the gross vertical surface[7] without deductions for openings because the wind is obstructed inside the structure by columns, beams, barriers and ramps.

Thermal loads are obtained by performing a volume analysis of the structure. To expedite this analysis, thermal strains are input into one of the many structural design computer programs. These thermal strains are presented in the open literature,[5] but are usually softened and do not take into account the high surface temperatures caused by direct sun exposure on the upper parking levels. For example, a design handbook[5] specifies a maximum climatic seasonal differential of 60°F. for mid-western U.S., while an actual project design guide for the same area specified 130°F. Consequently, it is a good practice for a firm involved in parking deck design to monitor existing structures.

Loads induced by volume changes are especially significant in pre-stressed concrete decks. Elastic, shrinkage and creep strains are readily available in the open literature.[5] These strains like thermal strains are used as input data for a computer aided analysis. Proper selection of expansion

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joints can reduce the build up of internal stresses, but not eliminate them.

STRUCTURAL STABILITY

In accordance with the AISC allowable stress design specification,[2,3] there are three basic types of steel connections: Type 1—Rigid connections; Type 2—Flexible connections; and Type 3—Semi-rigid connections. The three types of construction may be used alone or in combination, where permitted by building codes, to provide the structure with its lateral load-resisting capability.

Lateral loads acting on a structure include all external applied loads specified in the codes and the internal loads required to stabilize the structure. Prof. Galambos[14] established formulas to calculate these lateral loads necessary for stability.

In most cases, the structure must be completed before stability is achieved. Also, in composite design, the beams are often assumed to have full lateral support under dead and live loads. Consequently, it is essential these facts be clearly stated in the contract documents.

Composite beams obtain their lateral support from their embedded shear connections. Therefore, provisions must be made during the placement of the concrete deck to either laterally support the beam, reduce its span or reduce the applied dead load.

To laterally support the beams, some manufacturers and design-build companies developed form systems that support the concrete deck forms with joists attached to the composite beams. The end result is a horizontal diaphragm developed by the connections between the composite beams and joists and the stiffness of the forms.

The beam span is reduced by shoring the beam, while the dead load is reduced by shoring the deck forms. In either case, the need for shoring must be conveyed clearly on the drawings.

The question of lateral support and shoring must be addressed during the preliminary design stage. The designer should seek the input of the construction manager (if selected), local contractors, fabricators and form companies. Because the final course of action is usually based on economics, the input of contractors is so important.

POST-TENSIONED DECK

Post-tensioning has been very successful in parking structures because it can produce a crack-free parking surface. However, it requires proper detailing and rigid quality control. Consequently, the designer should develop an awareness for these points:

1. Proper detailing for volume changes
2. Separation of the main structure from stiff elements
3. Pour strips to reduce the creep and shrinkage accumulations
4. Well distributed mild reinforcing should be provided for crack control in the areas where cracks are expected.
5. Average prestress in the slab should be a minimum of 200 psi. in both directions. Under the real service load (25 to 30 psf), no tension should exist anywhere.
6. An expansion joint should, preferably, be provided at a maximum interval of 150 feet.
7. Shear walls when structurally required should be located as close as possible to the center of rigidity.

In a later publication,[16] the American Concrete Institute specifies an effective prestress of 100 psi in the shrinkage and temperature direction. This criteria is equally applicable to one-way concrete decks supported by steel beams. Furthermore, it reduces the effects of post-tension parallel to the beams.

Also, the following criteria, encountered by the author in a design guide package for a parking structure should be considered:

1. Minimum depth of slab = 5 in.
2. Maximum span to depth ratio of slabs should be 1/40 for simple spans and 1/45 for continuous spans.

In conjunction with these recommendations, a post-tensioned slab in a parking structure must be designed in accordance with the requirements of the Post-Tension Institute and the American Concrete Institute.[11,12,15,16]

BEAM DESIGN

In parking structures, the floor beams are designed as either composite or non-composite members. In both cases, the beams are commonly treated as simply supported members. Wind resistance and lateral stability are provided by bracing, shear walls, flexible or semi-rigid wind-resistant connections. Of the two modes of construction, composite construction is very economical for parking deck spans (50 to 60 ft). AISC[2] recommends the gross depth-to-span ratios of $f_Y/800$ (floors) and $f_Y/1000$ (roofs) be used as minimum guide.

Of particular concern in composite construction are the effects of differential volume changes between the concrete slab and the steel beams. These effects can be evaluated by using the following method. Unlike other methods, the most common being the "composite section method,"[1] this method takes into account the actual stiffnesses of the steel and concrete. The composite section method assumes the steel beam is infinitely rigid. Consequently, the restraining forces generated by the steel beam are extremely high and not realistic.

The following expressions for stress, moment and deflection are based upon the assumption that, under full composite action, the strain in the concrete slab equals that in the steel beam at the concrete/steel interface, i.e.

no slip. Hence,

$$STR^{C}_{TOT} = STR^{S}_{TOT} \tag{1.1}$$

which represents the compatibility equation.

The equilibrium expression (Fig. 1) is derived from the fact the unrestrained force (P_{TOT}) in the concrete slab due to volume changes is resisted by the internal forces of the steel beam (P_S) and concrete slab (P_C). Hence,

$$P_{TOT} = P_S + P_C \tag{1.2}$$

Equation 1.2, after substituting the appropriate areas, elastic modulus, strains and rearranging the terms becomes

$$A_C E_C STR_{TOT} - A_S E_S STR^{S}_{TOT} = A_C E_C STR^{C}_{TOT} \tag{1.3}$$

Substituting STR^{C}_{TOT} for STR^{S}_{TOT} to ensure compatibility, the above expression becomes

$$A_C E_C STR_{TOT} - A_S E_S STR^{C}_{TOT} = A_C E_C STR^{C}_{TOT} \tag{1.4}$$

After rearranging the terms in Eq. 1.4,

$$A_C E_C STR_{TOT} = STR^{C}_{TOT}\,(A_S E_S + A_C E_C) \tag{1.5}$$

Now solving for the concrete strain, Eq. 1.5 becomes,

$$(A_C E_C STR_{TOT})/(A_S E_S + A_C E_C) = STR^{C}_{TOT} = STR^{S}_{TOT} \tag{1.6}$$

In accordance with these equations and Fig. 1, the stresses and deflections due to differential volume changes are given by these equations: (Compression = positive)

Slab stresses

$$f_T(t/c) = P_C/A_C + P_S \times e/(S_{TR}(t/c) \times n) \tag{1.7}$$

$$f_B(b/c) = P_C/A_C + P_S \times e/(S_{TR}(b/c) \times n) \tag{1.8}$$

Beam stresses

$$f_T(t/s) = P_S/A_S + P_S \times e/S_{TR}(t/s) \tag{1.9}$$

$$f_T(b/s) = P_S/A_S - P_S \times e/S_{TR}(b/s) \tag{1.10}$$

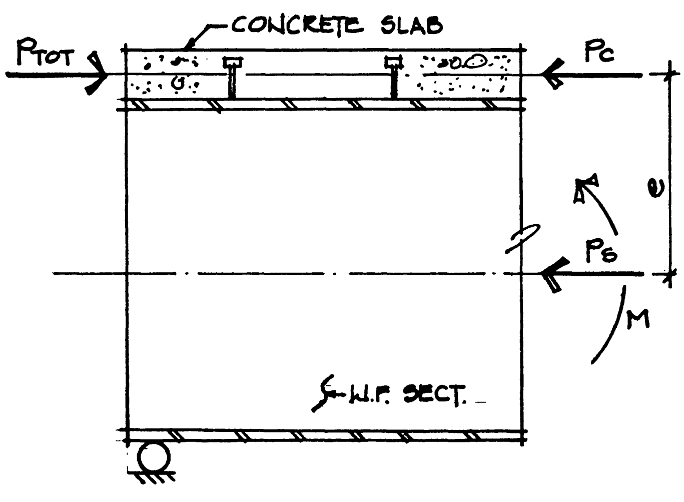

Fig. 1. Free body diagram

The mid-span deflection is evaluated assuming a constant moment, $P_S \times e$, by the equation:

$$D_{VOL} = P_S \times e \times (L)^2/8\ E_S I_{TR} \tag{1.11}$$

"Generally, the effects of differential shrinkage are relieved by the relaxation of concrete which depends on the creep characteristics of the concrete which can be described by the creep coefficient C_T. It has, therefore, been suggested that the calculated stresses and deflections be reduced by a factor R."[1]

After the long term stresses and deflections are quantified, the adequacy of the composite shear connectors should be evaluated. This evaluation is accomplished by determining the final slab axial force V_H:

$$V_H = .67 \times A_c \times (f_T(t/c) + f_T(b/c))/2.0 \tag{1.12}$$

where the factor .67 averages the flexural stresses in a parabolic distribution from zero stress at the beam's end to maximum stress at mid-span.

The axial force V_H must be resisted by the shear capacity Q of the shear connectors, where Q is defined by this expression:

$$Q = (N \times q)/2 \tag{1.13}$$

In addition to the shear connectors, the beam end connections should be investigated for wind, gravity and long-term effects. Because many connections and connection types exist, this discussion focuses on Type II connections only. This connection type is most often used in parking structures by designers because of ease of design.

In accordance with Blodgett,[2] two methods can be used to design Type II connections. Method 1 includes the gravity and wind loadings in the connection design. In Method 2, the connection is designed to resist wind moments while the larger moments induced by gravity loads are relieved by connection deformation. In either method, the loading due to long term effects should be included in the design. Refer to the design example in Appendix I.

SUMMARY

Long-term composite beam stresses and deflections were often underestimated in parking structures with conventional concrete decks.

Post-tension concrete decks are replacing the conventional concrete decks in new parking structures. The prestress forces in these decks minimize crack formation in the slab, but create differential volume changes between the deck and the beam. These volume changes, in turn, create long-term beam and slab stresses and beam deflections which can be evaluated by the preceding method.

APPENDIX I—DESIGN EXAMPLE

Figure 2 shows one half of the floor plan of a parking structure. An expansion joint is provided at mid-length (col. line 9) and a slip joint is provided at mid-width (col. line D). Both joints reduce the build up of stresses—especially at the far corner columns where biaxial bending is induced by two-dimensional volume changes. The absence of the slab joint produces an unbroken slab between column lines 1 and 3 of 240 ft, which is undesireable.

Methodology

The following design example illustrates the procedure used to determine the long-term stresses and deflections for a typical beam in the parking structure of Fig. 2.

The cross section of a typical interior beam is shown in Fig. 3.

Design Data

$L = 60'\text{-}0''$	$W33 \times 118$ with	$I_{TR} = 13{,}586$ in.^{4}
$s = 17'\text{-}6''$		$S_{TR} = 482$ in.^{3}
$f'_C = 4{,}000$ psi		$y_B = 28.2$ in.
$f_Y = 50{,}000$ psi		$I_S = 5{,}900$ in.^{4}
$t = 5$ in.		$A_{TR} = 89.8$ in.^{2}
$E_C = 3.64 \times 10^6$ psi		$A_S = 34.7$ in.^{2}

1. Dead load (non-composite action unshored):

The section properties of the steel beam alone are used to calculate the dead-load stresses and deflections during construction.

$$W_{DL} = 1{,}200 \text{ lbs./ft}$$
$$S_S = 359 \text{ in.}^3$$
$$I_S = 5{,}900 \text{ in.}^4$$

Hence, the stresses in the beam are

$$f_B = 1{,}200(60)^2(1.5)/359 = 18{,}050 \text{ psi} \quad \textbf{o.k.}$$

and the dead-load deflection is

$$D_{DL} = (5\times 1{,}200\times(60)^4\times(12)^3)/(384\times 30\times 10^6\times 5900) = 2.00 \text{ in.}$$

2. Live load:

The live-load stresses and deflections are based upon the transformed section properties:

$$W_{LL} = 525 \text{ lbs./ft}$$
$$S_{TR} = 482 \text{ in.}^3$$
$$I_{Tr} = 13{,}586 \text{ in.}^4$$
$$n = 8.3$$

Hence, the stresses in the transformed section are,

$$f_T(t/c) = (525(60)^2(1.5))/((8.3)(1306)) = 247 \text{ psi} \quad (C)$$
$$f_T(t/s) = 525(60)^2(1.5)/2830 = 1{,}002 \text{ psi} \quad (C)$$
$$f_T(b/c) = 1002/8.3 = 121 \text{ psi} \quad (C)$$
$$f_T(b/s) = 525(60)^2(1.5)/482 = 5{,}882 \text{ psi} \quad (T)$$

and the live load deflection is,

$$D_{LL} = (5(525)(60)^4(12)^3)/(384(30\times 10^6)(13{,}586)) = .38 \text{ in.}$$

3. Dead load (composite action):

In this example, the effects of dead load on the composite section for immediate and long term effects do not have to be considered. The steel section alone carries the dead load of the slab.

On the other hand, the immediate and long-term effects

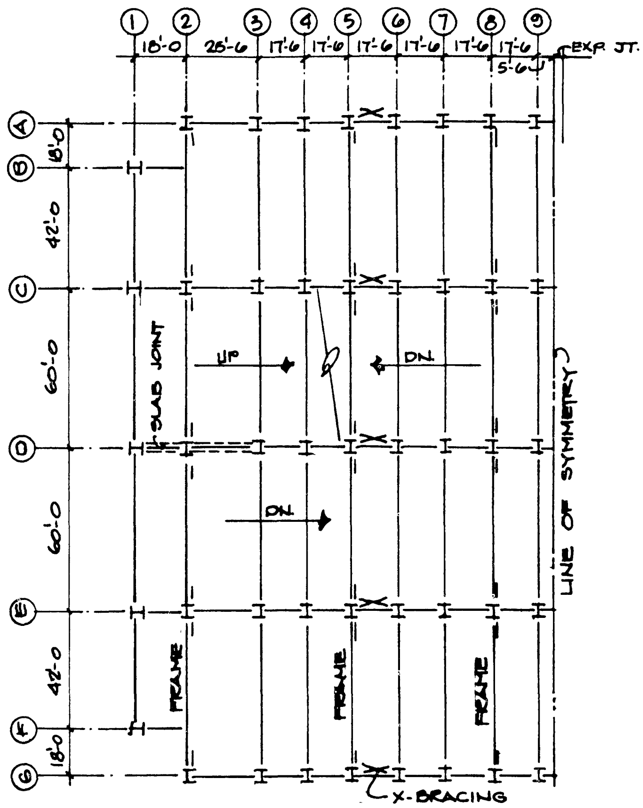

Fig. 2. Partial floor framing plan

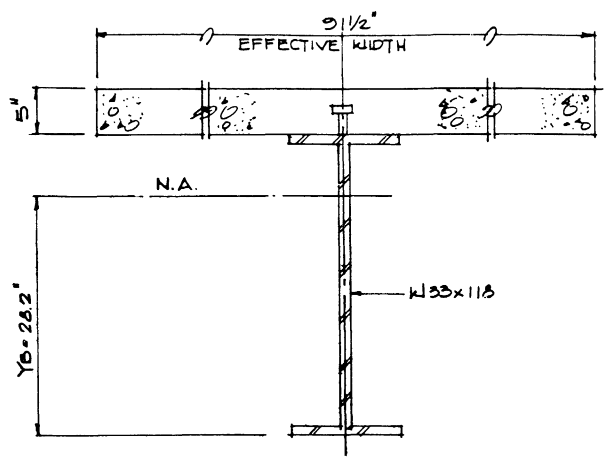

Fig. 3. Typical composite beam cross section

due to dead load must be considered for shored beams. After the shores are removed, the dead load is transferred to the transformed section. Consequently, the concrete slab is placed in long-term compression.

Although the dead-load stresses act on the steel section, the AISC Specification[3] permits the use of the transformed section properties to calculate and check dead-load stresses. Hence, dead-load stresses in the transformed section can be determined by multiplying the stresses in Sect. 2 (live load) above by the ratio of dead load to live load.

$f_T(t/c)$ = (1,200/525)247 = 565 psi (C)
$f_T(t/s)$ = (1,200/525)1,002 = 2,290 psi (C)
$f_T(b/c)$ = (1,200/525)121 = 277 psi (C)
$f_T(b/s)$ = (1,200/525)5,882 = 13,445 psi (T)

4. Stress summary (composite action):

$f_T(t/c)$ = 247 + 565 = 812 psi (C) less than $0.45 f'_C$ = 1,800 psi **o.k.**
$f_T(b/s)$ = 5,882 + 13,445 = 19,327 psi (T) less than 0.66(50,000 psi) = 33,000 psi **o.k.**

5. Elastic, shrinkage and creep strains:

The elastic strain produced by an effective post-tension stress of 100 psi for shrinkage and temperature control is,

STR_{EL} = 100 psi/3.64×10^6 = 2.8×10^{-5} in./in.

The shrinkage strain obtained from Table 4.4.1 of Ref. 5 is

$STR_{SH} = 5.10 \times 10^{-4}$ in./in.

However, correction factors of .63 = $(1-R)$ from p. 197[1] for creep relaxation and .90 from Table 4.4.4.[5] for a higher volume to surface ratio (V/S = 2.5) must be applied. Therefore,

$STR_{SH} = 5.10 \times 10^{-4}(.9)(.63) = 2.85 \times 10^{-4}$ in./in.

The creep strain from Table 4.4.11 of Ref. 5 is,

$STR_{CR} = 5.24 \times 10^{-4}$ in./in.

The correction factors for creep are based on the average[1] compressive stress in the slab due to dead load (0 psi) plus post-tensioning (100 psi). Therefore, the correction factors of .100 from Table 4.4.2.[5] for a lower equivalent prestress of 100 psi, .88 from Table 4.4.4.[5] for a higher volume to surface ratio and .63 from the above paragraph must be applied. Hence,

$STR_{CR} = (.63)(.10)(.88)5.24 \times 10^{-4}$ in./in.
$= 2.91 \times 10^{-5}$ in./in.

Then, the total differential strain is,

$STR_{TOT} = 3.42 \times 10^{-4}$ in./in.

6. Axial forces:

$P_{TOT} = 457.5 \times (3.64 \times 10^6) \times (3.42 \times 10^{-4})$
$= 5.70 \times 10^5$ lbs.
$A_SE_S = 34.7 \times (30 \times 10^6) = 1.04 \times 10^9$ lbs.
$A_CE_C = 457.5 \times (3.64 \times 10^6) = 1.67 \times 10^9$ lbs.
$STR^C_{TOT} = 5.70 \times 10^5/(1.04 \times 10^9 + 1.67 \times 10^9) =$
2.10×10^{-4} in./in.
$P_C = (1.67 \times 10^9)(2.10 \times 10^{-4}) = 3.51 \times 10^5$ lbs.
$P_S = (1.04 \times 10^9)(2.10 \times 10^{-4}) = 2.18 \times 10^5$ lbs.

7. Deflection:

$e = (5 + 32.86)/2 = 18.93$ in.
$D_{VOL} = (2.18 \times 10^5)(18.93)(60)^2(12)^2/(8(30 \times 10^6)(13{,}586)) = 0.66$ in.

8. Stresses due to volume changes:

Slab Stresses:

$f_T(t/c) = 3.51 \times 10^5/457.5 + (2.18\times10^5)(18.93)(9.66)/(8.3 \times 13{,}586)$
= 767 + 354 = 1,121 psi (C)

$f_T(b/c) = 767 + (2.18 \times 10^5)(18.93)(4.66)/(8.3 \times 13{,}586)$ = 767 + 171 = 938 psi (C)

Beam Stresses:

$f_T(t/s) = 2.18 \times 10^5/34.7 + (2.18 \times 10^5)(18.93)(4.66)/13{,}586$ = 6,282 + 1,415
= 7,697 psi (C)

$f_T(b/s) = 6{,}282 - (2.18 \times 10^5)(18.93)(28.2)/13{,}586$
= 6,282 − 8,566
= 2,284 (T)

9. Stress summary (long term):

In this section, the actual stresses acting on the concrete and steel sections are used.

Slab Stresses:

$f_T(t/c)$ = 247 + 1,121
= 1,368 psi (C) less than $0.45 f'_C$

$f_T(b/c)$ = 121 + 938
= 1,059 psi (C) less than 1,800 psi

Steel Stresses:

$f_T(t/s)$ = 18,050 + 1,002 + 7,697 = 26,749 psi (C)
$f_T(b/s)$ = 18,050 + 5,882 + 2,284 = 26,216 psi (T)

In both locations, the stress is less than 0.66(50,000) = 33,000 psi.

10. Deflection Summary:

D_{TOT} = 2.00 + .38 + 0.66 = 3.04 in.

Cambering the beam 2.00 in. results in a deflection of 1.04 in. (L/692).

11. Shear Studs:

For total composite action, the number of ¾-in. dia. shear studs required for one-half span is determined from the following equations:[3]

$N_s = (34.7 \times 50)/(2 \times 13.3) = 65.2$
$N_c = ((4 \times .85)/(13.3 \times 2)) \times 5 \times 91.5 = 59$

Therefore, the total number of studs for the full span is 118. From Ref. 3, the allowable shear load per stud is 13.3 kips. Hence, the shear capacity for one-half span Q from Eq. 1.8 is:

$Q = 118 \times 13.3/2 = 784.7$ kips

From Eq. 1.7, the maximum slab axial force V_H is:

$V_H = .67(5)(91.5)(1{,}368) + 1{,}059)/2 =$
371,968 lbs. or 372 kips less than 784.7 kips provided **o.k.**

12. Connection:

At the second level, the wind moment at the west end of the beam on column line 5 between column lines A and C is 60.0 kip ft. The positive moment due to dead plus live loads is 776.3 kip ft and the positive moment due to long term effects is 345.2 kip ft.

In accordance with Method 1 of Blodgett,[2] the force F on the top plate is:

$F = ((.25(776.3 + 345.2) + 60.0)12)/33 = 124$ kips

The top plate required area is:

$A_{PL} = 124 \text{ kips}/(1.33(.6)(36)) = 4.3$ sq. in.

Therefore, use a A36 plate 9 in. wide by ½ in. thick.

Method 1 is chosen simply to illustrate how the long-term effects are accounted for in connection design. Methods 1 and 2, presented by Blodgett,[2] are equally accepted by AISC.

APPENDIX II—REFERENCES

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APPENDIX III—NOTATION

1.	A_C	= Area of concrete
2.	A_{TR}	= Area of transformed section
3.	C_T	= creep coefficient[1]
4.	D_{DL}	= dead-load deflection
5.	D_{LL}	= live-load deflection
6.	D_{VOL}	= differential volume change deflection
7.	e	= distance between centroid of slab and centroid of steel beam
8.	E_C	= concrete modulus of elasticity
9.	E_S	= steel modulus of elasticity
10.	f_T	= flexural stresses in transformed section
11.	f_B	= flexural stress in non-composite steel section
12.	f'_C	= compressive strength of concrete at 28 days
13.	$f_T(b/c)$	= flexural stress at bottom of slab
14.	$f_T(b/s)$	= flexural stress at bottom of steel section
15.	$f_T(t/c)$	= flexural stress at top of slab
16.	$f_T(t/s)$	= flexural stress at top of steel section
17.	f_Y	= steel yield stress
18.	I_S	= moment of inertia of steel section
19.	I_{TR}	= moment of inertia of transformed section
20.	L	= span length of beam
21.	n	= ratio of E_S/E_C
22.	N	= total number of shear connectors

23. P_C = force in slab induced by differential volume change
24. P_S = force in beam induced by differential volume change
25. P_{TOT} = unrestrained force induced by differential volume change
26. q = allowable shear per connector
27. R = creep reduction factor = $(1 - e^{-C}T)/C_T$[1]
28. s = beam spacing
29. S_S = section modulus of steel section
30. S_{TR} = transformed section modulus
31. $S_{TR}(b/c)$ = transformed section modulus at bottom of concrete slab
32. $S_{TR}(b/s)$ = transformed section modulus at bottom of steel section
33. $S_{TR}(t/c)$ = transformed section modulus at top of concrete slab
34. $S_{TR}(t/s)$ = transformed section modulus at top of steel section
35. STR_{CR} = differential creep strain
36. STR_{EL} = differential elastic strain
37. STR_{SH} = differential shrinkage strain
38. STR_{TOT} = total differential strain
39. STR^C_{TOT} = total differential strain in slab
40. STR^S_{TOT} = total differential strain in beam
41. STR_{VOL} = differential strain induced by volume change
42. W_{DL} = dead load per foot
43. W_{LL} = live load per foot
44. y_B = location of transformed section center of gravity above bottom of steel section