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Development of prestressed composite floor slabs constructed with cold-formed steel decking

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Development of prestressed composite floor slabs

constructed with cold-formed steel decking

by

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DEDICATION

This thesis is dedicated to my parents, Boujemaa and Mabrouka Hedhli.

LIST OF SYMBOLS

a = Depth of equivalent rectangular stress, in.

A_c = Cross-sectional area of composite deck, in.^{2}

A_p = Cross-sectional area of precast deck, in.^{2}

A_{ps} = Prestressing steel area, in.^{2}

A_s = Cross-sectional area of steel deck, in.^{2}

b = Design width of the member, in.

b_1 = Width of down corrugation, in.

b_2 = Width of upper corrugation, in.

d_p = Distance from extreme compression fiber to centroid of prestressed reinforcement, in.

d_s = Distance from extreme compression, in.

e = Distance from neutral axis to centroid of prestressed reinforcement, in.

E_c = Modulus of elasticity of concrete, ksi.

E_s = Modulus of elasticity of steel deck, ksi.

F = Prestress force, kips.

f_{bc} = Stress in the bottom fiber of the composite deck cross section, ksi.

f_{bp} = Stress in the bottom fiber of the precast deck cross section, ksi.

f_{ps} = Stress in prestressed reinforcement at nominal strength, ksi.

f_{pu} = Ultimate strength of prestressing steel, ksi.

f_{se} = Effective stress in prestressing steel after losses, ksi.

f_{tc} = Stress in the top fiber of the composite deck cross section, ksi.

f_{tp} = Stress in the top fober of the precast deck cross section, ksi.

f'_c = Concrete compressive strength, psi.

f'_{ci} = Compressive strength of concrete at the time of initial prestress, psi.

H_1 = Depth of precast concrete above the steel corrugation, in.

H_2 = Depth of topping concrete, in.

H_s = Depth of the steel corrugation, in.

I_c = Moment of inertia of the composite deck, $in.^4$

I_p = Moment of inertia of the precast deck, $in.^4$

I_s = Moment of inertia of the steel deck, $in.^4$

L = Span length, ft.

LL = Allowable live load, $lbs./ft.^2$

M_p = Moment due to weight of the precast deck, k-ft.

M_t = Moment due to weight of the topping, k-ft.

M_u = Moment due to live load, k-ft.

M_n = Nominal moment strength of the composite deck, k-ft.

M_u = Applied factored moment at a section, k-ft.

n = Modular ratio = E_s/E_c.

N_o = Number of strands.

S_{bc} = Section modulus with respect to the bottom fiber of the composite deck cross section, $in.^3$

S_{bp} = Section modulus with respect to the bottom fiber of the composite deck cross section, $in.^3$

S_{tc} = Section modulus with respect to the top fiber of the composite deck cross section, $in.^3$

S_{tp} = Section modulus with respect to the top fiber of the precast deck cross section, $in.^3$

T_p = Tensile force at the prestressing tendons, kips.

T_s = Tensile force at the centroid of the steel deck, kips.

W = Width of steel deck, in.

W_{LL} = Allowable live load based on deflection limitations, $lbs./ft.^2$

W_p = Weight of precast deck, lbs./ft.

W_t = Weight of topping concrete, lbs./ft.

W_s = Weight of steel deck, lbs./ft.

W_c = Weight of composite deck, lbs./ft.

y_c = Distance from neutral axis of composite section to bottom fiber of the slab, in.

y_p = Distance from neutral axis of precast section to bottom fiber of the slab, in.

y_s = Distance from neutral axis of steel deck to bottom fiber of the slab, in.

ρ_p = A_{ps}/bd = ratio of prestressed reinforcement.

ϕ = Strength reduction factor.

1. INTRODUCTION

1.1. Background

Prestressing can be defined as the induction of compressive stresses in a concrete member prior to the application of dead and live loads. The purpose is to improve the strength and behavior of the member under service loads. Since the tensile strength of concrete is very small compared to its compressive strength, the effect of prestressing is to reduce the tensile stresses caused by external loading. This would improve the control of concrete cracking.

The idea of prestressing was applied by P. H. Jackson in the design of structural concrete about 1886. In 1928, Eugene Freyssinet originated the use of high strength steel wires in prestressing to minimize the effect of shrinkage and creep in the concrete [6]. Application of prestressed concrete in the construction of bridges was made possible after the development of end anchorage methods in 1939 by Freyssinet and in 1940 by Magnel, a professor from Belgium [6]. In the late 1940s, prestressed concrete began in the United States with the construction of the Walnut Lane Bridge in Philadelphia. Since then, the prestressed concrete industry has grown tremendously. In 1950, there was only one prestressing plant; 229 were completed by 1961, and 500 were operating in 1975 [7]. Figure 1.1 shows a graph of the dollar sales of prestressed concrete for the 25 year period, 1950-1975 [7].

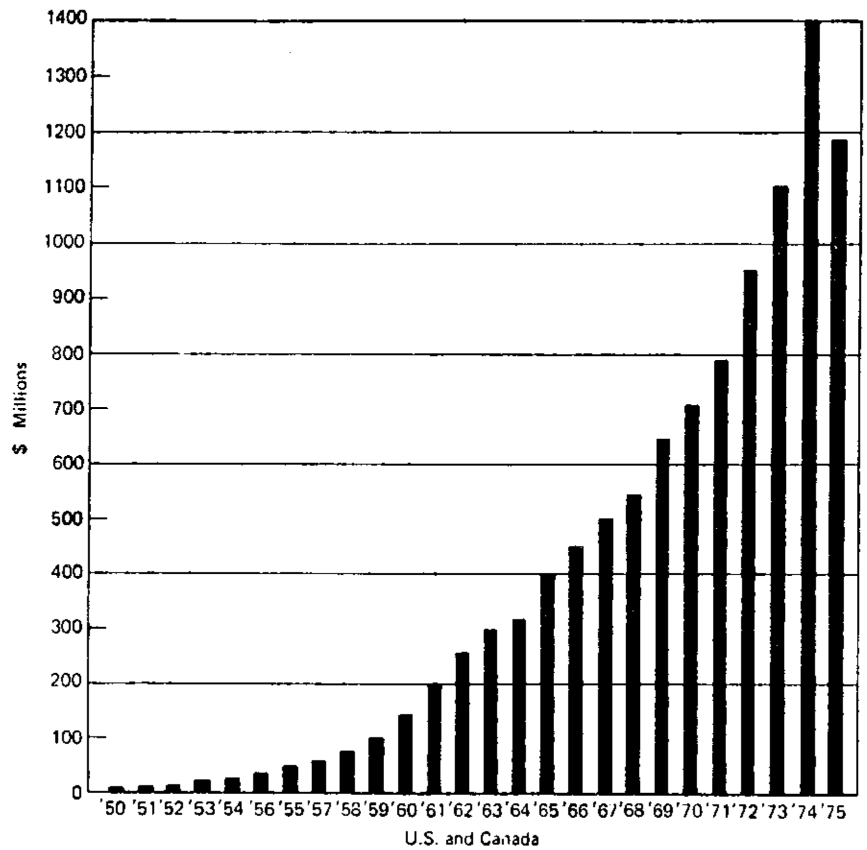


Figure 1.1. Dollar sales volume of precast and prestressed concrete for the United States and Canada

The rapid development of the prestressing industry was due to the numerous advantages prestressed concrete offered. Prestressed concrete structures are often more economical than reinforced concrete, particularly for long spans and heavy loads [17].

One of the most significant applications of prestressed concrete is the hollow-core slab systems, also referred to as hollow-core concrete planks. See Figure 1.2. A hollow-core plank is a precast, prestressed concrete member containing longitudinal voids throughout its length. Hollow-core planks have spans ranging from 18 ft. to 42 ft. and depths varying from 6 to 12 in. They are primarily used as floor and roof decks in buildings such as hotels, schools, hospitals, offices, shopping malls, etc. [13].

In recent years, due to the increase of construction costs, composite steel deck systems have been extensively used as floor or roof decks. A typical composite steel deck is shown in Figure 1.3 [15]. Bond action between concrete and the steel deck is attained by means of shear transferring devices, such as holes, inclined or longitudinal embossments and transverse wires. Using cold-formed steel decks for floor or roof systems yields many advantages. The steel deck serves as a permanent form during construction and later serves a positive reinforcement for the floor system. It is economical because it significantly reduces the time of construction.

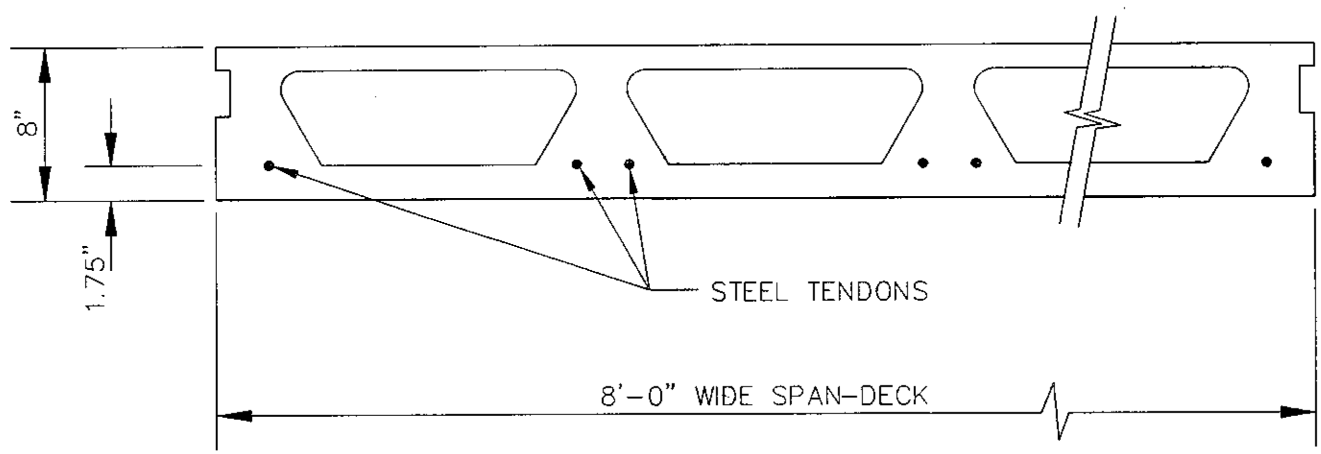


Figure 1.2. Untopped prestressed precast hollow-core plank

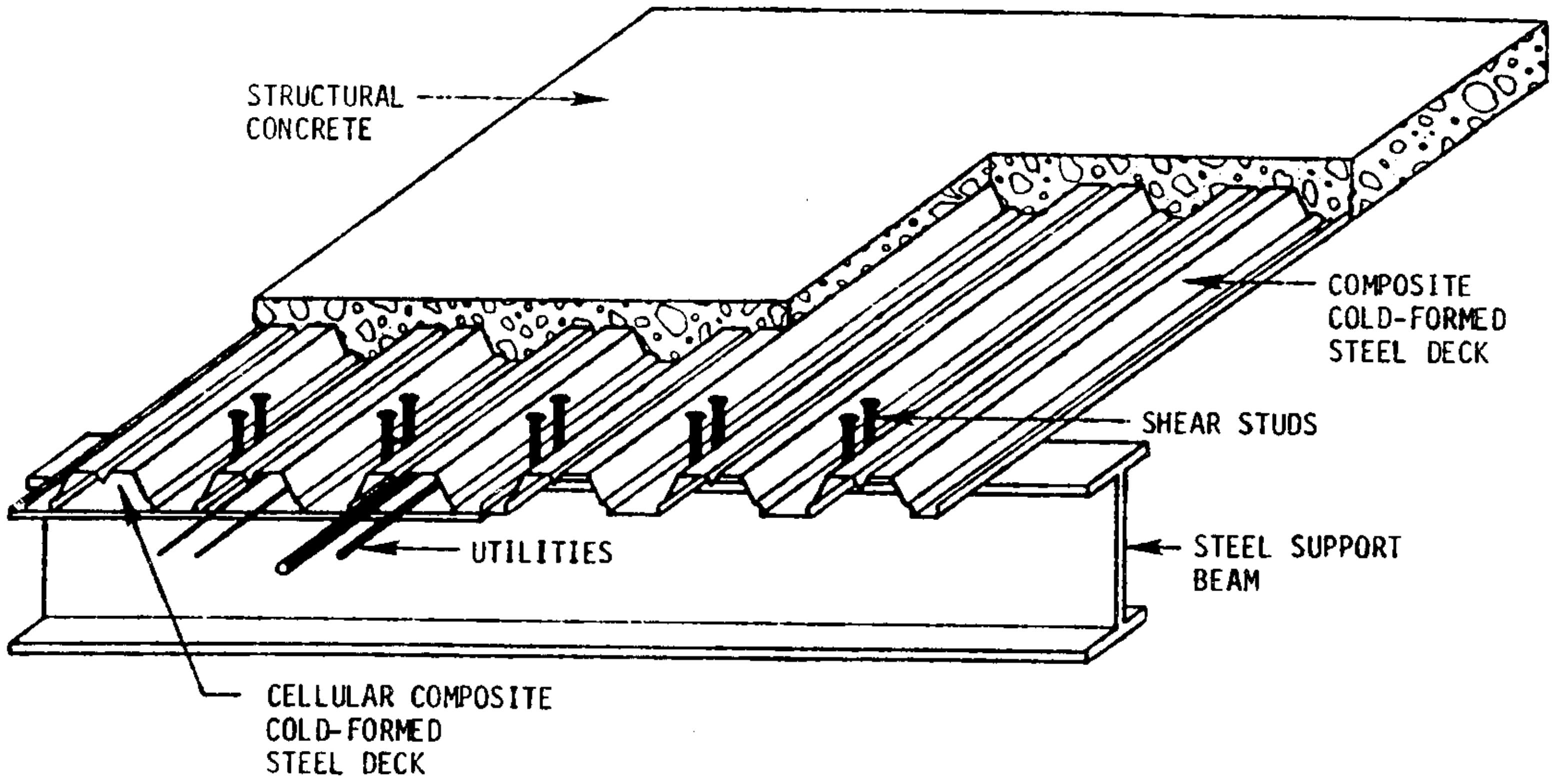


Figure 1.3. Typical building floor slab utilizing cold-formed steel decking (Reference 15)

1.2. Objective

The primary object of this investigation was to explore the feasibility of a composite prestressed concrete slab constructed with cold-formed steel decks in buildings. This was accomplished by conducting a theoretical analysis of composite prestressed concrete floor systems and comparing the results with similar spans utilizing hollow-core planks.

1.3. Scope

The above objective will be accomplished by analyzing two different slab systems. One system utilized a commercially available Bowman steel deck with a depth of 2½ in. and 20 gauge thickness (see Figure 1.4). The other, as illustrated in Figure 1.5, consisted of a generic 20 gauge steel deck, ranging in depth from 2 to 4 in. Parameters studied herein for both systems were restricted to thickness of precast concrete and the thickness of concrete placed in the field. Depth of corrugations for the generic deck were varied with span length. Span lengths analyzed herein ranged from 15 to 29 ft. The total depth of all slabs was kept constant and equal to 8 in. Comparisons were subsequently made with 8 in. deep hollow-core prestressed concrete slabs.

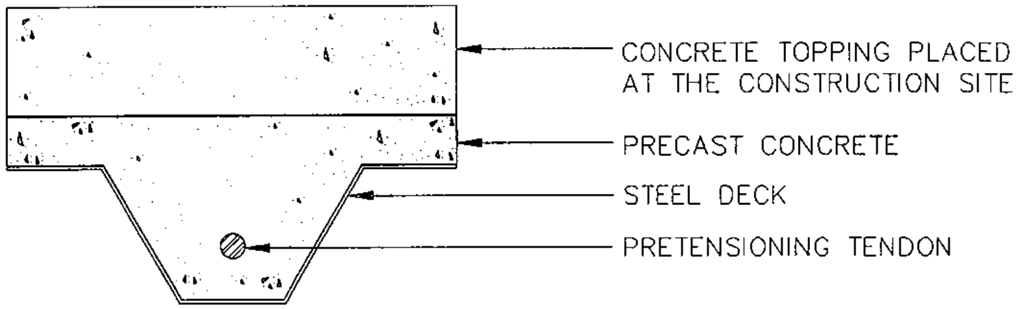


Figure 1.4. Cross section of the composite prestressed deck constructed with Bowman steel deck

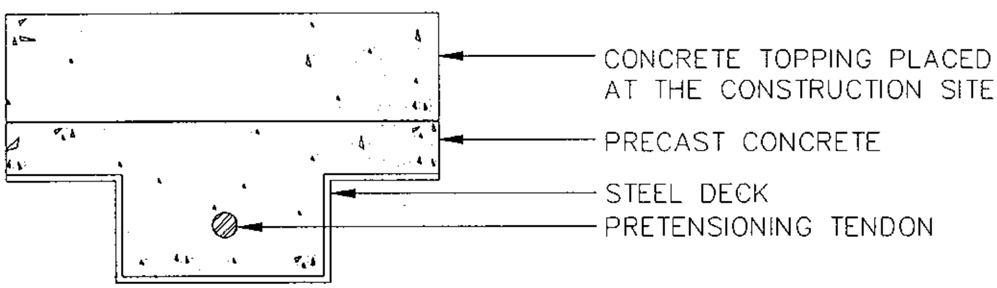


Figure 1.5. Cross section of the composite prestressed deck constructed with Generic steel deck

2. DESCRIPTION OF PRESTRESSED COMPOSITE SLAB SYSTEM

2.1. Introduction

The proposed prestressed composite slab systems can be constructed in two stages. During the first stage (Stage I), a composite, prestressed concrete member is constructed in a prestressing plant. The second stage (Stage II) involves transporting the precast member to the construction site and placing a concrete topping over it to provide the necessary stiffness and strength.

2.2. Stage I

The first stage entails pretensioning steel strands which are anchored against end abutments. The tendons are positioned within the down corrugations of the steel deck in a straight configuration (see Figure 2.1). Tendons are available in different diameters, i.e., 3/8, 7/16 and 1/2 in. diameters with a specified ultimate strength of 270 ksi. Smaller strands are also available in 1/4 and 5/16 in. diameters with an ultimate strength of 250 ksi [13]. The steel deck serves as the bottom form when the concrete is placed. Side and end forms are positioned to provide for concrete placement over the deck and in contact with the strands throughout the length of the member. The side forms are situated so that the concrete is not deposited over the outside 2 to 3 in. of the deck in order to utilize the interlocking mechanism along the edges which serves to connect adjacent units (see Figure 2.3).

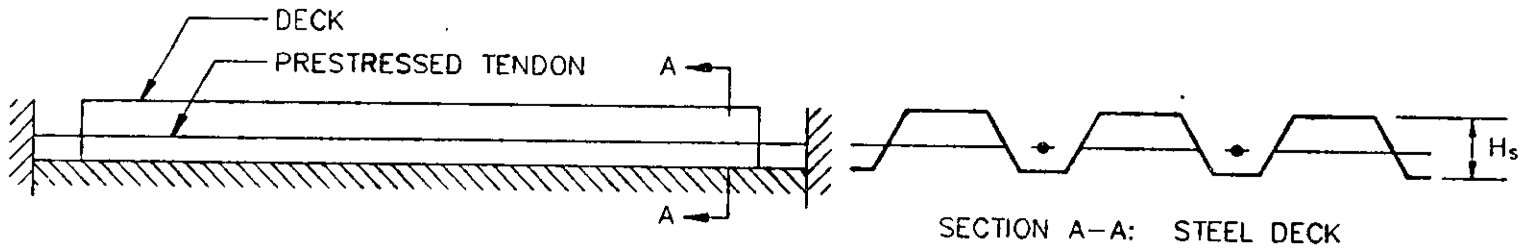


Figure 2.1. Elevation view of the steel deck in position for precasting

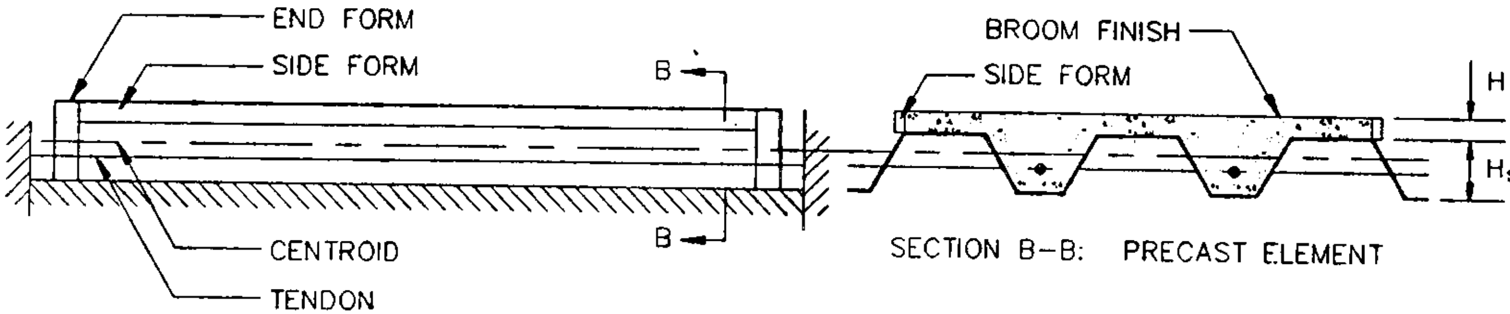


Figure 2.2. Elevation view of steel deck with forms in place

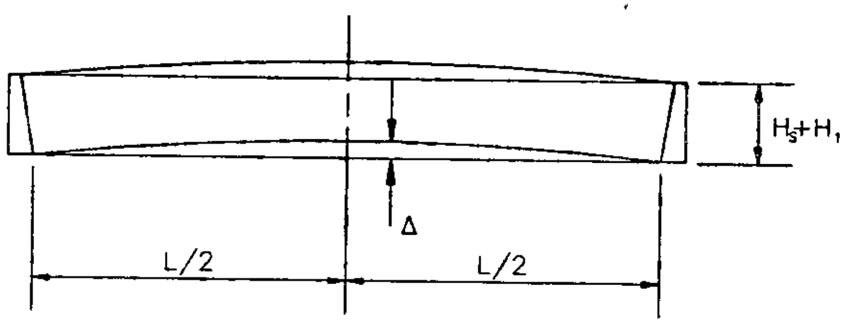


Figure 2.3. Elevation view of precast unit after release of prestress

While the tendons are stressed, the concrete is placed in contact with the tendons to a depth, H_1 above the up corrugation. During curing a bond is created between the tendons, the concrete and the steel deck. Once the concrete has been cured and reaches sufficient strength, the prestress is released. The tendons react against the concrete through the bond action. This results in a slight shortening of the member accompanied by an upward camber (see Figure 2.3). Composite action develops between the steel deck and the concrete by means of embossments which are uniformly spaced along the deck surface.

2.3. Stage II

The second stage occurs after the separate precast composite units have been positioned and connected together in the supporting structure. The precast deck is designed to have adequate rigidity and strength to support construction loads as well as the weight of the concrete topping that is subsequently placed. The purpose of the topping is to provide a sufficiently thick member to adequately resist the design live loads. Figure 2.4 shows a typical section consisting of two precast composite units with topping. The downward deflection due to the topping is expected to offset the upward camber of the precast deck, and thus cause the member to be essentially straight after application of topping. The final composite slab under the live load is shown in Figure 2.5.

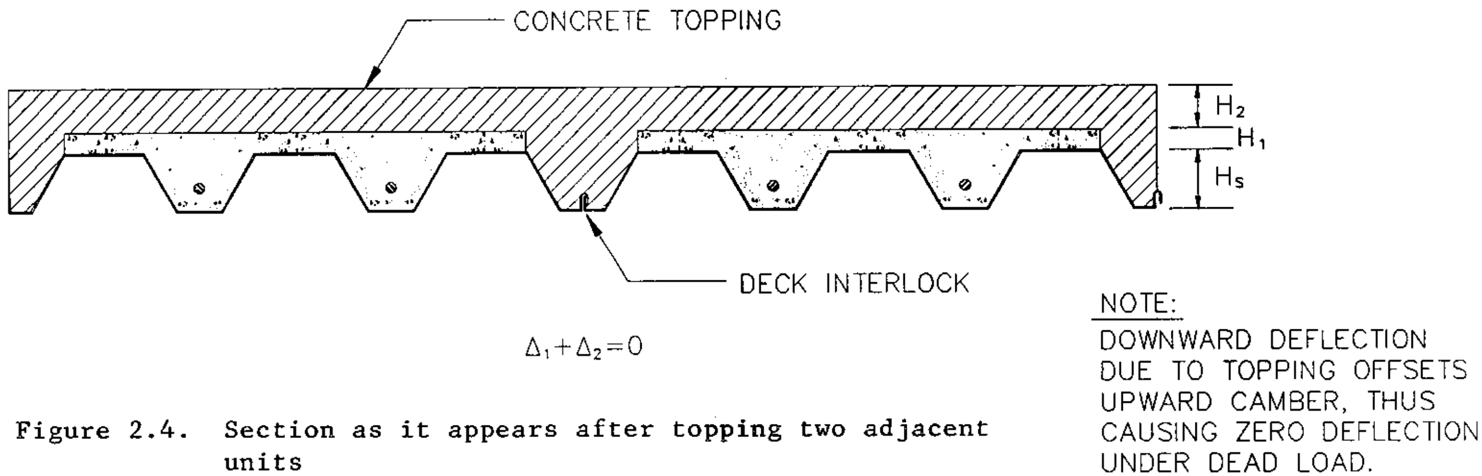


Figure 2.4. Section as it appears after topping two adjacent units

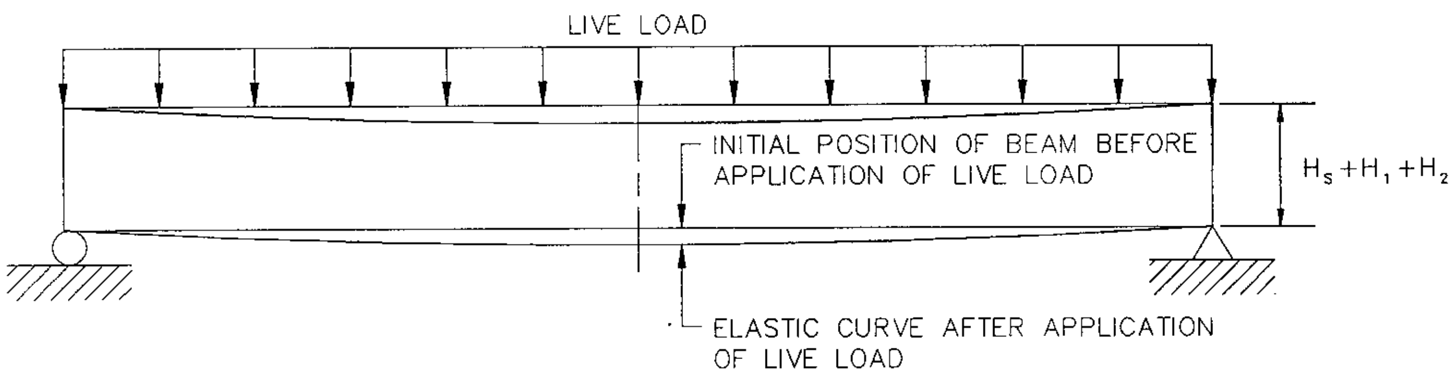


Figure 2.5. Elevation view of final composite member under design live load

2.4. Advantages of Prestressed Composite Deck

Some of the advantages of prestressed composite deck are listed below.

(1) During field erection, this system does not require shoring at the intermediate points on the span.

(2) Longer spans would be possible than are now feasible with other comparable systems, such as hollow-core planks.

(3) The precast portion of the composite prestressed system could be produced with normal operations in a prestressed concrete plant. The completed precast prestressed units would be lighter and thus more economically transported to the construction site, than would be the case for comparable hollow core planks.

(4) The attainment of a level profile under dead loads, including weight of topping, would simplify construction. Excess camber in prestressed hollow core planks can cause problems with doorways and other openings in a building. Sagging of non-prestressed composite systems constructed with light-gauge metal decking can cause a build-up of concrete over the middle portion of the span when the top surface of concrete is made level.

(5) The steel deck corrugations of the prestressed composite system provide an excellent connection between adjacent units.

3. ANALYSIS OF PROPOSED PRESTRESSED COMPOSITE DECKS

Two simply supported slab systems were selected for analysis. One system used a steel deck which is marketed by the Bowman Company and the other utilized a generic steel deck which is specially adapted for prestressing. The cross sections of these two systems are illustrated in Figures 3.1 and 3.2.

3.1. Analysis of the Bowman Composite Prestressed Decks

3.1.1. Dimensions and material properties

The steel deck used was a twenty-gauge Bowman section having a width of 25 3/4 in. and a 2½ in. depth. See Figure 3.3. The section properties of this steel deck were:

cross sectional area: 0.589 in.^{2}/ft.

centroid from the bottom fiber: 1.426 in.

Moment of Inertia: 0.518 in.^{4}/ft.

Modulus of Elasticity: 29×10^6 psi

Normal weight concrete having a unit weight of 145 lb./ft.^{3} and a compressive strength of 5000 psi was assumed throughout the analysis. The analysis is based on the use of Grade 250 strands which are used in normal prestressing operations. These strands range in size from 1/4 to 1/2 in. in 1/16 in. increments. Each tendon is positioned in a straight configuration with the centroid of its cross section located at 1 in. above the bottom fiber of the steel deck. The span lengths ranged from 15 to 29 ft. The thickness, H_1, of precast concrete above

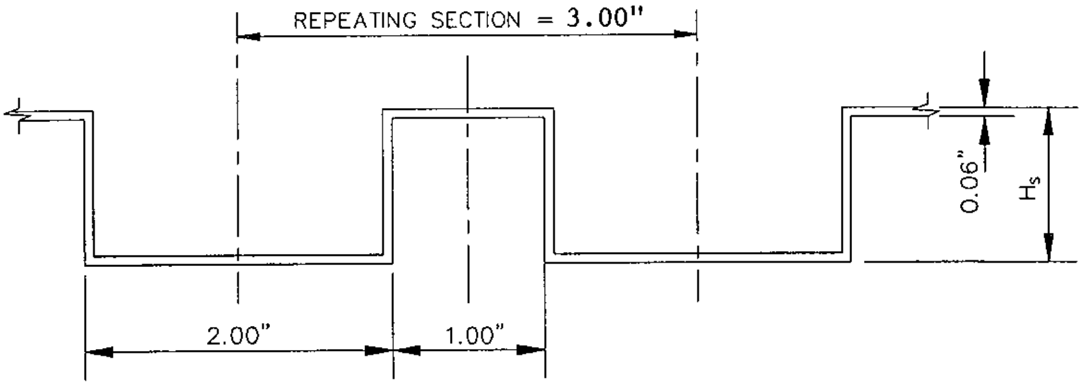


Figure 3.1. Generic steel deck cross section

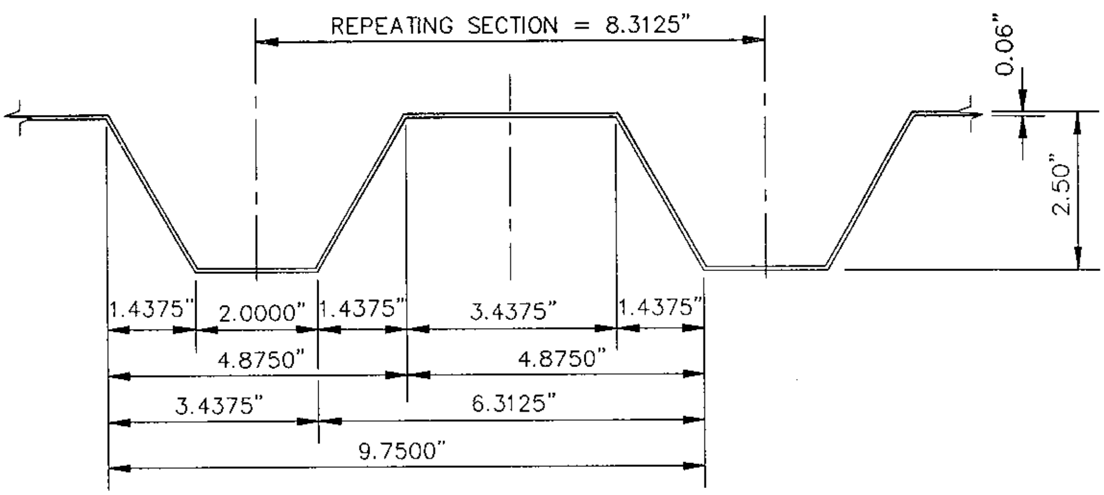


Figure 3.2. Bowman deck cross section

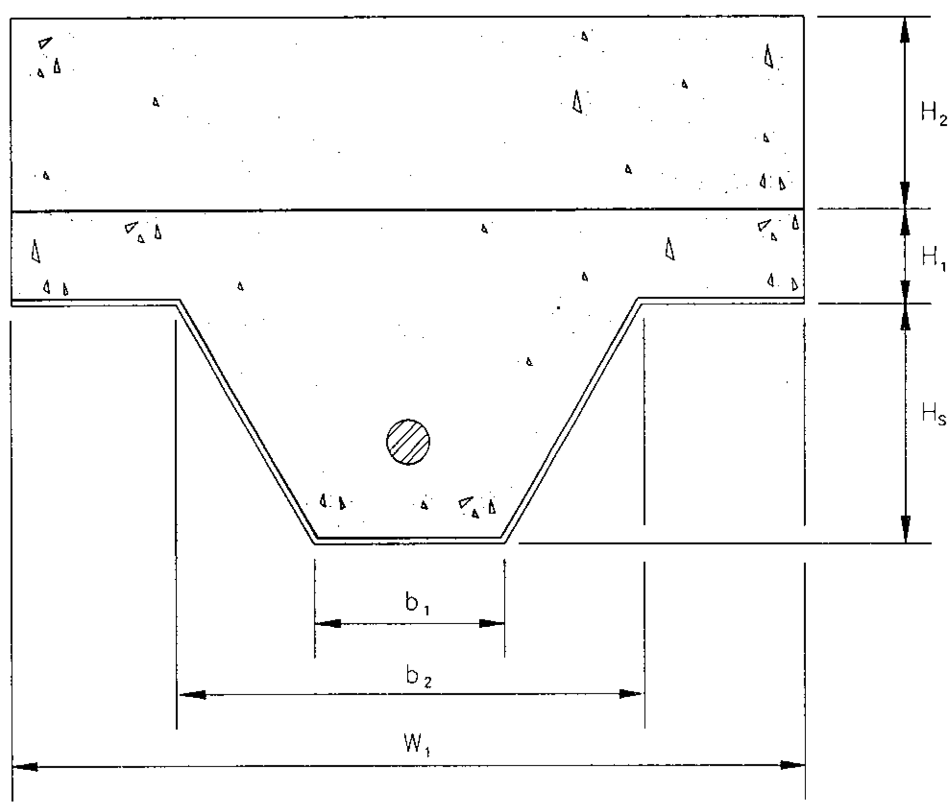


Figure 3.3. Cross section of one down corrugation of composite Bowman deck

the top corrugation, and the thickness, H_2, of the concrete placed in the field, were varied for a given span. The precast portion of the deck should be made as light as possible for handling and transportation purposes. Consequently, H_1 ranged from 1 to 2 in. The overall thickness of the prestressed composite deck was limited to 8 in. This limit was imposed in order to compare with 8 in. hollow-core slabs shown in Table 1, Appendix B [11].

3.1.2. Analysis

3.1.2.1. Assumptions An important aspect of the design is to utilize a precast section, which, when prestressed, will develop an upward camber that is sufficient to offset the downward deflection due to the later application of concrete topping in the field. The composite member, when subsequently subjected to live load, should not exceed a midspan deflection of L/360, where L is the length of the span. Careful attention must be paid to the stresses in the concrete at transfer of prestress and under live load in order to ascertain which stresses might be controlling. The critical sections are near the ends of the precast member (top and bottom fibers), at transfer of prestress, and at midspan of the composite section under live load (top and bottom fiber). These stresses should not exceed the following limiting stresses imposed by the ACI Building Code:

At transfer:

Extreme fiber stress in compression: $0.6\ f'_{ci}$

Extreme fiber stress in tension at ends: $6\sqrt{f'_{ci}}$

At service loads:

Extreme fiber stress in compression: $0.45\ f'_c$

Extreme fiber stress in tension: $12\sqrt{f'_c}$

where:

f'_{ci}: The compressive strength of concrete at time of initial prestress

f'_c: The specified compressive strength of the concrete.

3.1.2.2. Deflection, calculation and limitations The net deflection caused by the dead weight and the prestressing force is zero. The downward deflection due to live load was limited to L/360. In general, for a simply supported deck under uniformly distributed load, the maximum deflection occurs at the midspan and is given by:

$$\Delta_1 = \frac{5\ WL^4}{384\ EI} \tag{1}$$

where:

I: moment of inertia of the transformed section

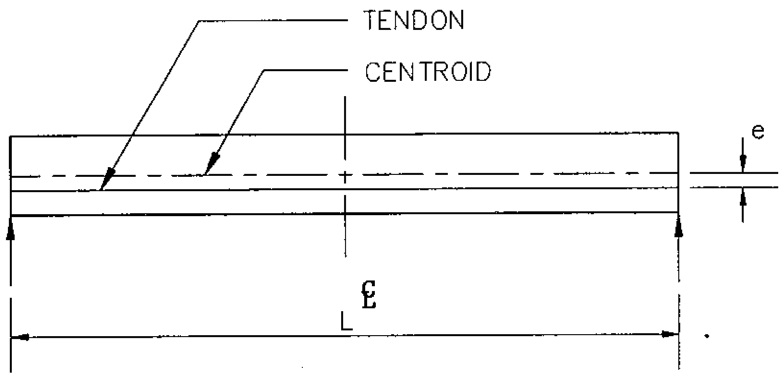
W: uniformally distributed load

L: span length

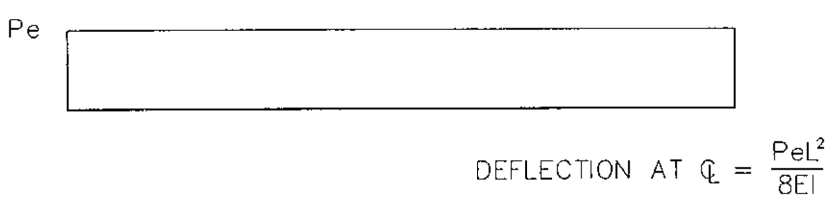
E: modulus of elasticity

The camber resulting from the prestressing is directly proportional to the eccentricity of the tendons. In this work the tendon configuration is straight with a constant eccentricity. Figure 3.4 indicates a resulting uniform moment of:

$$M = P \times e \tag{2}$$



Profile of Tendon



Moment Diagram

Figure 3.4. Profile of straight tendon and moment diagram due to prestressing

where

P = prestress force

e = eccentricity

The maximum upward deflection at midspan due to prestress force is then:

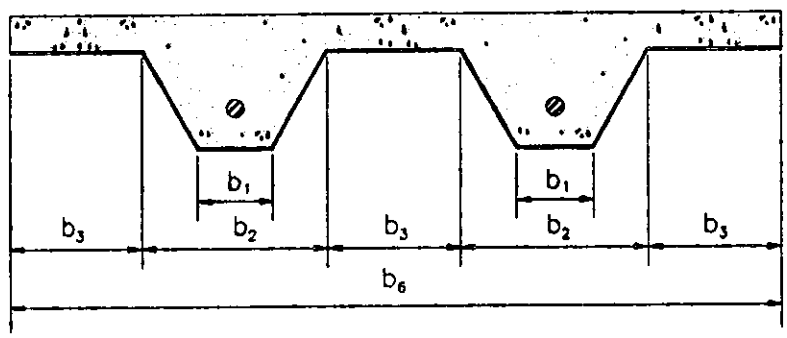
$$\Delta_2 = \frac{PeL^2}{8\ EL} \tag{3}$$

The downward deflection due to the topping and the precast section was equated to the upward camber due to prestress, thus causing the net deflection to be zero and the member to be essentially straight. The immediate midspan deflection due to live load was limited to that given in Reference [1], i.e.,

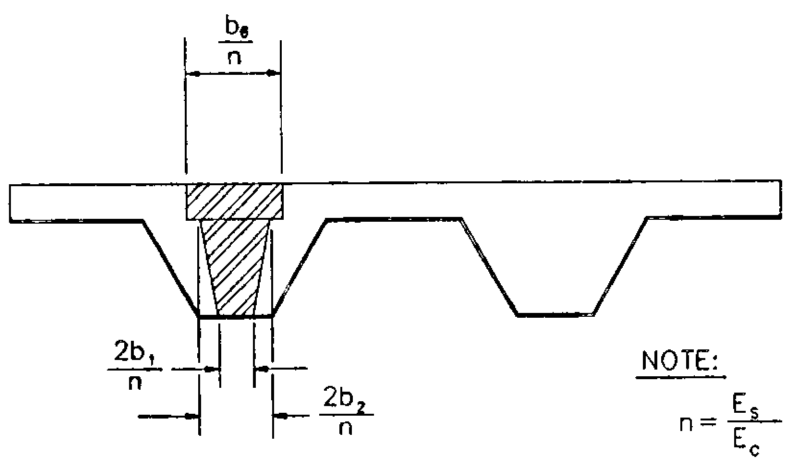
$$\Delta_3 = L/360 \tag{4}$$

3.1.2.3. Calculation of composite section properties The section properties of the precast composite system were determined by means of the transformed area concept. Due to the irregular configuration of the steel deck, it was most convenient to transform the precast section to an equivalent steel section as shown in Figure 3.5a. The properties for the composite section, after application of topping were determined from the corresponding transformed section shown in Figure 3.5b.

3.1.2.4. Prestress and live load computations The prestress force was determined from the assumption that the composite section

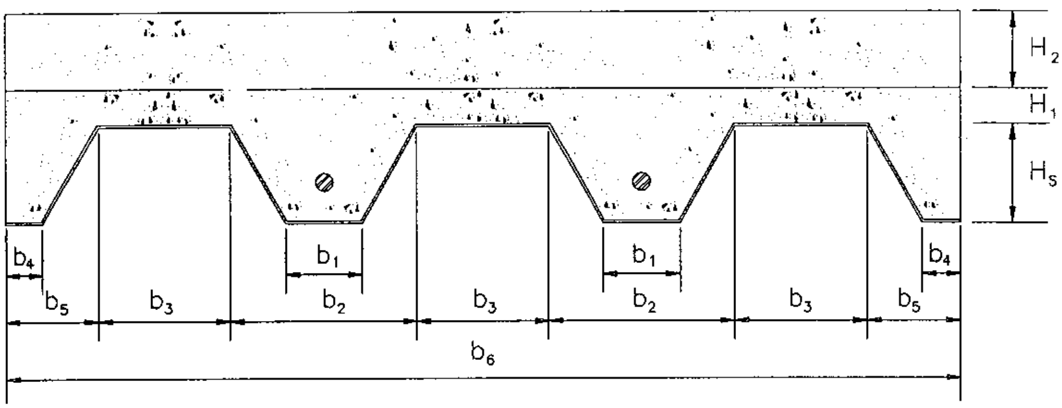


Actual section

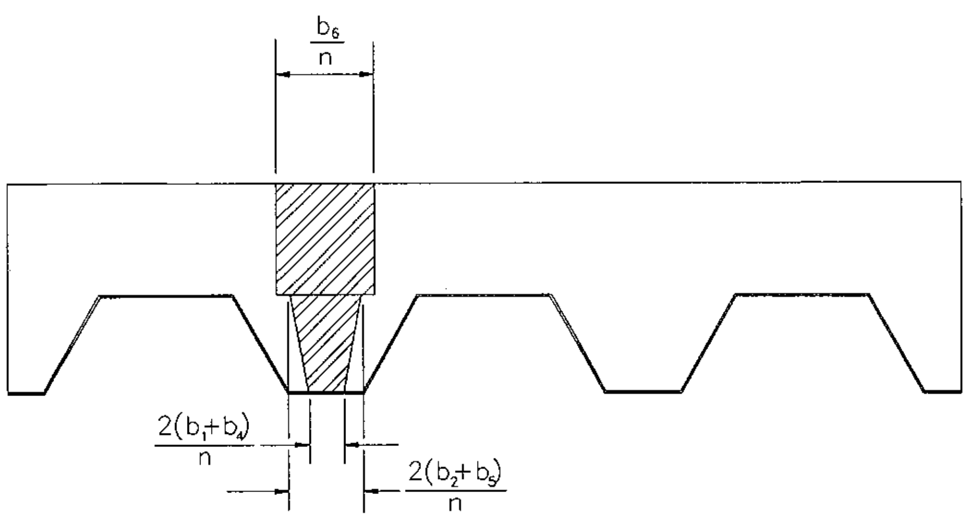


Transformed section

Figure 3.5a. Actual and transformed sections of the precast Bowman deck



Actual section



Transformed section

Figure 3.5b. Actual and transformed sections of the composite Bowman deck would be perfectly level under the combined effects of prestress and self weight. Thus,

$$\delta_1 - \delta_2 + \delta_3 = 0 \qquad (5)$$

where

δ_1 = downward deflection due to self-weight of the precast section

δ_2 = upward camber due to prestress force

δ_3 = downward deflection due to the action of concrete topping placed in the field.

The values of δ_1, δ_2 and δ_3 can be calculated from the following:

$$\delta_1 = \frac{5\ W_p L^4}{384\ E_s I_p} \qquad (6)$$

$$\delta_2 = \frac{P_e\ L^2}{8\ E_s I_p} \qquad (7)$$

$$\delta_3 = \frac{5}{384}\ \frac{W_t\ L^4}{E_s I_p} \qquad (8)$$

where:

Wp = unit weight of the precast section

E_s = modulus of elasticity of steel

I_p = moment of inertia of precast section transformed to steel

Thus,

$$\frac{5\ L^4}{384\ E_s I_p} \quad (W_p + W_s + W_t) = \frac{PeL^2}{8\ E_s I_p}$$

and

$$P = \frac{5}{48}\ \frac{(W_p + W_s + W_t)L^2}{e} \qquad (8a)$$

L = span length

W_t = weight of concrete topping

The maximum allowable live load was obtained by limiting the net deflection to: L/360, in accordance with the ACI Code [1]. This limitation was indeed controlling in certain instances where excessive live load stresses occurred in the concrete at midspan.

3.1.2.5. Flexural strength computations The nominal flexural strength was determined for all sections.

The actual stress distribution as well as the Whitney stress block are shown in Figure 3.6.

The forces acting on the section are expressed as follows:

$$T_s + T_p = F_C \tag{9}$$

Also, the nominal strength of the section can be calculated by

$$M_n = T_p\,[d_p - a/2] + T_s\,[d_s - a/2] \tag{10}$$

where:

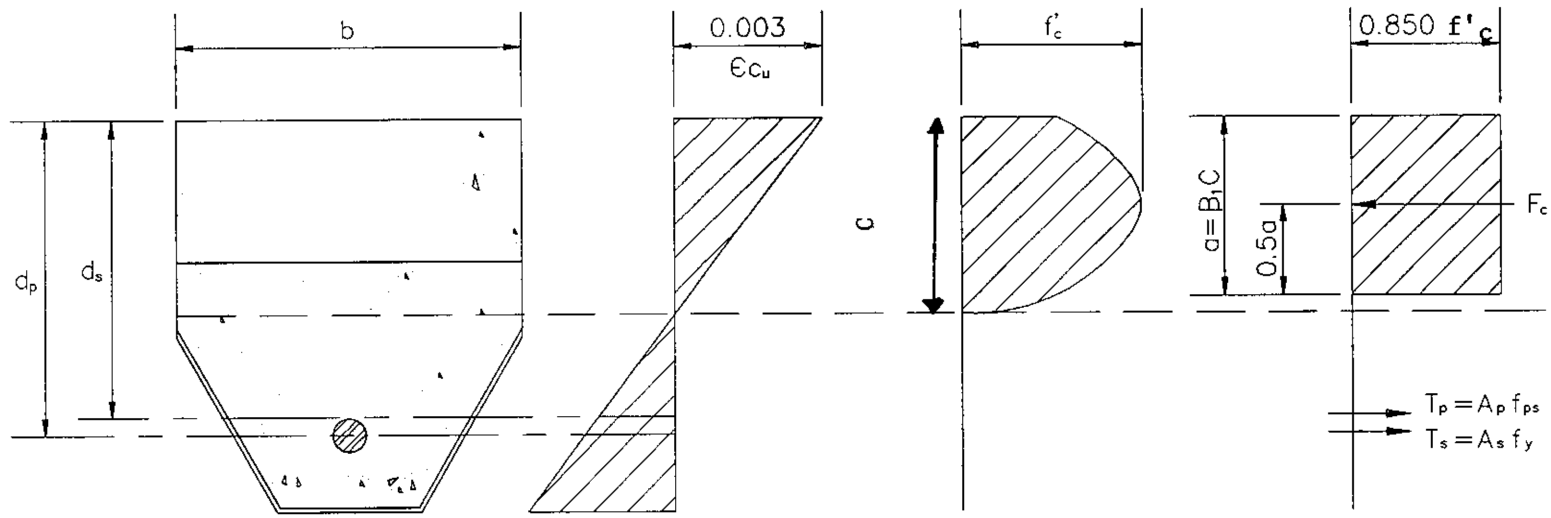
T_s = total tensile stress force of the steel deck

T_p = tensile force at the strands

F_C = compressive force in the concrete

d_p = distance from extreme compression fiber to centroid of prestressed reinforcement

d_s = distance from extreme compression fiber to centroid of steel deck



Cross section of slab
(a)

Assumed strain diagram
(b)

Actual stress
diagram
(c)

Whitney force
distribution
(d)

Figure 3.6. Stresses and strain distribution as assumed in ultimate strength computation

Assuming that the steel deck has yielded, one can calculate the depth of block by:

$$a = \frac{A_p\ f_{ps} + A_s\ f_y}{0.85\ (f'_c)b} \qquad (11)$$

where:

b = width of the deck

A_p = area of prestressed reinforcement

A_s = area of steel deck

f_y = specified yield strength of the steel deck

It is noteworthy that the required strength of the deck M_u must not exceed the nominal strength M_n reduced by applying a strength reduction factor ϕ [1]. A fully worked example of a prestressed composite deck computing the forces above and the flexural strength is illustrated in Appendix A.

3.1.3. Discussion of the results

The results of the analysis of the slab system with a Bowman deck are illustrated in Figures 3.7 and 3.8. Figure 3.7 shows the relationship between the prestress and the height, H_2. Figure 3.8 indicates the variation of live loads with height H_2. Each figure pertains to discrete values of H_1, varying from 1 to 2 in. and H_2, ranging between 2 and 5 in. Live load, W_{LL} and prestress force, F can be estimated as:

W_{LL} = 1000 ML/L^3 and

F = PF x L^2

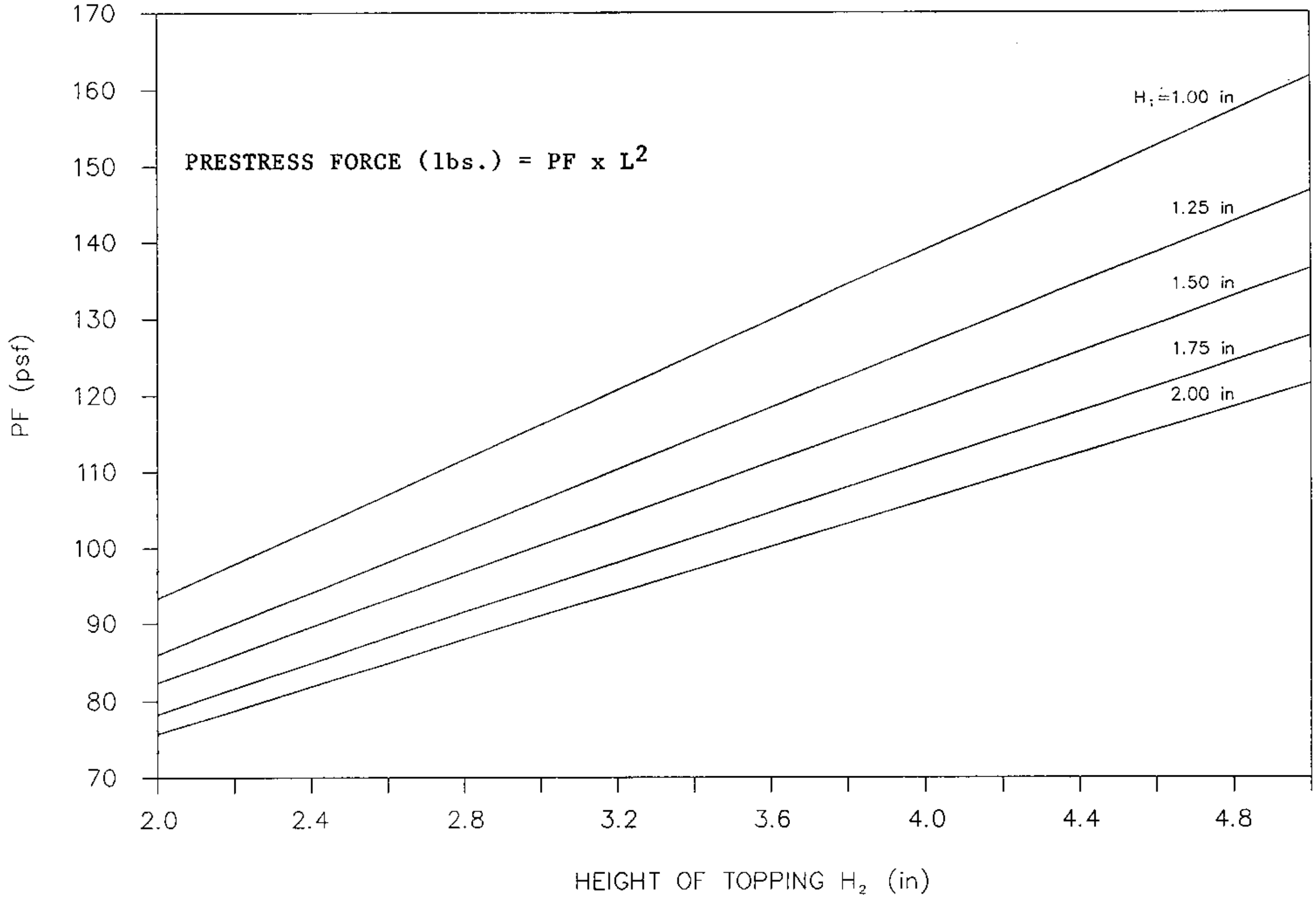


Figure 3.7. Prestress force required for the Bowman slab

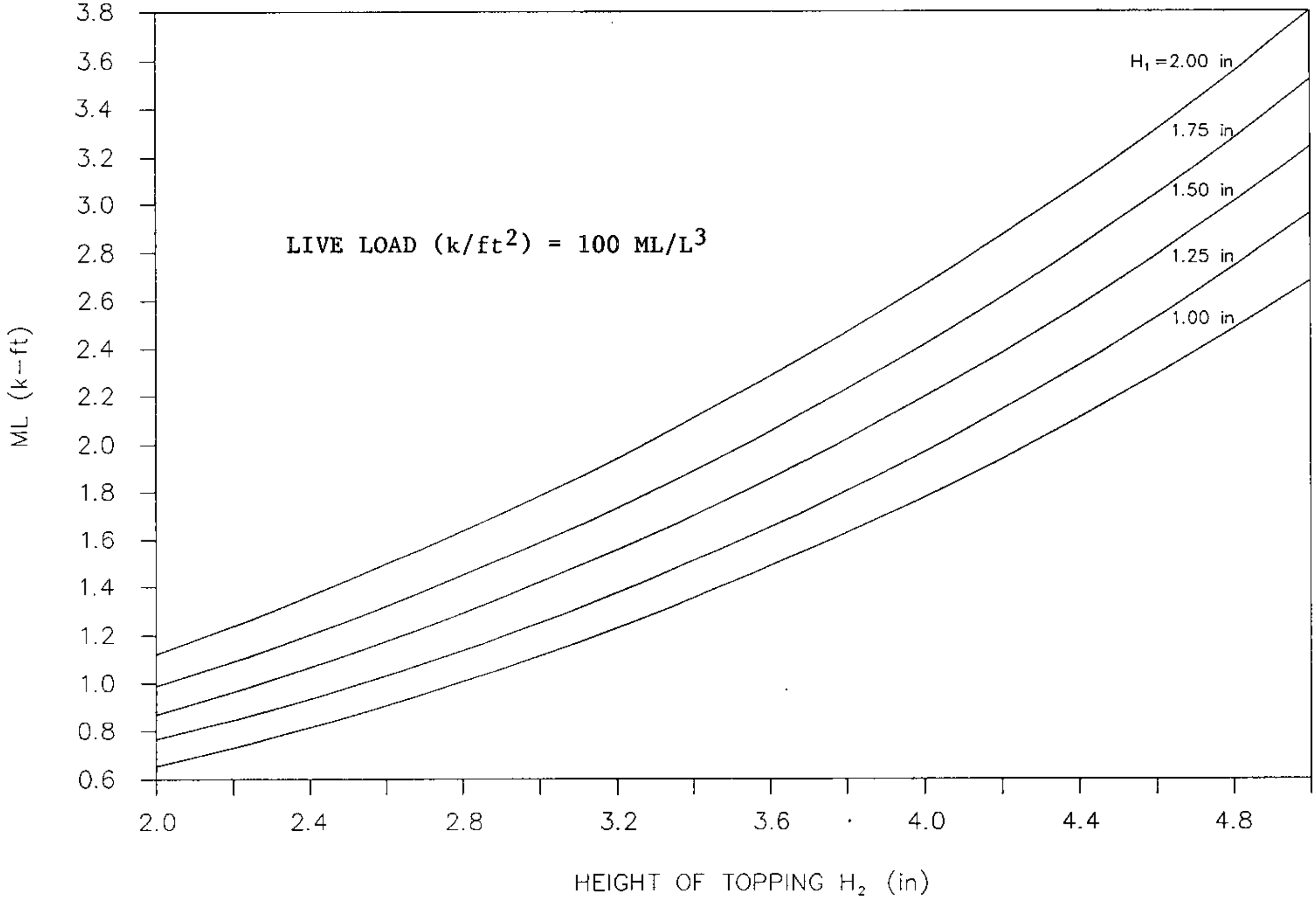


Figure 3.8. Allowable live load for the Bowman slab based on deflection limitation

where ML is a factor obtained from Figure 3.8 and PF is a factor found in Figure 3.7. As expected, the curves in Figure 3.7 are linear. For a constant height H_1, the required prestress force increases with height H_2. Consequently, for a given precast deck, the amount of prestressing required to attain a straight horizontal surface after placing the concrete topping, increases with weight of concrete placed. On the other hand, a shallow precast deck necessitates more prestress than a deep one regardless of the amount of topping placed. For H_2 equals 4 inches and H_1 equals 1 inch, the maximum required prestress force for a given span length L, is ($140 \times L^2$) lbs. The allowable live load that can be sustained by the floor deck likewise increases with height H_2. Decreasing the depth of the precast H_1 decreases the allowable live load. Thus, a heavier precast composite deck, i.e., with larger H_1, can support more live load than a lighter one. For H_2 and H_1 equal to 4 in. and 2 in. respectively, the maximum allowable live load is ($2600/L^3$) k/ft.^{2}

3.1.3.1. Design load tables Load tables for Bowman slabs with varying span lengths were prepared. See Table 2 in Appendix B. Listed in the tables are:

1. Dimensions of the given slab in inches
2. Required prestress force F in kips
3. Allowable superimposed live loads in k/ft.^{2}: LL
4. Number of strands needed, NO
5. Strength of the slab M_n in k-ft.

Each of these parameters cited above is a function of the slab thickness and span length shown in the tables. The span length varied from 15 to 27 ft. When varying the precast height H_1 from 1 to 2 in., end stresses at the bottom and top fibers at transfer exceeded the allowable values advised by the ACI building code (1), particularly for long-span slabs. Consequently, H_1 shown in the design load tables varied from 1.25 to 5.5 in. Since the Bowman steel deck is composed of two down corrugations, an even number of strands is required.

The values of safe superimposed service load are based on the capacity of the member as governed by Ref. [1], (as outlined in the previous sections), on service load flexural stresses, maximum deflections and flexural strength. The number of strands was computed based on the required prestress force. Two tendons of different diameters, and the resulting strengths of the slab are included in each design load table. Depending on the strength needed, one of these two combinations of tendons could be selected. Spacing requirements of the strands and concrete cover required by the ACI code for prestressed concrete controlled, when 1/2-in. and 0.6-in. strands are used. However, the proposed system is protected against exposure by the steel deck, and hence less concrete cover might be acceptable.

The following examples demonstrate the ways in which load tables in Table 2, Appendix B may be used.

Example 1: From Table 2, select a Bowman deck to carry a superimposed live load of 270 lbs/ft^2 for a 15-ft. span.

Answer: Select 2 - 7/16 in. diameter strands having a required capacity of 62.59 ft.-kip.

Example 2: For 23-ft. span, 8-in. Bowman slab, find the allowable live load that can be supported.

Answer: From Table 2, the maximum superimposed live load this slab can carry safely is: 90 lbs./ft.^{2}.

3.2. Analysis of the Generic Composite Prestressed Deck

In order to provide a section that may be more adaptable to prestressing, the generic deck shown in Figure 3.9 was considered.

A 20 gauge generic deck similar to the previously analyzed Bowman deck was selected. The height H_s ranged from 2 in. to 4 in. with 1/4 in. increments. The height H_1 was varied from 1 in. to 2 in. with the total height of the composite section kept constant at 8 in. The down corrugations had a constant width of 4 in. The analysis of this deck was carried out with the same material properties and assumptions as used for the Bowman deck. Figures 3.10 and 3.11 show the relationship between height H_s and prestress force and live load, respectively.

<u>3.2.1. Design Load Tables</u>

Load tables for the generic slabs were developed in accordance with the ACI building code. See Tables 3 through Table 5 in Appendix B. Each of the tables corresponds to a specified H_s: 2 in., 3 in. and 4 in. These values were selected as they represent a practical range of steel decks. For every slab of height H_s, the span length is varied from 15 ft. to 29 ft. The

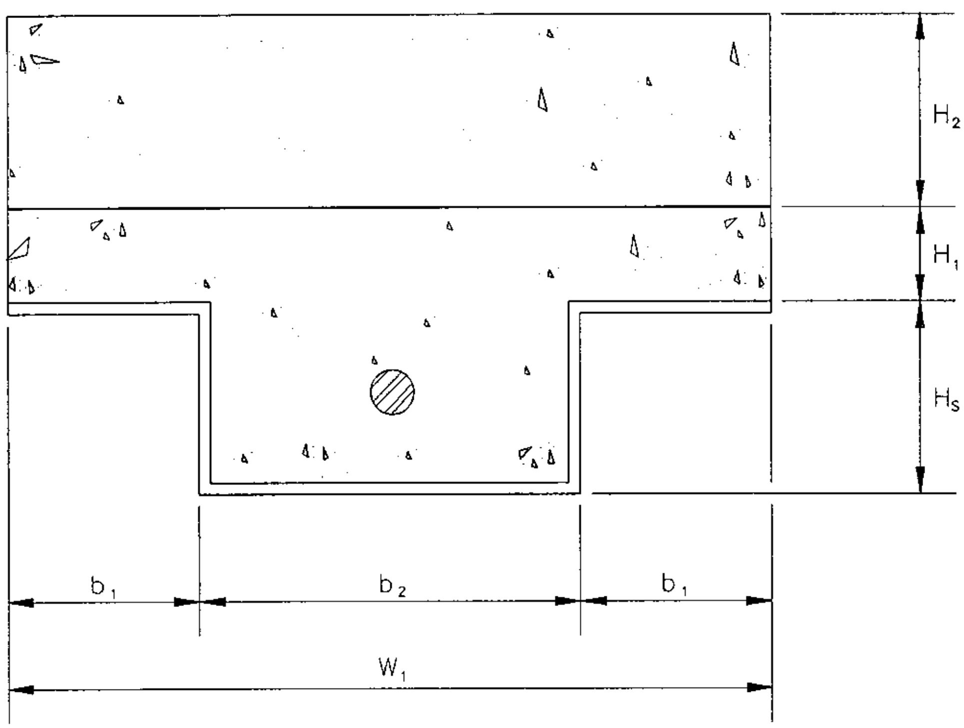


Figure 3.9. Cross section of one down corrugation of composite generic deck

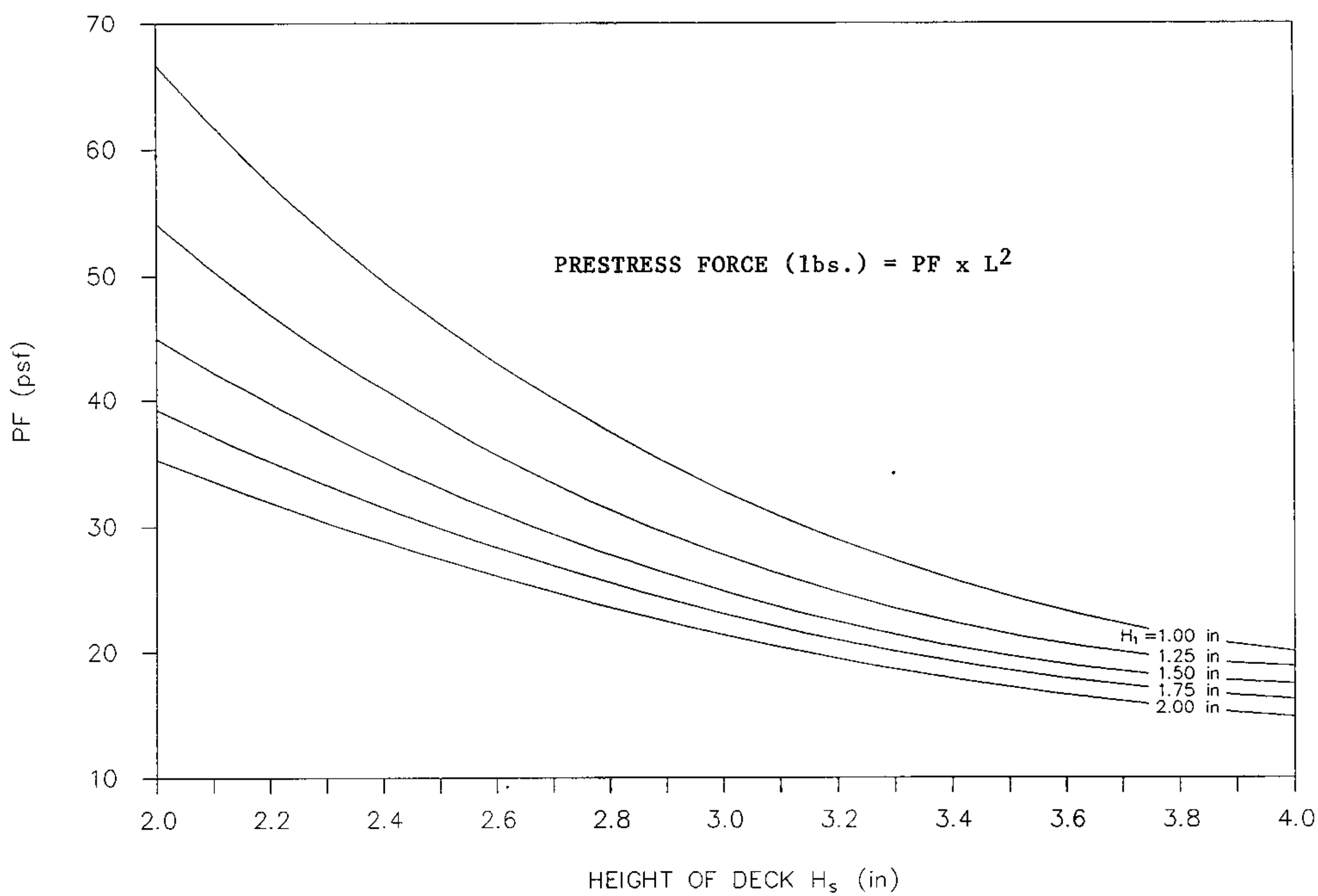


Figure 3.10. Prestress force required for Generic steel deck with variable steel deck height

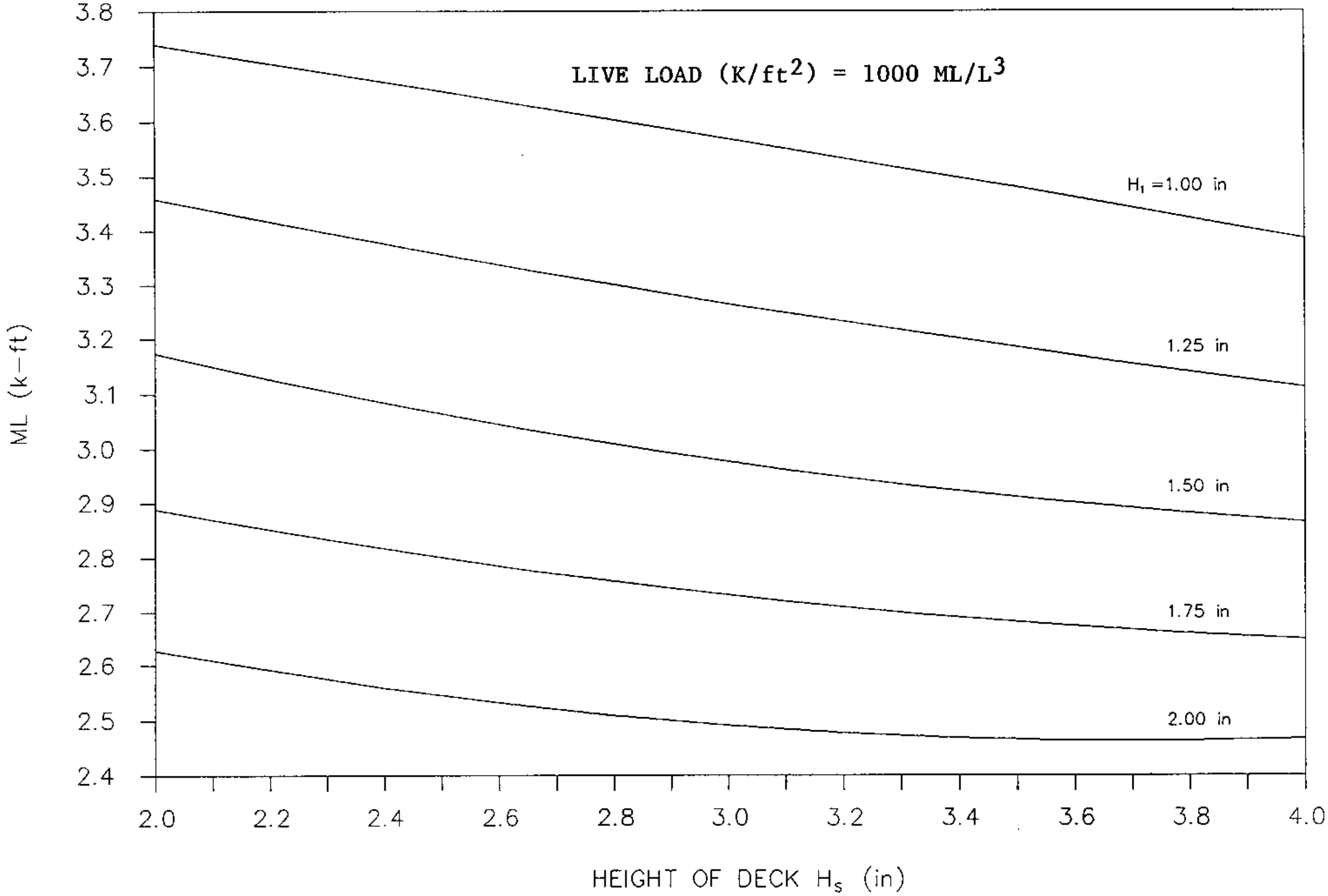


Figure 3.11. Allowable live load for the Generic slab based on deflection on limitation

precast depth H_1, ranged between 1 in. and 5 in. for short spans up to 21 ft. For longer spans, deeper and shallower steel decks, H_1-values (up to 6 in.) were used.

The load tables define the allowable live load that a given slab can safely support in addition to the slab self weight. The load capacity depends on the steel deck thickness and the amount of prestressing provided. The generic deck load tables may be used in the same manner as those developed for the Bowman systems.

3.3. Analysis of Prestressed Composite Decks Using Personal Computers

The Lotus spreadsheet is a matrix of spaces called cells. Each cell is defined by a row number and a column letter. The cells can store data such as numbers, letters or words, or formulas. The size of this spreadsheet is 2048 rows by 256 columns.

Lotus was used to determine the section properties, live loads, prestress forces of the composite decks, and strengths. In the analysis using Lotus worksheet, the required input data are:

1. Height of the steel deck, H_s
2. Height of the precast concrete, H_1
3. Height of the concrete placed in the field, H_2
4. Width of down corrugation, b_1
5. Width of upper corrugation, b_2
6. Unit weight of concrete, W_c
7. Modulus of elasticity of concrete, E_c
8. Modulus of elasticity of steel deck, E_s

9. Area of steel deck, A_s

10. Moment of inertia of steel deck, I_s

11. Span length of the deck, L

12. Number of down corrugations, m

The input data is set up on the left corner of the template. Once all required data are entered, the supporting calculations needed to compute the live load, the prestress force, stress, and the flexural strength are performed.

4. EXPERIMENTAL PROCEDURE AND TEST RESULTS

4.1. General Remarks

Due to lack of availability of funds, only one slab system was constructed and tested in the Iowa State Structural Engineering Laboratory. A 17 ft. span Bowman slab 20 in. wide and 8 in. deep was selected for investigation. The main purpose of this test was to corroborate the theoretical results and to ascertain the practicality of the proposed systems.

4.2. Specimen Preparation

The material used in the construction for this specimen consisted of corrugated cold-formed steel decking (Bowman), DYWIDAG single-bar tendons and normal-weight, high-strength concrete. The steel decking consisted of the twenty-gauge Bowman deck analyzed in Chapter 3. Two DYWIDAG single-bar tendons having a diameter of 5/8 in. were used to post-tension the slab.

4.2.1. Construction of the specimen

The composite prestressed specimen was constructed in two stages:

1. During Stage I, hollow conduits were placed within the down corrugations. Side forms were situated at 2.8 in. from the edges of the steel deck. Concrete was placed to a depth of 1 in. above the top of the steel deck. Vibration of the concrete was accomplished with a small laboratory type vibrator. The top surface was given a rough finish with a

wooden float in order to permit a satisfactory bond with concrete topping to be added later. The concrete was subsequently cured with moist burlap covered by plastic sheeting. The two DYWIDAG bars were then inserted into the conduits. The precast deck is shown in Figure 4.1.

2. During Stage II, which occurs after prestressing the precast deck, wood forms were placed around the clean deck. The concrete topping was placed over the precast section resulting in an 8-in. composite specimen as shown in Figure 4.2. The final specimen was moist cured for 7 days, followed by air curing.

4.3. Test, Equipment and Instrumentation

A manually operated hydraulic jacking unit was used to apply prestress to the DYWIDAG bars. The applied force was 20 kips per tendon. The loading apparatus, shown in Figure 4.3, provided two-point loading to the simply supported deck. Upward deflection of the precast deck during the prestressing operation was measured by a single dial gage placed at midspan. The vertical deflections of the completed composite member were measured with dial gages placed under the specimen at midspan and under the two load points. The end slip between the concrete and the steel deck was measured with dial gages at each end of the specimen (see Figure 4.4).

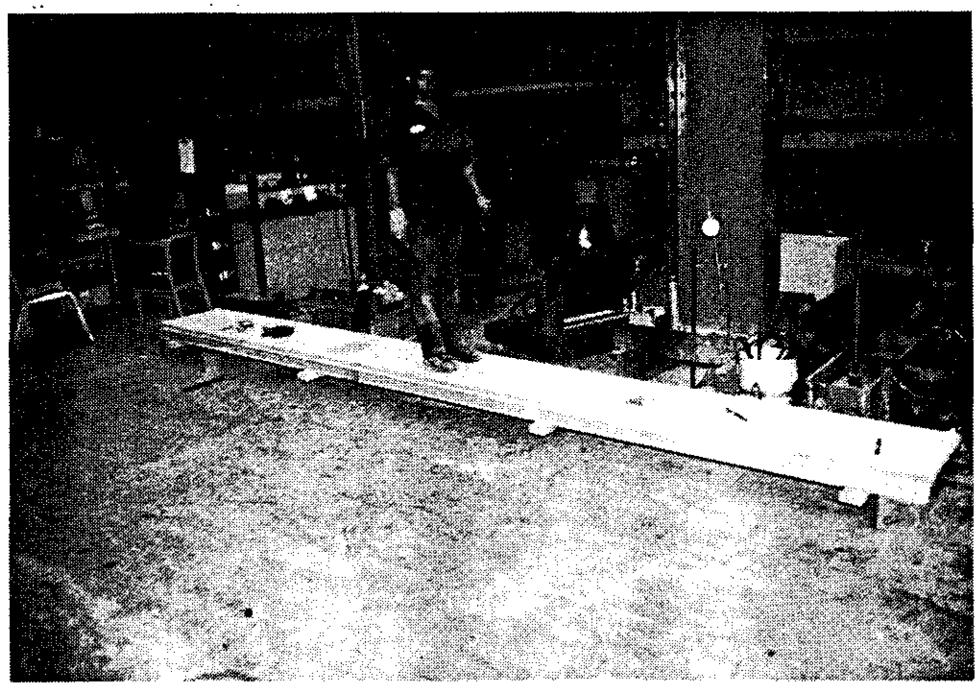


Figure 4.1. Precast Bowman deck

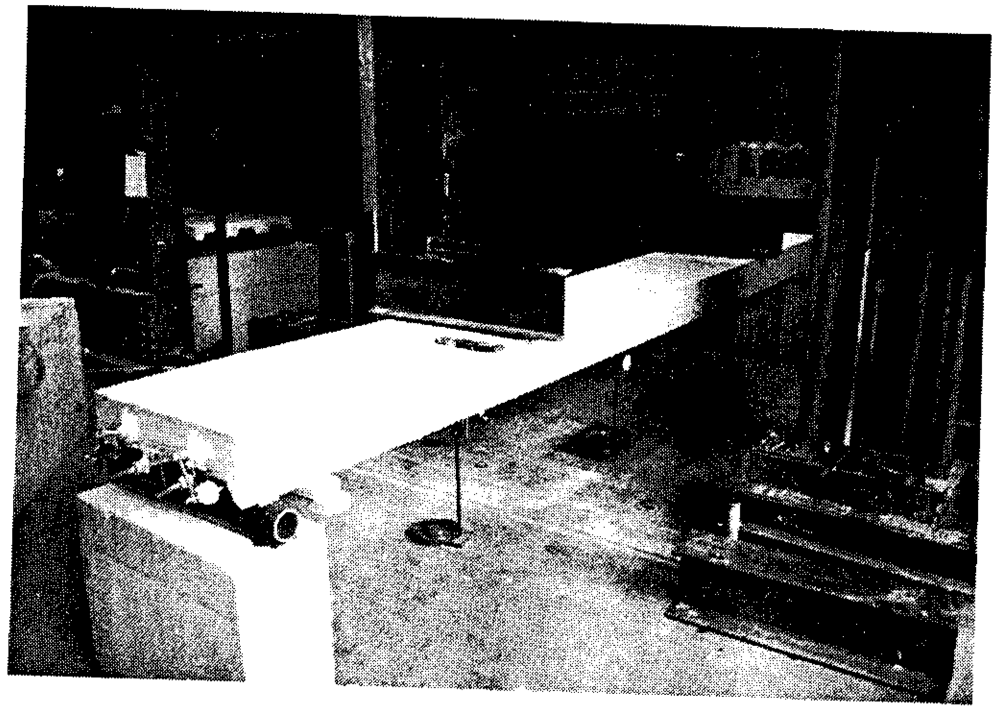


Figure 4.2. Composite Bowman deck during testing

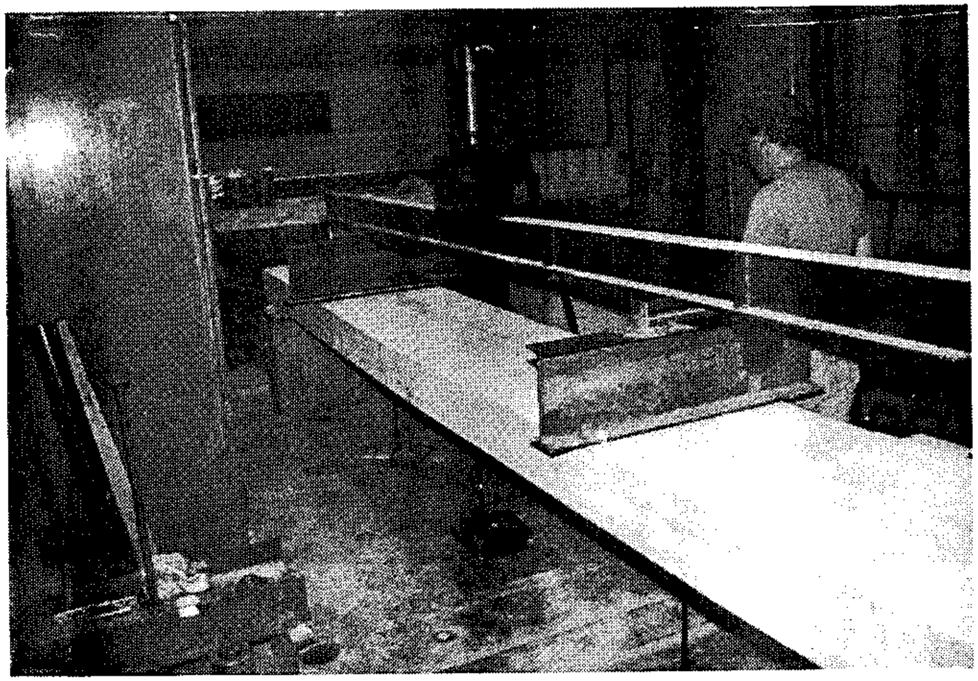


Figure 4.3. Testing arrangement of the composite specimen

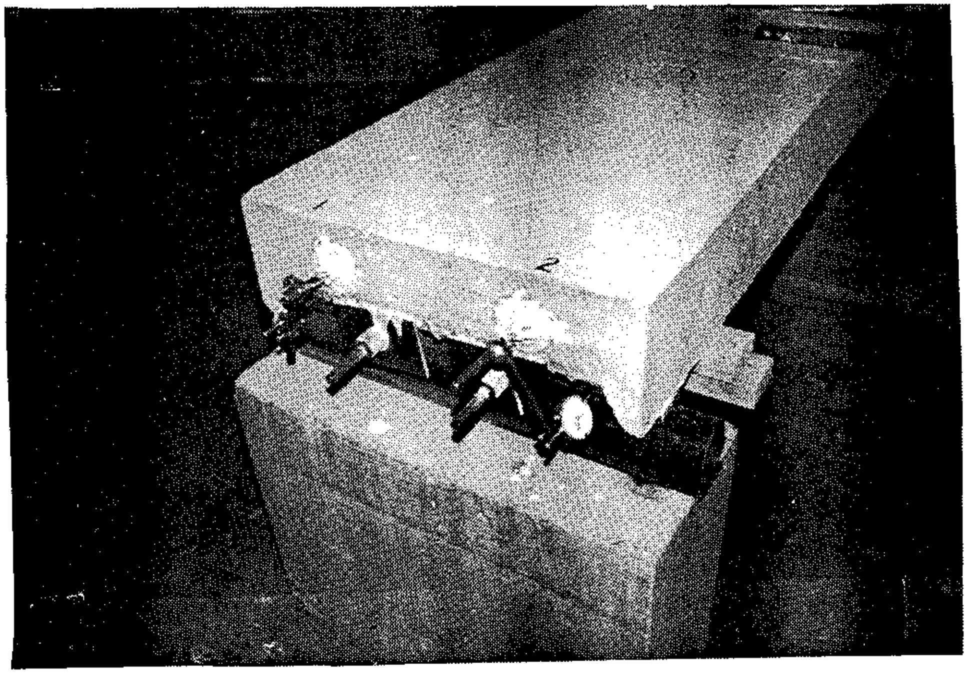


Figure 4.4. End view of the composite specimen showing slip dial gages

4.4. Test Procedure

The precast deck was subjected to three loading cycles. Loading was applied while maintaining a 1/10-in. deflection increment. The composite deck was tested in three stages. During the first two stages, the two point loadings were placed at a distance L′ equal to 4 ft. 1½ in. from the ends. The distance L′ was 6 ft. 3 in. during Stage III.

Loading on the composite slab was applied and maintained at each 1 kip increment level allowing for the necessary deflection readings. Cracking characteristics, and evidence of visual end-slip between concrete and steel deck were observed and recorded.

4.5. Test Results and Discussion

4.5.1. Precast post-tensioning results

The jacking force and the subsequent deflection or camber are presented in Table 6 in Appendix B. The net camber of the precast member resulting from jacking was: 0.878 in. during the first test and 0.922 in. during the second test, resulting in an increase in camber of 5%. The calculated camber was 1.02 in., which was 10% greater than the measured value for the second test.

4.5.2. Load-deflection results

Load vs. midspan deflection diagrams, shown in Figures 4.5 through 4.10 illustrate the behavior of the precast and the composite

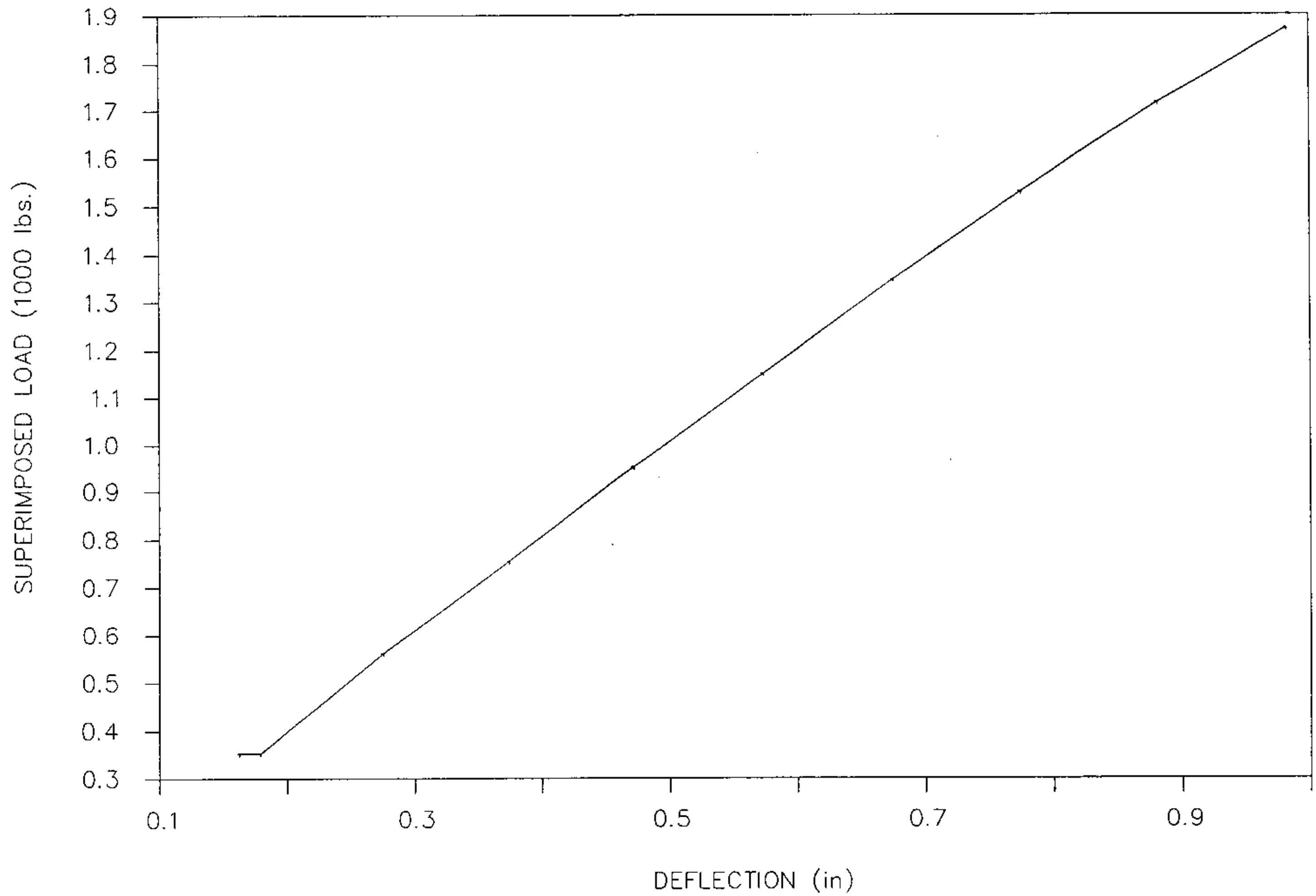


Figure 4.5. Load deflection diagram of the precast deck during test #1

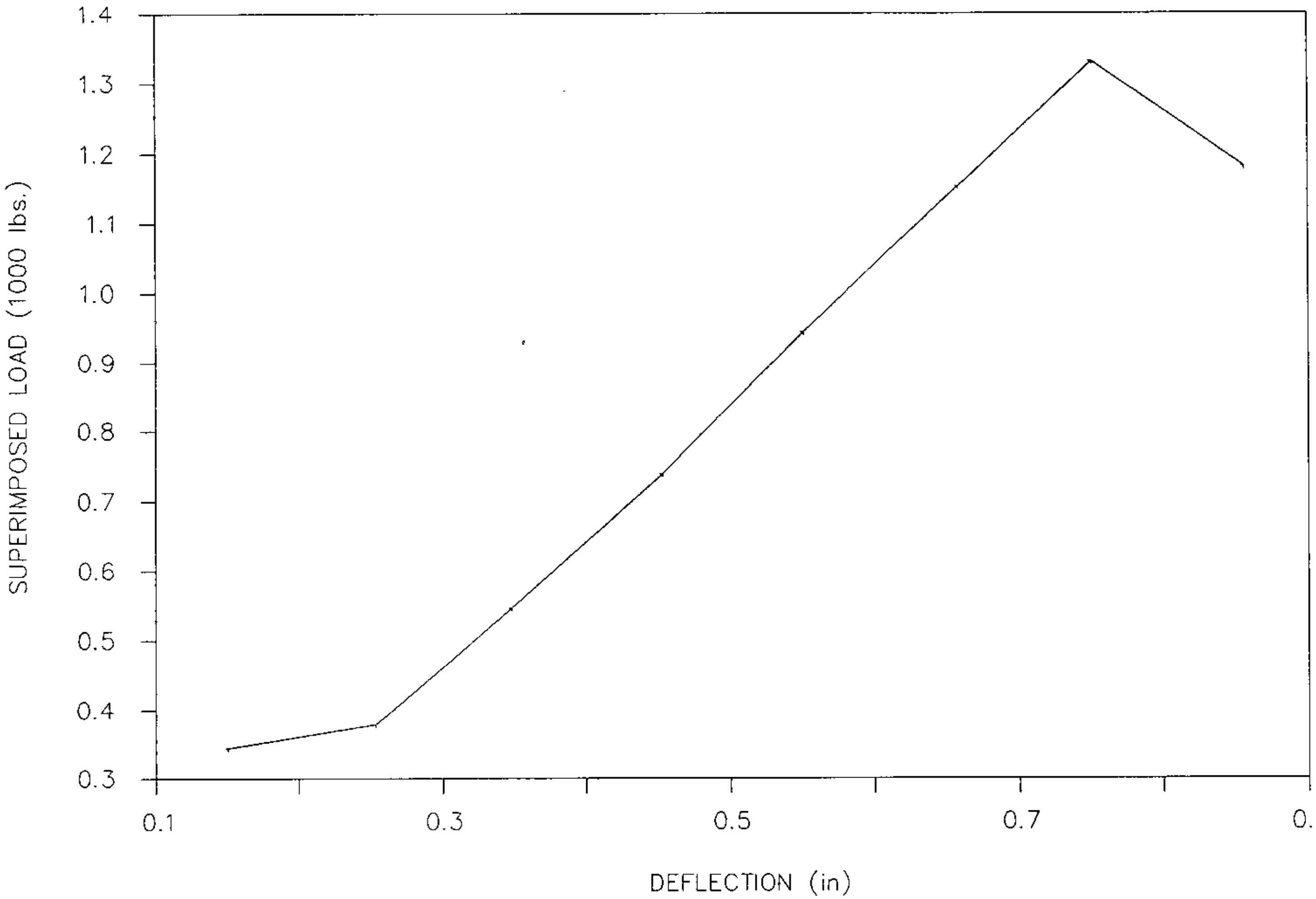


Figure 4.6. Load deflection diagram of the precast deck during test #2

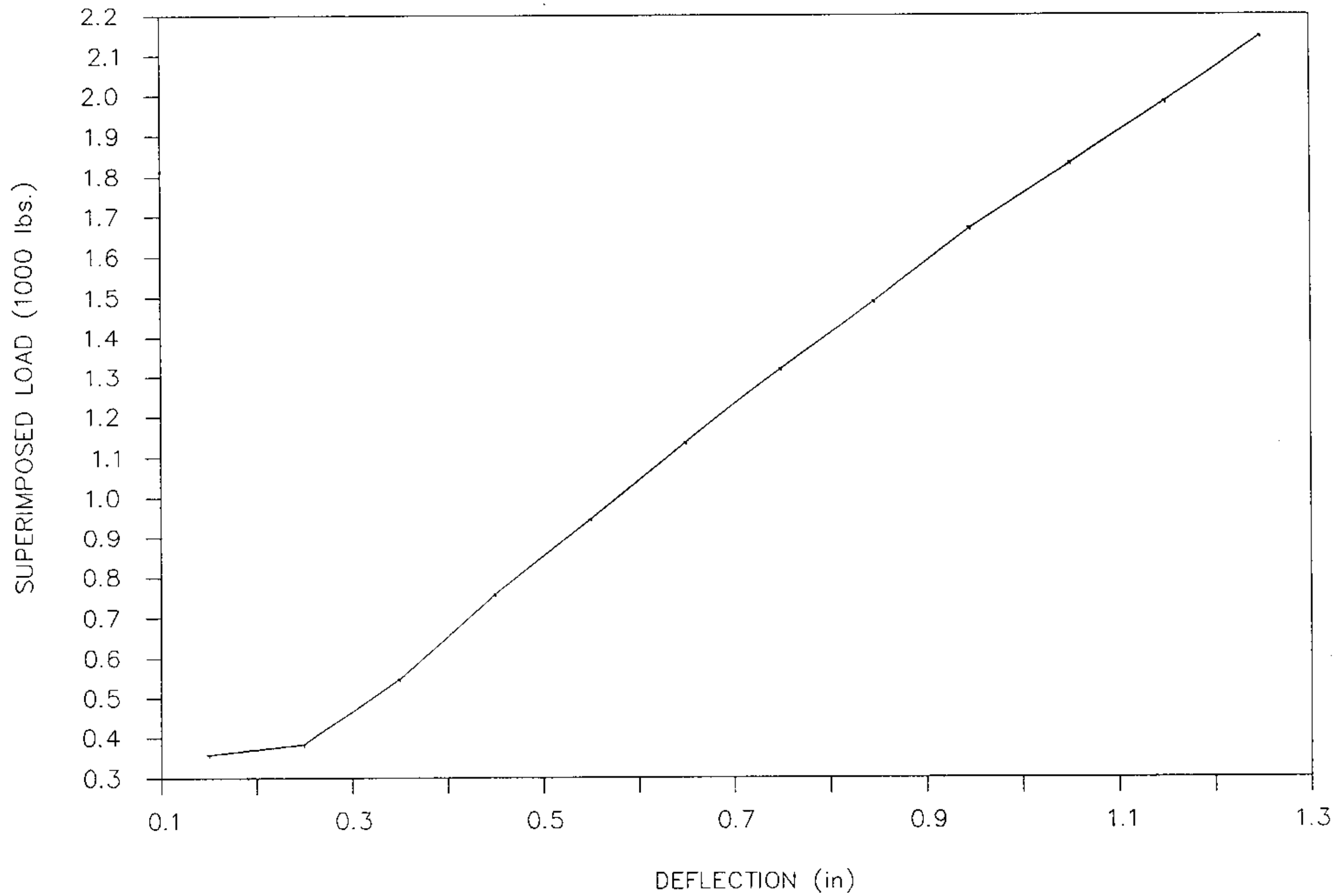


Figure 4.7. Load deflection diagram of the precast deck during test #3

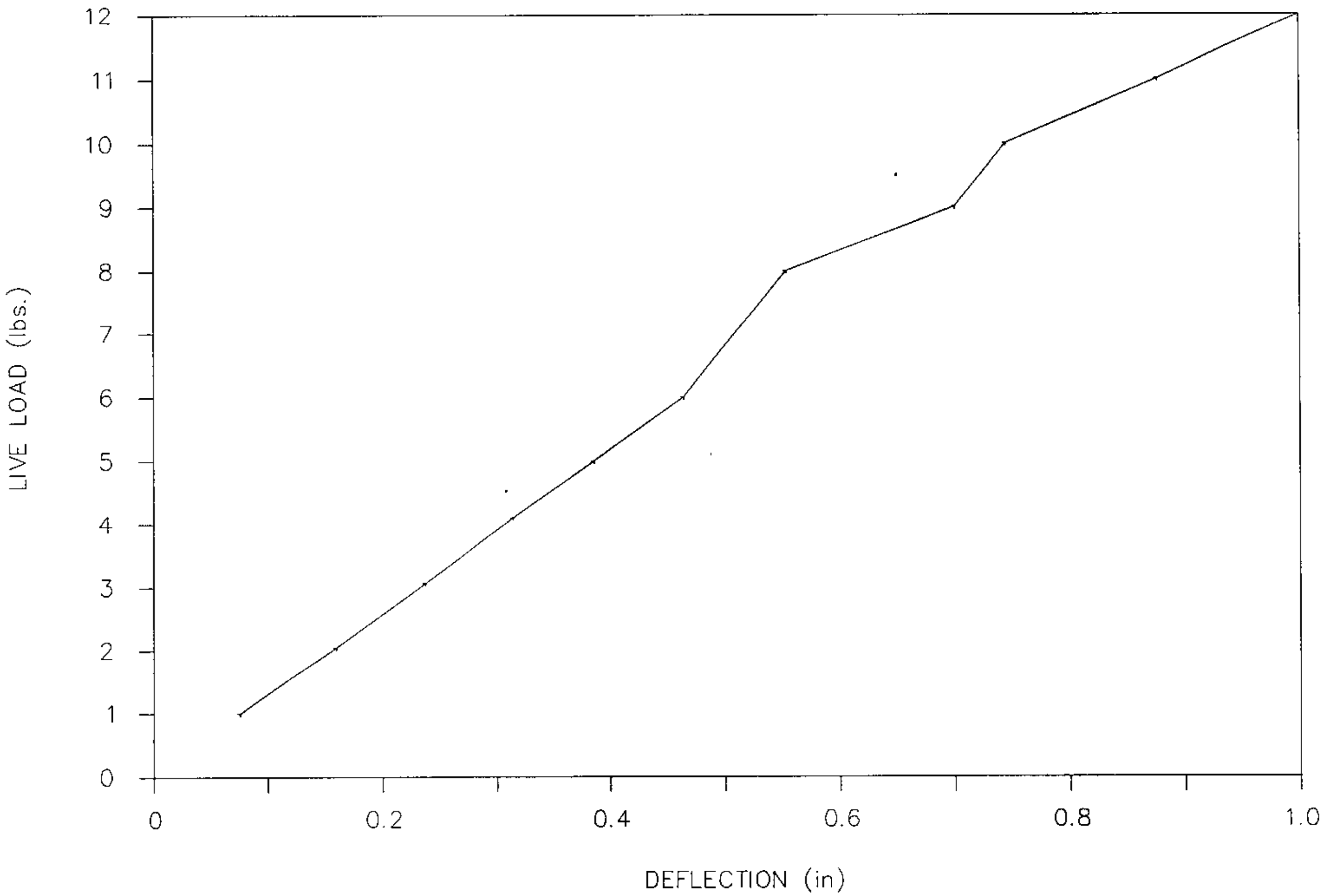


Figure 4.8. Load deflection diagram of the composite deck during test #1

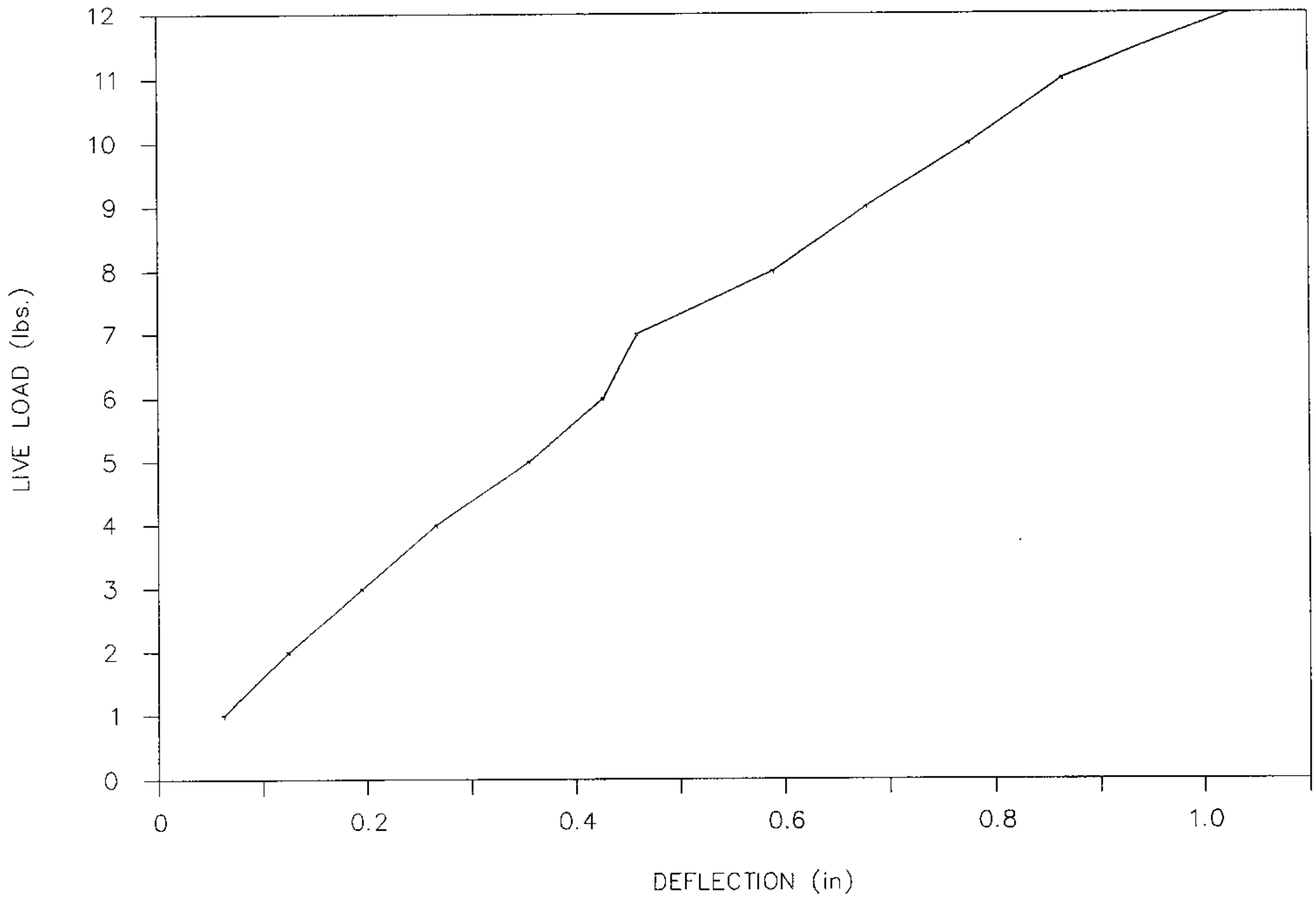


Figure 4.9. Load deflection diagram of the composite deck during test #2

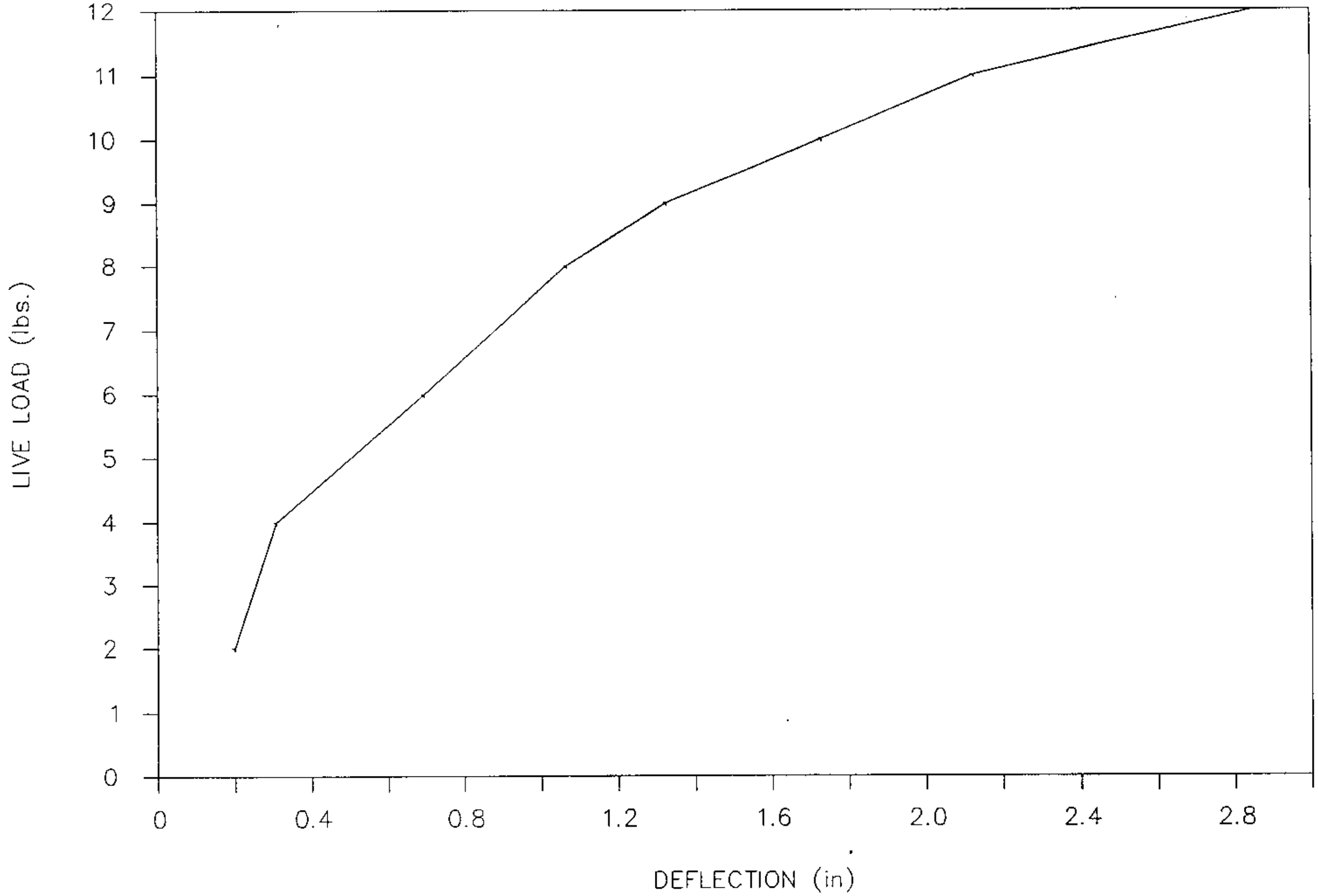


Figure 4.10. Load deflection diagram of composite deck during test #3

specimens under the loading. For the precast deck, the load varies almost linearly with the deflection during the three cycles.

When loading the composite deck, three stages were observed. During the first stage, before the formation of cracks, the concrete and steel deck acted as a composite section. During the second stage, a few cracks were developed and some end-slip was observed. The mechanical interlocking mechanism between concrete and steel deck neared their ultimate capacity. The two concentrated loads were moved closer to the midspan, during the third stages of loading. At the third stage, more cracks had formed, accompanied by larger end slips, and it became apparent that the member was approaching a failed condition.

5. SUMMARY AND CONCLUSIONS

5.1. Summary

In the past few decades, due to increases in cost of construction, numerous construction methods were developed for floors and roofs of buildings. Composite slabs, constructed using cold-formed steel decks and prestressed precast concrete hollow-core slabs, have frequently been used in buildings. The objective of this investigation was to determine the feasibility of using a prestressed composite slab in buildings. Such a construction system combines the features of the composite and prestressed systems.

Two systems using different steel decks were investigated. One of the steel decks was generic, having a simple rectangular cross section. The other one was manufactured by the Bowman Company. Design load tables were developed for each slab. Only one slab fabricated using a Bowman steel deck was tested in the lab. The proposed slab systems were then compared with the precast hollow core planks and the capacities and strengths are summarized in Tables 5.1 and 5.2 respectively.

5.2. Conclusions

Based on the results of this investigation illustrated in Figure 5.1, the following conclusions can be drawn:

1. Generic sections, exceed their hollow core slab counterparts for 15 ft., 17 ft. and 19 ft. spans in live load capacity.

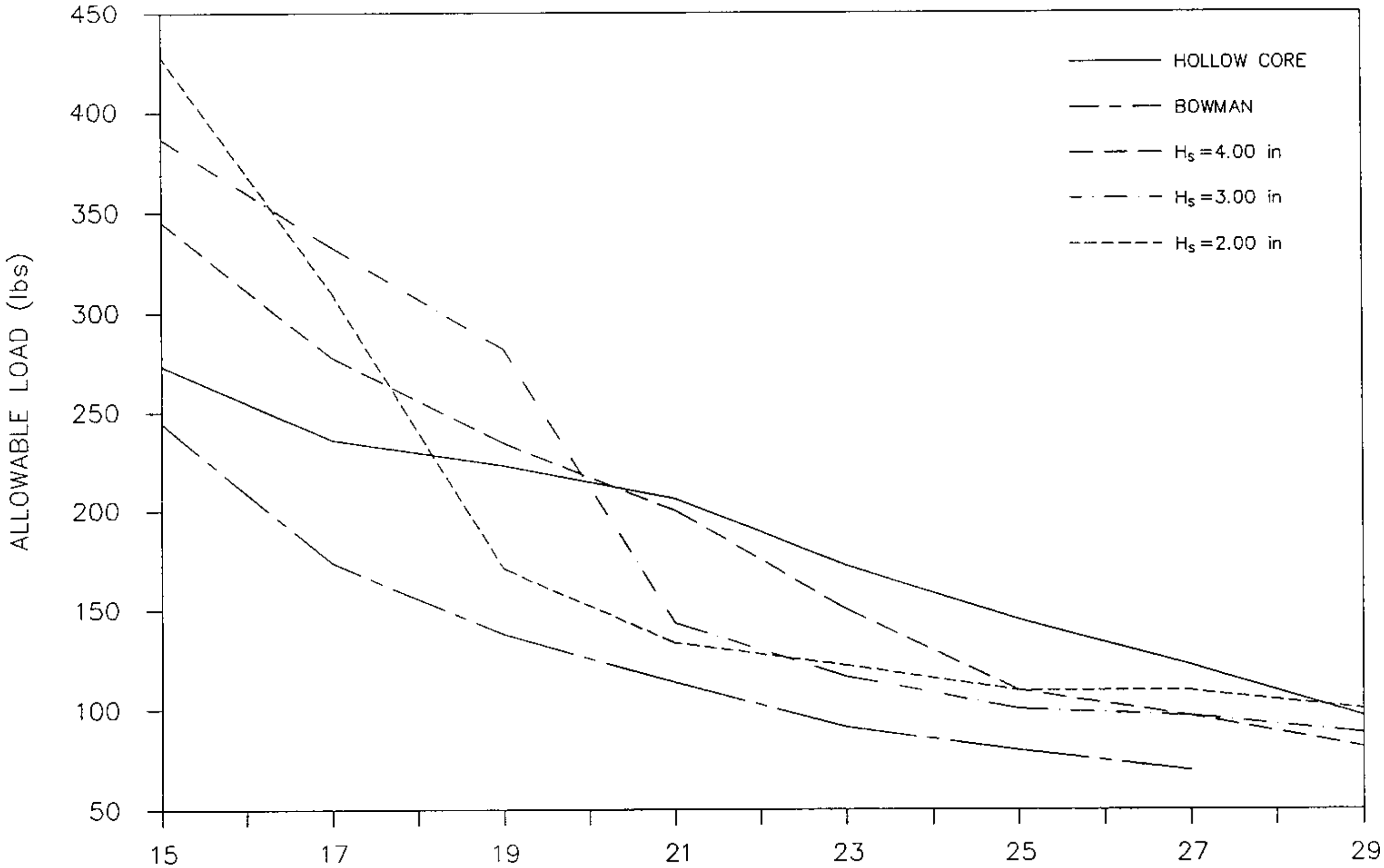


Figure 5.1. Span length vs. allowable load

Table 5.1. Allowable live loads (lb. per sq. ft.) proposed systems vs. hollow core

Span Length	Hollow Core LL	Bowman LL	Bowman H_l (in)	Generic H_s = 4" LL	Generic H_s = 4" H_l (in)	Generic, H_s = 3" LL	Generic, H_s = 3" H_l (in)	Generic, H_s = 2" LL	Generic, H_s = 2" H_l (in)
15	270	270-220	1.25-2.75	420-270	1-2.5	500-280	1.00-2.50	560-300	1.75-3.25
17	200-278	180-170	2.50-4.00	340-210	1-2.5	430-230	1.00-2.50	410-200	1.75-3.25
19	152-289	140-130	3.25-4.75	290-180	1-2.5	370-190	1.00-2.50	190-150	3.00-4.00
21	116-290	111	4.00-5.50	250-150	1-2.5	160-130	2.50-4.00	150-120	4.00-5.50
23	90-247	90	4.50-5.50	180-120	1.5-3.0	130-110	3.25-4.75	140-110	4.75-6.00
25	79-210	80	5.25-5.50	120-100	2.25-3.75	110-90	4.00-5.00	120-100	5.25-6.00
27	61-178	70	5.25-5.50	100-90	3.00-4.00	100-90	4.75-5.00	110	6.00
29	46-149	--[a]	--[a]	90-80	3.50-4.00	90	5.00	100	6.00

[a]Bowman deck is not feasible for a 29-ft. span.

Table 5.2. Flexural strength, k-ft. proposed systems vs. hollow core

Span Length	36-inch wide Hollow Core Strands	36-inch wide Hollow Core ϕM_n	20-inch wide Bowman Strands	20-inch wide Bowman ϕM_n	4-inch wide Generic H_s = 4" Strands	4-inch wide Generic H_s = 4" ϕM_n	4-inch wide Generic H_s = 3" Strands	4-inch wide Generic H_s = 3" ϕM_n	4-inch wide Generic H_s = 2" Strands	4-inch wide Generic H_s = 2" ϕM_n
15'	4-3/8"	45.1	2-7/16"	56.3	1-1/4"	20.01	1-3/8"	22.94	1-3/8"	22
17'	4-7/16"	59.4	2-7/16"	56.3	2-1/4"	22.54	1-3/8"	22.94	1-3/8"	22
19'	4-1/2"	76.7	2-1/2"	63.0	2-1/4"	22.54	1-3/8"	22.94	1-3/8"	22
21'	6-1/2"	105.3	2-1/2"	63.0	2-1/4"	22.54	1-3/8"	22.94	1-3/8"	22
23'	6-1/2"	105.3	2-1/2"	63.0	2-1/4"	22.54	1-3/8"	22.94	1-3/8"	22
25'	6-1/2"	105.3	2-1/2"	63.0	2-1/4"	22.54	1-3/8"	22.94	1-3/8"	22
27'	6-1/2"	105.3	2.06"	83.4	2-1/4"	22.54	1-3/8"	22.94	1-3/8"	22
29'	6-1/2"	105.3	2.06"	83.4	2-1/4"	22.54	1-3/8"	22.94	1-3/8"	22

For longer spans, generic sections can support almost the same live loads as the hollow core sections.

2. The generic sections with H_s = 2 in. are able to carry the highest loads. However, these sections are heavy, hence, not economical for longer spans. Generic sections with H_s = 2 in. necessitates a large mount of concrete topping.
3. Significantly high flexural strengths are attainable using the generic sections. See Table 5.2.
4. The Bowman deck appears to compete favorably for spans up to 15 ft. The live load capacity and the strength of the Bowman deck for 15-ft. spans are higher than the ones carried by a hollow core slab with the same span.
5. For decks of longer spans, the allowable live load is controlled by the allowable bottom fiber stresses.
6. The Bowman deck was not feasible with a span length of 29 ft.
7. Generic sections with depth H_s = 3 in. - 4 in. compete favorably with the hollow core slabs.

5.3. Recommendations

Further investigations should be carried on the proposed system before its use in building construction. First, more analytical work should be carried out with various configurations of generic steel decking in order to ascertain which is the most adaptable to prestressing. Plate theory should be used in further analytical work.

The generic configurations could be used as a basis for selecting certain commercially available steel decking that might be best-suited for use in composite prestressed concrete slabs. An analytical study of the most promising sections would then precede a program of laboratory testing. In the event that satisfactory steel decking could not be found in the marketplace, it might be feasible to actually fabricate a new steel deck which was patterned after the best of the generic decks. Laboratory tests could then be used to establish the viability of the generic deck.

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8. APPENDIX A: DESIGN EXAMPLES OF COMPOSITE PRESTRESSED SLABS

In order to illustrate the procedure used to develop design tables of composite prestressed decks, two examples follow:

8.1. Design of Composite Bowman Slab

The following is the design of the slab shown in Figure 9.1.

Step 1 - Design span

Span Length L = 15 ft.

Step 2 - Material properties

2.1. Precast and topping concrete

$f'_{ci} = 3500$ psi

$f'_c = 5000$ psi $\quad E_c = 33\, w_c^{1.5} \sqrt{f'_c}$

$W_L = 145$ pcf $\quad = 4,074,000$ psi

2.2. Prestressing steel

$f_{pu} = 250$ ksi

2.3. Steel deck

$f_y = 60$ ksi

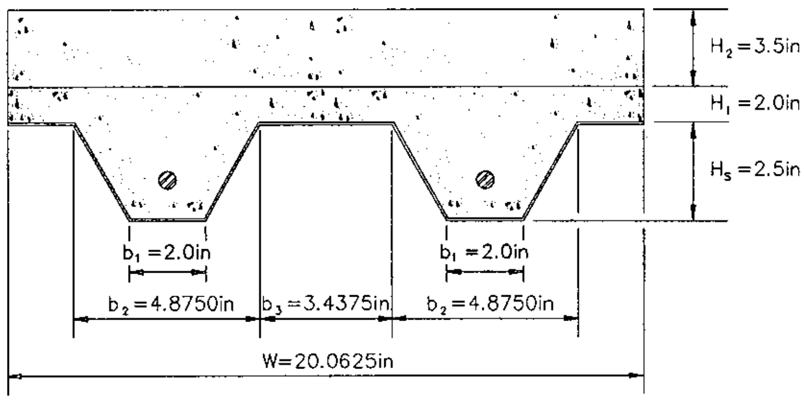
$E_s = 29 \times 10^6$ psi

$W_s = 490$ pcf

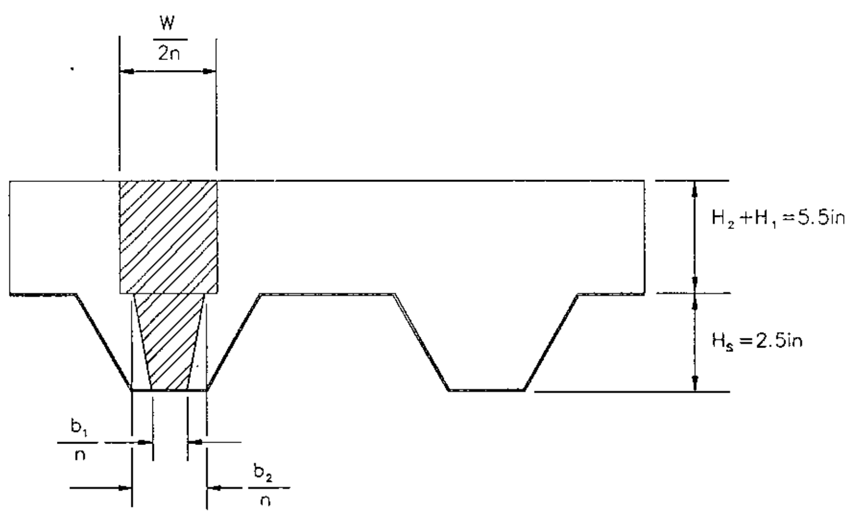
Step 3 - Allowable stresses

Type of stress	Temporary stress	Final stress
Compression	$0.6\, f'_{ci} = 2100$ psi	$0.45\, f'_c = 2250$ psi
Tension	$6\sqrt{f'_{ci}} = -355$ psi	$12\sqrt{f'_c} = -848$ psi

(At ends of simply supported members)



Actual section



Transformed section

Figure 8.1. Composite Bowman deck cross section

Step 4 - Determine the precast and composite section properties

The cross section of the Bowman deck and the transformed section are shown in Figure 9.1.

4.1. 4.5 in. deep precast section

Steel deck section properties

$t = 0.06$ in.

$A_s = 1.2639$ in.^{2}

$I_s = 1.1115$ in.^{4}

$y_s = 1.426$ in.

Placed concrete is transformed into steel

$E_c = 4.074 \times 10^6$ psi

$E_s = 29 \times 10^6$ psi

$n = E_s/E_c = 7.1174$

$1/n = 0.1405$

$H_1 = 2$ in.

By using the transformed area concept, the area, the centroid, and the moment of inertia of the precast composite section are found to be:

$A_p = \Sigma A = 9.32$ in.^{2}

$y_p = \Sigma (A_y)/\Sigma A = 2.68$ in.

$I_p = \Sigma (I + Ad^2) = 13.78$ in.^{4}

The moduli of elasticity of the precast section are:

$S_{bp} = [13.78 \text{ in.}^4/2.68 \text{ in.}] = 5.142$ in.^{3}

$S_{tp} = [13.78 \text{ in.}^4/[4.5 - 2.68] \text{ in.}] = 7.47$

4.2. 8-in. deep composite section

The section properties of the composite section are determined using the transformed area concept.

$A_c = \Sigma A = 19.2 \text{ in.}^2$

$y_c = [\Sigma AY/ A] = 4.52 \text{ in.}$

$I_c = 84.89 \text{ in.}^4$

The moduli of elasticity of the composite section are then:

$S_{bc} = [84.89/4.52] = 18.78 \text{ in.}^3$

$S_{tc} = [84.89/(8-4.52)] = 24.78 \text{ in.}^3$

Step 5. Compute the prestress force required

5.1. Deflections due to the weight of precast section and topping H_2, i.e. δ_1, δ_3 can be determined using these equations:

$$\delta_1 = \frac{5[W_p + W_s]L^4}{384\ E_s I_p}$$

$$\delta_3 = \frac{5\ W_t L^4}{384\ E_s I_p}$$

5.2. Deflection due to the prestress force: δ_2

$$\delta_2 = \frac{F_e L^2}{8\ E_s I_p}$$

The net deflection under the precast deck, the topping and the prestress is set equal to zero.

$$\delta_1 + \delta_3 - \delta_2 = 0$$

The equation expressing the prestress force F is then:

$$F = \frac{5}{48}\ \frac{[W_p + W_s + W_t]\ L^2}{e}$$

W_s = 4.3 lbs./ft.

W_p = 57.71 lbs./ft.

W_t = 70.71 lbs./ft.

[Assume centroid steel = 1 in. from bottom fiber]

$$e = y_p - 1 = [2.68 - 1] \text{ in.} = 1.68 \text{ in.}$$

$$F = \frac{5}{48} \left[\frac{4.3 + 57.71 + 70.71}{1.68}\right] [15]^2 \times [12]$$

$$F = 22.2 \text{ kips}$$

Step 6. Determine allowable live load based on deflection limitation

The maximum deflection due to live load is:

$$\delta_4 = L/360$$

Thus,

$$\delta_4 = \frac{5 W_{LL} L^4}{384 I_c E_s} = \frac{L}{360}$$

Solving for allowable live load:

$$W_{LL} = \frac{384 I_c E_s}{360 \times 5 \times L^3}$$

$$W_{LL} \text{ (lbs./ft.)} = \left[\frac{384 \times 84.89 \times 29{,}000}{360 \times 5 \times [15]^3 \times 144}\right]$$

$$W_{LL} \text{ (lbs./ft.}^2) = \left[\frac{384 \times 84.89 \times 29{,}000}{360 \times 5 \times [15]^3 \times 144}\right] \times \frac{12}{20.0625}$$

$$W_{LL} = 650 \text{ lbs./ft.}^2$$

$$= .65 \text{ k/ft.}^2$$

Step 7. Calculate moments and stresses

The following moments and resulting stresses in the precast deck and the composite slab must be calculated.

1. In the precast under its own weight
2. In the precast due to the weight of concrete topping
3. In the precast due to the prestress force
4. In the composite due to live load

7.1. Find moment and stress under weight of precast

Maximum moment under precast weight it

$$M_p = \frac{[W_p + W_s]\ L^2}{8}$$

$$= \frac{[4.3 + 57.71]}{1000} \times \frac{15^2}{8}$$

$$= 1.74 \text{ k-ft.}$$

Top and bottom stresses at midspan due to weight of precast are:

$$f_{tp} = M_p/S_{tp}$$

$$= \left[\frac{1.74}{7.57}\right] \times 12 = 2.77 \text{ ksi}$$

$$f_{bp} = M_p/S_{bp}$$

$$= \left[\frac{1.74}{5.142}\right] \times 12 = 4.07 \text{ ksi}$$

7.2. Find moment and stresses due to the weight of topping

Maximum moment due to topping weight is

$$M_t = \frac{W_t \times L^2}{8}$$

$$= \left[\frac{70.71 \times 15^2}{1000 \times 8}\right] = 1.99 \text{ k-ft.}$$

Top and bottom stresses at midspan due to weight of topping are:

$$f_{tp} = \frac{M_t}{S_{tp}} = 3.16 \text{ ksi}$$

$$f_{bp} = \frac{M_t}{S_{bp}} = 4.64 \text{ ksi}$$

7.3. Find moments and stresses due to the prestress force

<u>Axial stress due to prestress</u>:

$$f_p = \frac{F}{A_p} = \frac{22.2}{9.32} = 2.4 \text{ ksi}$$

Stresses due to moment caused by prestress:

$$f_{tp} = \frac{F_e}{S_{bp}} = \frac{22.2 \times 1.68}{5.142} = 4.93 \text{ ksi}$$

7.4. Find moment and stresses under live load computed by deflection limitation

Maximum moment under live load is:

$$M_{LL} = W_{LL} \text{ (k/ft.)} \times \frac{L^2}{8}$$

$$M_{LL} = [0.65 \times \frac{20.0625}{12} \times \frac{15^2}{8}$$

$$= 30.56 \text{ k-ft.}$$

The top and bottom stresses due to live load are:

$$f_{tc} = [\frac{30.56}{24.4}] \times 12 = 15 \text{ ksi}$$

$$f_{bc} = [\frac{30.56}{18.78}] \times 12 = 19.5 \text{ ksi}$$

Step 8. Check stresses

The stresses calculated in Step 7 were based on the transformed section properties. Consequently, the stresses should be divided by the ratio n to be compared to the allowable stresses.

8.1. Check stresses at transfer at ends of precast deck

f_{tp} (ends) = [2.4 - 4.92] x 0.1405

= .355 ksi

f_{tp} (allowable) = -.355 ksi OK

f_{bp} (ends) = [2.4 + 7.25] x 0.1405

= 1.36 ksi

f_{bp} (allowable) = 2.100 ksi OK

8.2. Check stresses at service loads at midspan

These stresses caused by the prestress, the weight of precast and topping, and the live load.

f_{tp} (midspan, top of precast) = [2.77 + 3.16 + 2.4 - 4.93]

= 0.477 ksi < 0.45 f_c' =

2.25 ksi (good)

f_{tc} (midspan, top of composite) = [15 x 0.1405] = 2.113 ksi

< 0.45 f_c' = 2.25 ksi

f_{bc} (at midspan) = [-4.07 - 4.64 + 7.25 + 2.4 - 19.5] x 0.1405

= -2.608 ksi

f_{bc} (allowable) = -12 f'_c = -848 psi

f_{bc} allowable < f_{bc} (at midspan)

No good

Step 9. Find allowable live load based on service load stress limitations

f_{bc} (allowable) = -.848 ksi

$$f_{bc} \text{ (allowable)} = \left[-4.07 - 8.64 + 7.25 + 2.4 - \frac{M_{LL}}{S_{bc}}\right] \times 0.1405$$

M_{LL} = 6975.6 x S_{bc} =

$$= \left[\frac{6975.6}{1000} \times \frac{18.78}{12}\right] = 10.92 \text{ k x ft.}$$

$$LL = \frac{10.92 \times 8}{[15]^2} \times \frac{12}{20.0625} = 0.232 \text{ K/ft}^2$$

Step 10. Find flexural strength of the composite slab

The flexural strength of the composite deck can be calculated using the following equation:

$$M_n = A_{ps} f_{ps} [d_p - a/2] + A_s f_y [d_s - a/2]$$

10.1. Find number of 3/8-strands needed assuming a 10% loss of prestress

$$f_{se} = \frac{F}{(No)\ A_{ps}}$$

F = 22.1 k

A_{ps} = 0.08 in.^{2}

f_{se} = 0.7 x 0.9 x 250 ksi = 157.5 ksi

$$No = \left[\frac{22.1}{157.5 \times 0.08}\right] = 1.76$$

Use 2 - 3/8 strands

10.2. Find f_{ps}: Stress in prestressed strands at nominal strength

$$f_{ps} = 250\ [1 - 0.5\rho_p \times \frac{f_{pu}}{f'_c}]$$

$$\rho_p = [A_{ps}/bd] = \frac{0.08 \times 2}{7 \times 20.0625} = 0.0011$$

$$f_{ps} = 250\ [1 - 0.5 \times 0.0011 \times \frac{250}{5}]$$

$$= 243 \text{ ksi}$$

10.3. Compute depth of equivalent rectangular stress block

$$a = \frac{A_p f_{ps} + A_s f_y}{0.85\ f'_c \times b}$$

$$a = \frac{243 \times 0.08 \times 2 + 1.2639 \times 60}{0.85 \times 5 \times 20.0625}$$

$$a = 1.346 \text{ in.}$$

10.4. Compute flexural strength, M_n

$$M_n = 243 \times 0.08 \times 2\ [7 - 1.346/2] + 60 \times 1.2639\ [6.57 - \frac{1.346}{2}]$$

$$= 56.5 \text{ K ft.}$$

<u>Step 11. Check strength capacity vs. ultimate capacity</u>

$$\phi M_n = 56.5 \times 0.9 = 50.85 \text{ K ft.}$$

$$M_u = 1.4\ [M_D] + 1.7\ [M_{LL}]$$

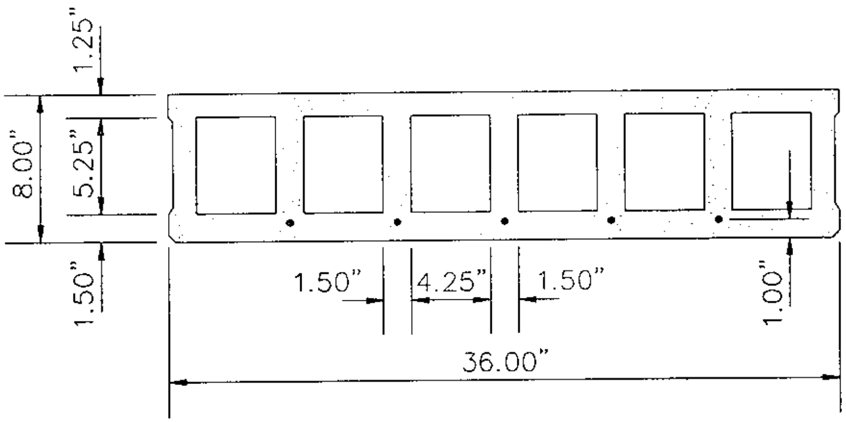
$$= 1.4\ [1.74 + 1.99] + 1.7\ [10.85] \text{ K ft.}$$

$$= 23.67 \text{ K ft.}$$

$\phi M_n > M_u$ Good

9. APPENDIX B: DESIGN TABLES

Table 1. Design load tables for the hollow core planks [11]



Section Properties

A = 154 in.^{2}
I = 1224.5 in.^{4}
b_w = 10.5 in.
y_b = 3.89 in.
S_b = 314.8 in.^{3}
S_t = 297.9 in.^{3}
wt = 53.5 psf

		Spans, ft.									
Strands, 270LR	ϕM_n, ft-k	14	15	16	17	18	19	20	21	22	23
4-3/8"	45.1	317	270	232	200	174	152	133	116	102	90
6-3/8"	65.4			356	311	272	240	212	188	168	150
4-7/16"	59.4			320	278	243	214	189	167	148	132
6-7/16"	85.0					343[1]	311[1]	283[1]	258	231	208
4-1/2"	76.7					327	289	257	229	204	183
6-1/2"	105.3							317[1]	290[1]	267[1]	247[1]

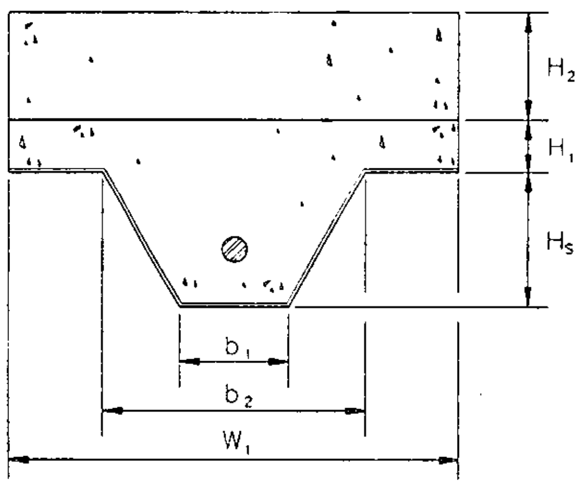
Table 1. (Continued)

		Spans, ft.						
Strands, 270LR	ϕM_n, ft-k	24	25	26	27	28	29	30
4-3/8"	45.1	79	79	69	61	53	46	
6-3/8"	65.4	134	120	108	97	87	78	70
4-7/16"	59.4	118	105	94	84	75	67	59
6-7/16"	85.0	187	169	153	139	126	114	104
4-1/2"	76.7	165	148	134	121	109	99	90
6-1/2"	105.3	227[1]	210[1]	195[2]	178[2]	163[2]	149[2]	137[2]

[1]Values are governed by shear strength.
[2]Values are governed by allowable tension.
[3]Table based on 5000 psi concrete with 6 f'_c allowable tension. Unless noted, values are governed by strength design.

Table 2. Design live loads for the Bowman slab

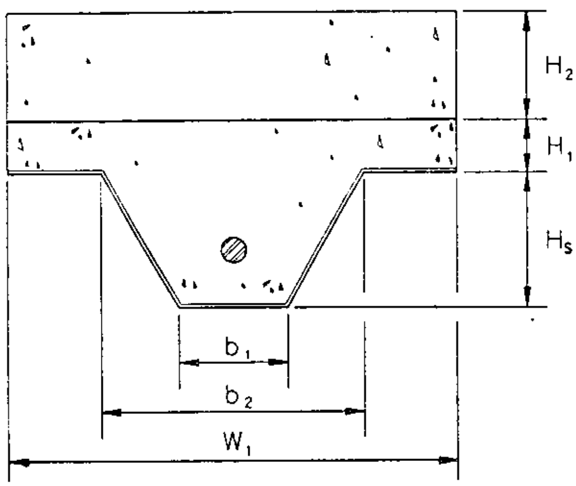
Span Length = 15 ft.



H1 in.	H2 in.	F kip	LL k/ft2	3/8-strands NO	3/8-strands Mn k-ft	7/16-strands NO	7/16-strands Mn k-ft
1.25	4.25	29.65	0.27	3	65.11	2	62.59
1.50	4.00	26.60	0.25	3	65.11	2	62.59
1.75	3.75	24.17	0.24	2	56.52	2	62.59
2.00	3.50	22.17	0.23	2	56.52	2	62.59
2.25	3.25	20.50	0.23	2	56.52	2	62.59
2.50	3.00	19.08	0.22	2	56.52	2	62.59
2.75	2.75	17.86	0.22	2	56.52	2	62.59

Table 2. (Continued)

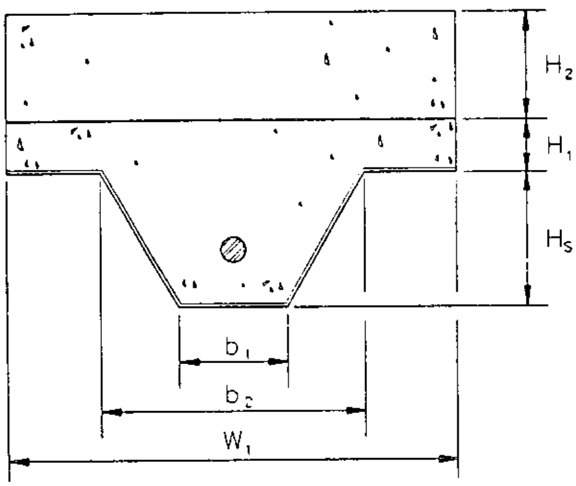
Span Length = 17 ft.



H1 in.	H2 in.	F kip	LL k/ft2	3/8-strands NO	3/8-strands Mn k-ft	7/16-strands NO	7/16-strands Mn k-ft
2.50	3.00	24.51	0.18	2	56.52	2	62.59
2.75	2.75	22.94	0.17	2	56.52	2	62.59
3.00	2.50	21.57	0.17	2	56.52	2	62.59
3.25	2.25	20.36	0.17	2	56.52	2	62.59
3.50	2.00	19.28	0.17	2	56.52	2	62.59
3.75	1.75	18.31	0.17	2	56.52	2	62.59
4.00	1.50	17.45	0.17	2	56.52	2	62.59

Table 2. (Continued)

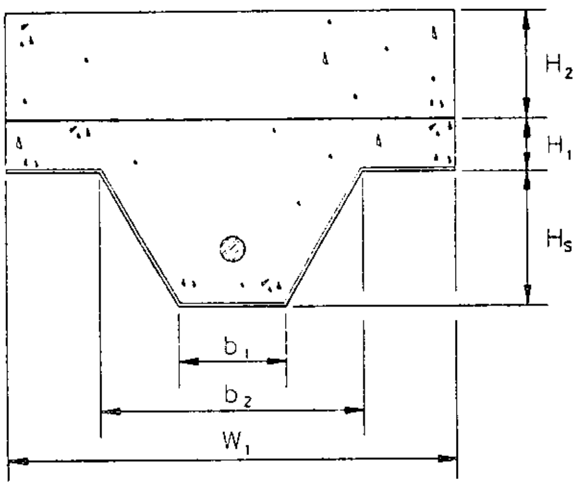
Span Length = 19 ft.



H1 in.	H2 in.	F kip	LL k/ft2	1/2-strands NO	1/2-strands Mn k-ft	7/16-strands NO	7/16-strands Mn k-ft
3.25	2.25	25.43	0.14	2	69.97	2	62.59
3.50	2.00	24.08	0.14	2	69.97	2	62.59
3.75	1.75	22.88	0.14	2	69.97	2	62.59
4.00	1.50	21.79	0.13	1	54.73	2	62.59
4.25	1.25	20.81	0.13	1	54.73	2	62.59
4.50	1.00	19.91	0.13	1	54.73	2	62.59
4.75	0.75	19.09	0.13	1	54.73	2	62.59

Table 2. (Continued)

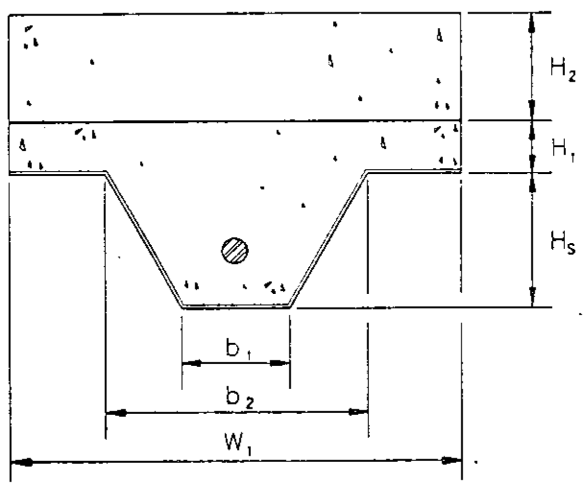
Span Length = 21 ft.



H1 in.	H2 in.	F kip	LL k/ft2	1/2-strands NO	1/2-strands Mn k-ft	7/16-strands NO	7/16-strands Mn k-ft
4.00	1.50	26.62	0.11	2	69.97	2	62.59
4.25	1.25	25.42	0.11	2	69.97	2	62.59
4.50	1.00	24.33	0.11	2	69.97	2	62.59
4.75	0.75	23.32	0.11	2	69.97	2	62.59
5.00	0.50	22.40	0.11	1	54.73	2	62.59
5.25	0.25	21.56	0.11	1	54.73	2	62.59
5.50	0.00	20.77	0.11	1	54.73	2	62.59

Table 2. (Continued)

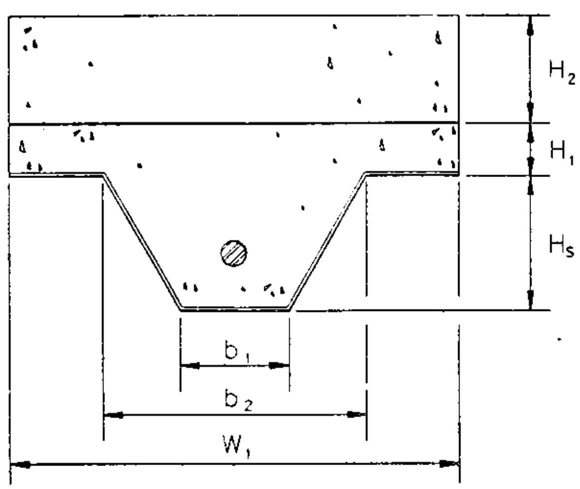
Span Length = 23 ft.



H1 in.	H2 in.	F kip	LL k/ft2	1/2-strands NO	1/2-strands Mn k-ft	7/16-strands NO	7/16-strands Mn k-ft
4.50	1.00	29.18	0.09	2	69.97	2	62.59
4.75	0.75	27.98	0.09	2	69.97	2	62.59
5.00	0.50	26.88	0.09	2	69.97	2	62.59
5.25	0.25	25.86	0.09	2	69.97	2	62.59
5.50	0.00	24.92	0.09	2	69.97	2	62.59

Table 2. (Continued)

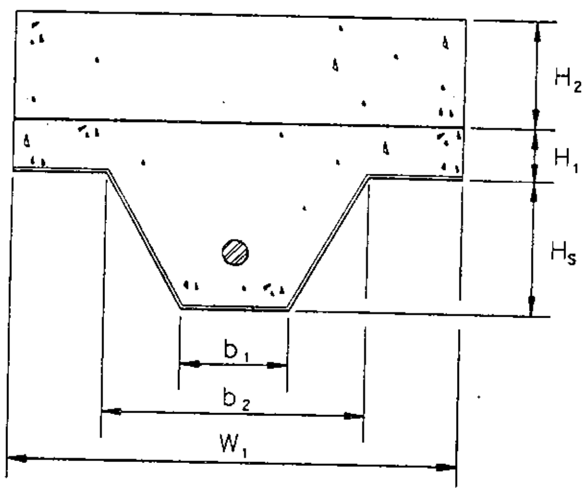
Span Length = 25 ft.



H1 in.	H2 in.	F kip	LL k/ft2	1/2-strands NO	1/2-strands Mn k-ft	7/16-strands NO	7/16-strands Mn k-ft
5.25	0.25	30.55	0.08	2	69.97	2	62.59
5.50	0.00	29.44	0.08	2	69.97	2	62.59

Table 2. (Continued)

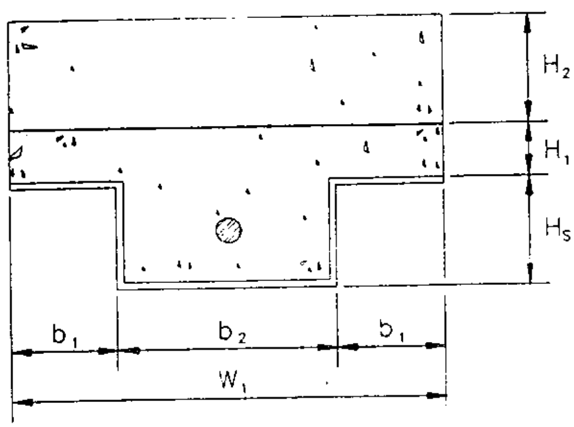
Span Length = 27 ft.



H1	H2	F	LL	0.6-strands		7/16-strands	
				NO	Mn	NO	Mn
in.	in.	kip	k/ft2		k-ft		k-ft
5.25	0.25	35.63	0.07	2	83.38	3	73.49
5.50	0.00	34.34	0.07	2	83.38	3	73.49

Table 3. Design live load tables for a 2-inch deep deck

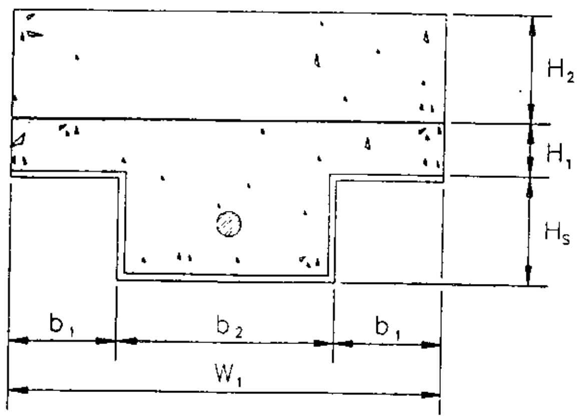
Span Length = 15 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
2.00	1.50	10.23	0.56	2	21.38	1	22.00
2.00	1.75	8.82	0.48	2	21.38	1	22.00
2.00	2.00	7.76	0.42	2	21.38	1	22.00
2.00	2.25	6.93	0.37	2	21.38	1	22.00
2.00	2.50	6.26	0.33	2	21.38	1	22.00
2.00	2.75	5.71	0.30	2	21.38	1	22.00

Table 3. (Continued)

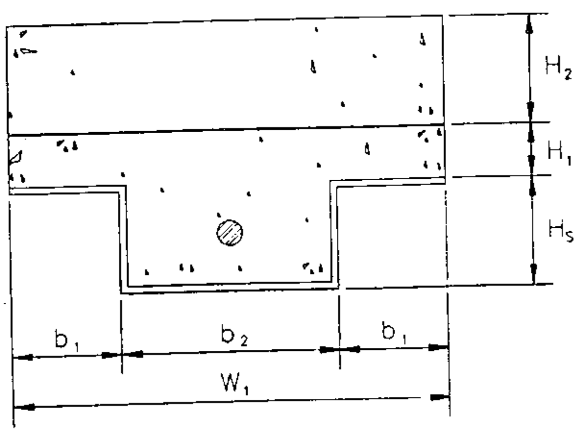
Span Length = 17 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
2.00	1.75	11.33	0.41	2	21.38	1	22.00
2.00	2.00	9.97	0.36	2	21.38	1	22.00
2.00	2.25	8.90	0.31	2	21.38	1	22.00
2.00	2.50	8.04	0.28	2	21.38	1	22.00
2.00	2.75	7.34	0.25	2	21.38	1	22.00
2.00	3.00	6.74	0.22	2	21.38	1	22.00
2.00	3.25	6.24	0.20	2	21.38	1	22.00

Table 3. (Continued)

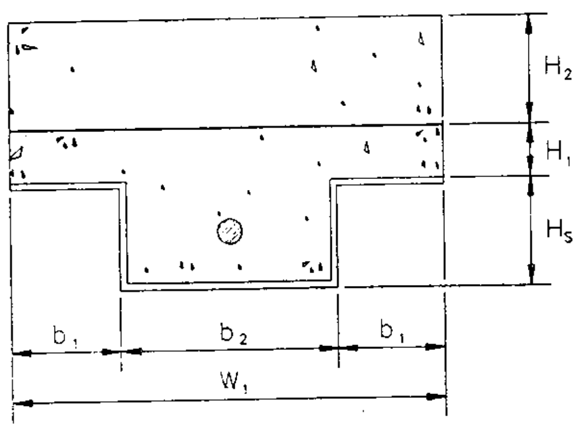
Span Length = 19 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
2.00	3.00	8.42	0.19	2	21.38	1	22.00
2.00	3.25	7.80	0.17	2	21.38	1	22.00
2.00	3.50	7.26	0.16	2	21.38	1	22.00
2.00	3.75	6.79	0.15	2	21.38	1	22.00
2.00	4.00	6.38	0.14	2	21.38	1	22.00
2.00	4.25	6.01	0.14	2	21.38	1	22.00
2.00	4.50	5.69	0.15	2	21.38	1	22.00

Table 3. (Continued)

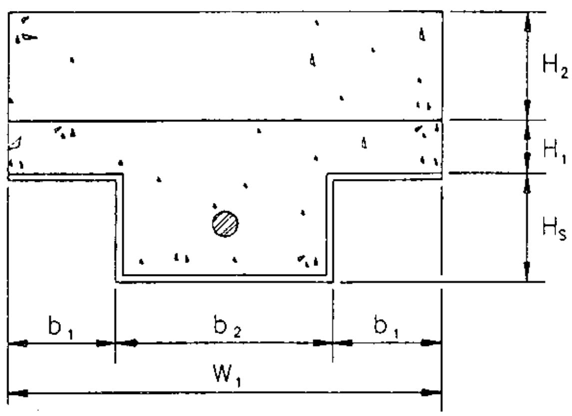
Span Length = 21 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
2.00	4.00	7.79	0.12	2	21.38	1	22.00
2.00	4.25	7.34	0.12	2	21.38	1	22.00
2.00	4.50	6.95	0.12	2	21.38	1	22.00
2.00	4.75	6.59	0.13	2	21.38	1	22.00
2.00	5.00	6.27	0.13	2	21.38	1	22.00
2.00	5.25	5.98	0.14	2	21.38	1	22.00
2.00	5.50	5.71	0.15	2	21.38	1	22.00

Table 3. (Continued)

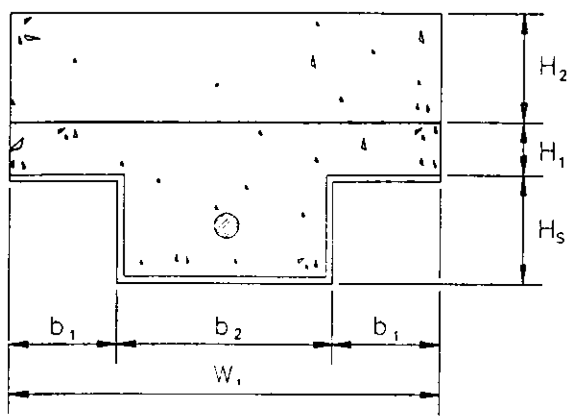
Span Length = 23 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
2.00	4.75	7.91	0.11	2	21.38	1	22.00
2.00	5.00	7.52	0.11	2	21.38	1	22.00
2.00	5.25	7.17	0.12	2	21.38	1	22.00
2.00	5.50	6.85	0.13	2	21.38	1	22.00
2.00	5.75	6.56	0.13	2	21.38	1	22.00
2.00	6.00	6.30	0.14	2	21.38	1	22.00

Table 3. (Continued)

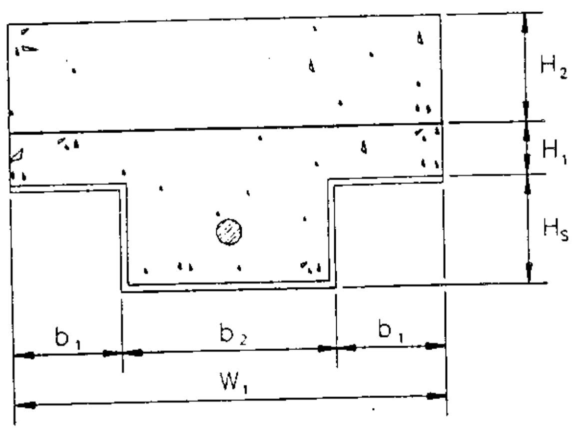
Span Length = 25 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
2.00	5.25	8.47	0.10	2	21.38	1	22.00
2.00	5.50	8.10	0.11	2	21.38	1	22.00
2.00	5.75	7.76	0.12	2	21.38	1	22.00
2.00	6.00	7.44	0.12	2	21.38	1	22.00

Table 3. (Continued)

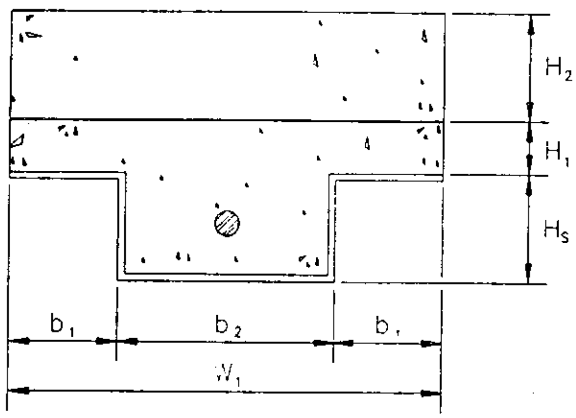
Span Length = 27 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
2.00	6.00	8.68	0.11	2	21.38	1	22.00

Table 3. (Continued)

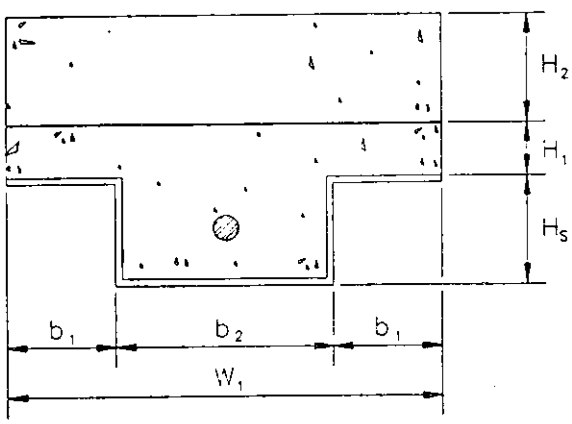
Span Length = 29 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
2.00	6.00	10.01	0.10	2	21.38	1	22.00

Table 4. Design live loads for a 3 inch deep generic deck

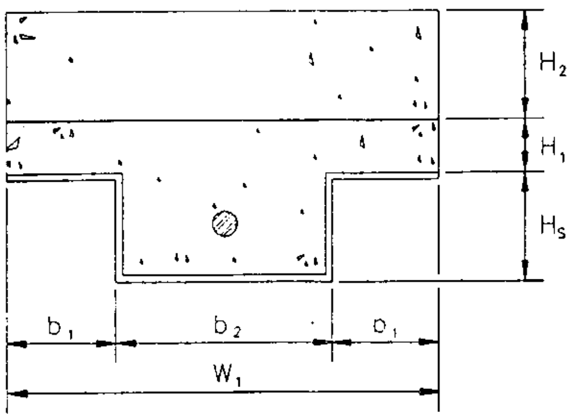
Span Length = 15 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
3.00	1.00	7.16	0.50	2	22.39	1	22.94
3.00	1.25	6.36	0.45	2	22.39	1	22.94
3.00	1.50	5.72	0.41	2	22.39	1	22.94
3.00	1.75	5.21	0.37	1	19.57	1	22.94
3.00	2.00	4.78	0.33	1	19.57	1	22.94
3.00	2.25	4.42	0.31	1	19.57	1	22.94
3.00	2.50	4.11	0.28	1	19.57	1	22.94

Table 4. (Continued)

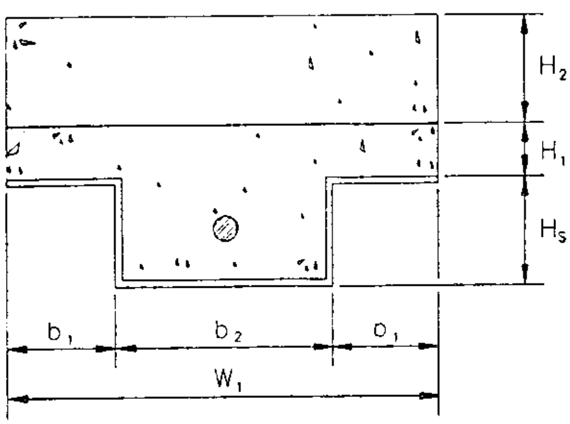
Span Length = 17 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
3.00	1.00	9.20	0.43	2	22.39	1	22.94
3.00	1.25	8.17	0.38	2	22.39	1	22.94
3.00	1.50	7.35	0.34	2	22.39	1	22.94
3.00	1.75	6.69	0.30	2	22.39	1	22.94
3.00	2.00	6.14	0.27	2	22.39	1	22.94
3.00	2.25	5.68	0.25	2	22.39	1	22.94
3.00	2.50	5.28	0.23	1	19.57	1	22.94

Table 4. (Continued)

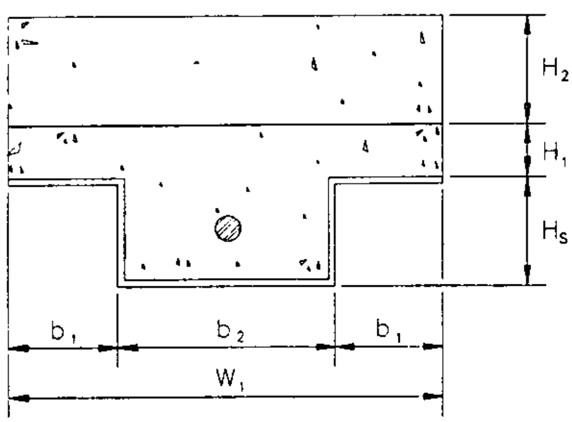
Span Length = 17 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
3.00	1.00	9.20	0.43	2	22.39	1	22.94
3.00	1.25	8.17	0.38	2	22.39	1	22.94
3.00	1.50	7.35	0.34	2	22.39	1	22.94
3.00	1.75	6.69	0.30	2	22.39	1	22.94
3.00	2.00	6.14	0.27	2	22.39	1	22.94
3.00	2.25	5.68	0.25	2	22.39	1	22.94
3.00	2.50	5.28	0.23	1	19.57	1	22.94

Table 4. (Continued)

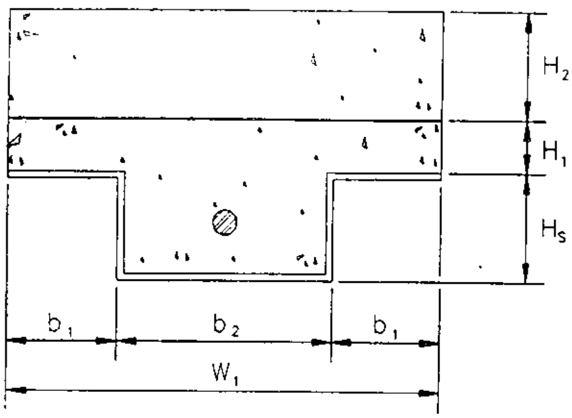
Span Length = 19 ft.



				1/4-strands		3/8-strands	
HS in.	H1 in.	F kip	LL k/ft2	NO	Mn k-ft	NO	Mn k-ft
3.00	1.00	11.49	0.37	3	24.71	1	22.94
3.00	1.25	10.20	0.32	2	22.39	1	22.94
3.00	1.50	9.18	0.29	2	22.39	1	22.94
3.00	1.75	8.36	0.25	2	22.39	1	22.94
3.00	2.00	7.67	0.23	2	22.39	1	22.94
3.00	2.25	7.10	0.21	2	22.39	1	22.94
3.00	2.50	6.60	0.19	2	22.39	1	22.94

Table 4. (Continued)

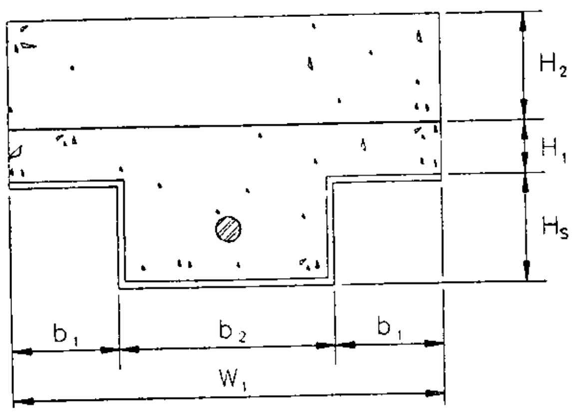
Span Length = 21 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
3.00	2.50	8.06	0.16	2	22.39	1	22.94
3.00	2.75	7.54	0.15	2	22.39	1	22.94
3.00	3.00	7.09	0.14	2	22.39	1	22.94
3.00	3.25	6.68	0.13	2	22.39	1	22.94
3.00	3.50	6.32	0.13	2	22.39	1	22.94
3.00	3.75	6.00	0.13	2	22.39	1	22.94
3.00	4.00	5.71	0.13	2	22.39	1	22.94

Table 4. (Continued)

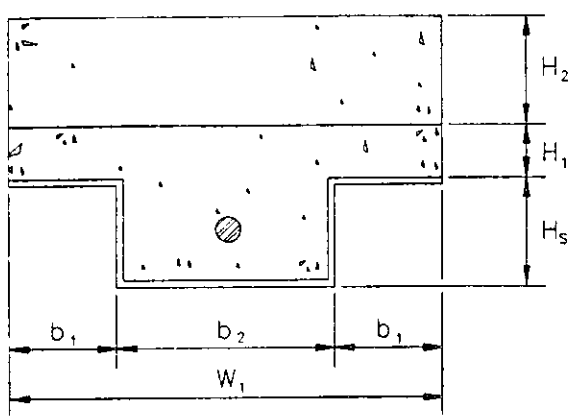
Span Length = 23 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
3.00	3.25	8.01	0.11	2	22.39	1	22.94
3.00	3.50	7.59	0.11	2	22.39	1	22.94
3.00	3.75	7.20	0.11	2	22.39	1	22.94
3.00	4.00	6.85	0.11	2	22.39	1	22.94
3.00	4.25	6.54	0.11	2	22.39	1	22.94
3.00	4.50	6.25	0.12	2	22.39	1	22.94
3.00	4.75	5.99	0.13	2	22.39	1	22.94

Table 4. (Continued)

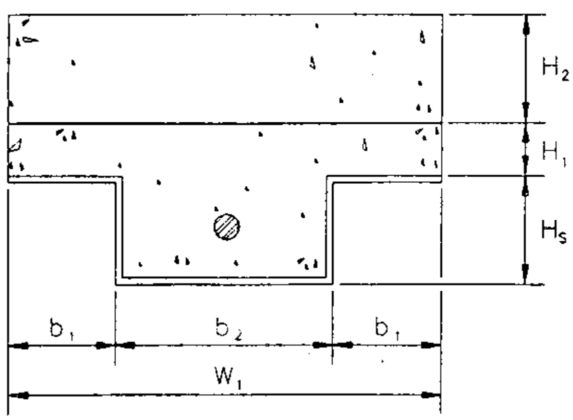
Span Length = 25 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
3.00	4.00	8.10	0.10	2	22.39	1	22.94
3.00	4.25	7.72	0.10	2	22.39	1	22.94
3.00	4.50	7.39	0.10	2	22.39	1	22.94
3.00	4.75	7.08	0.11	2	22.39	1	22.94
3.00	5.00	6.79	0.11	2	22.39	1	22.94

Table 4. (Continued)

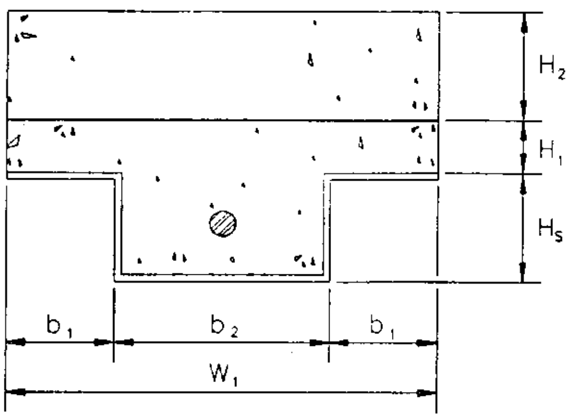
Span Length = 27 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
3.00	4.75	8.25	0.09	2	22.39	1	22.94
3.00	5.00	7.92	0.10	2	22.39	1	22.94

Table 4. (Continued)

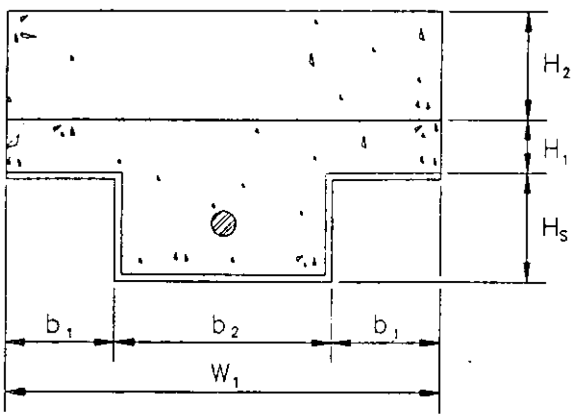
Span Length = 29 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft	3/8-strands NO	3/8-strands Mn k-ft
3.00	5.00	9.14	0.09	2	22.39	1	22.94

Table 5. Design live loads for 4-inch deep generic deck

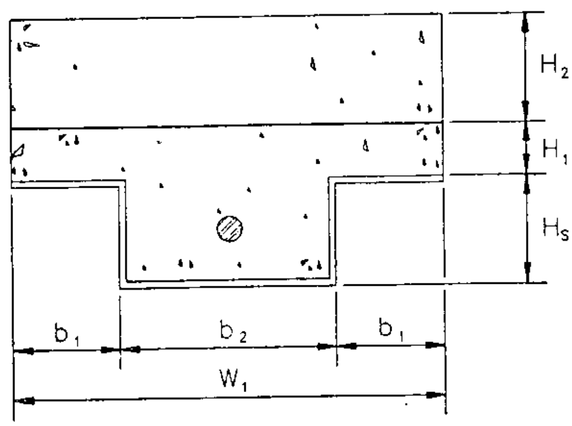
Span Length = 15 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft
4.00	1.00	4.53	0.42	1	20.01
4.00	1.25	4.16	0.38	1	20.01
4.00	1.50	3.85	0.35	1	20.01
4.00	1.75	3.59	0.33	1	20.01
4.00	2.00	3.36	0.30	1	20.01
4.00	2.25	3.16	0.28	1	20.01
4.00	2.50	2.98	0.27	1	20.01

Table 5. (Continued)

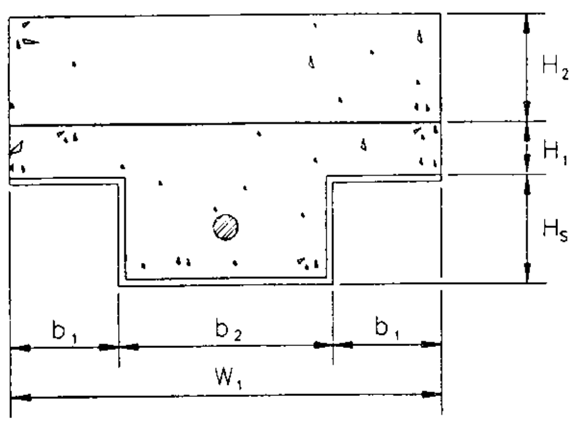
Span Length = 17 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	Mn k-ft
4.00	1.00	5.81	0.34	2	22.54
4.00	1.25	5.34	0.31	1	20.01
4.00	1.50	4.94	0.28	1	20.01
4.00	1.75	4.61	0.26	1	20.01
4.00	2.00	4.31	0.24	1	20.01
4.00	2.25	4.06	0.23	1	20.01
4.00	2.50	3.83	0.21	1	20.01

Table 5. (Continued)

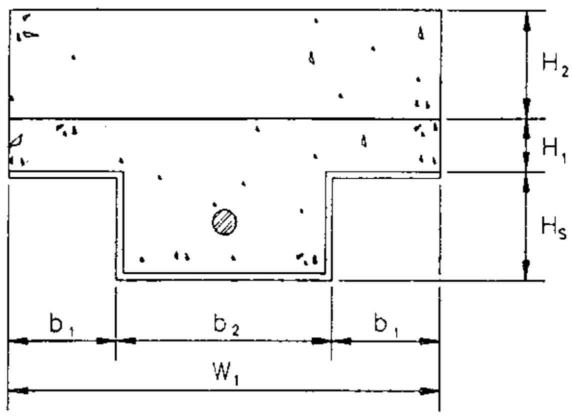
Span Length = 19 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft
4.00	1.00	7.26	0.29	2	22.54
4.00	1.25	6.67	0.26	2	22.54
4.00	1.50	6.18	0.24	2	22.54
4.00	1.75	5.75	0.22	2	22.54
4.00	2.00	5.39	0.20	1	20.01
4.00	2.25	5.07	0.19	1	20.01
4.00	2.50	4.79	0.18	1	20.01

Table 5. (Continued)

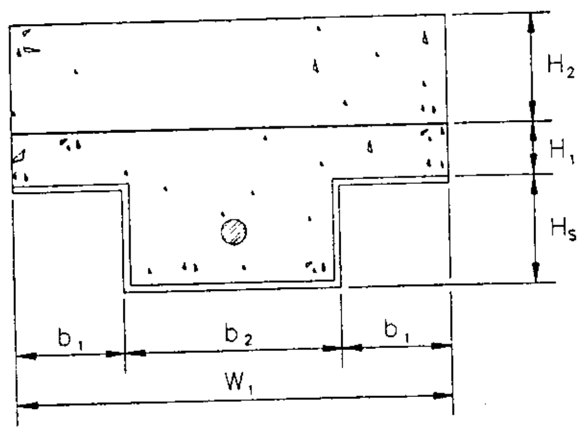
Span Length = 21 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	Mn k-ft
4.00	1.00	8.87	0.25	2	22.54
4.00	1.25	8.15	0.22	2	22.54
4.00	1.50	7.55	0.20	2	22.54
4.00	1.75	7.03	0.18	2	22.54
4.00	2.00	6.58	0.17	2	22.54
4.00	2.25	6.19	0.16	2	22.54
4.00	2.50	5.85	0.15	2	22.54

Table 5. (Continued)

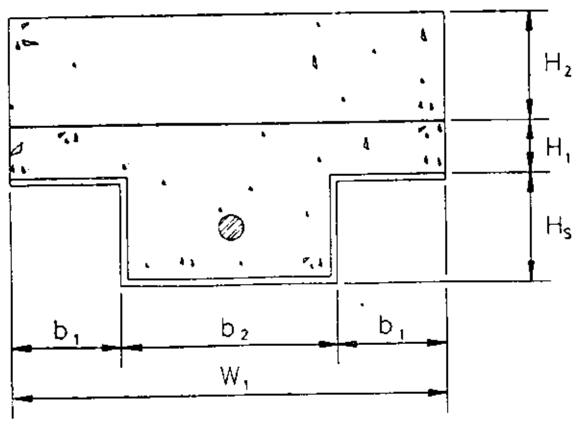
Span Length = 23 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	Mn k-ft
4.00	1.50	9.05	0.18	2	22.54
4.00	1.75	8.43	0.16	2	22.54
4.00	2.00	7.90	0.15	2	22.54
4.00	2.25	7.43	0.14	2	22.54
4.00	2.50	7.02	0.13	2	22.54
4.00	2.75	6.65	0.12	2	22.54
4.00	3.00	6.32	0.12	2	22.54

Table 5. (Continued)

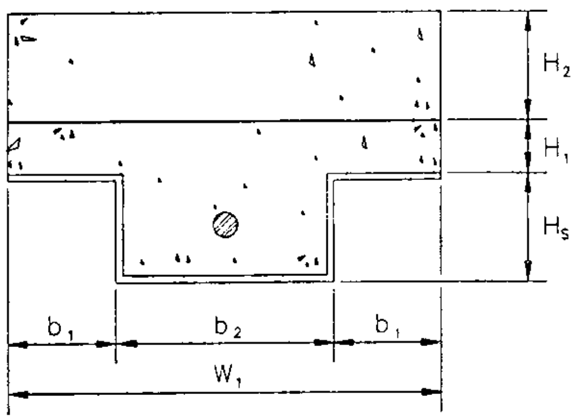
Span Length = 25 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	Mn k-ft
4.00	2.25	8.78	0.12	2	22.54
4.00	2.50	8.29	0.11	2	22.54
4.00	2.75	7.86	0.11	2	22.54
4.00	3.00	7.47	0.10	2	22.54
4.00	3.25	7.12	0.10	2	22.54
4.00	3.50	6.80	0.10	2	22.54
4.00	3.75	6.52	0.11	2	23.54

Table 5. (Continued)

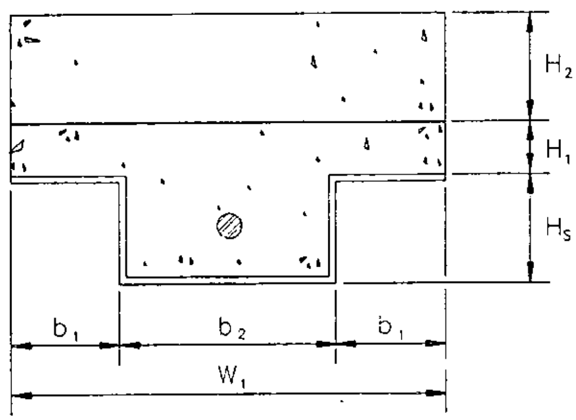
Span Length = 27 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	Mn k-ft
4.00	3.00	9.71	0.09	2	22.54
4.00	3.25	8.31	0.09	2	22.54
4.00	3.50	7.94	0.09	2	22.54
4.00	3.75	7.60	0.09	2	22.54
4.00	4.00	7.29	0.10	2	22.54

Table 5. (Continued)

Span Length = 29 ft.



HS in.	H1 in.	F kip	LL k/ft2	1/4-strands NO	1/4-strands Mn k-ft
4.00	3.50	9.16	0.08	2	22.54
4.00	3.75	8.77	0.08	2	22.54
4.00	4.00	8.41	0.09	2	22.54

Table 6. Jacking forces and corresponding camber

Test 1		Test 2	
Def.	Pres.	Def.	Pres.
0	0	0	0
0.317	5	0.327	5
0.541	10	0.466	10
0.668	15	0.694	15
0.878	20	0.922	20