ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

FACULTY OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

DESIGN CHARTS FOR COMPOSITE SLAB OF 80mm AND 100mm CONCRETE THICKNESSES

A thesis submitted to the school of Graduate Studies in Partial fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Structures)

By

Michael Abebe

Advisor: **Dr. Shifferaw Taye**

July 2005

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In loving memory of my Mother

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A_{webs} = Area of the webs of the steel deck per unit width of the slab

A_{bf} = Area of bottom flange of the steel deck per unit width of the slab.

A_p = Profile steel deck cross sectional area per unit width

A_{pe} = Profile steel deck effective cross sectional area per unit width

A_r = Total stiffened area comprising the flange stiffener plus the two adjacent effective portions of the flange

$A_{r,eff}$ = Effective area of flange stiffener

b = Width of slab

b_o = Mean width of the rib

b_{eff} = Effective width of a compression element

B_f = Width of a flange for flange curling

b_p = Top width of the flange in profile sheet

b_u = Bottom width of the flange in profile sheet

C_s = Cell /trough/ spacing

D = Nominal total slab depth

d_c = Depth of concrete slab

d_d = Depth of steel deck

d_p = Distance of the steel deck centroid to the top surface of the slab (effective depth)

d_r = Depth of stiffener

D_w = Sloping distance between the intersection points of the web and flange

e = Distance from the centroid of the effective area of the steel sheet to its underside

e_r = Distance from the centroid of the flange stiffener to its underside

E_{ap} = Modulus of elasticity of steel deck

E_c = Modulus of elasticity of Concrete

E_s = Modulus of elasticity of reinforcing steel

f_c = Stress in deck due to casting

f_{ck} = Concrete compressive strength

f_{yc} = Yield stress of deck, corrected to account for casting stresses

f_{yp} = Yield stress of deck

I_{avg} = average cross sectional stiffness

I_{cc} = Cracked cross-sectional stiffness

I_{cu} = Uncracked cross-sectional stiffness

I_{eff} = Effective second moment area of steel deck

I_r = Second moment area of a flange stiffener, about its own centroid

k = Coordinate intercept of reduced experimental shear-bond line

K = Bond force transfer property, $K_3/(K_1 + K_2)$

K_1 = Coefficient that measures the influence of the steel deck depth on the development of the shear bond along the shear span

K_2 = Coefficient that indicates the mechanical bond performance along the shear span L.

K_3 = Coefficient that accounts for the increase in efficiency of the embossment with increasing slab width

l_{nf} = Clear span length

L_f = Length of shear span (¼ L for uniformly loaded slabs)

m = Slope of reduced experimental shear-bond line

M_{et} = First Yield Moment per cell, first yield method

M_{sd} = The design value of bending due to load

$M_{p.Rd}$ = The design value of the positive bending resistance of the section

M_t = Bending moment, modified for bond limitations, ASCE method

n = Modular ratio, E_s / E_c

N_{cf} = Compression force in concrete slab over the width b

N_p = Tensile force in the profile steel deck

Q = Live load intensity

$S_{eff,t}$ = Effective section modulus of top section

$S_{eff,b}$ = Effective section modulus of bottom section

t_s = Thickness of the steel deck

$t_{s,eff}$ = Effective thickness of the steel deck

V_u = Ultimate transverse shear-bond resistance (per unit width)

W_{ser} = Serviceability loads

x_c = Position of the elastic neutral axis of the composite slab to the upper side of the slab, cracked section analysis

x_u = Position of the elastic neutral axis of the composite slab to the upper side of the slab, uncracked section analysis

x_{pl} = Stress block depth

z = Lever arm between compressive and tensile forces

ε = $\sqrt{280 / f_{yp}}$, (f_{yp} in N/mm^2)

ρ = Reinforcement ratio

ρ_b = Balanced reinforcement ratio

δ = Deflection

ABSTRACT

The use of steel deck in the construction of floors began in the 1920's. The concept of using steel deck to act compositely with the concrete slab began in the 1950's. A composite slab comprises steel decking, reinforcement and cast in situ concrete. Modern profiled steel are mostly designed to act as both formwork and Composite slabs.

Composite construction in Ethiopian has not yet developed. This can be due to many factors. But to name a few, unavailability of the profile sheet locally and limited exposure to composite steel decks are some factors.

In this thesis work, three analytic methods to calculate the capacity of composites is used. These are the full flexural method, first yield method and ASCE appendix D method.

The full flexural method assumes that full interaction is present between the concrete section and profile steel sheet. The First Yield Method predicts the slab capacity to be the load that causes the bottom flange of the deck to reach yield stress. The Alternate Appendix D Method considers the shear transfer ability of different decks by the application of relaxation factors that describe the deck and embossment properties. The design strength is the multiplication product of the first yield strength and the relaxation factor.

Based on these three analytic methods, design charts for different combinations of cross sectional values and material properties are developed.

1 INTRODUCTION

1.1 GENERAL

The use of steel deck in the construction of floors began in the 1920's. The deck commonly was the main structural component for the floors of steel framed buildings. The addition of concrete cover provided no structural strength, but rather served the purposes of fire protection, a means to level the top surface of the floor, and a means to distribute the load.

The concept of using steel deck to act compositely with the concrete slab began in the 1950's. A composite slab comprises steel decking, reinforcement and cast in situ concrete as shown in Figure 1-1. When the concrete has hardened, it behaves as a composite steel-concrete structural element. Modern profiled steel are mostly designed to act as both permanent formwork during concreting and tension reinforcement after the concrete has hardened. After construction, the composite slab consists of a profiled steel sheet and an upper concrete topping which are interconnected in such a manner that horizontal shear forces can be transferred at the steel-concrete interface.

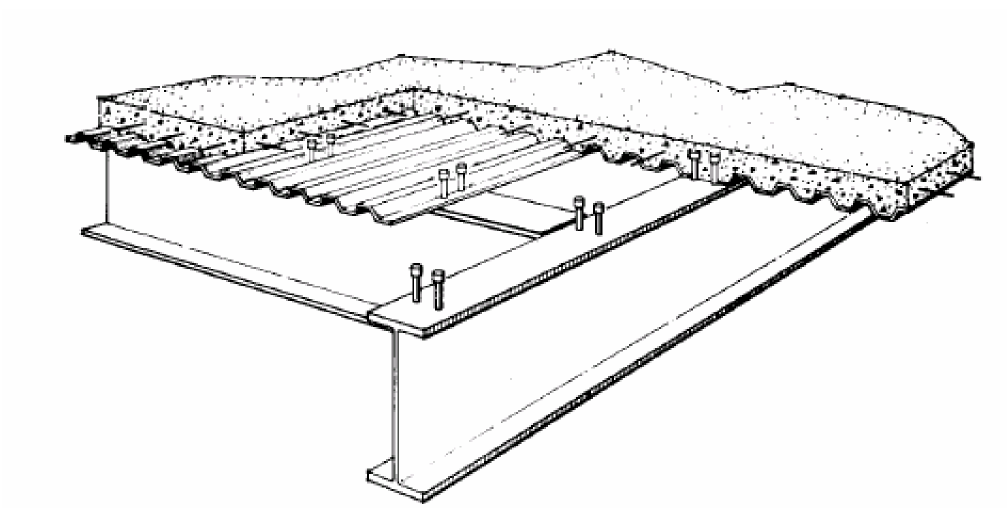


Figure 1-1: Composite slab with profiled sheeting

Composite floor construction is essentially an overlay of one-way spanning structural elements. The slabs span between the secondary floor beams, which span transversely between the primary beams. The latter in turn span onto the columns. This set of load paths leads to rectangular grids, with large spans in at least one direction [Structual Steel Eurocodes, 2001]. Typical example of composite construction is shown in Figure 1-2.

Figure 1-2: A typical example of composite construction, showing deck placing on a steel frame

Composite slabs are supported by steel beams, which normally also act compositely with the concrete slab.

1.2 ADVANTAGES AND DISADVANTAGES OF COMPOSITE SLABS

Composite floor construction used for commercial and other multi-storey buildings, offers a number of important advantages to the designer and client.

The advantages of composite slabs are

- Simplicity of construction
- Acts as stay-in-place formwork and offers an immediate working platform
- Lighter construction than a traditional concrete building
- Less on site construction
- Acts as slab reinforcement
- Ease of transportation and installation
- Strict tolerances achieved by using steel members manufactured under controlled factory conditions to established quality procedures.

Another benefit is that the cellular shape of some decks provides room for the flush fitting of ceiling fixtures. They are easier and faster to construct than traditional reinforced concrete slabs because it is easier to install the deck, which acts as the reinforcing, than to lay out a series of reinforcing bars.

Some of the disadvantages are

- Need extra care in areas of concentrated traffic or storage so that the steel deck is not damaged
- Prior to concreting, the steel-deck panels must be cleaned of all dirt, debris, oil and all foreign matter
- High cost of materials for both the steelwork members and the necessary fire protection system.
- The non-availability of materials locally and therefore total reliance on overseas suppliers for delivery, quality control, etc.

1.3 MOTIVATION

Structural Engineering is a field in which there is a constant revolution-taking place. New and innovative structural forms and technological developments are being created everyday.

Following this technological development, there is an enhanced intention imposed on building construction industry to improve time, economy and structural efficiency of structures. Especially when high-rise buildings are constructed it is true that a lot of time, money and labor is needed.

In Ethiopia, building construction meant for different purposes is being carried out by different institution. In order to get the aforementioned benefits the desired profiled steel sheet should be produced and applied locally.

Thus, this study is intended in order to prepare design charts using spreadsheet software that will be used as a reference for designers and local manufacturers in the country. Spreadsheets offer simple method of obtaining solutions to a variety of problems and are very efficient with regard to the amount of required to obtain these solutions.

1.4 OBJECTIVE AND SCOPE OF RESEARCH

The objective of this research is to prepare design tables and charts by determining cross sectional dimensions of composite slabs for different loading and span.

The research is based on available analytic methods to determine the capacity of composite slabs. Hence laboratory works are not included.

1.5 OVERVIEW

This thesis is organized as follows. Chapter two contains a summary of previous and current research on the design, testing, and behavior of composite slabs. This section discusses on how the resistance of composite cross sections is determined. In addition, the available analytic method to determine the capacity of composite slabs at the construction and final stages are discussed.

Chapter three elaborates the methods used and followed on developing the design tables and charts. As an introductory, one of the design tables is used to show the different layouts developed.

The last chapter, Chapter four, gives conclusions and recommendations with some recommendation on future work.

On the accompanying CD, the complete design tables and charts for the concrete and steel sections specified are included. They are saved using PDF format.

2 STEEL – CONCRETE COMPOSITES: DESIGN AND CONSTRUCTION

2.1 GENERAL

The development of composite steel deck began in the 1950's [Samuel, 1992 and Shen, 2001]. Composite floors are systems in which the steel deck acts as the primary tensile reinforcement because there is some form of mechanical interlocking between the steel and concrete.

Profiled decking is cold formed: a galvanized steel coil goes through several rolls producing successive and progressive forming.

2.2 CONCEPTS

2.2.1 PROFILED DECKING TYPES

The two basic deck profile types are trapezoidal and re-entrant, as illustrated in Figure 2-1. In some cases, indentations or embossments of various shapes are pressed into the deck to provide shear resistance. The addition of shear studs also aids composite action [Widajaja, 1997].

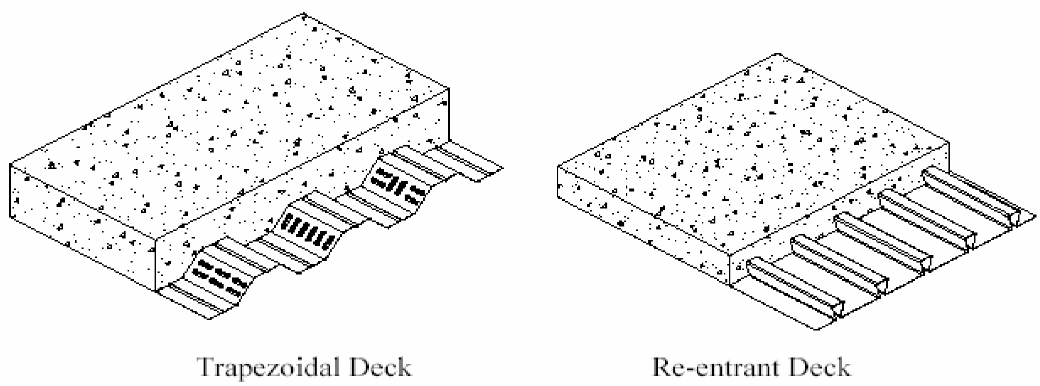


Figure 2-1: Typical Types of Composite Deck and Shear Transfer Devices

The profiled sheeting characteristics are generally the following [Structual Steel Eurocodes, 2001 and Steelbiz, 2000]:

- Thickness between 0.75mm and 1.5mm and in most cases between 0.9mm and 1.2mm;
- Depth between 40mm and 80mm;
- Standard protection against corrosion by a thin layer of galvanizing on both faces.

2.2.2 STEEL TO CONCRETE CONNECTION

The profiled sheeting should be able to transfer longitudinal shear to concrete through the interface to ensure composite action of the composite slab. The adhesion between the steel profile and concrete is generally not sufficient to create composite action in the slab and thus an efficient connection is achieved with one or several of the following [Porter and Ekberg, 1976 and Structual Steel Eurocodes, 2001]

- Appropriate profiled decking shape (re-entrant trough profile), which can effect shear transfer by frictional interlock;
- Mechanical anchorage provided by local deformations (indentations or embossments) in the profile;
- Holes or incomplete perforation in the profile;
- Anchorage element fixed by welding and distributed along the sheet;
- End anchorage provided by welded studs or another type of local connection between the concrete and the steel sheet;
- End anchorage by deformation of the ribs at the end of the sheeting

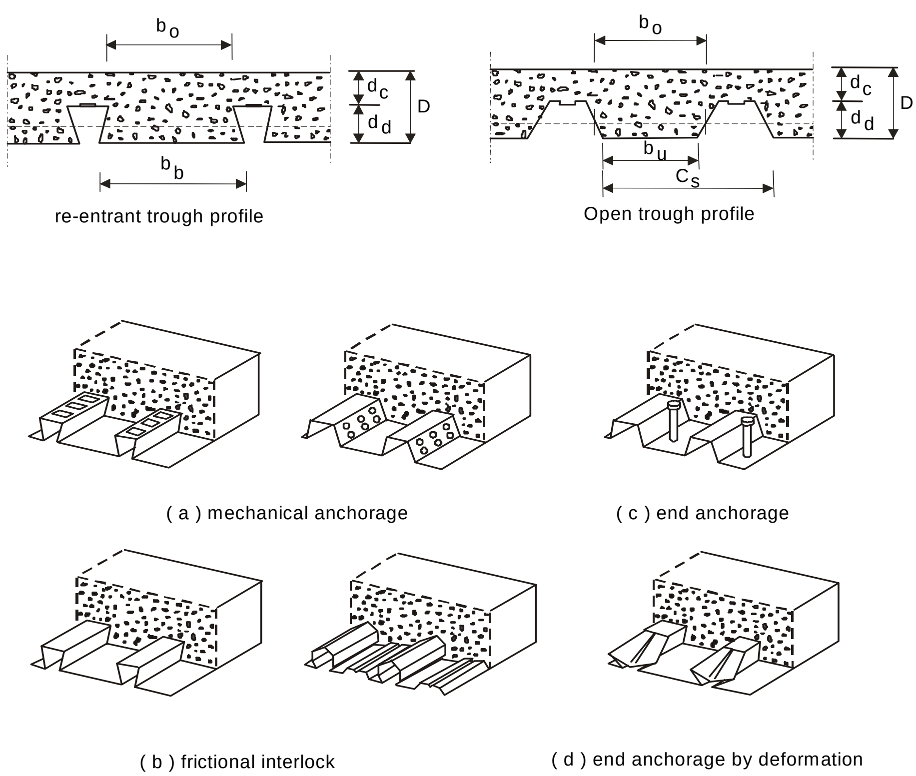


Figure 2-2: Typical forms of interlock in composite slabs

2.2.3 MECHANICAL PROPERTIES

2.2.3.1 Partial Safety factors

Based on values tabulated [EBCS 2, 1995] and taking Class 1 works, the partial safety factors employed in the thesis work are summarized in Table 2-1

Table 2-1: Partial safety factors

	Material Safety Factor				**Load Safety factor**	
	Concrete	Profile sheet	Reinforcing steel	Shear stud	Dead	Live
γ	1.5	1.10	1.15	1.25	1.3	1.6

2.2.3.2 Materials

2.2.3.2.1 Profiled sheeting

Steel used for the fabrication of profiled sheeting has minimum nominal yield strength of 220N/mm^2. The nominal values of yield strength and secant modulus of elasticity for the steels used are shown in Table 2-2.

Table 2-2: Steel grades and associated properties of profile sheet

Steel grade	**f_{yp} [N/mm^2)**	**E_{ap} [N/mm^2]**
Fe360	235	205 000
Fe430	275	

2.2.3.2.2 Concrete

The Concrete used for composite slabs is made with normal weight aggregate. The most commonly used grades of concrete [EBCS 2, 1995] are given in Table 2-3, which also gives the following properties: characteristic cylinder 28 days compressive strength, f_{ck}; mean tensile strength, f_{ctm}, characteristic tensile strength of concrete, f_{ctk} and the secant modulus of elasticity, E_{cm}.

Table 2-3: Concrete grades and associated properties of concrete

Concrete grade	**C20/25**	**C25/30**
f_{ck} [N/mm^2]	20	25
f_{ctm} [N/mm^2]	2.2	2.5
f_{ctk} [N/mm^2]	1.5	1.7
E_{cm} [kN/mm^2]	28.9	30.2

2.2.4 ACTIONS

Verification for the ultimate and serviceability limit states is done in accordance with part 4 of BS5950.

For the situation where the profiled sheeting acts as formwork, the following loads would be considered in the calculations:

- Self-weight of the profiled sheeting.
- Weight of the wet concrete.
- Construction load and temporary storage load, if applicable.

Construction loads represent the weight of the operatives and concreting plant and take into account any impact or vibration that may occur during construction. According to BS5950-4, basic construction load on one span of the sheeting should be taken as not less than 1.5kN/m^2. The other spans should be taken as either loaded with the weight of the wet concrete slab plus a construction load of one-third of the basic construction load, or unloaded apart from the self-weight of the profiled steel sheets, whichever is the more critical (Figure 2-3). The loading pattern shown in Figure 2-3 is for a continuous slab. The research is primarily considering simple span only. In such instances, to have the maximum action, the whole span would be loaded.

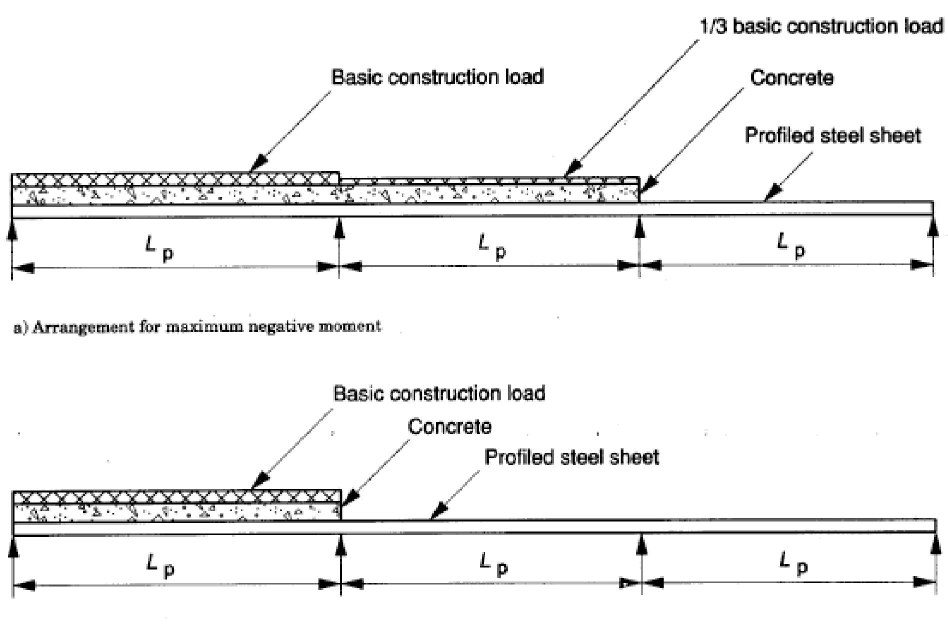


Figure 2-3: Load arrangements for sheeting acting as shuttering (Continuous slab case)

For the situation where the steel and the concrete act compositely /composite stage/, the loads acting on the slab are [BS5950 Part 4, 1990 and BS5950 Part 6, 1999]

- Self-weight of the slab (profiled sheeting and concrete);
- Weight of floor finishes
- Imposed loads
- Other permanent self-weight loads (not load carrying elements);
- Climatic actions (temperature, wind...).

For typical buildings, temperature variations are generally not considered.

2.3 RESEARCHES CARRIED OUT

Since the inception of composite deck, many institutions have investigated its behavior. In the early stages, as manufacturers created new deck designs, they had to determine the strength of their product experimentally by performing numerous full-scale tests. These tests were costly and tedious because they had to be very specific to the configurations under consideration (concrete strength, gage of deck, profile shape, etc).

In 1967 the American Iron and Steel Institute (AISI) sponsored research at Iowa State University to develop a standard design procedure for composite slab floors. Ekberg directed the testing of 353 full-scale composite floor specimens with various configurations. From the performance of the tested slabs, Porter and Ekberg (1975; 1976) made numerous observations on the behavior of composite slabs. They determined that failure occurs in three modes:

- longitudinal shear bond failure
- flexure of an under-reinforced section
- flexure of an over-reinforced section

Numerous tests have shown that longitudinal shear failure is most often the governing mode (Porter and Ekberg 1975; 1976; Ong and Mansu 1986). Therefore, shear bond strength is the focus of much of the research.

In the early 1970’s the Steel Deck Institute sponsored research at West Virginia University to better predict composite slab strengths in the field. This research involved testing 25 slabs (both single span and two span continuous) of varying width in which embossed deck acted as the only reinforcing. The embossments were generally horizontal or vertical.

The analysis was based on the limitation of stresses at the extreme fibers and on the consideration of shear bond failure (Luttrell and Davison 1973). Luttrell performed an exhaustive analysis on the results of these and other experimental tests from eighteen years of testing at West Virginia University. Failure of the embossed decks occurred more gradually than plain deck, the slab continued to sustain load even after slip initiated. He postulated that the embossments not only increased strength by providing mechanical shear resistance, but also increased the stiffness of the webs. Greater web stiffness increased the resistance to over-ride, which occurs when the concrete moves vertically to go over the deck as it slips horizontally.

Deeper slabs also seemed to provide better shear resistance allowing it to more closely approach full bending capacity (Luttrell 1987). Luttrell also made some observations regarding the end conditions of the slab specimens. He determined that continuous specimens did have capacities about 10%-15% greater than simple span specimens. The use of shear studs did significantly increase strength.

The investigation of end anchorage conditions later continued at Virginia Tech with multi-span tests consisting of different end conditions such as shear studs, and a welded angle. These end anchorage devices did improve performance. If the system had a sufficient number of shear studs, the slab did attain its maximum plastic bending capacity (Easterling and Young 1992; Terry 1994)

When comparing decks with embossments of mostly vertical shape versus mostly horizontal, Luttrell concluded that the vertical embossments are almost 50% more effective in shear resistance than the horizontal ones. The decks with only horizontal embossments did not sustain much load after the chemical bond was destroyed. However, the horizontal embossments helped resist vertical separation, thereby also contributing to composite interaction. (Luttrell and Davison 1973; Luttrell 1987)

Stark (1978) also experimentally investigated the behavior of composite slabs. He classified slabs to be either ductile or brittle. Brittle behavior occurs when the maximum flexural strength is reached soon after shear failure or slip initiates. The sustained load drops suddenly. A ductile slab, however, continues to sustain load even after slip initiates. The curvature of the slab increases and the steel and concrete components no longer have a common neutral axis.

Luttrell and Prasannan (1984) reconsidered the assumption that in the flexure mode, the slab behaves like a reinforced concrete section with the deck's tensile force acting at its centroid. They argued that the steel deck behaves differently than embedded reinforcing bars because the deck is only bonded on one surface and is free to deflect on the other surface. Therefore the geometry of the deck has a great effect on the shear resistance.

Full-scale specimens are very costly and time-consuming to test, so attention turned to the development of small-scale tests. The intent of these smaller specimens was to test the shear bond specifically, which consistently was the governing mode of failure. The development of shear bond tests stemmed from existing push-off tests that measure the shear connection between a slab and a girder in composite systems. The test set-ups all consisted of concrete cast onto the steel deck and the application of a load to shear the concrete off the deck. [Daniel and Crisinel, 1993]

Several researchers considered the effect of deck and embossment parameters using shear bond tests. A summary of their findings is as follows [Shen, 2001]:

- Embossments with the greatest vertical face against the concrete were the most effective in shear resistance, even if that face did not have the greatest overall vertical height. (Jolly and Zubair 1987)
- Discontinuities of embossment shape (such as a cross) caused more flexibility of the deck, promoting concrete over-ride (Jolly and Zubair 1987).
- Increasing embossment frequency, which required decreasing size did not improve shear resistance (Jolly and Zubair 1987).
- Increased embossment depths were most effective, but caution should be taken against tear during production. Depth was the most influential shape factor. (Jolly and Zubair 1987; Makelainen and Sun 1998, and Crisinel and Schumaker 2000)
- The optimal embossment location was at the middle of the web. Embossments at the corner of the web and flange were very difficult to construct and did not display much improvement in shear resistance. Embossments in the tension

flange flattened upon loading, decreasing their effectiveness. Those in the compression flange acted as initial deformations, which promote buckling. (Jolly and Zubair 1987, Makelainen and Sun 1998)

- Increasing embossment length increased performance, but there seemed to be a length limit when improvement ceased (Makelainen and Sun 1998).
- Penetrant embossments greatly improved shear resistance by the concrete that entered the holes (Makelainen and Sun 1998).
- Increased deck thickness also improved shear resistance (Jolly and Zubair 1987, Makelainen and Sun 1998).
- Re-entrant profiles improved performance by 63%-88%, with a linear dependence of performance verses the area of concrete under the re-entrant portion. However, unembossed deck of this type still provided only 50% the shear strength of embossed deck. (Wright and Essawy 1996).
- A study of flat plates with embossments showed failure occurred by bearing due to the local crushing of concrete when the embossment height to spacing ratio was less than 0.10. When this ratio was greater than 0.19, the specimen failed in concrete shearing at a plane that extends from the top of one embossment to the next. (Kitoh and Sonoda 1996)

Flexural failure

Flexural failure occurs when the slab has full composite interaction. This mode of failure is divided into under-reinforced slabs or over-reinforced slabs.

Longitudinal Shear Failure

This method is based on the Porter and Ekberg m-k method. It uses graphs of data from full-scale simple span slab tests. A typical graph is shown in Figure 2-4. The design reduces the regression line by 15% to account for scatter of the test results.

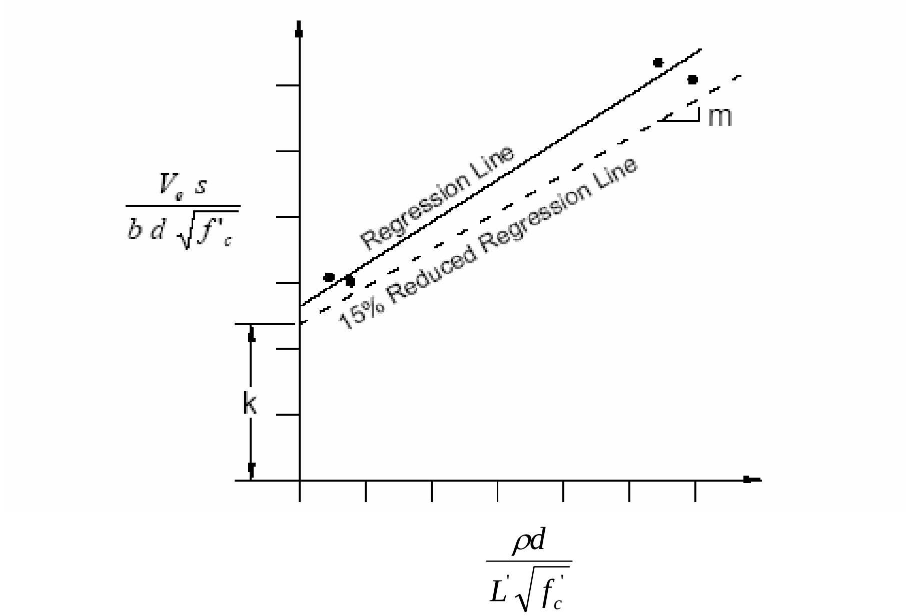


Figure 2-4: Typical m-k Graph (Porter and Ekberg 1975)

ASCE also includes an alternative method of design in Appendix D of the Standard, which was based on Luttrell's design recommendations that the tensile forces should be divided into components at each of the deck flanges and at the web. Design capacity is determined to be when the bottom flange of the deck first reaches yield stress.

W. Samuel Easterling after conducting a number of tests concluded that an acceptable lower-bound strength can be predicted for a composite slab, using elastic principles to calculate the first-yield moment. Based on results of the test conducted, the strength of composite slab used in conjunction with shear studs can be calculated based on the under-reinforced flexural limit.

Daniels and Crisinel developed a numerical analysis method that experimentally requires only shear bond tests and is applicable to continuous and single span slabs. The advantage is that this method does not require full-scale slab tests. A "pull-out test" gives the embossment resistance and a "push-off test" gives the end anchorage resistance.

Widjaja at Virginia Tech used the Partial Shear Connection theory to develop two strength methods that preclude the use of full-scale slab tests by using only shear stress versus slip data from shear bond tests similar to Daniels.

Grace Shen carried out full scale laboratory tests and concluded on the Evaluation of Analysis Methods:

- The First Yield Method of Design and the ASCE Appendix D Alternate Method of Design provided satisfactory lower bound strength predictions when compared to the experimentally determined ultimate strength.
- The accuracy of the strength prediction given by the ASCE Appendix D Method was not compromised for slabs with embossments deeper than the recommended limits for use of this method. Therefore, it appears that this method is valid for embossment depths up to 3.5mm.
- The ASCE Appendix D Method is slightly more accurate than the First Yield Method, but requires slightly more complicated calculations.

2.4 BEHAVIOR, ANALYSIS AND DESIGN CONSIDERATIONS

2.4.1 PROFILE SHEETING

2.4.1.1 Behaviour

During execution when the concrete is wet, the profiled sheeting alone resists the exterior loads. The profiled sheeting is subjected mainly to bending and shear; compression due to bending may arise in either the flanges or the web; shear occurs essentially near the supports. The thin-plate elements, which make up the profiled sheeting, may buckle prior to yield under these compressive and shear stresses, thereby reducing the resistance and stiffness of the sheeting.

2.4.1.2 Analysis for internal forces and moments

For analysis elastic analysis methods are recommended to be used due to the slenderness of the sheeting cross-section [BS5950 Part 4, 1990 and Structual Steel Eurocodes, 2001]

2.4.1.3 Design considerations

Verifications at the ultimate limit state and the serviceability limit state are required, with respect to the safety and serviceability of the profiled sheeting acting as formwork for the wet concrete. Although the steel deck may be propped temporarily during construction it is preferable if no propping is used [BS5950 Part 4, 1990 and Structual Steel Eurocodes, 2001]. In this research, un-propped construction situation is assumed.

2.4.2 Composite Slab

2.4.2.1 Behaviour

Composite behavior is that which occurs after a floor slab comprising of a profiled steel sheet, plus any additional reinforcement and hardened concrete have combined to form a single structural element. The profiled steel sheet should be capable of transmitting horizontal shear at the interface between the steel and the concrete. Under external loading, the composite slab takes a bending deflection and shear stresses appear at the steel-concrete interface.

The Cross section of a composite slab is shown in Figure 2-5.

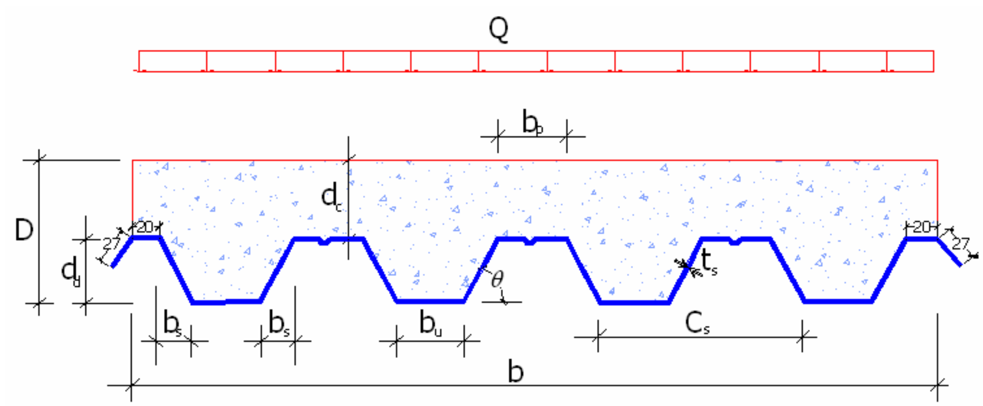


Figure 2-5: Cross section of a composite slab

The components of the composite slab are

d_c: Depth of Concrete	C_S: Pitch/ trough
d_d: Depth of Profile sheet	Q: Live load
b_u: Bottom flange	t_s: Profile thickness
b_p: top opening	θ: Angle of inclination of deck web
b_s: Horizontal dimension of web	

2.4.2.2 Steel – Concrete Interface

Two types of movement can be identified at the steel-concrete interface [Structual Steel Eurocodes, 2001]:

- Local micro-slip that cannot be seen by the naked eye. This micro-slip is very small and allows the development of the connection forces at the interface;

- Interface global *macro-slip* that can be seen and measured and depends on the type of connection between the concrete and steel.

Three types of behavior of the composite slab can be identified (Figure 2-6) [Structual Steel Eurocodes, 2001]:

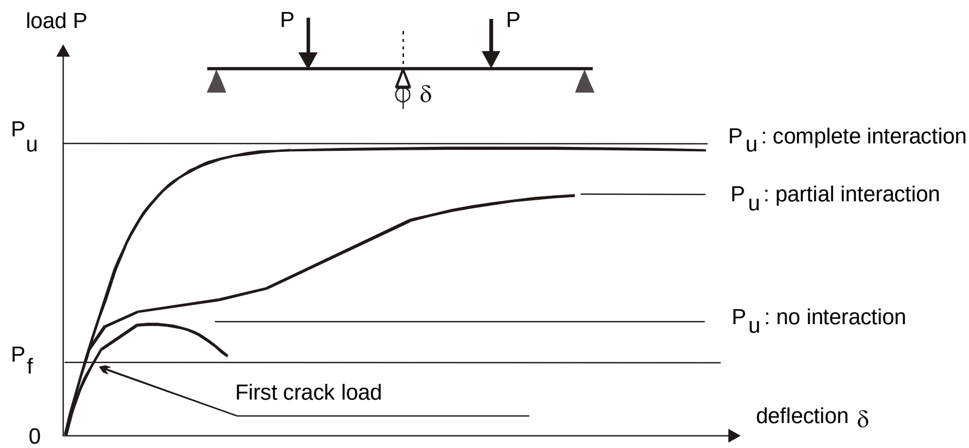


Figure 2-6: Composite slab behaviour

- Complete interaction between steel and concrete: no global slip at the steel-concrete interface exists. The horizontal transfer of the shear force is complete and the ultimate load P_u is at its maximum, the composite action is complete.
- Zero *interaction* between concrete and steel: global slip at the steel-concrete interface is not limited and there is almost no transfer of shear force. The ultimate load is at its minimum and almost no composite action is observed.
- *Partial interaction* between concrete and steel: global slip at the steel-concrete interface is not zero but limited. The shear force transfer is partial and the ultimate lies between the ultimate loads of the previous cases. The failure can be *brittle* or *ductile*.

Three types of link exist between steel and concrete [Structual Steel Eurocodes, 2001]:

- Physical-chemical link which is always low but exists for all profiles;
- Friction link which develops as soon as micro slips appear;
- Mechanical anchorage link which acts after the first slip and depends on the steel-concrete interface shape (embossments, indentations etc)

2.4.2.3 Analysis for internal forces and moments

The following methods of analysis may be used [Structual Steel Eurocodes, 2001]:

- *Linear analysis without moment redistribution* at internal supports if cracking effects are considered (Continuous slab modeled as a series of simple span);
- *Linear analysis with moment redistribution* at internal supports (limited to 30 %) without considering concrete cracking effects ;
- *Rigid-plastic analysis* provided that it can be shown that sections where plastic rotations are required have sufficient rotation capacity ;
- *Elastic-plastic analysis* taking into account non-linear material properties.

The application of linear methods of analysis is suitable for the serviceability limit states as well as for the ultimate limit states. A continuous slab may be designed as a series of simply supported spans. In such a case, nominal reinforcement should be provided over intermediate supports [Structual Steel Eurocodes, 2001].

2.4.2.4 Design considerations

Verifications at the ultimate limit state and the serviceability limit state are required, with respect to the safety and the serviceability of the composite slab after composite behavior has commenced.

2.4.2.5 Design Requirements

Referring to Figure 2-5 the overall depth of the composite slab, D should not be less than 80 mm. The thickness d_c of concrete above the ribs of the decking should be greater than 40mm to ensure ductile behavior of the slab. If the slab acts compositely with a beam, or is used as a diaphragm, the minimum total depth D is 90mm and the minimum concrete thickness d_c above decking is increased to 50mm [Structual Steel Eurocodes, 2001].

2.4.2.6 Elastic properties of composite slab cross-sections

Elastic analysis is normally used to calculate the deflection of slabs at the serviceability limit state. Elastic stiffness calculations for composite slabs is made based on simply supported boundary conditions and with the concrete transformed into an equivalent steel member [EBCS 4, 1995, BS5950 Part 4, 1990, Easterling, 1992 and Structual Steel Eurocodes, 2001]. The stiffness calculations are made using the average cross sectional stiffness, $I_{avg.}$ This value is calculated by taking the average of the cracked, I_{cc}, and uncracked, I_{cu}, cross-sectional stiffness.

$$I_{avg} = \frac{I_{cc} + I_{cu}}{2} \qquad \text{2-1}$$

In a cross-section where the concrete in tension is considered as cracked, as the cross-section shown in Figure 2-7(a) under sagging loading, the second moment of area I_{cc} per unit width of slab can be obtained from:

$$I_{cc} = \frac{bx_c^3}{3n} + A_{pe}(d_p - x_c)^2 + I_{ap} \qquad \text{2-2}$$

With:

x_c : position of the elastic neutral axis to the upper side of the slab obtained by the next formula :

$$x_c = \frac{nA_{pe}}{b}\left(\sqrt{\frac{2bd_p}{nA_{pe}} + 1} - 1\right) \qquad \text{2-3}$$

I_{ap}: Second moment of area of the profiled sheeting;

n: Modular ratio

A_{pe}: Effective area of steel deck

d_p: Effective depth

b: Width of slab

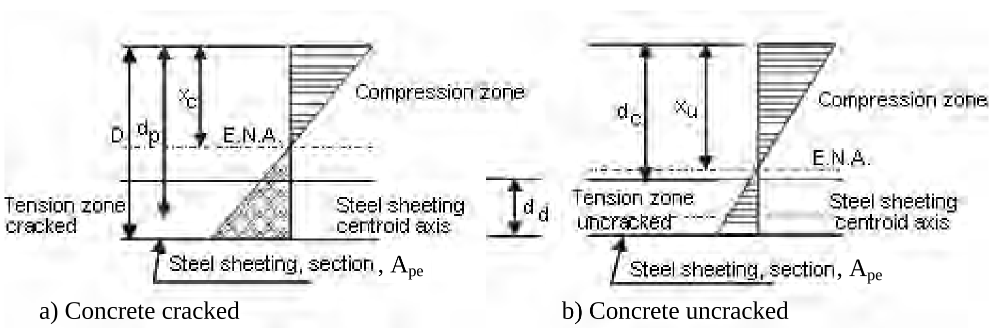


Figure 2-7: Second moment of Inertia calculation for cracked and un-cracked cross-sections under sagging moment

In a section under sagging moment, considering the concrete in tension as not cracked Figure 2-7(b), the second moment of area I_{cu} per unit width of slab is given by:

$$I_{cu} = \frac{bd_c^3}{12n} + \frac{bd_c(x_u - \frac{d_c}{2})^2}{n} + \frac{bd_d}{nC_s}\left(\frac{b_s d_d^2}{18} + \frac{b_u d_d^2}{12} + b_s(D - x_u + \frac{d_d}{3})^2 + b_u(D - x_u - \frac{d_d}{2})^2\right) \text{... 2-4}$$

$$+ A_{pe}(d_p - x_u)^2 + I_{ap}$$

where

x_u: is the position of the elastic neutral axis to the upper side of the slab and is calculated as.

$$x_u = \frac{\sum A_i Z_i}{\sum A_i} = \frac{\frac{bd_c}{n}\left(d_c/2\right) + A_{pe}d_p + \frac{b_s bd_d}{nC_s}\left(\frac{d_d}{3} + d_c\right) + \frac{b_u bd_d}{nC_s}\left(\frac{d_d}{2} + d_c\right)}{\frac{bd_c}{n} + A_{Pe} + \frac{bd_d(b_u + b_s)}{nC_s}} \text{.....................2-5}$$

C_s: Cell /trough/ spacing

D: Overall depth

d_c: Depth of concrete slab

d_d: Depth of steel deck

b_u: Bottom flange dimension

b_s: Horizontal width of the inclined web

In these formulae giving the second moment of area, the modular ratio n is calculated as:

$$n = \frac{E_{ap}}{E_{cm}} \text{.. 2-6}$$

2.5 RESISTANCE AND VERIFICATION OF PROFILE DECK

2.5.1 RESISTANCE OF PROFILED STEEL DECK

The construction load case is one of the most critical. The sheeting, which is a thin steel element, should resist to construction and wet concrete loads.

Current design procedures rely on the concept of effective width, to provide a method for the calculation for this type of thin-walled member [BS5950 Part 4, 1990, BS5950 Part 6, 1999, Structual Steel Eurocodes, 2001 and Brockenbrough etal, 1999]. Method of calculating the effective cross sectional properties of the profile sheet are discussed in section 2.5.1.1 and section 2.5.1.2 below. Clearly, the effective width of the compression flange depends upon the maximum stress imposed on the flange, which in turn depends on the location of the neutral axis of the cross-section.

2.5.1.1 *Effective width of a stiffened flange element*

2.5.1.1.1 General

The Effective Design Width, b_{eff}, is defined as a reduced design width for computing sectional properties of flexural and compression members [Yu, 2000].

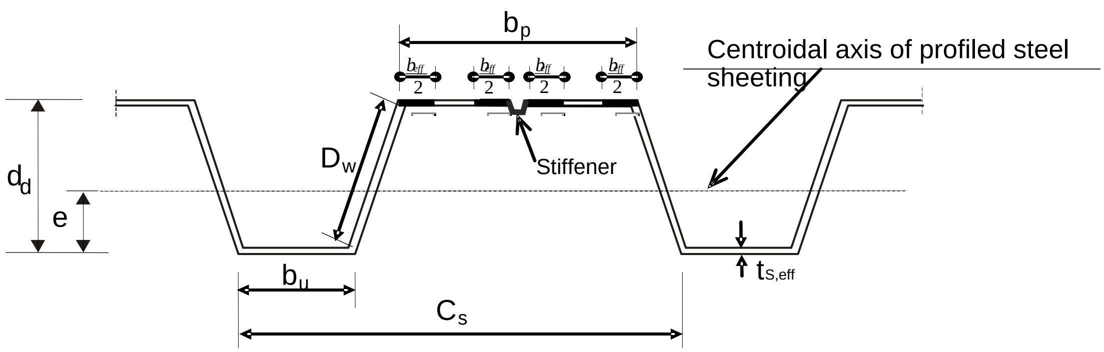


Figure 2-8: Labelling to calculate Effective cross section parameter

2.5.1.1.2 Flange with a single stiffener at the center

A Stiffened Element is an element that is stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners which are parallel to the direction of stress (Figure 2-9). The portion between adjacent stiffeners or between a web and an intermediate stiffener or between an edge and an intermediate stiffener is called a ‘‘sub element.’’ [Yu, 2000].

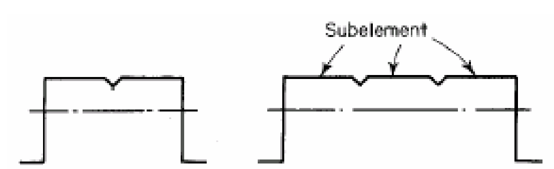


Figure 2-9: Sections with multiple-stiffened elements

The effective cross section of a multiple-stiffened flange with a single central stiffener should be obtained in the form of the separate proportions indicated in Figure 2-10 as follows [BS5950 Part 6, 1999]

- Two effective portions each of effective width $b_{eff}/2$, one adjacent to each web, where b_{eff} is the effective portion of a flat width of the flange element b,
- The effective area of the flange stiffener $A_{r,ef}$

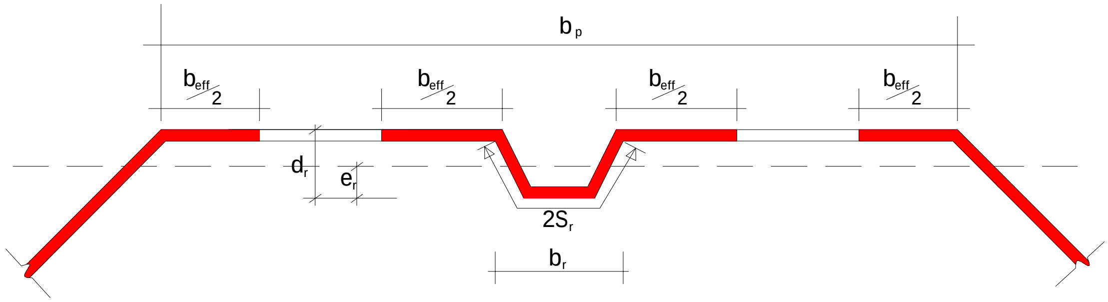


Figure 2-10: Effective cross section of a flange with one intermediate stiffener

Effective area of the flange stiffener

The effective area $A_{r,ef}$ of the flange stiffener should be determined from [BS5950 Part 6, 1999]

$$A_{r,ef} = \beta A_r \text{ .... 2-7}$$

where

f_c: the applied compressive stress on the effective element

A_r: total stiffened area comprising the flange stiffener itself plus the two adjacent effective portions of the flange each with an effective width $b_{eff}/2$

β: the reduction factor for stiffener effectiveness

The reduction factor β for stiffener effectiveness should be determined from the following

- If $\alpha \leq 0.65$: $\beta = 1.0$
- If $0.65 < \alpha \leq 1.38$: $\beta = 1.47 - 0.723\alpha$
- If $\alpha > 1.38$: $\beta = 0.66/\alpha$

where

α is the elastic critical strength ratio

For trapezoidal elements with flat web elements, the value α is determined from

$$\alpha = \sqrt{f_{yb} / \Pr,cr} \text{ .... 2-8}$$

Buckling strength of a single central flange stiffener

The elastic critical buckling strength $P_{r,cr}$ of a single central stiffener plus the two adjacent effective portions of the flange, treated as a strut supported by an elastic medium, should be determined from [BS5950 Part 6, 1999]

$$P_{r,cr} = \frac{4.2K_w E}{A_r}\sqrt{\frac{I_r t_{s,eff}^{\ 3}}{8b_r^{\ 3}(1+{3s_r}/{b})}} \quad \text{2-9}$$

where

I_r = The second moment of the Area flange stiffener, about its centroidal axis, parallel to the center line of the flange

$$= t_{s,eff}\left[2\frac{b_{eff}}{2}(d_r - e_r)^2 + (b_r - d_h)e_r^2 + l_i(e_r - {d_r}/{2})^2 + {l_i d_r^2}/{12}\right] \quad \text{2-10}$$

b_r = Flat width of the flange between stiffener

e_r = Distance of the flange stiffener from the centroid to its underside

d_r = Depth of stiffener

K_w = Web restraint coefficient

l_i = Inclined length of stiffener

s_r = Is the semi-perimeter of the stiffener (See Figure 2-10)

The value of the web restraint coefficient K_w may conservatively be taken as 1.0 representing nil restraint.

The compressive strength of a thin plate is presented in terms of an effective width of plate acting at its full yield strength. The effective width of a flat width is defined as an element that can be considered to resist compression effectively. The following empirical formula is employed to calculate effective width [BS5950 Part 6, 1999]:

For $f_c/P_{cr} \leq 0.123$

$$b_{eff}/b' = 1 \quad \text{2-11}$$

For $f_c/P_{cr} > 0.123$

$$b_{eff}/b' = \left\{1 + 14\left(\sqrt{f_c/p_{cr}} - 0.35\right)^4\right\}^{-0.2}$$ 2-12

where

f_c: is the applied compressive stress in the effective element

P_{cr}: is the local buckling strength of the element (N/mm^2)

$$P_{cr} = 0.904EK\left(t/b'\right)^2$$ 2-13

where

K: the relevant local buckling coefficient, taken as 4.0

t: net thickness of the steel material

b': flat width of the element /Between stiffened elements/

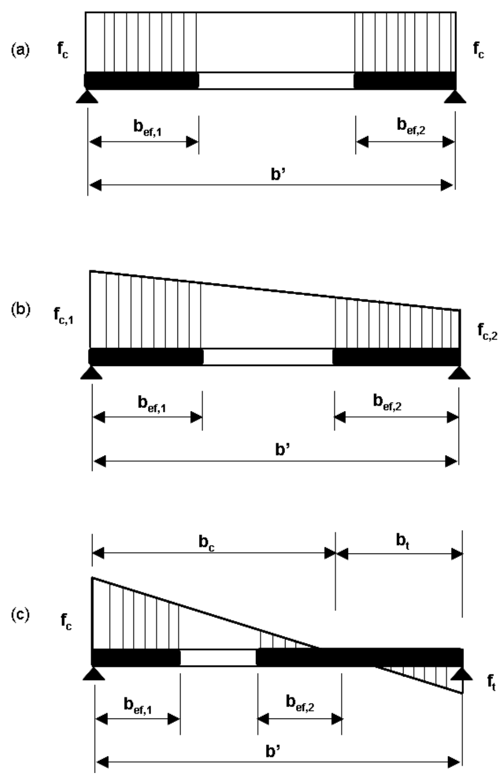


Figure 2-11: Effective width for a stiffened flange element

2.5.1.2 Effective width of a flat web element

BS5950 part 6 gives the following recommendation.

- The webs of trapezoidal profiles in which the web depth to thickness ratio of $D_w/t \le 70\varepsilon$ may be taken as fully effective. $\{\varepsilon=(280/f_{yp})^{0.5}\}$

In all other cases the effective width of a web in which the stress varies linearly as shown in Figure 2-12, should be obtained in two portions, one adjacent to each edge as follows.

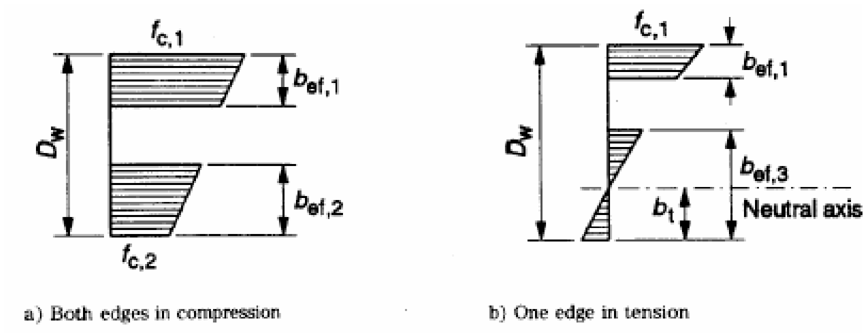


Figure 2-12: Stress distribution over effective portions of web

Both edges in compression (Figure 2-12a)

$$b_{ef,1} = 0.76t\sqrt{E/f_{c,1}} \quad \text{2-14}$$

$$b_{ef,2} = \left(1.5 - 0.5\, f_{c,2}/f_{c,1}\right) b_{ef,1} \quad \text{2-15}$$

where

$f_{c,1}$: is the larger compressive stress

$f_{c,2}$: is the smaller compressive stress

$b_{ef,1}$: is the portion of the effective width adjacent to the more compressed edge

$b_{ef,2}$: is the portion of the effective width adjacent to the less compressed edge

If $b_{ef,1}$ and $b_{ef,2} \ge D_w$ the whole web is effective

One edge in tension (Figure 2-12b)

$$b_{ef,1} = 0.76t\sqrt{E/f_{c,1}} \quad \text{2-16}$$

$$b_{ef,3} = b_t + 1.5 b_{ef,1} \quad \text{2-17}$$

where

b_t: is the width subject to tension

$b_{ef,3}$: is the portion of the effective width adjacent to the tension edge

If $b_{ef,1}$ and $b_{ef,3} \ge D_w$ the whole web is effective

Both edges in tension

The whole web is effective

2.5.1.3 *Capacity of Profile Deck*

After calculating the effective widths of all the planar elements in compression (flange and web), the determination of the cross-section properties (neutral axis depth, effective second moment of area I_{eff} and effective section modulus S_{eff}) is obtained as follows.

$$e = \frac{d_d\left[(2b_{eff} - t_{s,eff}) + (D_w - t_{s,eff})\right]}{b_u + 2b_{eff} + 2D_w} \quad \text{2-18}$$

$$I_{eff} = \frac{b}{C_s}\left\{t_{s,eff}\left[b_u e^2 + 2b_{eff}(d_d - t_{s,eff} - e)^2 + \frac{2D_w d_d^2}{12} + 2D_w\left(\frac{d_d}{2} - t_{s,eff} - e\right)^2\right]\right\} + I_r + \frac{b}{C_s}\left[A_{r,ef}(d_d + d_r - e_r)^2\right] \quad \text{2-19}$$

$$S_{eff,t} = \frac{I_{eff}}{d_d - t_{s,eff} - e} \quad \text{2-20}$$

$$S_{eff,b} = \frac{I_{eff}}{e} \quad \text{2-21}$$

Bending moment resistance of the section is given by:

$$M_{p,Rd} = \frac{f_{yp}}{\gamma_{ap}}\left\{lesser \quad of \begin{pmatrix} S_{eff,t} \\ S_{eff,b} \end{pmatrix}\right\} \quad \text{2-22}$$

where

b_{eff} = Effective width of a compression element

D_w = Sloping distance between the intersection points of the web and flange

e = Distance from the centroid of the effective area of the steel sheet to its underside

$M_{p,Rd}$ = Moment resistance of steel deck

$t_{s,eff}$ = Effective thickness of steel deck

$S_{eff,t}$, $S_{eff,b}$ = Effective section modulus of top and bottom section, respectively

$A_{r,ef}$, e_r, I_r and d_r as defined in section 2.5.1.1.2.

Other notations are as defined in previous sections.

The thickness $t_{s,eff}$ used in the calculation of sectional properties and the design of cold-formed sections is the thickness of base steel. Any thickness of coating material should be deducted from the overall thickness of steel [Structural Steelwork Eurocodes, 2001 and Yu, 2000].

2.5.2 VERIFICATION OF THE PROFILE DECK

Two verifications are necessary to ensure that safety and serviceability requirements are met:

- Verification at the ultimate limit states
- Verification at the serviceability limit states

2.5.2.1 Verification at ULS

Flexure check is made at the section of maximum positive moment. The condition can be expressed as:

$$M_{Sd} \leq M_{p,Rd} \qquad \text{2-23}$$

where

M_{sd} = The design value of bending due to load

$M_{p,Rd}$ = The design value of the positive bending resistance of the section.

2.5.2.2 Verification at SLS

For uniformly distributed loading, BS5950 part 4 gives the following approximate expression to calculate mid-span deflection

$$\delta = \frac{5}{384}\frac{W_{ser}L_p^3}{EI_{eff}} \qquad \text{2-24}$$

where

W_{ser}: Serviceability load

L_p: Effective span

I_{eff} : Effective second moment of area of the sheeting (Section 2.5.1)

The effective span, L_p, of profiled steel sheet is defined as [BS5950 Part 4, 1990], the smaller of

- Distance between centers of permanent or temporary supports, and
- Clear span between centers of permanent or temporary supports plus overall depth of profiled sheet, d_d

During execution, deflection (δ) of the profiled sheeting under loads due to self-weight and wet concrete, must not exceed a limiting value.

The deflections should not normally exceed the following [BS5950 Part 4, 1990]:

- $L_p/180$ (but ≤ 20mm) when the effect of ponding are not taken into account
- $L_p/130$ (but≤ 30mm) when the effects of ponding are taken into account

where: L_p is the effective span of the profiled steel sheets

If the central deflection, δ, of the sheeting under its own weight plus that of the wet concrete, calculated for serviceability, is less than 1/10 of the slab depth, the ponding effect may be ignored in the design of the steel sheeting. If this limit is exceeded, this effect would be allowed for; for example by assuming in design, that the nominal thickness of the concrete is increased over the whole span by 0.7δ [Structural Steelwork Eurocodes, 2001].

2.6 Resistance and Verification of Composite Slab

2.6.1 Resistance of Composite Slabs

Composite slab failure can happen according to one of the following collapse modes [BS5950 Part 4, 1990 and Structural Steelwork Eurocodes, 2001]

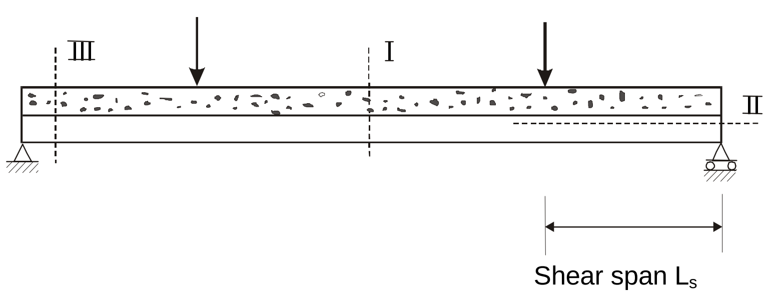


Figure 2-13: Composite slab failure mode types

- **Failure type I**: The failure is due to an excessive sagging moment (section I), that is the bending resistance of the slab $M_{p,Rd}$; this is generally the critical mode for moderate to high spans with a high degree of interaction between the steel and concrete (With end anchorages, embossments).

- **Failure type II**: The failure is due to excessive longitudinal shear; the ultimate load resistance is reached at the steel concrete interface. This happens in section II along the shear span L_s.
- **Failure type III**: The failure is due to an excessive vertical shear near the support (section III) where vertical shear is important. This is only likely to be critical for deep slabs over short spans and subject to heavy loads

2.6.1.1 Flexural Resistance

Type I failure is due to sagging bending resistance. That failure mode is reached if the steel sheeting yields in tension or if concrete attains its resistance in compression. In sagging bending regions, supplementary reinforcement in tension may be taken into account in calculating the composite slab resistance.

2.6.1.2 Vertical Shear Resistance

Type III failure corresponds to the resistance to vertical shear. In general the vertical shear resistances are assumed to be given by the concrete section since the contribution of the steel sheeting is neglected. This type of failure can be critical where steel sheeting has effective embossments (thus preventing Type II failure) and is characterized by shearing of the concrete and oblique cracking as is observed for reinforced concrete beams.

The vertical shear design resistance, over a width equal to the distance between centers of ribs has the value [EBCS 2, 1995, BS5950 Part 4, 1990 and Structural Steelwork Eurocodes, 2001]:

$$V_{v,Rd} = b_o\, d_p\, \tau_c \qquad \text{2-25}$$

where

τ_c is the limiting shear stress appropriate for composite slabs (γ_c included).

$$\tau_c = \tau_{Rd}\, k_1\, k_2 \qquad \text{2-26}$$

where

b_o : mean width of the concrete ribs (Figure 2-14);

τ_{Rd} : basic shear strength to be taken as 0.25 f_{ctk}/γ_c;

f_{ctk} : approximately equal to $0.21 f_{ck}^{2/3}$

A'_{pe} : is the effective area of the steel sheet in tension within the considered width b_o

$k_1 = 1.2 + 40\ \rho_o$

$k_2 = 1.6 - d_p \geq 1.0$ (d_p in m)

$\rho_o = A'_{pe}/b_o d_p < 0.02$

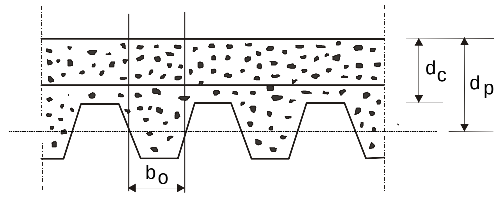


Figure 2-14: Cross section used for vertical shear resistance

2.6.1.3 Punching Shear Resistance

Under a heavy concentrated load, the slab can fail by punching i.e. vertical shear on the perimeter of the concentrated load. Figure 2-15 represents that kind of failure. The punching shear resistances is assumed to be given by the concrete section since the contribution of the steel sheeting is neglected.

The punching shear resistance $V_{p,Rd}$ of a composite slab at a concentrated load is determined from:

$$V_{p,Rd} = C_p\ d_c\ \tau_c \quad \text{......} \quad 2\text{-}27$$

where

C_p is the critical perimeter determined as shown in Figure 2-15.

d_c is the thickness of the concrete slab (above the ribs).

τ_c is the limiting shear stress given above.

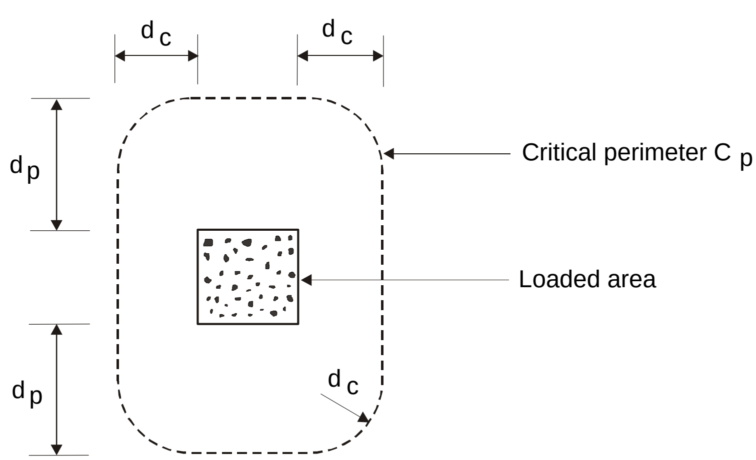


Figure 2-15 Critical perimeter for punching shear

2.6.1.4 Longitudinal Shear Resistance

Resistance to longitudinal shear in composite slabs is due to the steel-concrete bond at the interface of these two materials, established by friction, embossments or connectors placed at the ends of the spans. The capacity of a composite slab for longitudinal shear requires are established based on results from laboratory tests. In this research, the shear studs and embossments are assumed to be enough in providing longitudinal shear resistance.

2.6.2 VERIFICATION OF COMPOSITE SLABS

Two verifications are necessary to ensure that safety and serviceability requirements are met:

- Verification at the ultimate limit states
- Verification at the serviceability limit states

2.6.2.1 Verification at ULS

2.6.2.1.1 Flexure

This check is made for the Composite slab is made at the section of maximum positive moment. The condition can be expressed as:

$$M_{Sd} \le M_{p,Rd} \qquad \text{2-28}$$

where

M_{sd} = The design value of bending due to load

$M_{p,Rd}$ = The design value of the positive bending resistance of the section.

2.6.2.1.2 Vertical shear

This check is made for the composite slab and is rarely critical; however, it may be critical in the case of deep slabs with loads of relatively large magnitude. This condition may occur at end supports where the bending moment is zero. The condition is expressed as:

$$V_{Sd} \le V_{v,Rd} \qquad \text{2-29}$$

where

V_{Sd} is the design value of the vertical shear due to load.

$V_{v,Rd}$ is the design value of the vertical shear resistance of the section.

2.6.2.1.3 Longitudinal shear

This check is often the determining factor for composite slabs with profiled sheeting without end anchorage. It implies that overall failure of the slab occurs by failure of the shear bond. The bending resistance at section I cannot then be attained.

If the empirical "m-k" method is used, the condition can be expressed as:

$$V_{Sd} \leq V_{l,Rd} \quad \text{2-30}$$

where

V_{Sd} is the design value of the vertical shear (equivalent span).

V_{lRd} is the design value of the shear resistance.

2.6.2.2 Verification at SLS

The behavior of the composite slab under permanent loads and variable service loads must fall within accepted limits. The following states would be verified:

- Concrete cracking restricted to a limited width (corrosion of reinforcement, appearance).
- Deflection or variation of deflection, within the permissible limit (use of slab, damage of non-structural elements, appearance, etc).

2.6.2.2.1 Cracking

Given that there is a profiled sheet on the lower surface of the concrete slab, only concrete cracking at the supports must be verified. In normal circumstances when, for example, the slab is designed as a series of simply supported beams, minimum reinforcement placed at the supports is sufficient.

Normal circumstances are [Structural Steelwork Eurocodes, 2001]:

- no exposure to aggressive physical or chemical environments;
- no damage other than cracking;

The amount of minimum reinforcement is given by the following [Structural Steelwork Eurocodes, 2001]

- for slabs propped at the time of concreting:

$$\rho_{\min} = \frac{A_s}{bd_c} = 0.4\% \qquad 2\text{-}31$$

- for slabs unpropped at the time of concreting:

$$\rho_{\min} = \frac{A_s}{bd_c} = 0.2\% \qquad 2\text{-}32$$

2.6.2.2.2 Deflection

Deflections in the composite state must be limited, in order that the slab may fulfill its intended function and that any other elements in contact with it (false ceilings, pipework, screens, partitions) will not be damaged. Deflection limits should, therefore, be considered relative to the use of the slab, the execution procedure and architectural aspects (aesthetics).

The deflection of the profiled steel sheeting would be calculated using the serviceability loads as recommended in BS5950, i.e. the deflection of the profiled steel sheeting due to its own weight and the weight of wet concrete should not be included as this deflection already exists when the construction work is completed. The following empirical formula will be used

$$\delta = \frac{5}{384}\frac{W_{ser}L_p^3}{EI_{av}} \qquad 2\text{-}33$$

where

$W_{ser:}$ Serviceability load

L_s: Effective span

I_{av}: Average second moment of area for composite stage

The deflection of the composite slab should not normally exceed the following [BS5950 Part 4, 1990]

- Deflection due to imposed load: L_s/350 or 20mm, whichever is smaller
- Deflection due to the total load less the deflection due to the self weight of the slab: L_s/250

where: L_s is Effective span of composite slab

The effective span, L_s, of composite slab sheet is defined as [BS5950 Part 4, 1990], the smaller of

- Distance between centers of permanent supports, and
- Clear span between centers of permanent supports plus effective depth of composite slab, d_p

2.7 ANALYSIS METHODS

2.7.1 GENERAL

In this section, available analytic methods to evaluate the capacity of the composite slab will be outlined. The following analysis are usually carried out

- Consideration of the performance of the steel sheeting as shuttering during construction and as reinforcement to the hardened concrete slab.
- The shear connection between the steel sheeting and concrete is of particular importance. This is usually determined by tests. In this research it is assumed that enough longitudinal resistance is achieved by embossments and end anchorages (Shear studs).

Three different composite slab strength prediction methods were used for the deck types under consideration.

2.7.2 FULL FLEXURAL CONDITION

The full flexural condition is when we have full interaction between the concrete and profile steel sheet. The full flexural condition /moment capacity of composite slab/ should be taken as the upper bound to the capacity of composite slab. This moment capacity should be calculated as for reinforced concrete, with the profiled steel sheets acting as tensile reinforcement [EBCS 4, 1995].

In the full flexural condition, material behavior is generally idealized with rigid plastic "stress-block" diagrams [EBCS 4, 1995, EBCS 2, 1995, BS5950 Part 4, 1990 and Structural Steelwork Eurocodes, 2001]. At the ultimate limit state, the steel stress is the design yield strength f_{yp}/γ_{ap}, the concrete stress is its design strength $0.85 f_{ck}/\gamma_c$ and the reinforcement steel stress is also its design strength f_{sk}/γ_s.

Two cases have to be considered according to the position of the plastic neutral axis.

Case 1 – Plastic neutral axis above the sheeting

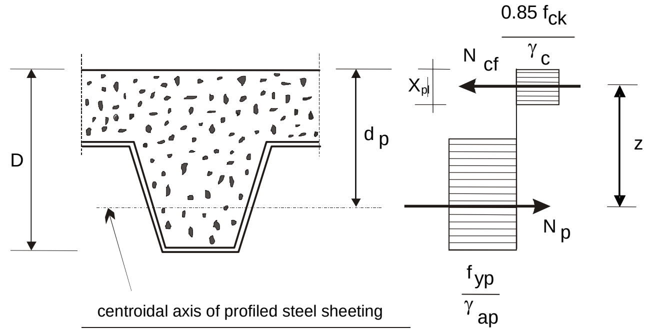


Figure 2-16: Stress distribution for sagging bending if the neutral axis is above the steel sheet

The resistance of concrete in tension is taken as zero. The resulting tension force N_p in the steel sheeting is calculated with the characteristics of the effective steel section A_{pe}. This force is equated to the resulting compression force in the concrete N_{cf} corresponding to the force acting on the width b of the cross-section and the depth x_{pl} with a stress equal to the design resistance:

$$N_p = A_{pe} \frac{f_{yp}}{\gamma_{ap}} \quad \text{2-34}$$

and

$$N_{cf} = b x_{pl} \frac{0.85 f_{ck}}{\gamma_c} \quad \text{2-35}$$

Equilibrium gives x_{pl} as:

$$x_{pl} = \frac{\dfrac{A_{pe} f_{yp}}{\gamma_{ap}}}{\dfrac{0.85 b f_{ck}}{\gamma_c}} \quad \text{2-36}$$

The lever arm z is then:

$$z = d_p - 0.5 x_{pl} \quad \text{2-37}$$

and the design resistance moment is equal to:

$$M_{p,Rd} = N_p \, z \quad \text{2-38}$$

or

$$M_{p,Rd} = A_{pe} \frac{f_{yp}}{\gamma_{ap}} \left(d_p - \frac{x_{pl}}{2} \right) \qquad \text{2-39}$$

where

N_{cf} = Compression force in concrete slab over the width b

N_p = Tensile force in the profile steel deck

x_{pl} = Stress block depth

The effective area A_{pe} of the steel decking is the net section obtained without considering the galvanizing thickness. A zinc coating of 275g/m^2 total, including both sides is normally specified for internal floors in a non-aggressive environment. A 275g/m^2 coating adds approximately 0.040mm to the bare metal thickness, 0.02mm on each side [BS 5950 Part 4, 1990].

Case 2 – Plastic neutral axis in steel sheeting

If the plastic neutral axis intercepts the steel sheeting, a part of the steel sheeting section is in compression to keep the equilibrium in translation of the section. For simplification, the concrete in the ribs as well as the concrete in tension is neglected.

As shown in Figure 2-17, the stress diagram can be divided in to two diagrams each representing one part of the design resistant moment $M_{p,Rd}$:

- The first diagram depicts the equilibrium of the force N_{cf}, corresponding to the resistance of the concrete slab (depth d_c) balanced by a partial tension force N_p in the steel sheeting. The lever arm z depends on the geometrical characteristics of the steel profile. The corresponding moment to that diagram is $N_{cf}.z$. The calculation of the lever arm z by an approximate method is explained below.
- The second diagram corresponds to a pair of equilibrating forces in the steel profile. The corresponding moment is M_{pr}, called the *reduced plastic moment* of the steel sheeting, and must be added to $N_{cf} Z$.

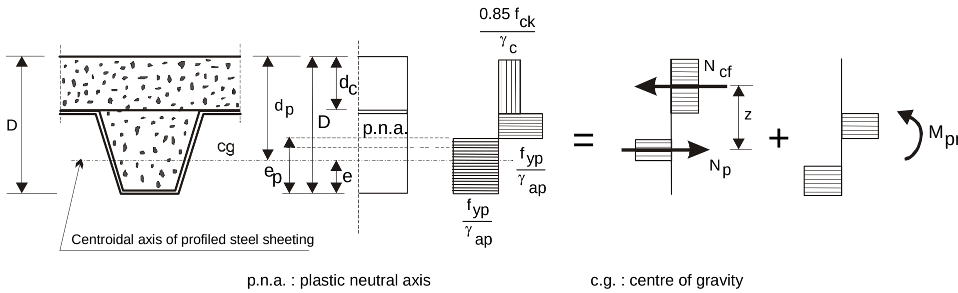


Figure 2-17: Stress distribution for sagging bending if the neutral axis is inside the steel sheet

The bending resistance is then:

$$M_{p,Rd} = N_{cf} z + M_{pr} \quad \text{2-40}$$

Compression force in the concrete is:

$$N_{cf} = \frac{0.85 f_{ck}}{\gamma_c} b d_c \quad \text{2-41}$$

Some authors have proposed an approximate formula where M_{pr}, reduced (resistant) plastic moment of the steel sheeting can be deduced from M_{pa}, design plastic resistant moment of the effective cross-section of the sheeting. That formulae calibrated with tests on 8 steel profile types is the following:

$$M_{pr} = 1.25 M_{pa} \left(1 - \frac{N_{cf} / A_{pe} f_{yp}}{\gamma_{ap}} \right) \leq M_{pa} \quad \text{2-42}$$

The lever arm is calculated with the following formula:

$$z = D - 0.5 d_c - e_p + (e_p - e) \frac{N_{cf}}{\frac{A_{pe} f_{yp}}{\gamma_{ap}}} \quad \text{2-43}$$

2.7.3 FIRST YIELD METHOD (HEAGLER 1992)

This method limits the predicted strength of the composite slab, which is considered fully composite, to the load that causes first yield of the bottom flange of the deck. The concrete is assumed to have cracked so that only concrete above the neutral axis contributes to compressive strength and the tensile loads are distributed into components in the top flange (T_1), the web (T_2), and the bottom flange (T_3) of the deck as shown in Figure 2-18. The effectiveness of embossments is not considered in this method.

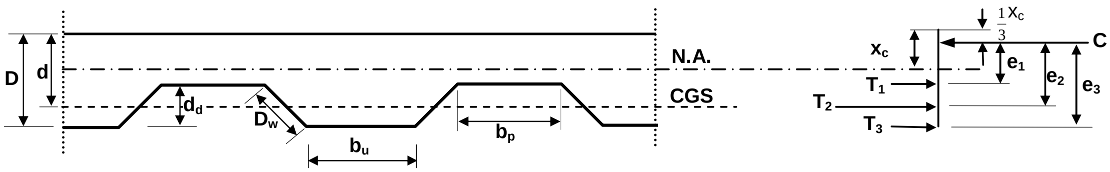


Figure 2-18: Deck Cross section and Force Distribution

The first yield moment per unit cell is

$$M_{et} = (T_1e_1 + T_2e_2 + T_3e_3) \quad \text{2-44}$$

$$e_1 = e_3 - d_d \quad \text{2-45}$$

$$e_2 = e_3 - d_d/2 \quad \text{2-46}$$

$$e_3 = D - {x_c}/{3} \quad \text{2-47}$$

$$T_1 = f_{yc}(B_t t)\left[\frac{D - x_c - d_d}{D - x_c}\right] \quad \text{2-48}$$

$$T_2 = f_{yc}(2D_w t)\left[\frac{D - x_c - {d_d}/{2}}{D - x_c}\right] \quad \text{2-49}$$

$$T_3 = f_{yc}(B_b t) \quad \text{2-50}$$

$$f_{yc} = F_y - f_c \quad \text{2-51}$$

where

- M_{et}: First yield moment per cell
- f_{yc} : Yield stress of deck, corrected to account for casting stresses
- F_y : Minimum yield stress of deck
- f_c : Stress in deck due to casting
- x_c : Position of the elastic N.A. to the upper side of the slab

2.7.4 ASCE APPENDIX D ALTERNATE METHOD (STANDARD 1994)

This method was developed by Luttrell and Prassanan (1984) at West Virginia University and is based on the shear bond limit state behavior found from numerous full-scale slab tests of deck with various characteristics. A statistical analysis was performed on the test results to determine the effect of the various deck characteristics on strength, resulting in the development of three “relaxation factors."

The slab strength is determined by applying these relaxation factors to the first yield strength described in Section 2.7.3. The dimension designations used in the design equations are illustrated in Figure 2-19.

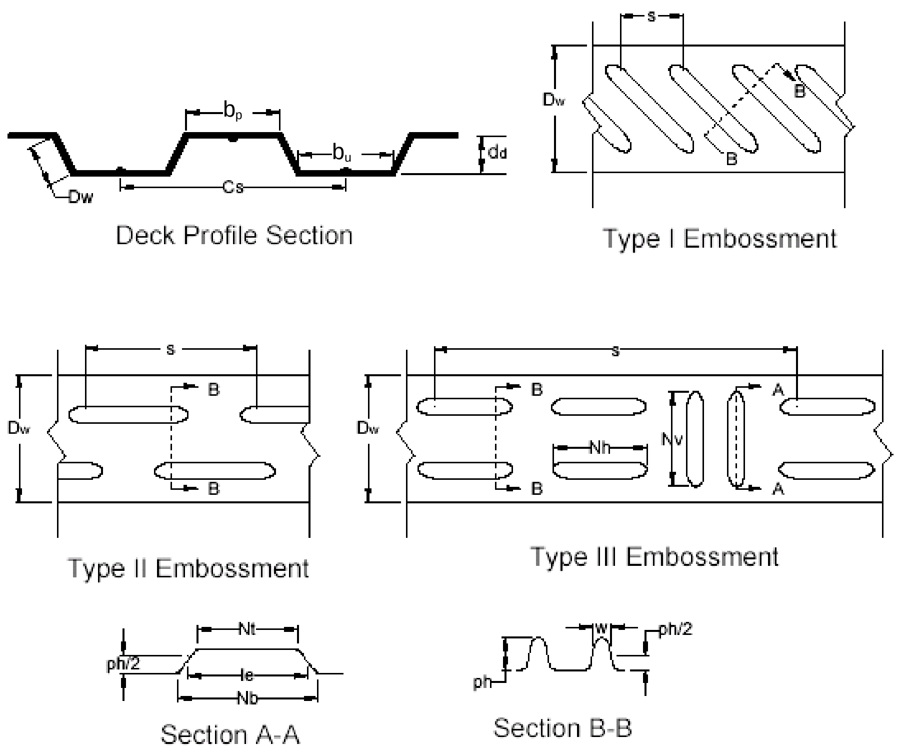


Figure 2-19: Dimension Designations and Types of embossments for ASCE Appendix D method

The Flexural Strength is:

$$M_t = KM_{et}\left(\frac{1000}{C_s}\right) \quad \text{2-52}$$

K: bond force transfer property,

$$K = \frac{K_3}{K_1 + K_2} \quad \text{2-53}$$

K_1: measures the influence of the steel section depth on bond development along the shear span:

$$K_1 = \sqrt{\frac{d_d}{7.8}} \quad \text{2-54}$$

K_3: a reflection of the increased effectiveness of multi-paneled specimens,

$$K_3 = 0.87 + 0.0688N - 0.00222N^2 \leq 1.4 \quad \text{.......... 2-55}$$

$$N = \frac{b}{C_s} \quad \text{.......... 2-56}$$

K_2: measures the performance of the embossments and is determined from a series of equations dependant on the type of embossment pattern used.

$$K_2 = \frac{D_w^{0.8}\left(\frac{K_3}{SS\,1}\right)}{1 + 60\,(P_h^2\sqrt[3]{P_s})} \quad \text{.......... 2-57}$$

Type I or Type III Deck:

$$SS1 = \left(\frac{3l_{nf}}{70}\right)(l_{nf} - 14) + 3.6 \quad \text{.......... 2-58}$$

$$P_s = \frac{12l_e}{s} \text{ (type I)} \quad \text{.......... 2-59}$$

$$P_s = \frac{12(N_v l_e + N_h w)}{s} \text{ (type III)} \quad \text{.......... 2-60}$$

Type II Deck:

$$K_2 = \frac{627\; t^2 SS^{\;2}}{d_d e^x} + t\left(\frac{7}{d_d}\right)^2 \quad \text{.......... 2-61}$$

$$SS2 = \frac{f_c^{'}}{5000} + \frac{\sqrt{\frac{12L}{L^{'}}}}{3.6} \quad \text{.......... 2-62}$$

where

C_s	= Cell /trough/ spacing, mm
b	= Width of the composite slab, mm
d_d	= Profile deck depth, inch
$f_c^{'}$	= Compressive strength of concrete, psi
L'	= Shear span, (for uniform load is equal to one quarter of the span), ft.
L	= Length of span, ft
l_{nf}	= Clear span length, ft

l_e = Average embossment length, inch

p_h = Embossment height, inch

N_v = Length of vertical embossment, inch

N_h = Length of horizontal embossment, inch

s = Spacing of shear transferring mechanisms, inch

w = Average embossment height, inch

x = $25p_{h,}$ inch

NB: Notations and units shown above are for this section only

- To change mm to inch, divide by 25.4
- To change mm to ft, divide by 2.117
- To change N/mm^2 to psi, multiply by 145

The applicability of the above formulas has been previously tested experimentally within the following limitations [Shen, 2001]:

$0.89 \text{ mm} \leq P_h \leq 2.67 \text{ mm}$

55° ≤ angle of inclination of the deck web ≤ 90°

Webs have no re-entrant bends in their flat width

$d_d \leq 77$ mm.

$C_s \leq 305$mm

$17.25\text{MPa} \leq f_{ck} \leq 41.4\text{Mpa}$

3 DESIGN TABLES AND CHARTS

3.1 PROBLEM FORMULATION

In order to calculate sectional properties, load capacity of the profile sheet, Microsoft EXCEL program was used. Spreadsheets offer simple method of obtaining solutions to a variety of problems and are very efficient with regard to the amount of effort required to obtain these solutions.

3.1.1 DESIGN DATA

In this section the data required to go on the preparation of the design tables is inserted into their respective cells. These are

- The different material properties
- Factors of safety /material and load/

In addition, it is in this part of the spreadsheet lay out that we specify parameters of the deck and concrete slab that are not variable. These include

- Concrete grade, f_{ck}
- Profile sheet yielding stress, f_{yp}
- Thickness of the profile sheet, t_s
- Depth of profile deck, d_d
- Depth of concrete slab, d_c
- Center to center span width, L
- Flat width of steel deck, B, which usually is 1000mm
- Stiffener data

Once these data are entered, the program will evaluate the following parameters

- Effective span (Section 2.6.2.2)
- Width of slab, b (Taking into consideration of end lap width taken and corrugations). It is calculated using the following formula

$$b = \left(\frac{C_s}{2D_w + b_u + b_p} \right) * \left(B - 2 * lap\_width \right) \qquad \text{3-1}$$

  - The lap width is taken as 30mm

- Overall depth, $D = d_c + d_p$
- Effective thickness of steel, $t_{s,eff} = t_s - 0.04$
- Angle of inclination of the deck web, θ

The layout of the design data is as shown below.

Table 3-1: Design data layout

Design Data

	Grade	f_i	E (N/mm^2)	γ (Kg/m^3)	f_{id}
Concrete	C25	20	28900	2350 / 2400	11.33
Profile sheet	Fe360	235	205000	7850	213.64
Reinforcing Bar	S275	275	210000	7850	239.13

Factor of Safety

n (Es/Ec) = 8.00

Material	Class I Works
Concrete	1.50
Structural Steel	1.10
Reinforcing Steel	1.15
Shear Stud	1.25

Load	
Dead	1.30
Live	1.60

Concrete Slab & Profile Sheet Data (mm)

d_c	80.00
d_d	50.00
t_s	1.10
t_{gal}	0.04
C_s	250.00
L	3500.00

Bearing length	50.00
b_s	20.00
B	1000.00

L_p	3450.00
L_s	3500.00
b	739.68
D	130.00
$t_{s,eff}$	1.06
θ	68.23

Stiffener data

depth, d_r	10.00
horizontal len: d_h	5.00
b_r (Flat width)	24.00
l_i, Inclined length	11.18

3.1.2 CONSTRUCTION STAGE

Loads

In this part of the table, the expected dead and live loads are calculated or entered, as the case may be. The dead load has two components.

- Steel deck
- Wet concrete

Generally, the action due to the unfactored dead loads is calculated as follows

$$G_i = \frac{g * \gamma_i * d_i}{1{,}000{,}000} \qquad \text{3-2}$$

where

Gi = Dead load (kN/m^2)

g = Acceleration due to gravity, (m/s^2)

γ_i = Unit weight of material (Steel deck or wet concrete) [kg/m^3]

d_i = Depth of section (d_c or d_d) [mm]

The live load used is as per the recommendation in BS5950 part 4, i.e.

$Q = 1.5 kN/m^2$

These values are not yet factored. The partial safety factors entered in the design data will now be applied to find the total factored load. Once this value is calculated, it would be possible to evaluate the resulting positive moment and shear force.

It is necessary to note here that, since the parameters needed to make up the profile sheet (b_u and b_p) are not defined or evaluated yet, the moment and shear force calculated above are not the maximum. With this preliminary calculation we now move on to the next steps.

Initial/ Starting values for b_u and b_p

In order to have a starting value for one of the sides, b_p (top flange), the program determines a combination of values for the top and bottom flanges that satisfy the following conditions

- $b_{eff/2} > 25\%$: Minimum effective cross section is above half of the top flange
- Maximum Moment capacity
- Minimum bottom side dimension = 50mm
- $b_p/t < 250\varepsilon$: This value is entered to control flange curling
- Deflection is not above the value recommended

With the above given constraints, the program will determine a value for b_p (top flange) and b_u (bottom flange). This is done using the built-in solver toolpak in EXCEL. The first value that would be calculated that satisfies the above conditions will generally have decimal values. Such materials may not be easily fabricated. In the next combination of values, the first value calculated for the top flange (b_P) will be rounded to the nearest whole number and the bottom segment (b_u) will be calculated as follows

$$b_u = C_s - b_p - 2b_s \qquad \text{3-3}$$

The other combinations that follow have an increment of 5mm on the bottom flange (b_u) and on the top flange (b_p) a reduction by 5mm. The program automatically calculates the following values.

- e = NA depth of the steel deck from bottom flange (Section 2.5.1)
- I_{eff} and I_{eff} per unit width = Effective second moment of area (Section 2.5.1) per cell and per unit width of slab respectively
- $S_{eff,t}$ and $S_{eff,b}$ = Effective section modulus (Section 2.5.1)
- $f_{c,b}$ = Compressive stress in compression plate (top flange) = $M_{p,s} / S_{eff,t}$
- $M_{p,Rd}$ = Moment resistance of steel deck (Section 2.5.1)
- δ = Central deflection of steel deck
- M_{ps}= Maximum positive moment in section
- Section status: This cell shows whether the required parameters for verification condition are satisfied, i.e. section flexural capacity is enough, whether deflections are under the specified limit. It returns a value of 1 for sections that satisfy the conditions and 0 for those not satisfying.

It is worth noting that in this part that the case of ponding is considered as recommended in BS5950-Part 4. The following table shows the design layout of the construction stage.

Table 3-2: Construction stage design table layout

Construction Stage:

Load(Kn/m^2)

Dead load		Live load					
Profile sheet:	0.08	Construction load	1.50	Total unfactored load=	**3.47**	Positive moment: (kN.m)	5.62
Wet concrete:	1.88						
Total:	1.97			Total Factored load=	**4.96**	Shear (kN)	8.68

Verification parameter

Deflection (mm)	L/130<20mm	**20.00**	Maximum deflection limit for Ponding effect (mm): D/10	13.00

Moment of Inertia, NA depth of Profile Sheet & Section flexural capacity determination

No	b_u (mm)	b_p (mm)	e (mm)	I_{eff} (N/mm^4) per trough	I_{eff} per width of slab (mm^4)	$S_{eff,t}$ (mm^3)	$S_{eff,b}$ (mm^3)	$f_{c,b}$ (N/mm^2)	$M_{p,Rd}$ (kN.m)	**δ (mm)**	$M_{ps\,(kN.m)}$	Section Status
First trial	50.00	160.00	22.28	369,701.34	1093848.4728	41,035.58	49,086.93	141.88	8.77	8.27	6.96	1
1.00	60.00	150.00	21.79	372,016.36	1100697.9983	40,548.37	50,503.03	138.67	8.66	8.22	7.00	1
2.00	65.00	145.00	21.57	371,778.97	1099995.6049	40,192.59	50,992.10	137.66	8.59	8.22	7.02	1
3.00	70.00	140.00	21.36	370,631.09	1096599.3363	39,762.20	51,336.38	137.03	8.49	8.25	7.03	1
4.00	75.00	135.00	21.16	368,624.97	1090663.7769	39,260.91	51,543.38	136.75	8.39	8.29	7.05	1
5.00	80.00	130.00	20.97	365,833.29	1082403.9277	38,694.53	51,624.29	136.77	8.27	8.36	7.06	1
6.00	85.00	125.00	20.78	362,340.20	1072068.7941	38,070.08	51,592.38	137.08	8.13	8.44	7.07	1
7.00	90.00	120.00	20.60	358,233.41	1059917.8940	37,395.14	51,461.66	137.64	7.99	8.53	7.08	1
8.00	95.00	115.00	20.42	353,597.92	1046202.6938	36,677.21	51,245.75	138.41	7.84	8.65	7.09	1
9.00	100.00	110.00	20.24	344,892.06	1020444.3518	35,550.68	50,427.02	140.84	7.59	8.86	7.10	1
10.00	105.00	105.00	20.06	338,430.75	1001327.0635	34,667.65	49,925.59	142.43	7.41	9.03	7.11	1
11.00	110.00	100.00	19.88	331,691.92	981388.6291	33,766.48	49,375.54	144.17	7.21	9.22	7.12	1
12.00	115.00	95.00	19.69	324,729.49	960788.6630	32,852.33	48,785.05	146.07	7.02	9.42	7.13	0
13.00	120.00	90.00	19.51	317,584.52	939648.5900	31,929.23	48,160.20	148.11	6.82	9.63	7.13	0
14.00	125.00	85.00	19.33	310,286.05	918054.3454	31,000.08	47,505.02	150.29	6.62	9.85	7.14	0
15.00	130.00	80.00	19.14	302,852.37	896060.0538	30,066.79	46,821.75	152.62	6.42	10.10	7.15	0
16.00	135.00	75.00	18.95	295,292.51	873692.4178	29,130.44	46,111.02	155.10	6.22	10.35	7.15	0
17.00	140.00	70.00	18.76	287,608.19	850956.5543	28,191.41	45,372.15	157.74	6.02	10.63	7.16	0
18.00	145.00	65.00	18.56	279,798.06	827848.4454	27,249.79	44,603.88	160.58	5.82	10.93	7.16	0
19.00	150.00	60.00	18.36	271,860.51	804363.3399	26,305.67	43,804.82	163.62	5.62	11.25	7.17	0
20.00	155.00	55.00	18.16	263,793.88	780496.3020	25,359.14	42,973.46	166.89	5.42	11.59	7.17	0

Legend: 0=Section does not satisfy verification parameters & 1=Section satisfies

Table 3-2, the calculation of b_{eff} (carried out in another table) requires iteration. Since the program can handle a large number of iteration cycles, it was decided to limit this based on the time required to carry out and available memory, the number of maximum iterations and maximum change to 200 and 0.00001 respectively.

3.1.3 COMPOSITE STAGE

For the composite stage, three methods that were discussed in section 2.7 are used to determine the capacity of the composite section.

3.1.3.1 Full flexural case

The first table prepared in the composite stage is for the full flexural method. The following parameters are calculated in Table 3-3.

- Idealized stress block depth, xpl (Section 2.7.2)
- NA depth of steel deck from top of slab: d_b (Section 2.7.2)
- Lever arm: z (Section 2.7.2)
- Tension force in steel deck, N_p (Section 2.7.2)
- $V_{v,Rd}$ =Vertical shear resistance (Section 2.6.1.2)
- $M_{p,Rd}$ = Moment resistance of composite slab at full interaction (Section 2.7.2)
- Load capacity: This cell estimates the total factored load capacity of the composite slab = $8 * M_{p,Rd,} / L^2$ in kN/m^2
- LL: This cell estimates the unfactored live load that can be applied to the section (kN/m^2)
- δ, LL: The deflection resulting at serviceability (Section 2.6.2.2.2)

Table 3-3: Layout of Full flexural capacity calculation

FULL FLEXURAL CAPACITY (FFC)

No	X_{pl} (mm)	d_p (mm)	z (mm)	N_p (kN)	$V_{v,Rd}$ (kN)	$M_{p,Rd}$ (kN.m)	Load capacity (KN/m^2)	LL (KN/m^2)	δ, LL (mm)
First trial	29.20	107.72	93.12	208.08	100.82	19.38	12.65	5.12	1.04
1	29.92	108.21	93.25	213.17	103.20	19.88	12.98	5.33	1.00
2	30.19	108.43	93.33	215.15	104.38	20.08	13.11	5.41	0.98
3	30.42	108.64	93.43	216.76	105.55	20.25	13.23	5.48	0.97
4	30.60	108.84	93.54	218.04	106.72	20.40	13.32	5.54	0.95
5	30.74	109.03	93.66	219.02	107.88	20.51	13.40	5.59	0.93
6	30.84	109.22	93.80	219.75	109.04	20.61	13.46	5.63	0.91
7	30.91	109.40	93.95	220.27	110.19	20.69	13.51	5.66	0.90
8	30.96	109.58	94.10	220.63	111.34	20.76	13.56	5.69	0.88
9	30.99	109.76	94.27	220.85	112.50	20.82	13.60	5.71	0.87
10	31.01	109.94	94.44	220.99	113.65	20.87	13.63	5.73	0.85
11	31.03	110.12	94.61	221.08	114.80	20.92	13.66	5.75	0.84
12	31.03	110.31	94.79	221.12	0	0	0	0	0
13	31.03	110.49	94.97	221.14	0	0	0	0	0
14	31.04	110.67	95.16	221.15	0	0	0	0	0
15	31.04	110.86	95.34	221.15	0	0	0	0	0
16	31.04	111.05	95.53	221.15	0	0	0	0	0
17	31.04	111.24	95.73	221.15	0	0	0	0	0
18	31.04	111.44	95.92	221.15	0	0	0	0	0
19	31.04	111.64	96.12	221.15	0	0	0	0	0
20	31.04	111.84	96.32	221.15	0	0	0	0	0

Legend: 0=Section does not satisfy required parameters

In the table above, a value of **0** is displayed as output if the section does not satisfy the construction stage or if the section capacity in load is below the load present at composite stage.

3.1.3.2 *First yield method*

In Table 3-4 estimates the capacity of the composite slab based on the First Yield Method are given.

In the first yield method, the dead load due to casting of the concrete was applied to the deck alone to determine the casting stresses, f_c. The additional applied load capacity was then determined by subtracting the casting stresses from the composite strength.

The following parameters are calculated in Table 3-4.

- Elastic NA depth of steel deck from top flange, x_c (Section 2.4.2.6)
- First yield moment per trough, M_{et} (Section 2.7.3)

The others as defined in the previous sections.

Table 3-4: Layout of First Yield method

No	x_c (mm)	M_{et} (per trough)	M_{et} (kN.m)	Load capacity (KN/m^2)	LL (KN/m^2)	δ, LL (mm)
First trial	38.25	3.01	8.91	5.82	0.85	0.17
1	38.73	3.14	9.30	6.08	1.01	0.19
2	38.91	3.21	9.50	6.21	1.10	0.20
3	39.07	3.28	9.71	6.34	1.18	0.21
4	39.21	3.35	9.91	6.47	1.26	0.22
5	39.32	3.42	10.12	6.61	1.35	0.22
6	39.41	3.49	10.33	6.74	1.43	0.23
7	39.49	3.56	10.53	6.88	1.52	0.24
8	39.55	3.63	10.74	7.02	1.60	0.25
9	39.61	3.70	10.96	7.15	1.69	0.26
10	39.66	3.77	11.17	7.29	1.77	0.26
11	39.70	3.85	11.38	7.43	1.86	0.27
12	39.75	0	0	0	0	0
13	39.79	0	0	0	0	0
14	39.83	0	0	0	0	0
15	39.87	0	0	0	0	0
16	39.91	0	0	0	0	0
17	39.95	0	0	0	0	0
18	40.00	0	0	0	0	0
19	40.04	0	0	0	0	0
20	40.08	0	0	0	0	0
Legend: 0=Section does not satisfy required parameters						

In the table above, a value of 0 is displayed as output if the section does not satisfy the construction stage or if the section capacity in load is below the load present at composite stage.

3.1.3.3 *ASCE method*

In Table 3-5 estimates the capacity of the composite slab as recommend in ASCE appendix D method. The following are calculated

- Bond force transfer property, K
- Modified bending moment for bond limitations, M_t

The others are as defined in the previous sections. In order to carry out on developing the design table, some other parameters are needed to be defined. These were established based on the recommended ranges as stated in section 2.7.4. The values are (in mm)

N_b: 43.85	N_t 39.85
Embossment height: p_h= 1.00	Spacing of shear transferring mechanisms, s=25.40

$N_b$and N_h are shown in Figure 2-19

These values are entered in SI units and are converted into imperial units so that they are readily used in the formula.

Table 3-5: Layout of ASCE analysis method

No	K	M_t (kN.m)	Load capacity (KN/m^2)	LL (KN/m^2)	δ, LL (mm)
First trial	0.89	10.68	6.97	1.57	0.32
1	0.89	11.15	7.28	1.77	0.33
2	0.89	11.39	7.44	1.86	0.34
3	0.89	11.63	7.59	1.96	0.35
4	0.89	11.87	7.76	2.06	0.35
5	0.89	12.12	7.92	2.16	0.36
6	0.89	12.37	8.08	2.27	0.37
7	0.89	12.62	8.24	2.37	0.38
8	0.89	12.87	8.41	2.47	0.38
9	0.89	13.13	8.57	2.57	0.39
10	0.89	13.38	8.74	2.68	0.40
11	0.89	13.64	8.90	2.78	0.40
12	0.89	0	0	0	0
13	0.89	0	0	0	0
14	0.89	0	0	0	0
15	0.89	0	0	0	0
16	0.89	0	0	0	0
17	0.89	0	0	0	0
18	0.89	0	0	0	0
19	0.89	0	0	0	0
20	0.89	0	0	0	0
Legend: 0=Section does not satisfy required parameters					

In the table above, a value of **0** is displayed as output if the section does not satisfy the construction stage or if the section capacity in load is below the load present at composite stage.

3.1.3.4 *Design graph*

Based on the values calculated in the previous three sections, the design chart is prepared for the values given in the design data section. The graph shown in Figure 3-1 gives the live load capacity of a composite slab for the three methods. This means for a given value of the bottom deck size (b_u), it is possible to estimate the live load capacity of the section.

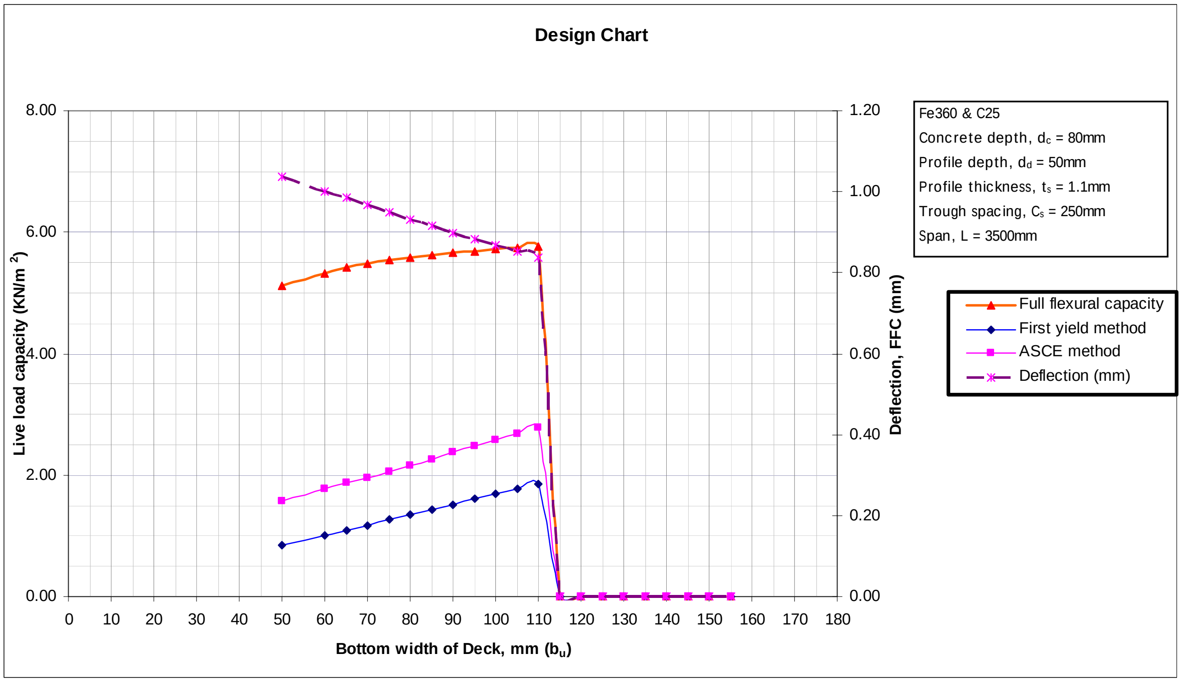


Figure 3-1: Design Graph

3.2 DESIGN TABLES AND GRAPHS

3.2.1 GENERAL

The following flow chart depicts the hierarchy employed while developing the tables and charts.

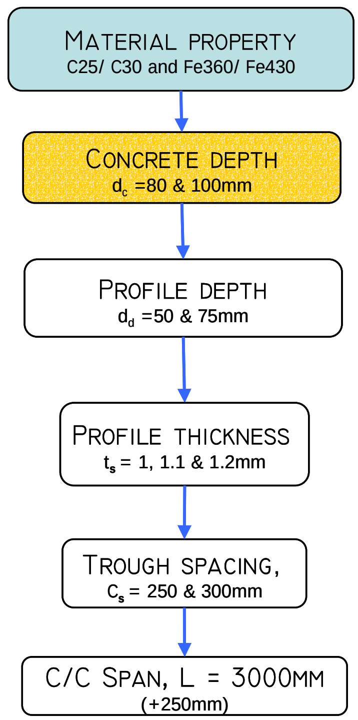


Figure 3-2: Flow chart for Design table and chart development

A sample design graph is inserted for the two profile depths, i.e. 50 and 75mm. The sample is selected to have the same profile thickness, trough spacing and span. The values are

- Profile thickness =1.2mm
- Trough spacing = 300mm
- Span = 3500mm

3.2.2 DESIGN TABLE AND GRAPHS FOR C25 AND FE360

3.2.2.1 Concrete thickness of 80mm

3.2.2.1.1 Profile depth of 50mm

In this section, different design tables and graphs for a concrete depth of 80 mm and profiled depth of 50mm are prepared. The tables are arranged as per the flow chart shown below.

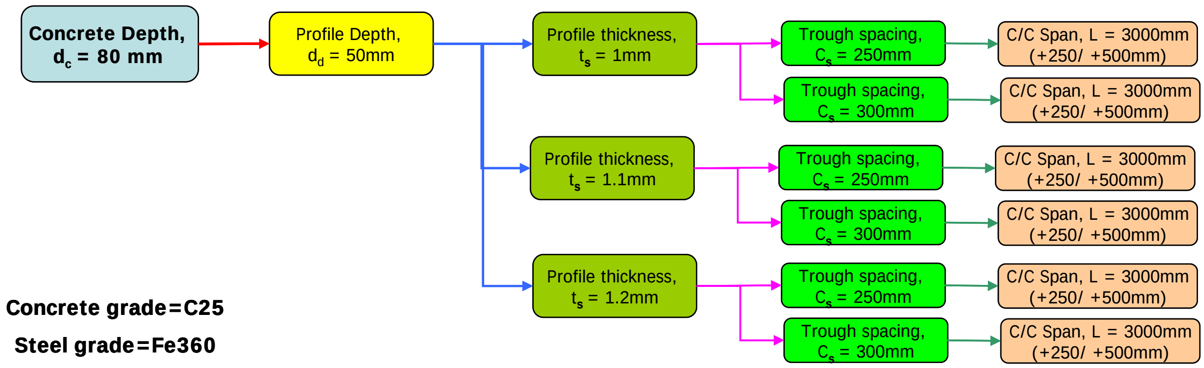
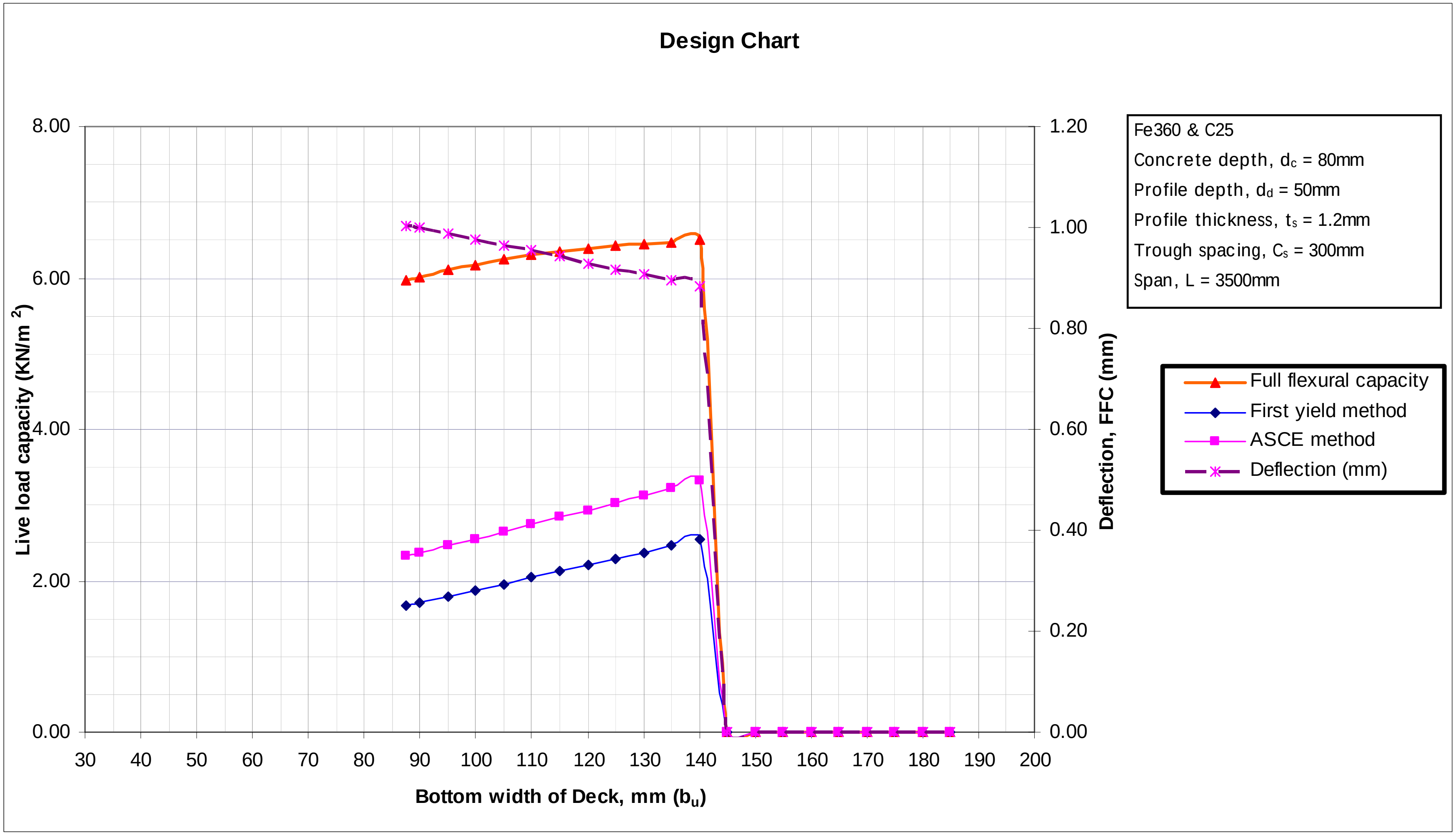


Figure 3-3: Flow chart for d_c=80mm and d_d=50mm


Design Chart
Live load capacity (KN/m ^{2})
Deflection, FFC (mm)
Bottom width of Deck, mm (b_u)
8.00
6.00
4.00
2.00
0.00
1.20
1.00
0.80
0.60
0.40
0.20
0.00
30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200
Fe360 & C25
Concrete depth, d_c = 80mm
Profile depth, d_d = 50mm
Profile thickness, t_s = 1.2mm
Trough spacing, C_s = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)

3.2.2.1.2 *Profile depth of 75mm*

In this section, different design tables and graphs for a concrete depth of 80 mm and profiled depth of 75mm are prepared. The tables are arranged as per the flow chart shown below.

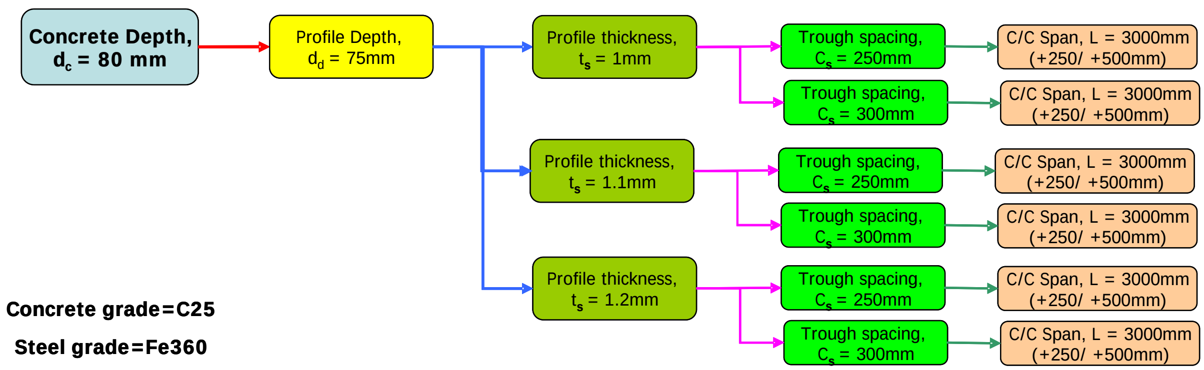
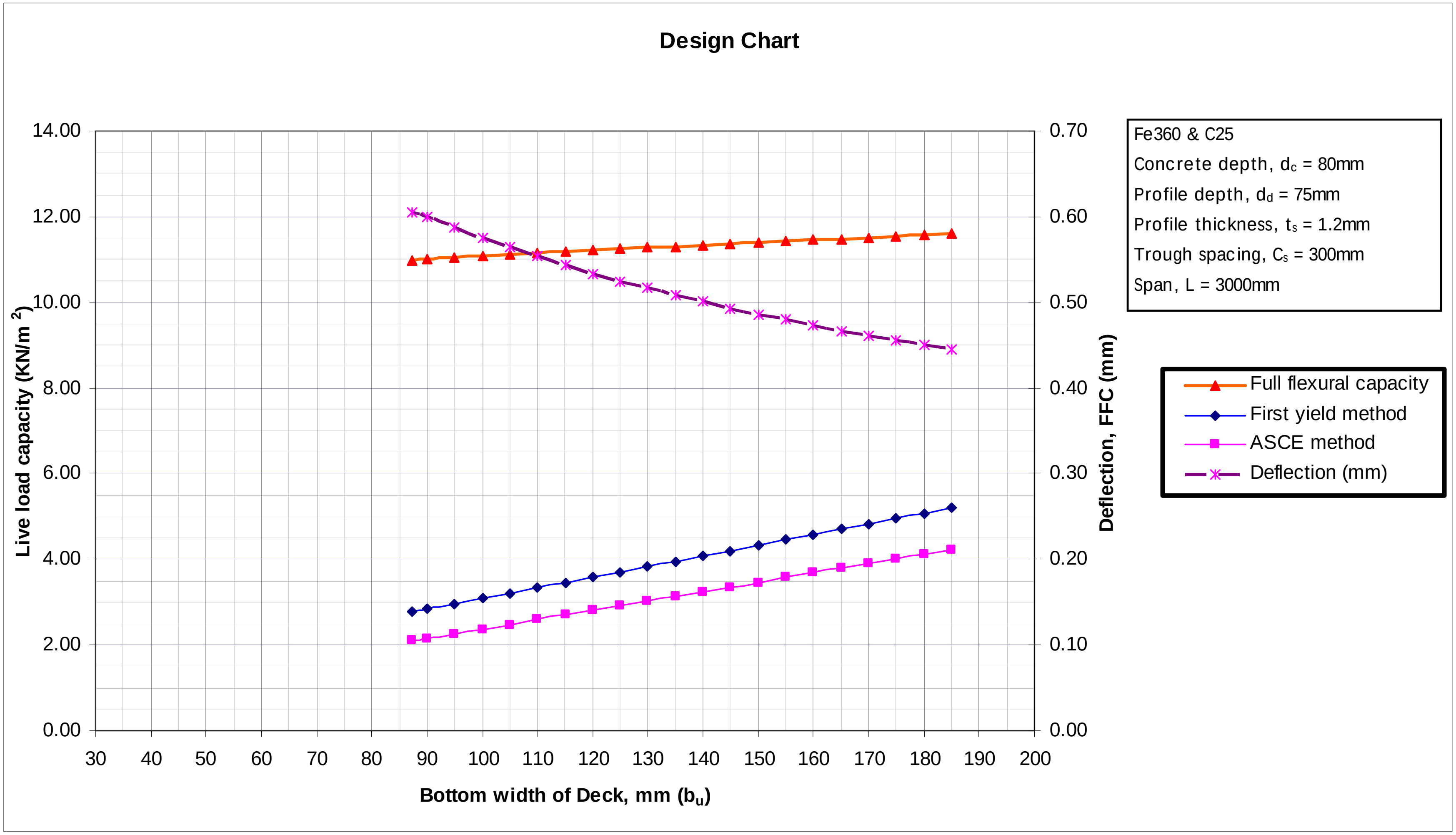


Figure 3-4: Flow chart for d_c=80mm and d_d=75mm



Design Chart
Fe360 & C25
Concrete depth, dc = 80mm
Profile depth, dd = 75mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3000mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)
Live load capacity (KN/m 2)
Deflection, FFC (mm)
Bottom width of Deck, mm (bu)
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200

3.2.2.2 *Concrete thickness of 100mm*

3.2.2.2.1 Profile depth of 50mm

In this section, different design tables and graphs for a concrete depth of 100 mm and profiled depth of 50mm are prepared. The tables are arranged as per the flow chart shown below.

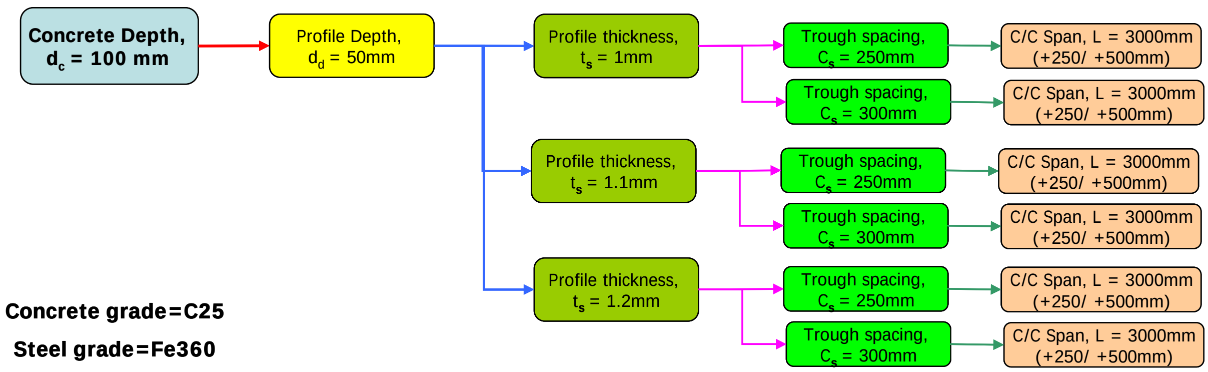
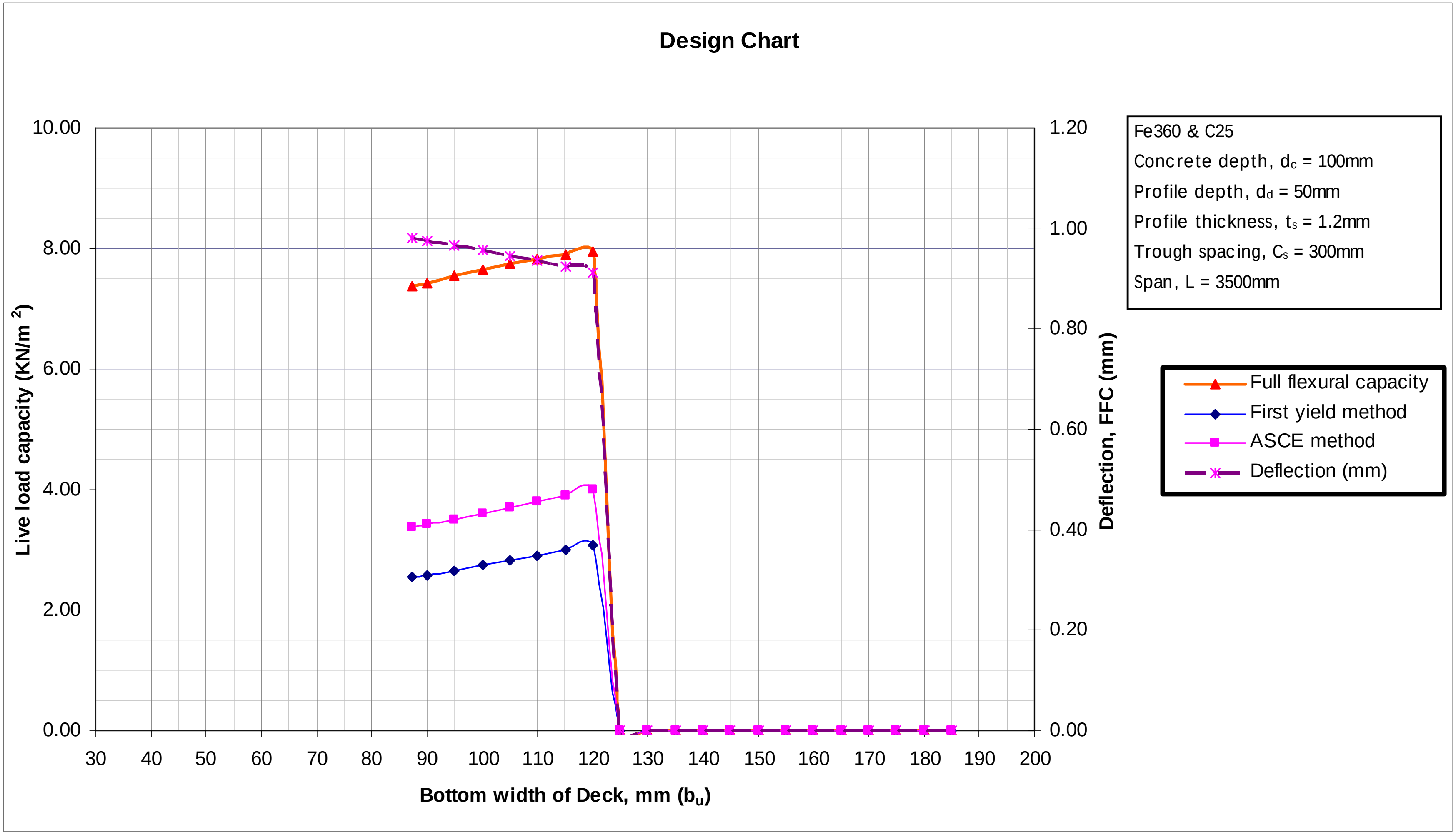


Figure 3-5: Flow chart for d_c=100mm and d_d=50mm



Design Chart
Fe360 & C25
Concrete depth, dc = 100mm
Profile depth, dd = 50mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)
Live load capacity (KN/m 2)
Deflection, FFC (mm)
Bottom width of Deck, mm (bu)
10.00
8.00
6.00
4.00
2.00
0.00
1.20
1.00
0.80
0.60
0.40
0.20
0.00
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200

3.2.2.2.2 Profile depth of 75mm

In this section, different design tables and graphs for a concrete depth of 100 mm and profiled depth of 75mm are prepared. The tables are arranged as per the flow chart shown below.

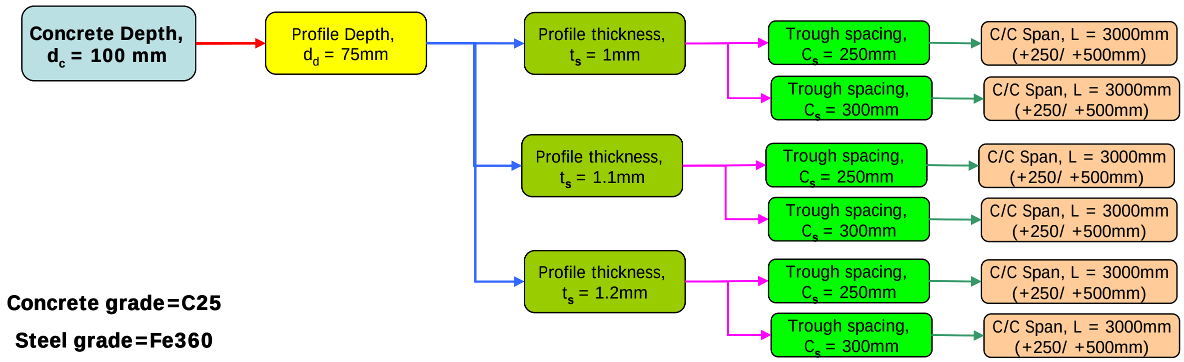
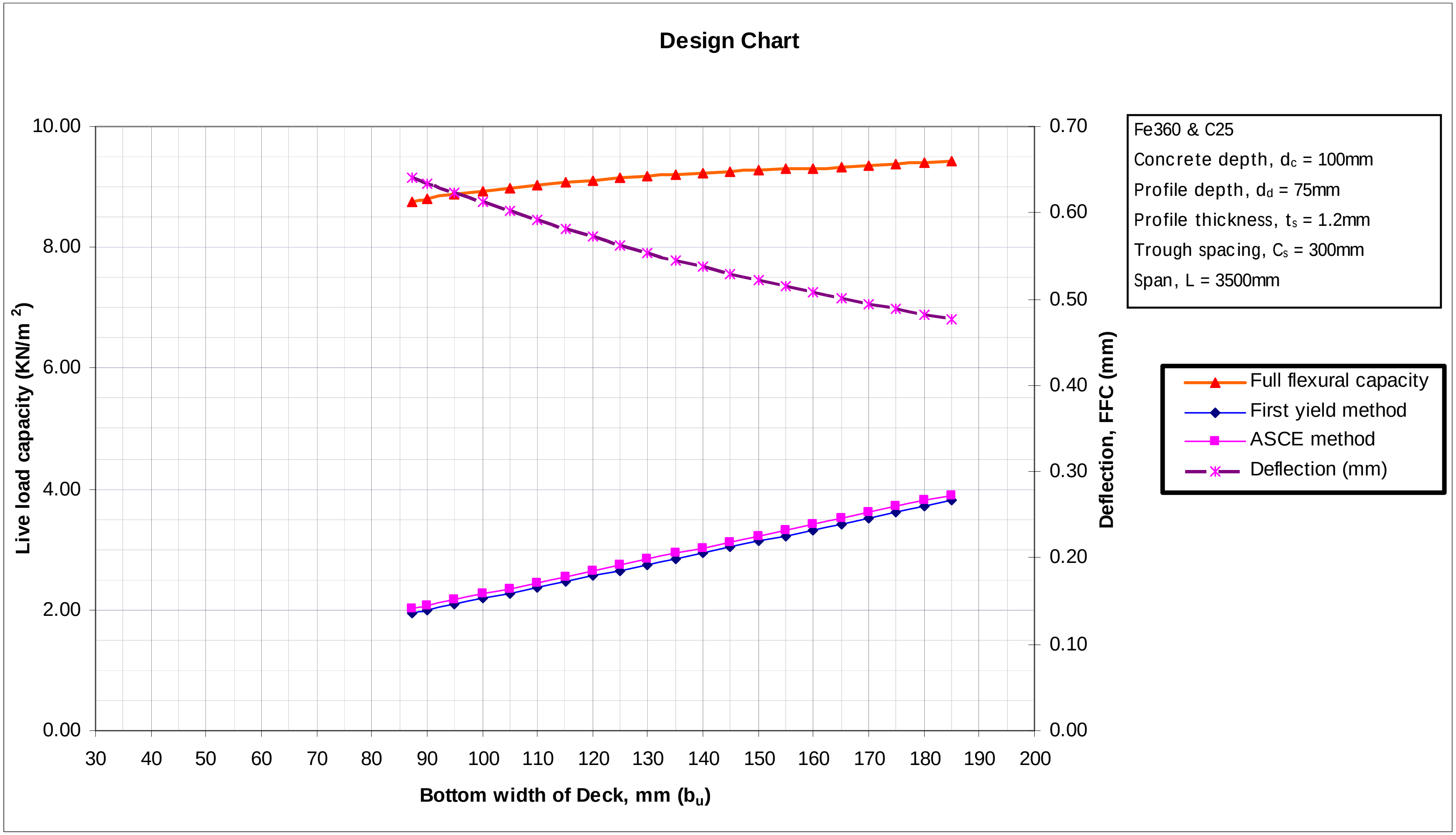


Figure 3-6: Flow chart for d_c=100mm and d_d=75mm


Design Chart
Live load capacity (KN/m 2)
10.00
8.00
6.00
4.00
2.00
0.00
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
Deflection, FFC (mm)
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
Bottom width of Deck, mm (bu)
Fe360 & C25
Concrete depth, dc = 100mm
Profile depth, dd = 75mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)

3.2.3 DESIGN TABLE AND GRAPHS FOR C25 AND FE430

3.2.3.1 Concrete thickness of 80mm

3.2.3.1.1 Profile depth of 50mm

In this section, different design tables and graphs for a concrete depth of 80 mm and profiled depth of 50mm are prepared. The tables are arranged as per the flow chart shown below.

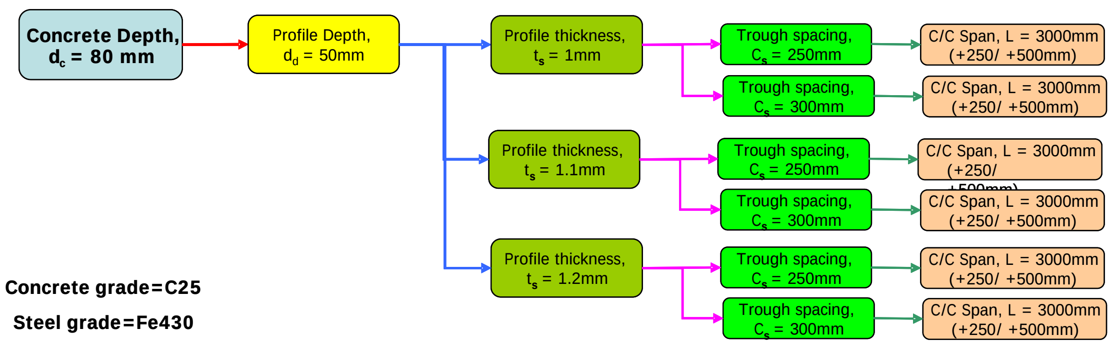
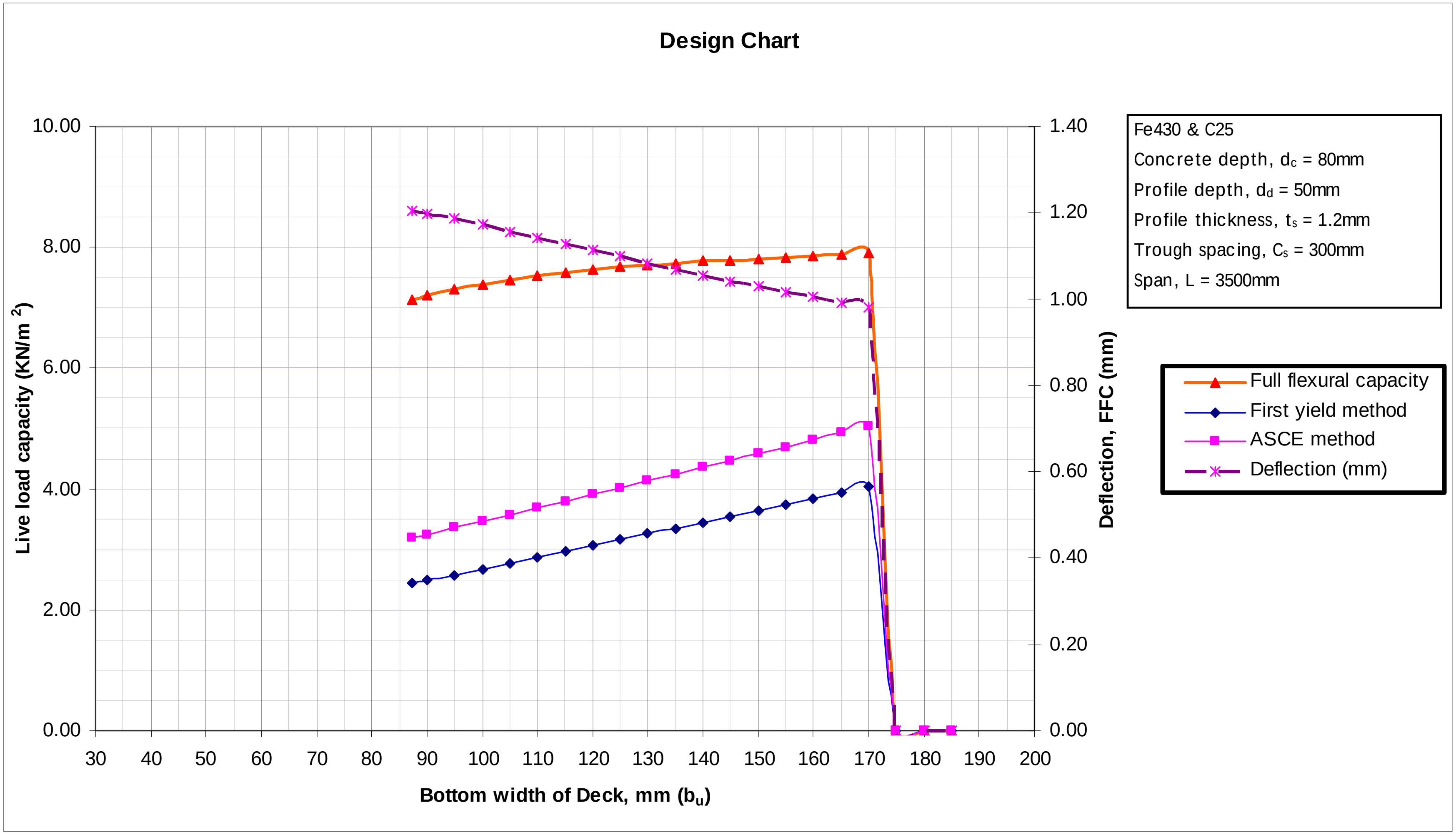


Figure 3-7: Flow chart for d_c=80mm and d_d=50mm


Design Chart
Fe430 & C25
Concrete depth, dc = 80mm
Profile depth, dd = 50mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)
Live load capacity (KN/m 2)
Deflection, FFC (mm)
Bottom width of Deck, mm (bu)
10.00
8.00
6.00
4.00
2.00
0.00
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200

3.2.3.1.2 Profile depth of 75mm

In this section, different design tables and graphs for a concrete depth of 80 mm and profiled depth of 75mm are prepared. The tables are arranged as per the flow chart shown below.

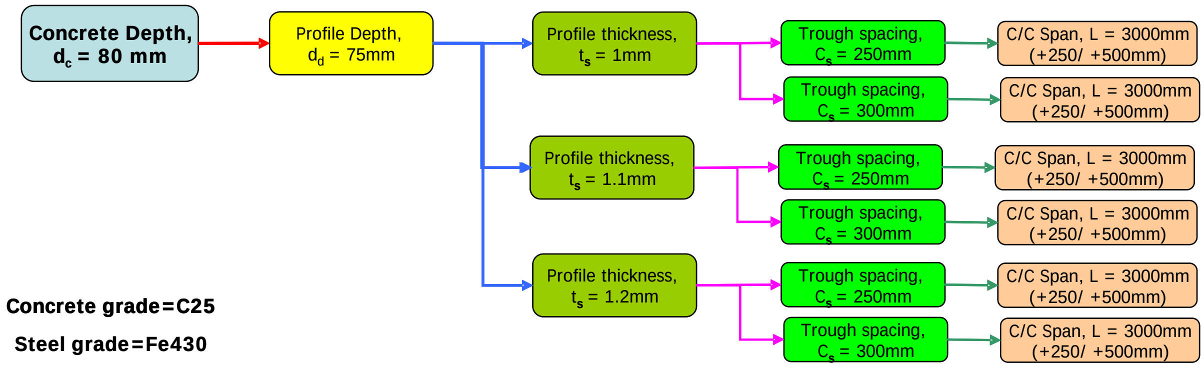
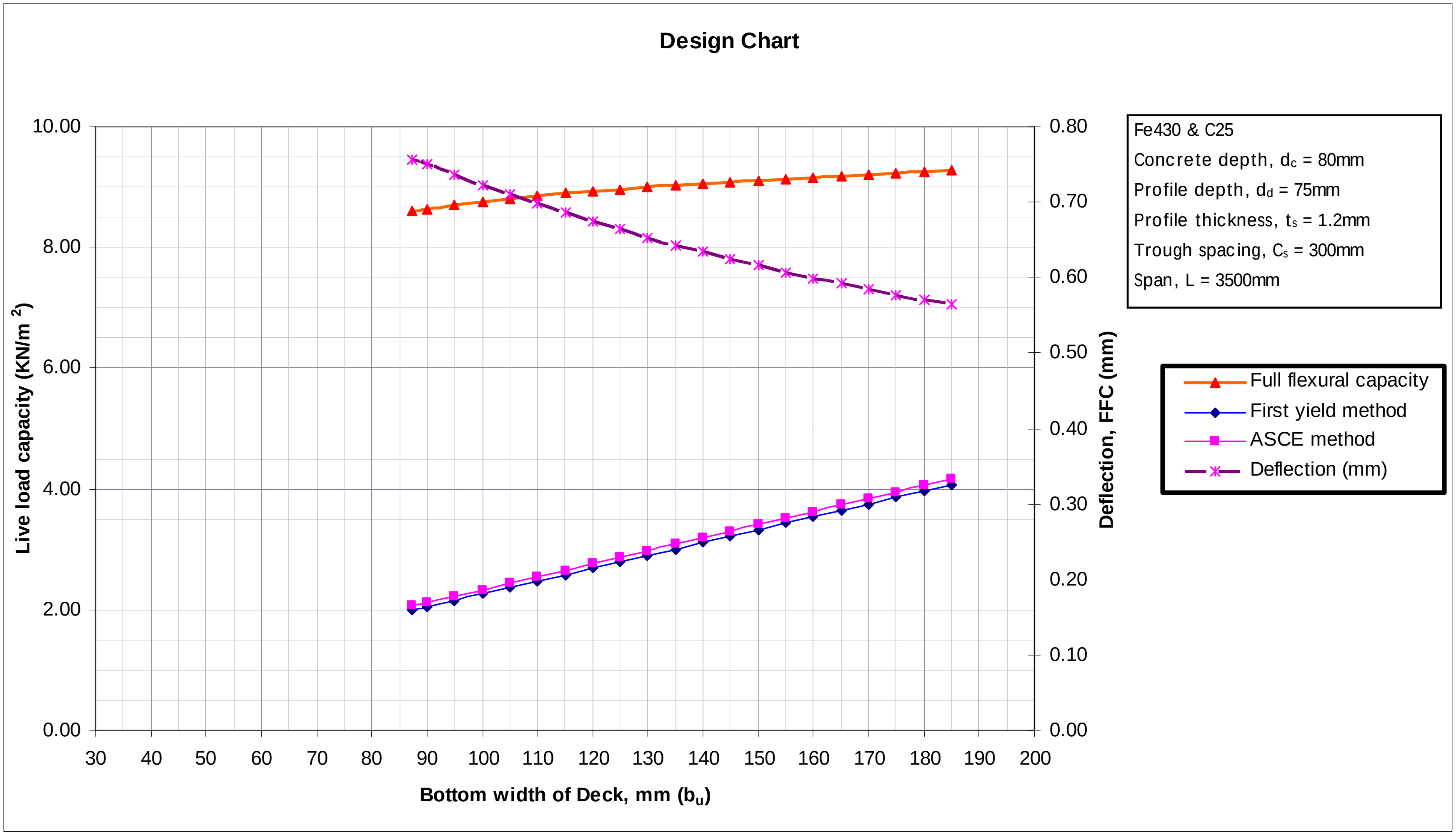


Figure 3-8: Flow chart for d_c=80mm and d_d=75mm


Design Chart
Fe430 & C25
Concrete depth, dc = 80mm
Profile depth, dd = 75mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)
Live load capacity (KN/m 2)
Deflection, FFC (mm)
Bottom width of Deck, mm (bu)
10.00
8.00
6.00
4.00
2.00
0.00
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200

3.2.3.2 Concrete thickness of 100mm

3.2.3.2.1 Profile depth of 50mm

In this section, different design tables and graphs for a concrete depth of 100 mm and profiled depth of 50mm are prepared. The tables are arranged as per the flow chart shown below.

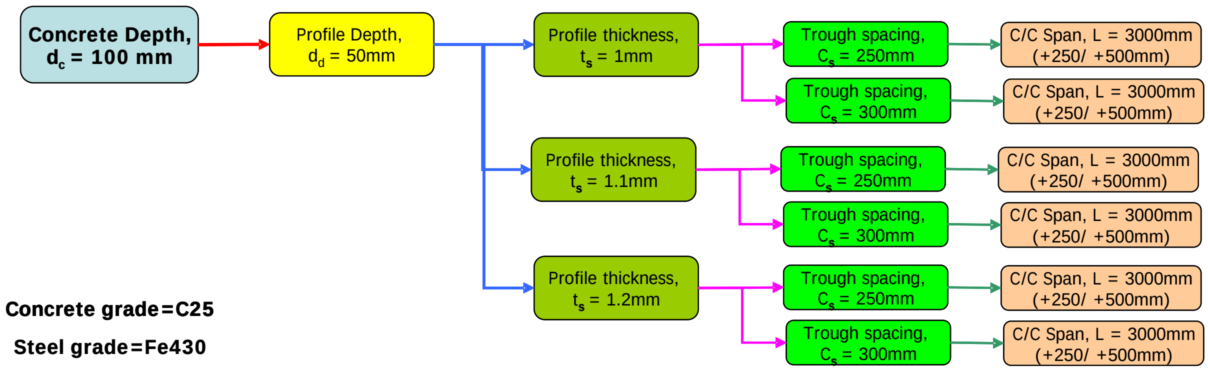
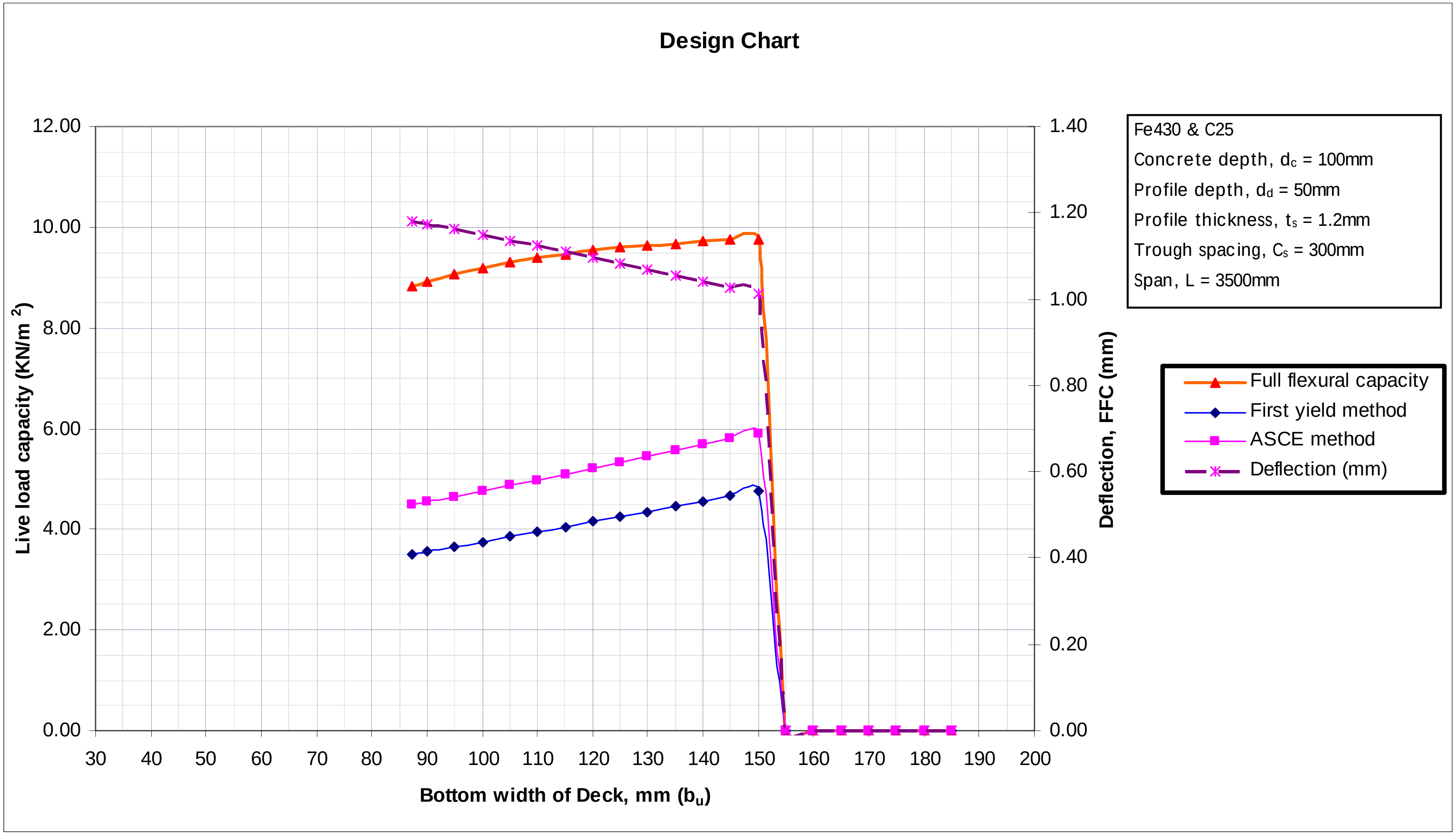


Figure 3-9: Flow chart for d_c=100mm and d_d=50mm


Design Chart
Fe430 & C25
Concrete depth, d_c = 100mm
Profile depth, d_d = 50mm
Profile thickness, t_s = 1.2mm
Trough spacing, C_s = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)
Live load capacity (KN/m ^{2})
Deflection, FFC (mm)
Bottom width of Deck, mm (b_u)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200

3.2.3.2.2 Profile depth of 75mm

In this section, different design tables and graphs for a concrete depth of 100 mm and profiled depth of 75mm are prepared. The tables are arranged as per the flow chart shown below.

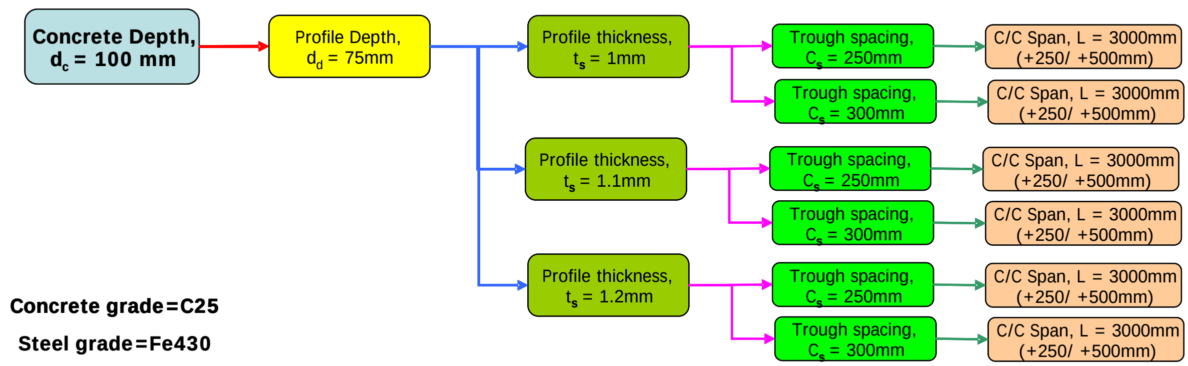
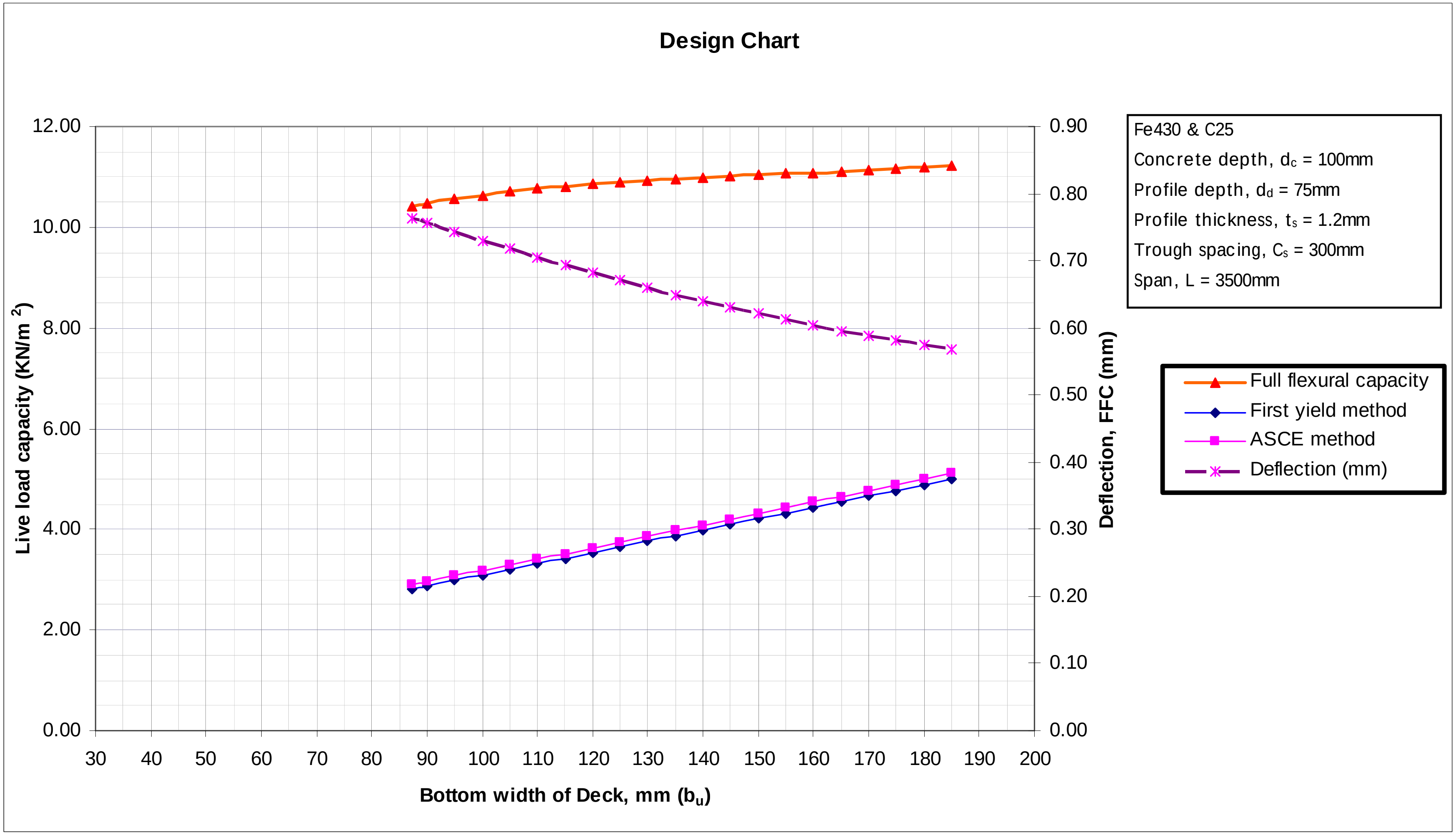


Figure 3-10: Flow chart for d_c=100mm and d_d=75mm



Design Chart
Fe430 & C25
Concrete depth, dc = 100mm
Profile depth, dd = 75mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)
Live load capacity (KN/m 2)
Deflection, FFC (mm)
Bottom width of Deck, mm (bu)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200

3.2.4 DESIGN TABLE AND GRAPHS FOR C30 AND FE360

3.2.4.1 Concrete thickness of 80mm

3.2.4.1.1 Profile depth of 50mm

In this section, different design tables and graphs for a concrete depth of 80 mm and profiled depth of 50mm are prepared. The tables are arranged as per the flow chart shown below.

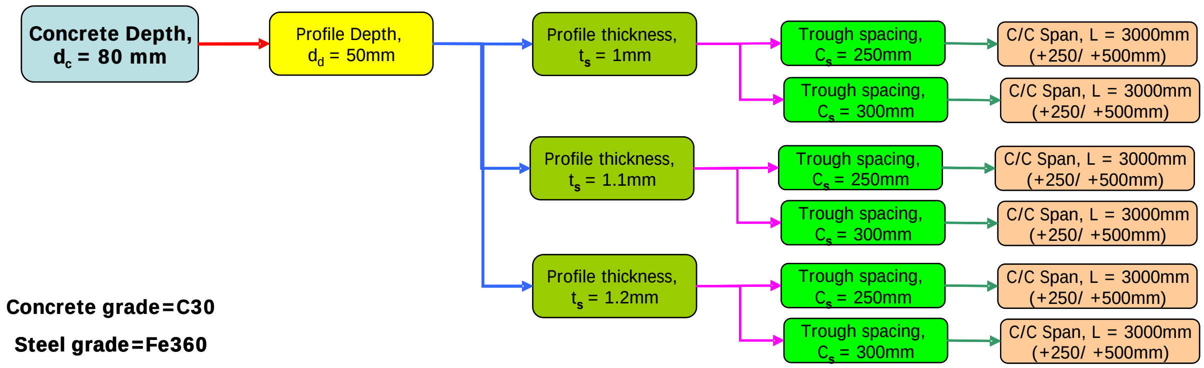
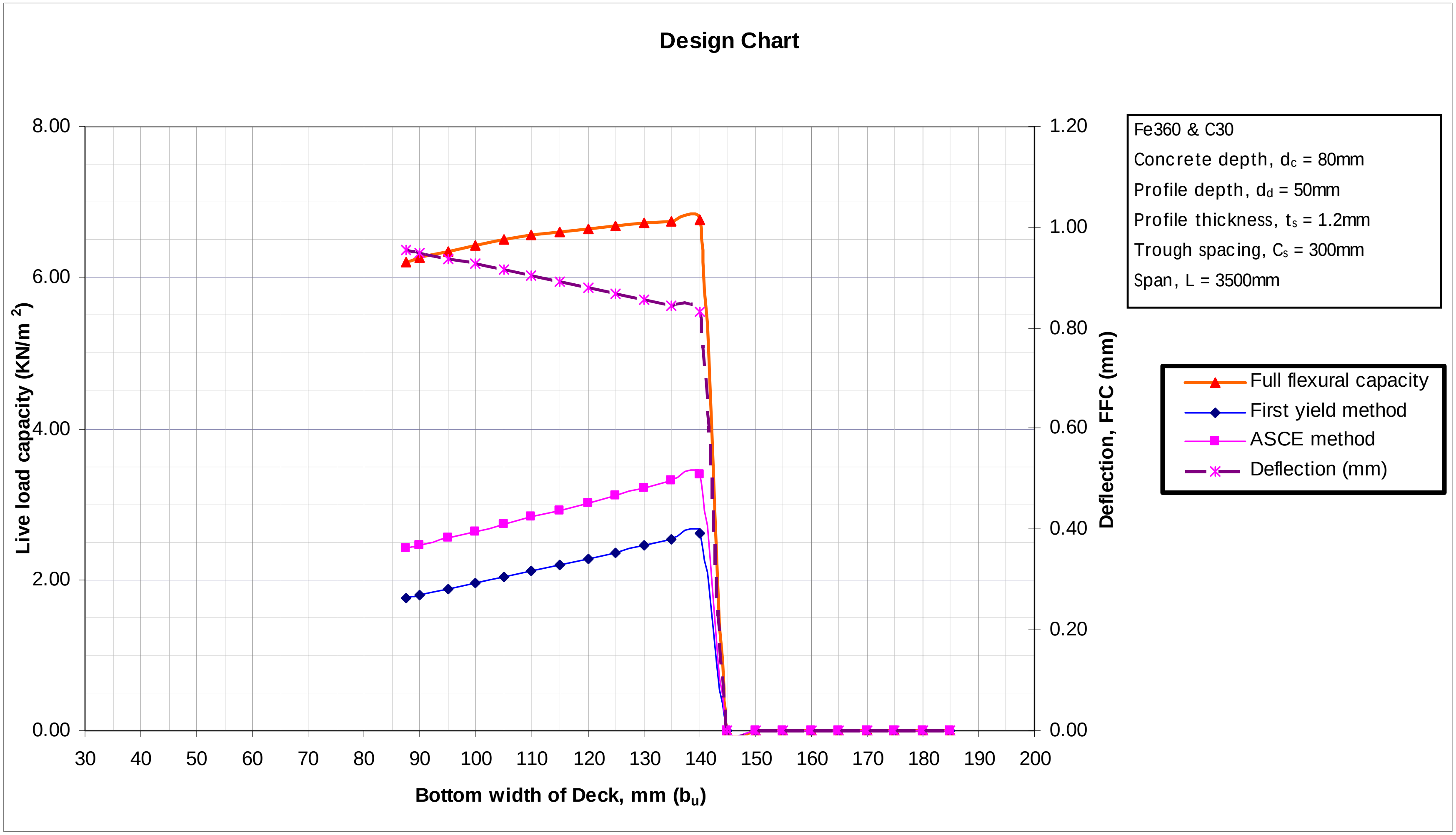


Figure 3-11: Flow chart for d_c=80mm and d_d=50mm



Design Chart
Fe360 & C30
Concrete depth, dc = 80mm
Profile depth, dd = 50mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)
Live load capacity (KN/m 2)
Deflection, FFC (mm)
Bottom width of Deck, mm (bu)
0.00
2.00
4.00
6.00
8.00
0.00
0.20
0.40
0.60
0.80
1.00
1.20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200

3.2.4.1.2 Profile depth of 75mm

In this section, different design tables and graphs for a concrete depth of 80 mm and profiled depth of 75mm are prepared. The tables are arranged as per the flow chart shown below.

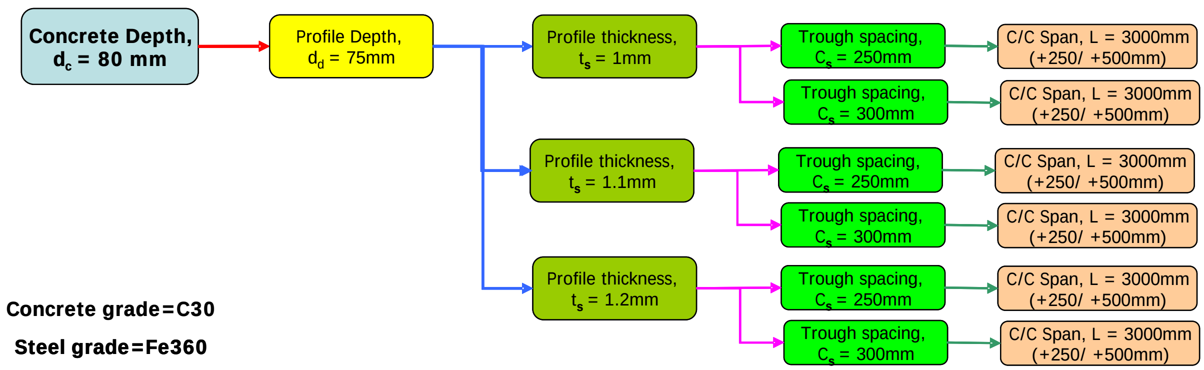
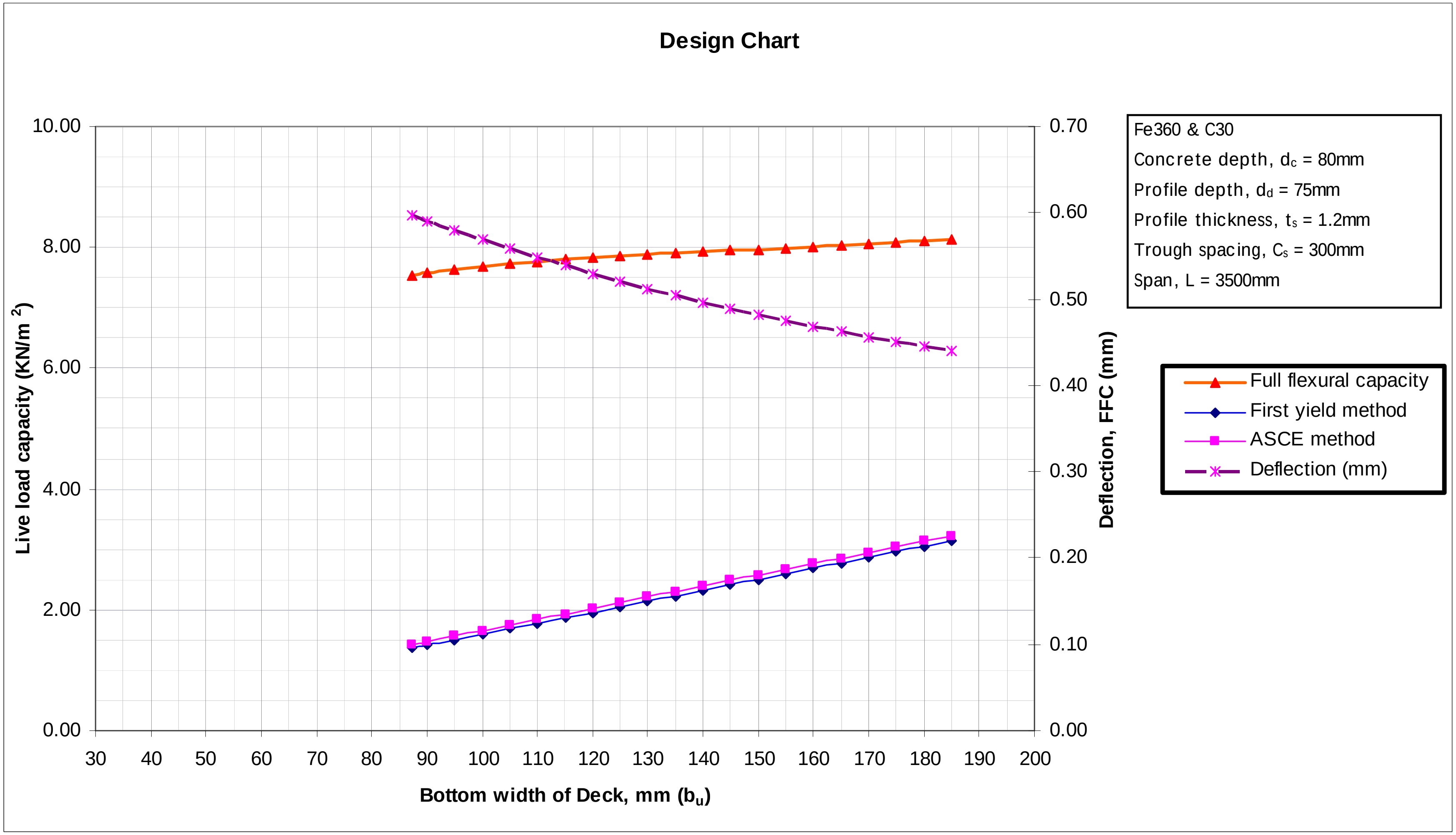


Figure 3-12: Flow chart for d_c=80mm and d_d=75mm



Design Chart
10.00
8.00
6.00
4.00
2.00
0.00
Live load capacity (KN/m 2)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
Deflection, FFC (mm)
30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200
Bottom width of Deck, mm (bu)
Fe360 & C30
Concrete depth, dc = 80mm
Profile depth, dd = 75mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)

3.2.4.2 *Concrete thickness of 100mm*

3.2.4.2.1 Profile depth of 50mm

In this section, different design tables and graphs for a concrete depth of 100 mm and profiled depth of 50mm are prepared. The tables are arranged as per the flow chart shown below.

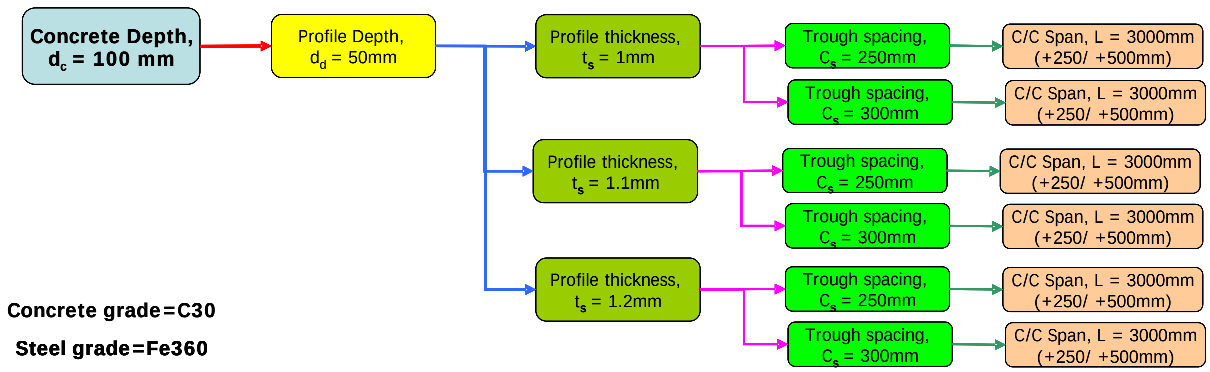
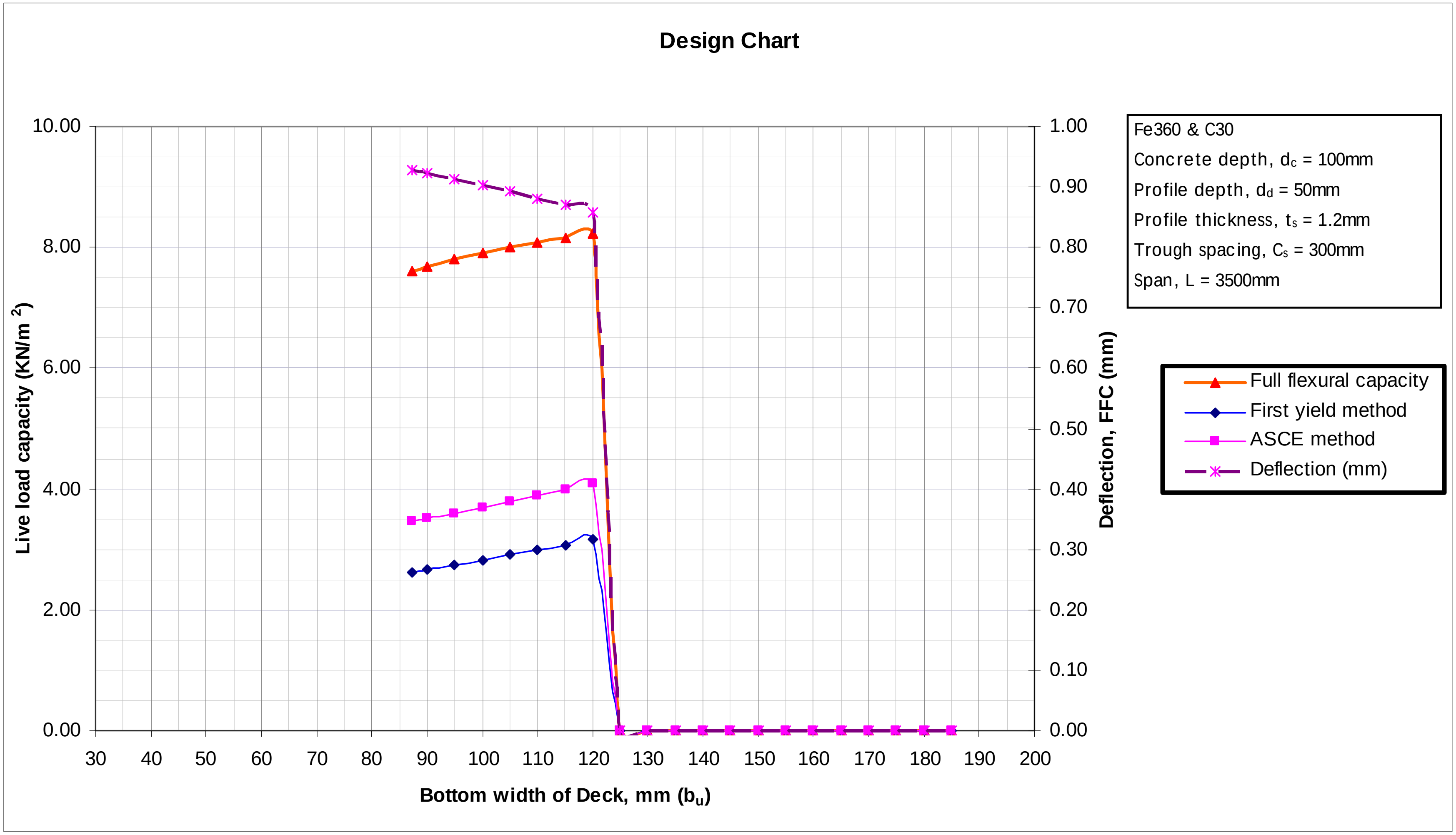


Figure 3-13: Flow chart for d_c=100mm and d_d=50mm


Design Chart
Fe360 & C30
Concrete depth, dc = 100mm
Profile depth, dd = 50mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)
Live load capacity (KN/m 2)
Deflection, FFC (mm)
Bottom width of Deck, mm (bu)
10.00
8.00
6.00
4.00
2.00
0.00
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200

3.2.4.2.2 Profile depth of 75mm

In this section, different design tables and graphs for a concrete depth of 100 mm and profiled depth of 75mm are prepared. The tables are arranged as per the flow chart shown below.

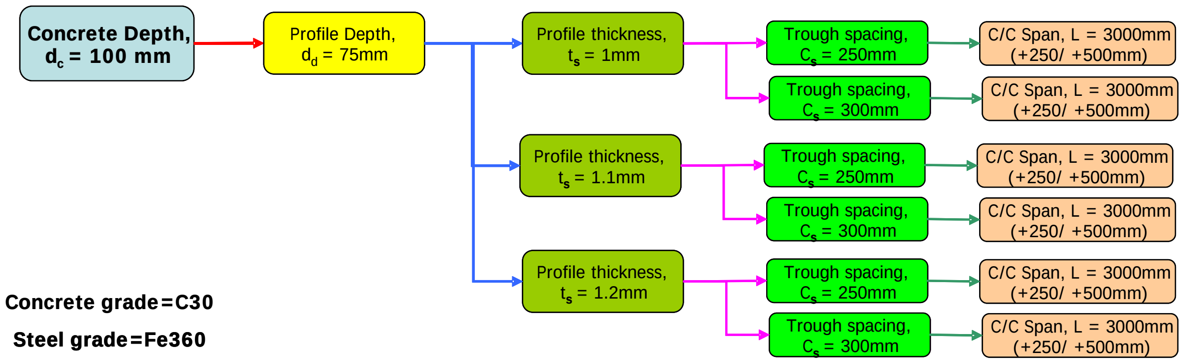
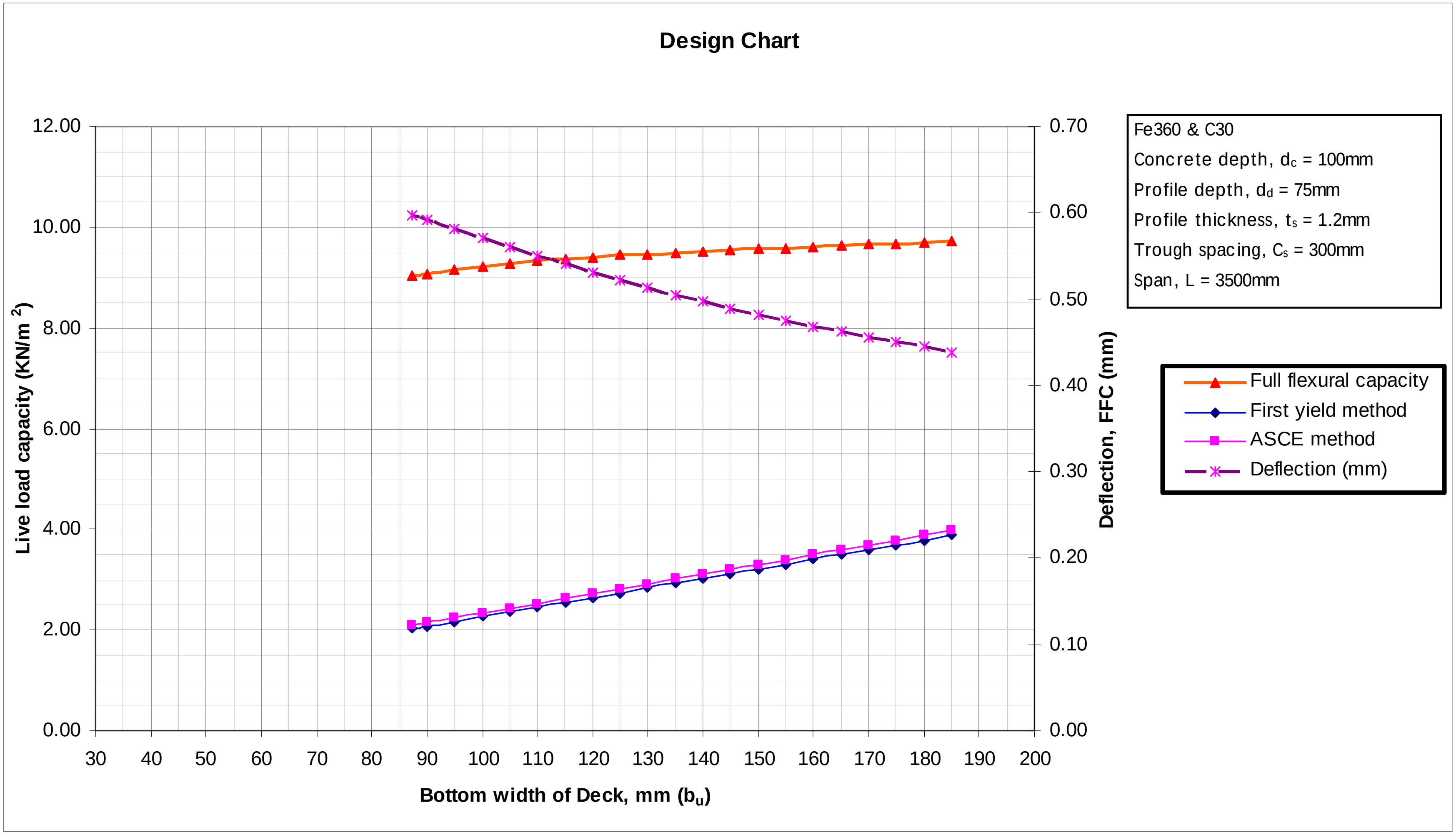


Figure 3-14: Flow chart for d_c=100mm and d_d=75mm



Design Chart
Fe360 & C30
Concrete depth, dc = 100mm
Profile depth, dd = 75mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)
Live load capacity (KN/m 2)
Deflection, FFC (mm)
Bottom width of Deck, mm (bu)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200

3.2.5 DESIGN TABLE AND GRAPHS FOR C30 AND FE430

3.2.5.1 Concrete thickness of 80mm

3.2.5.1.1 Profile depth of 50mm

In this section, different design tables and graphs for a concrete depth of 80 mm and profiled depth of 50mm are prepared. The tables are arranged as per the flow chart shown below.

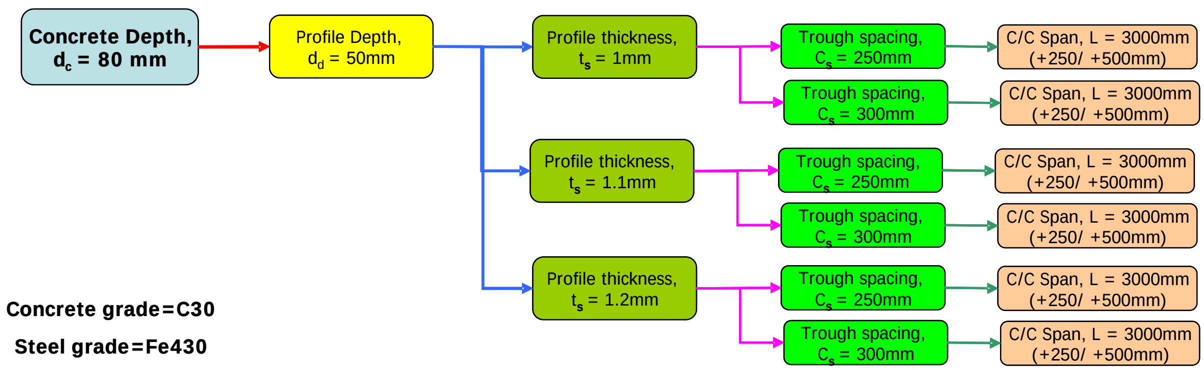
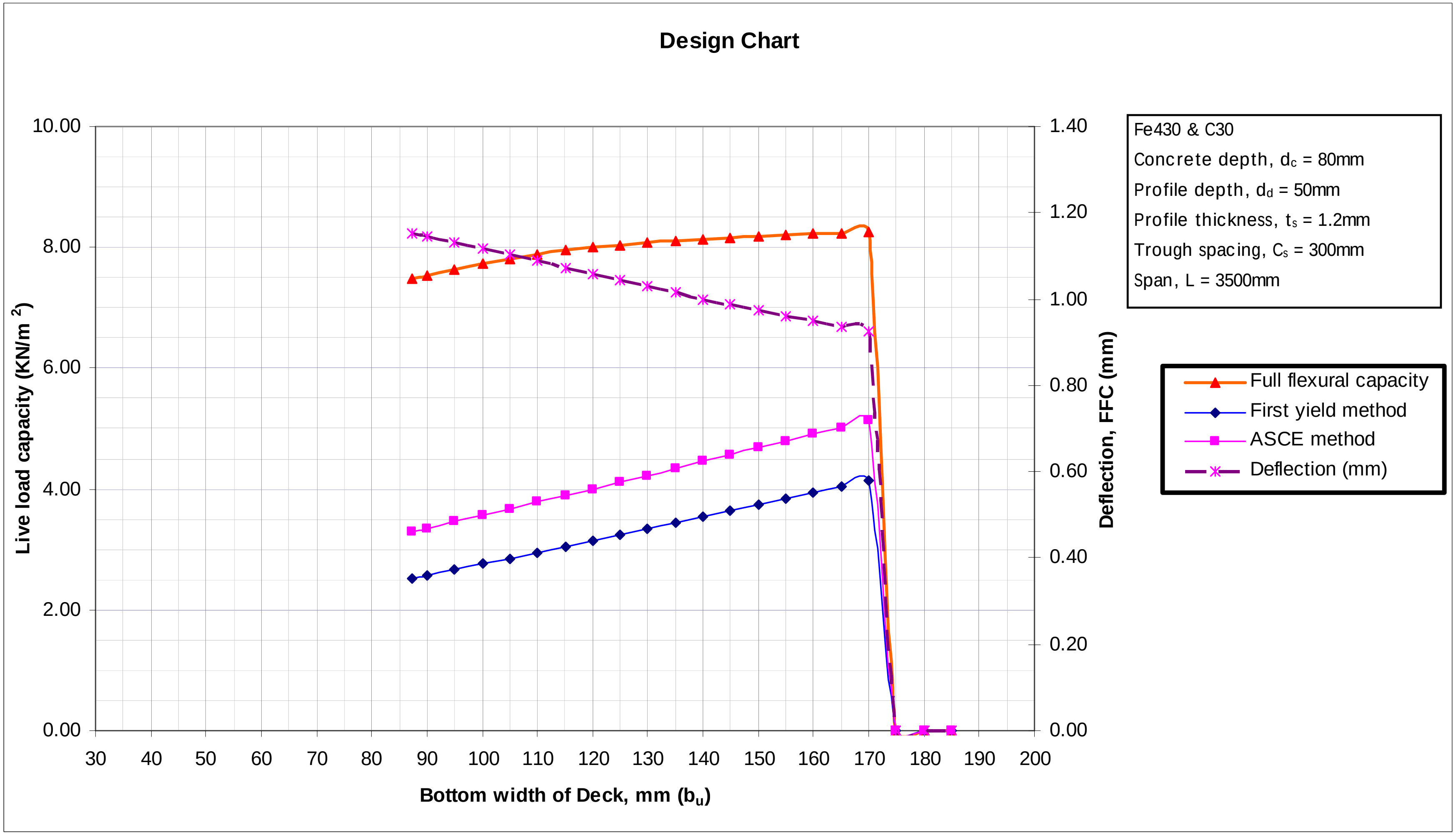


Figure 3-15: Flow chart for d_c=80mm and d_d=50mm



Design Chart
Live load capacity (KN/m 2)
Deflection, FFC (mm)
Bottom width of Deck, mm (bu)
10.00
8.00
6.00
4.00
2.00
0.00
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200
Fe430 & C30
Concrete depth, dc = 80mm
Profile depth, dd = 50mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)

3.2.5.1.2 Profile depth of 75mm

In this section, different design tables and graphs for a concrete depth of 80 mm and profiled depth of 75mm are prepared. The tables are arranged as per the flow chart shown below.

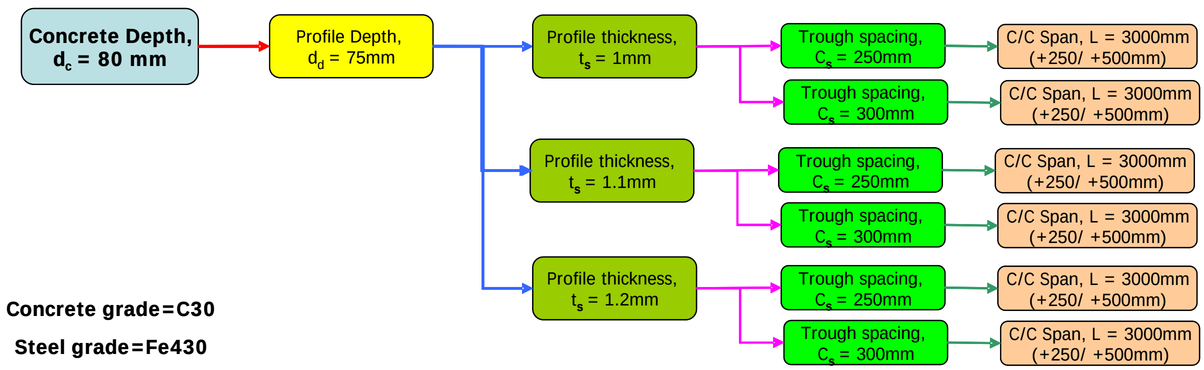
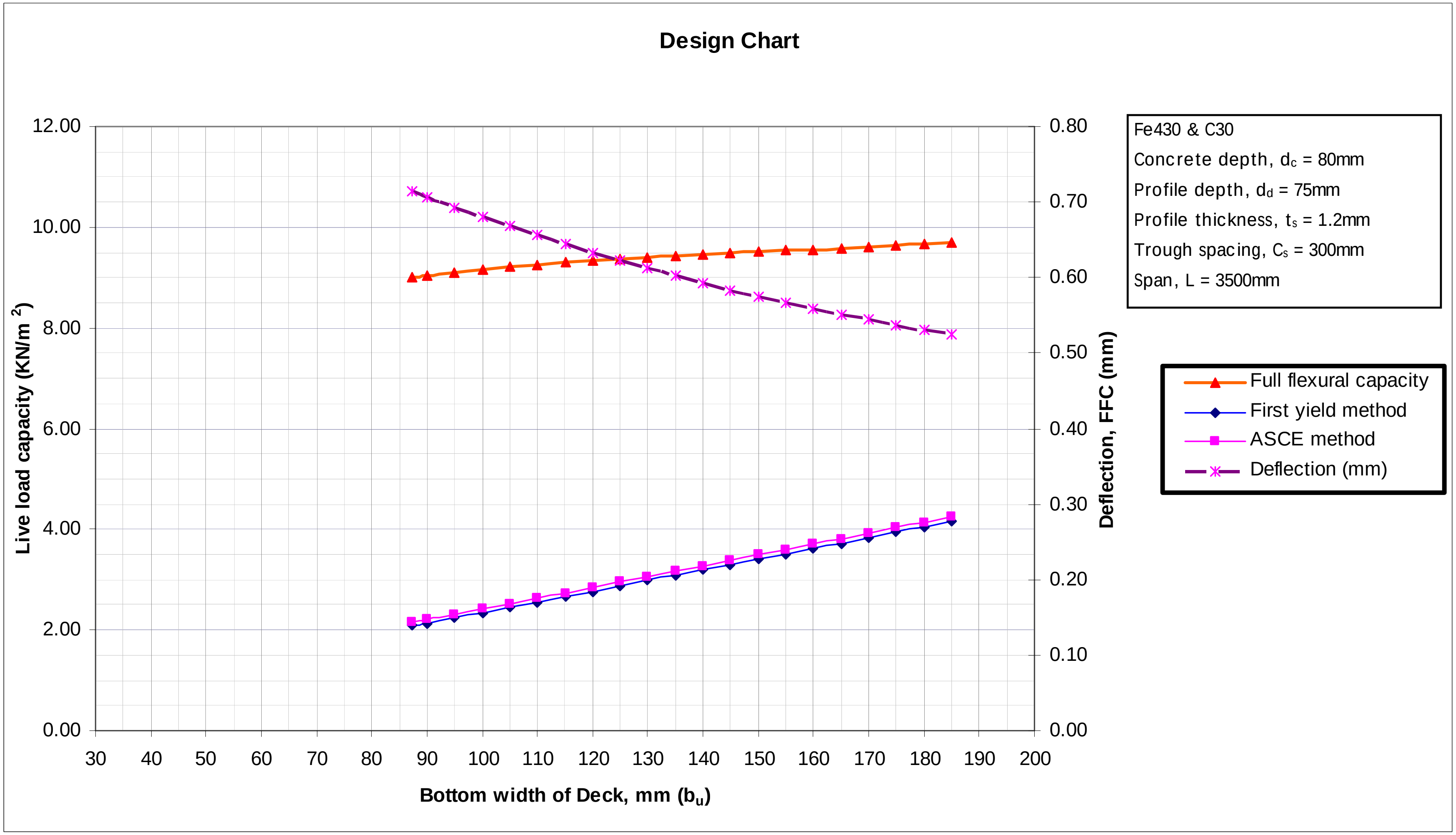


Figure 3-16: Flow chart for d_c=80mm and d_d=75mm



Design Chart
Fe430 & C30
Concrete depth, dc = 80mm
Profile depth, dd = 75mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)
Live load capacity (KN/m 2)
Deflection, FFC (mm)
Bottom width of Deck, mm (bu)
0.00
2.00
4.00
6.00
8.00
10.00
12.00
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200

3.2.5.2 Concrete thickness of 100mm

3.2.5.2.1 Profile depth of 50mm

In this section, different design tables and graphs for a concrete depth of 100 mm and profiled depth of 50mm are prepared. The tables are arranged as per the flow chart shown below.

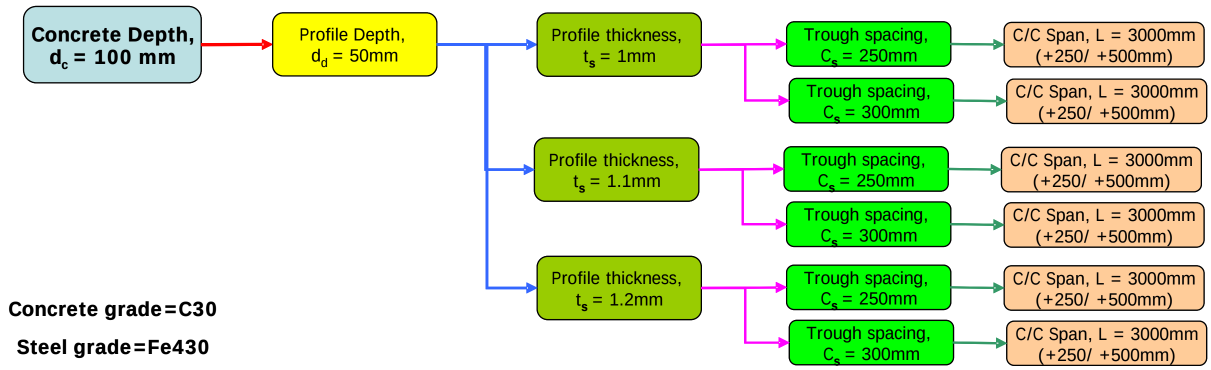
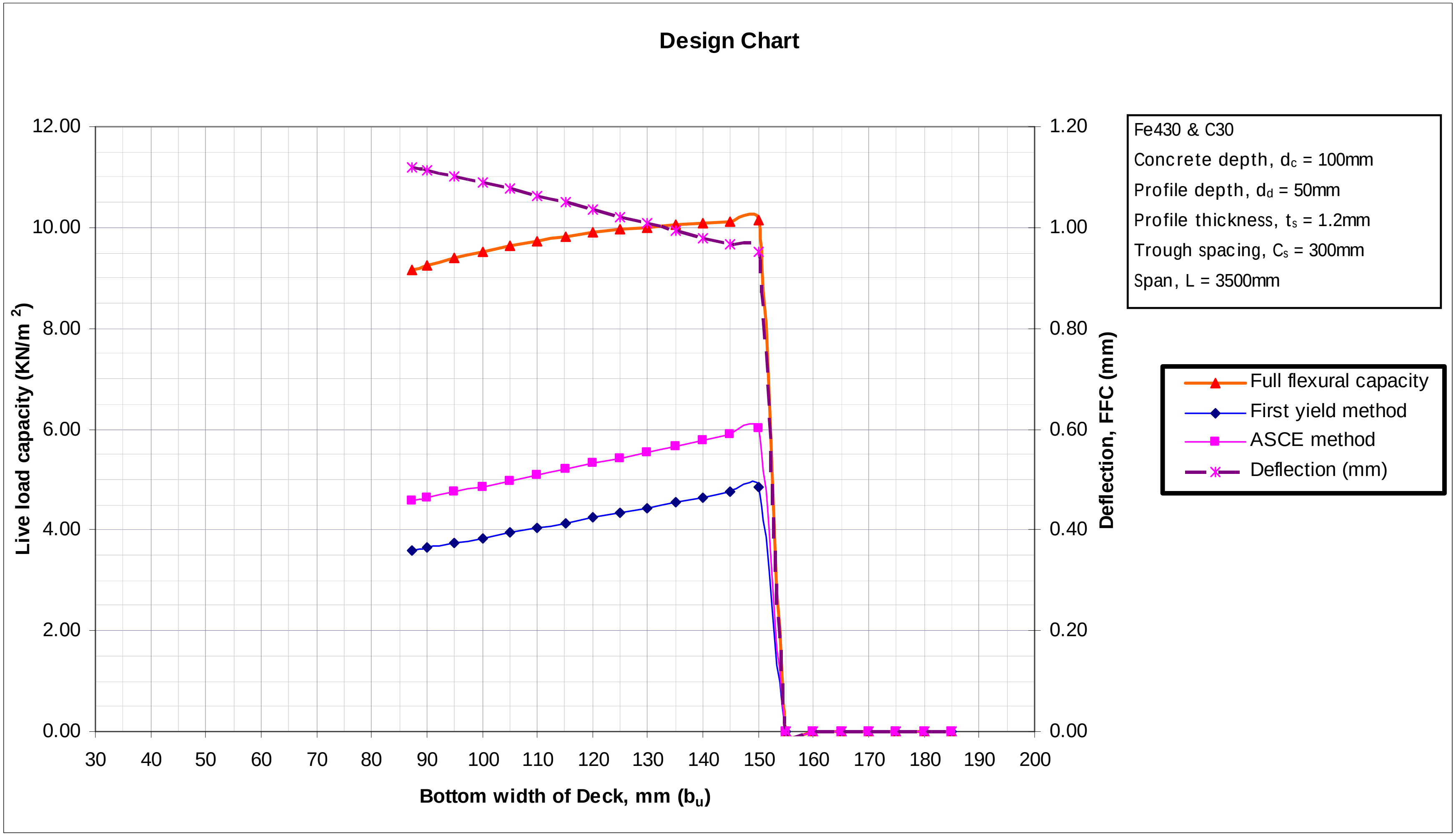


Figure 3-17: Flow chart for d_c=100mm and d_d=50mm


Design Chart
Fe430 & C30
Concrete depth, dc = 100mm
Profile depth, dd = 50mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)
Live load capacity (KN/m 2)
Deflection, FFC (mm)
Bottom width of Deck, mm (bu)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
1.20
1.00
0.80
0.60
0.40
0.20
0.00
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200

3.2.5.2.2 Profile depth of 75mm

In this section, different design tables and graphs for a concrete depth of 100 mm and profiled depth of 75mm are prepared. The tables are arranged as per the flow chart shown below.

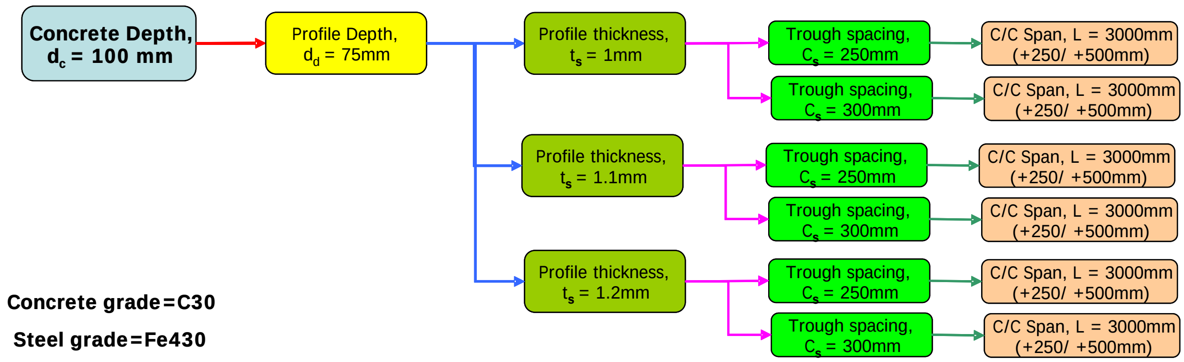
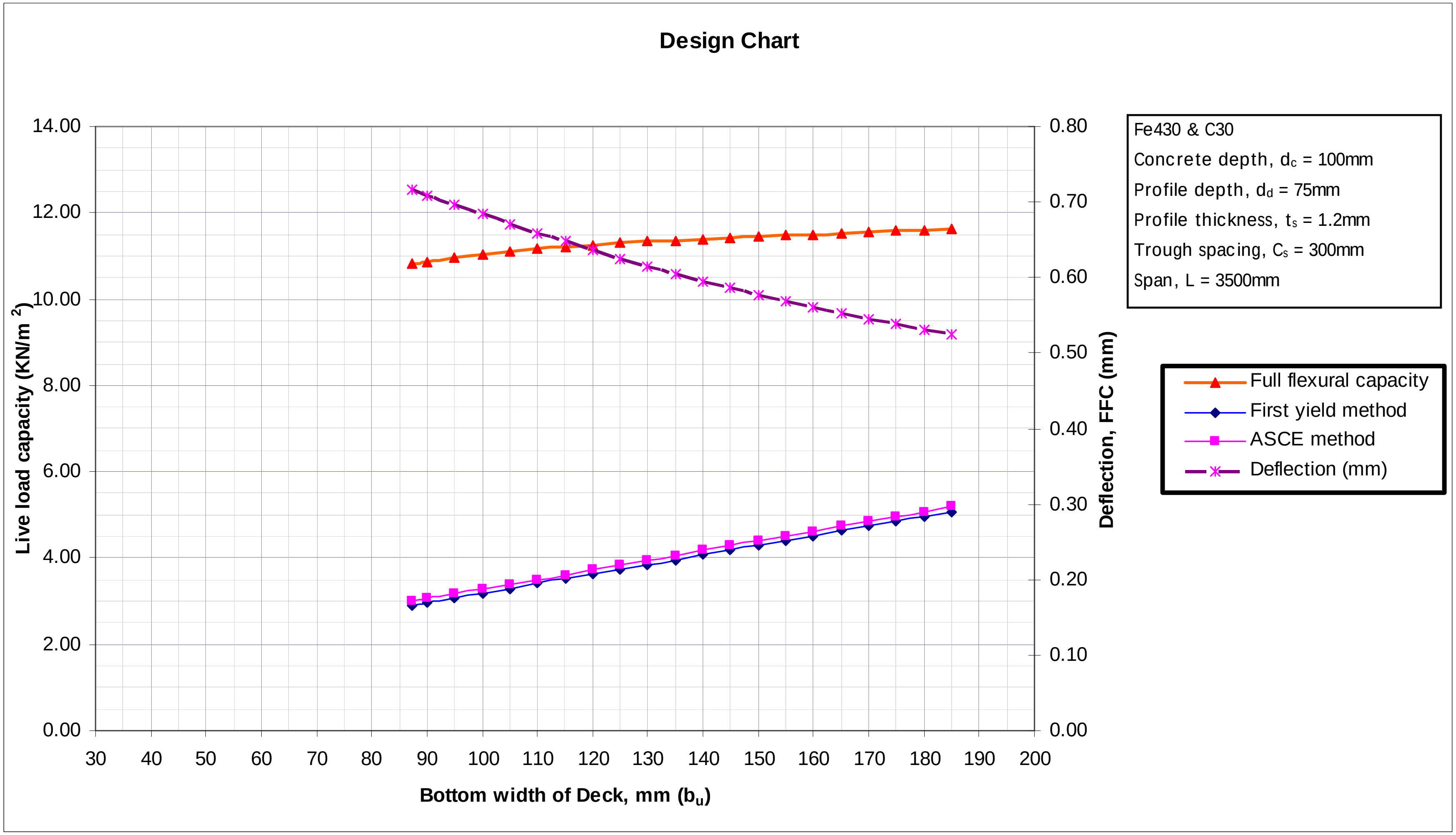


Figure 3-18: Flow chart for d_c=100mm and d_d=75mm


Design Chart
Live load capacity (KN/m 2)
Deflection, FFC (mm)
Bottom width of Deck, mm (bu)
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
Fe430 & C30
Concrete depth, dc = 100mm
Profile depth, dd = 75mm
Profile thickness, ts = 1.2mm
Trough spacing, Cs = 300mm
Span, L = 3500mm
Full flexural capacity
First yield method
ASCE method
Deflection (mm)

4 CONCLUSION AND RECOMMENDATION

4.1 CONCLUSION

A number of design tables and graphs have been prepared for this thesis work. The total number is summarized in the table below.

Table 4-1: Summary of design graphs

No	Concrete Grade	Steel Grade	Concrete depth, d_c (mm)	Profile depth, d_p (mm)	No of Design graphs
1	C25	Fe360	80	50	18
				75	25
			100	50	15
				75	25
2	C25	Fe430	80	50	20
				75	27
			100	50	20
				75	31
3	C30	Fe360	80	50	20
				75	24
			100	50	15
				75	25
4	C30	Fe430	80	50	20
				75	27
			100	50	20
				75	31

The rigid-plastic analysis method is utilized for evaluating the section capacity while using the full flexural analysis method. The linear analysis method is used while deterimining the section capacity for the first yield method and ASCE appendix D method. Hence the full flexural method gives the maximum capacity of the section assuming full interaction between the steel and concrete and as well as the section can attain its plastic moment capacity.

The first yield method and the ASCE method are basically based on the Working Stress principle, i.e, actual stress is below yielding stress.

From the charts prepared, it can be seen that there is a large difference in the load capacity for the full flexural method as opposed to the first yield method and the ASCE method. For the latter two methods, the difference in load capcity is not that large. The difference arouse due to the approach the full flexural method and the other methods use while evaluating the section capacity. The full flexural uses the rigid-plastic approach while the other two methods use the linear elastic method.

Based on available literature, it can be said that both the first yield method and the ASCE method give lower bound values. It can be seen in most of the charts that the ASCE gives relatively higher values than the first yield method, since this method takes into consideration the indentations of the profile deck. Here, the full flexural capacity should not be compared with the other two values for the reasons earlier discussed.

As can be seen from the graphs, the following conclusion can be arrived at

Span

- From the graphs, we can conclude that mostly the capacity is limited below 4m.

Load capacity

- In most of the charts, it is shown that the live load capacity is generally from 2 to 4 kN/m^2.

Generally, better result is found when

- Trough spacing = 250mm
- Steel class = Fe430
- Profile thickness = 1.2mm

In these combinations, the clear span to be covered is larger.

4.2 RECOMMENDATION

It is recommended that, once an optimal cross section based on a set of requirements is found, to conduct laboratory tests and generate design tables. This way it is possible to have cross-sectional parameters will be defined that are optimal and be able to be produced locally.

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DECLARATION

This thesis work is my original work and has not been presented for a degree in any other university, and that all source of material are duly acknowledged.

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Title of Thesis: **DESIGN CHARTS for COMPOSITE SLAB of 80mm and 100mm CONCRETE THICKNESSES**

Department: **CIVIL ENGINEERING**

Signature: ______________________________

Advisor: **Dr. Shifferaw Taye**

Signature: ______________________________