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EUROPAISCHE KONVENTION FÜR STAHLBAU

ECCS - Technical Committee 3
- Fire Safety of Steel Structures

Technical Note

Calculation of the Fire Resistance of
Centrally Loaded
Composite Steel-Concrete Columns
Exposed to the Standard Fire.

FIRST EDITION

1988

N° 55

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CONTENTS

	pages
- FOREWORD	I to II
Chapter 1 : INTRODUCTION	1.1 to 1.6
Chapter 2 : BASIC PRINCIPLES OF TEMPERATURE AND LOAD BEARING CALCULATIONS	2.1 to 2.11
Chapter 3 : FIRE RESISTANCE OF COMPOSITE COLUMNS WITH ROLLED PROFILES ENCASED IN CONCRETE	3.1 to 3.8
Chapter 4 : FIRE RESISTANCE OF COMPOSITE COLUMNS WITH ROLLED PROFILES CONCRETED BETWEEN THE FLANGES	4.1 to 4.9
Chapter 5 : FIRE RESISTANCE OF COMPOSITE COLUMNS WITH CONCRETE FILLED HOLLOW STRUCTURAL SECTIONS	5.1 to 5.8
- GENERAL NOTE ON PRACTICAL FIRE DESIGN DIAGRAMMS	III
Appendix A: FIRE DESIGN DIAGRAMMS FOR ROLLED PROFILES ENCASED IN CONCRETE	A.1 to A.9
Appendix B: FIRE DESIGN DIAGRAMMS FOR ROLLED PROFILES CONCRETED BETWEEN THE FLANGES	B.1 to B.35
Appendix C: FIRE DESIGN DIAGRAMMS FOR CONCRETE FILLED HOLLOW STRUCTURAL SECTIONS	C.1 to C.31
Appendix D: THERMO-MECHANICAL MATERIAL PROPERTIES AT ELEVATED TEMPERATURES	D.1 to D.8

FOREWORD

This Technical note is to be considered as a supplement to the European Recommendations for the Fire Safety of Steel Structures also prepared by Technical Committee 3 of the European Convention for Constructional Steelwork. The ECCS-Recommendations indeed do not cover composite steel-concrete structures.

The use of composite steel-concrete building components is becoming increasingly important today. By combining steel and concrete in one structural component, the advantages of each material can be fully taken into account. Compared with more traditional steel or concrete structural elements, the composite steel concrete system offers the following benefits:

- reduced overall dimensions,
- easy prefabrication and erection,
- good resistance to instability and
- permanent fire resistance.

All these benefits are certainly achieved for composite columns. In the first place however it should be underlined, that composite action automatically increases the fire resistance of unprotected steel columns. If necessary, the fire resistance can be increased to practically any desired level by simple and reliable means.

Until recently the structural fire engineering design of composite columns could only be based on fire resistance tests. Such a method is time consuming, rather expensive, and sometimes gives rise to anomalies due to variations in test conditions.

For these reasons it is felt within the ECCS-Technical Committee 3 that there is a strong need for spreading the available calculation methods by which the fire resistance of composite columns can be determined. This should lead on one side to more uniform levels of safety and on the other side to a more simple and systematic design procedure, stimulating the use of structural steel.

In order to facilitate the analytical fire engineering design of composite columns in practice, the present Technical Note was prepared offering, besides comprehensive background information like correlation between test and calculation results, a wide range of practical fire design diagrams for different types of composite columns exposed to standard fire.

The present membership of ECCS Technical Committee 3 "Fire Safety of Steel Structures" is as follows:

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Editing has been made mainly at the Department for Research and Promotion of Steel Structures of ARBED Research Centre, Esch-sur-Alzette, Luxembourg.

It is the hope of Technical Committee 3 that this Technical Note will be used and be of value not only to ECCS Member Countries, but also to other Nations.

London, September 1988

Mrs. M. LAW

Chairman of the ECCS-Technical Committee 3

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Chapter 1: INTRODUCTION

Four different types of composite columns should be considered in relation to structural performance in fire (Fig. 1.1):

- a) rolled H-profiles encased in concrete
- b) rolled H-profiles with concrete between the flanges
- c) concrete filled circular or rectangular hollow sections with and without reinforcing bars
- d) steel core columns embeded in concrete with outer circular or square steel sheet

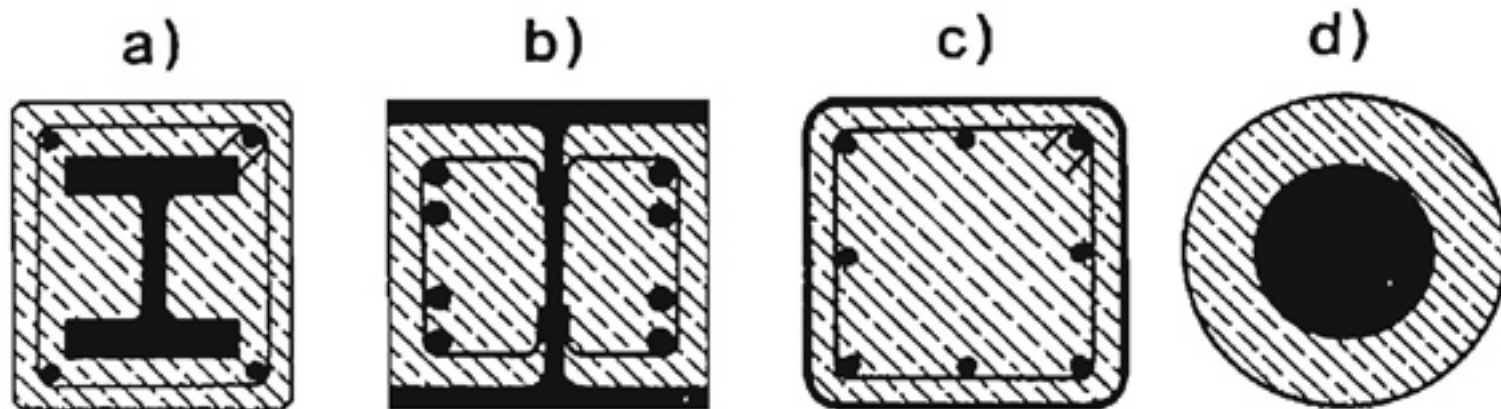


FIG. 1.1: Typical cross sections of composite columns

One of the great advantages of composite columns are their uniform outside dimensions in multi-storey buildings. By varying the thickness of the steel section, the material qualities of both steel and concrete, and the percentage of reinforcement, the cross section of the column may be adapted to support an increased load without significant changes in the outer dimensions. Each type of composite columns has specific advantages and ranges of application.

The oldest type of composite column is the steel section encased in concrete. Its advantages are a highly allowable load level in fire conditions and a high load carrying capacity not only for centrally applied loads, but also for bending moments (1.1).

The second type, namely the steel section with concrete between the flanges, can support considerable central loads and high bending moments. The amount of shuttering is significantly reduced and, if the section is concreted before erection, is even superfluous. Other advantages are a good resistance to mechanical damage without the need for corner reinforcement and the ability to use conventional steel connections between columns and steel beams even when the columns are concreted before erection (1.2).

It should be mentioned that these two types of composite sections are also used as beams. Furthermore they both have an excellent resistance to earthquakes.

The load carrying capacity of the concrete filled hollow sections is strongly influenced by fire exposure, but can also be improved by using reinforcement and high values of concrete compressive strength. The use of reinforcing bars allows one to consider bending moments or small load eccentricities under fire action (1.3).

The steel core column is a further development of the concrete filled hollow section, but with the main part of the load carrying steel cross section protected against fire by layers of concrete. However, this column type should only be used as centrally loaded member with small load eccentricity (1.4).

The latter types of composite columns have a considerable advantage compared with the reinforced concrete columns. The outside casing will prevent spalling of the concrete and the core will remain better protected against fire.

The range of use of the four types of composite columns is very wide. Some examples of columns with outer maximum dimensions of 300 mm x 300 mm are given in Fig. 1.2. Generally speaking these composite columns support centrally applied loads between 100 kN and 20000 kN for the fire class F 120. Higher load carrying capacities can be obtained by using special composite column types similar to those given in (1.5).

The calculation of the temperature distribution in the cross-section and consequently of the buckling load of **composite columns** under fire conditions is not possible with the approximation methods (1.6, 1.7), but must take into account the **unequal distribution of temperatures in the cross section**.

Chapter 2 presents the basic principles for the calculation of the temperature distribution in the cross section of composite columns and for the calculation of their load bearing capacity under fire action.

Chapters 3 to 5 give more information about the calculation of this load bearing capacity for the first three composite column types described above, the fire tests for comparing experimental results with theoretical calculations, and some examples of completed structural designs. Composite column section d, which has been developed and tested by a Steel Constructor in Switzerland, will not be explained in detail as the analysis is not available for publication.

Practical fire design diagrams for the different fire classes will be given in appendices A, B and C for the composite column types a, b and c. As the fire class F30 is automatically guaranteed for column types a, b and c, the F30 diagrams have been deleted from appendices A, B and C.

All columns are uniformly heated around their surface according to the standard ISO-834 heating curve (1.8).

Appendix D describes thermal and mechanical material properties of concrete and steel at elevated temperatures, applicable to the structural elements freely expandable as those covered by this technical note.

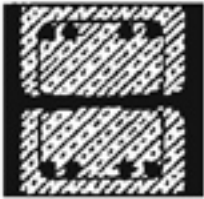
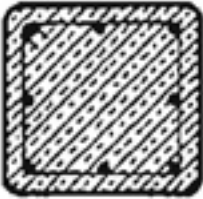
SECTION Type	DIMENSIONS (mm) Steel Profile Re-bars	FIRE RESISTANCE: F60 $L_{cr}=3000$ mm	FIRE RESISTANCE F90 $L_{cr}=3000$ mm	CONSTRUCTIONAL CONDITIONS
	300x300 HEM 200 ** 4 Ø 20	3610	2820	Steel profile: $\sigma_y = 355$ N/mm ² Reinforcement: $\sigma_y = 400$ N/mm ² ; $\mu \leq 3\%$ Stirrups..... Concrete: $\beta_{cy} = 30$ N/mm ²
	299x306 HP 310x310x79 ** 8 Ø 20	2890	2050	Steel profile: $\sigma_y = 355$ N/mm ² Reinforcement: $\sigma_y = 500$ N/mm ² ; $\mu \leq 3\%$ Stirrups..... Concrete: $\beta_{cy} = 40$ N/mm ²
	300x300 300x300x7,1 8 Ø 20	2600	1810	Hollow section: $\sigma_y = 235$ N/mm ² Reinforcement : $\sigma_y = 400$ N/mm ² ; $\mu \leq 5\%$ Stirrups..... Concrete: $\beta_{cy} = 40$ N/mm ² ***
	300x300 300x300x6 core Ø 200	4700	4250	Core steel: $\sigma_y = 195$ N/mm ² Concrete : $\beta_{cy} = 30$ N/mm ²

Fig. 1.2; EXAMPLES FOR ULTIMATE AXIAL LOAD $N_{cr,\theta}$ OF COMPOSITE COLUMNS IN KN (*)

* it should be noted that the full design loads at 20°C, $N = N_{cr}/\gamma_0$, could be smaller than the there ultimate loads $N_{cr,\theta}$ for elevated temperature.

*** for design at normal temperature only 3 % maximum should be considered (1.10)

** see (1.9)

NOTATIONSCapital Roman letters

A - area, section
 E - modulus of elasticity
 F - surface area of the composite section per unit length exposed to fire
 I - moment of inertia
 L - length of column, L_{cr} -buckling length of column
 M - bending moment
 N - normal design force, N_p -ultimate plastic load, N_E -Euler buckling load,
 N_{cr} -ultimate, critical or buckling load, N_g -loading during fire
 Q - heat flow
 R }
 S } re-bar geometrical positions
 T }
 V - volume of the composite member per unit length

Lower-case Roman letters

a - width, depth of cross section
 b - width of structural shape
 c - specific heat
 d - concrete cover
 e - eccentricity
 f - function of
 h - depth of structural shape
 q - free distance between re-bars
 t - time, t_u - failure time
 u - hook of half stirrups
 v }
 w } deflections
 x }
 y } coordinates or axis
 z }

Lower-case Greek letters

- α - convection heat transfer coefficient
 β - concrete design strength
 β_{cu} - minimum guaranteed compressive strength of concrete, measured on cubic specimens of 200 mm side dimensions, after 28 days
 β_{cy} - minimum guaranteed compressive strength of concrete, measured on cylindric specimens of 150 mm diameter and 300 mm height, after 28 days
 ($\beta_{cy} = 0,83 \cdot \beta_{cu}$)
 γ_0 - design safety factor > 1
 γ_{θ} - fire related partial safety factor = 1,0
 ε - strain, resultant emissivity
 η - percentage of design load
 θ - temperature
 κ - reduction factor for buckling
 λ - thermal conductivity, $\bar{\lambda}$ - reference slenderness ratio
 μ - percentage of reinforcement
 ρ - density
 σ - stress, σ_y - yield point, σ_{cr} - buckling stress,
 σ_p - proportional steel limit, σ^* - Stefan Boltzmann Constant
 Φ - diameter of reinforcement
 Ψ - reduction factor function of the thermal stress field
-

As - subscript or superscript

- c - concrete, convection
 cr - buckling
 f - fire, fire condition
 fl - flange
 r - reinforcing bar, radiation
 s - steel, steel profile, column surface
 u - ultimate, failure
 w - web
 $\left. \begin{matrix} y \\ z \end{matrix} \right\}$ eccentricity according either y, either z
 θ - temperature

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Chapter 2: BASIC PRINCIPLES OF TEMPERATURE AND LOAD BEARING**CALCULATIONS****2.1. Load bearing capacity at room temperature (20°C)**

Load bearing capacity of composite columns is based on the principle of the addition of load bearing capacity of its individual components, steel profile s , concrete c and reinforcement r . Plastic load bearing capacity N_p of a centrally loaded composite cross section can be calculated as

$$N_p = \sum_i N_{p,i} \quad [2.1.1]$$

$$= \sum_{i,j} (A_i \cdot \beta_i + A_j \cdot \sigma_{y,j}) \quad [2.1.2]$$

$$= N_{p,s} + N_{p,c} + N_{p,r}. \quad [2.1.3]$$

Factors in equation [2.1.2] represent cross sectional areas of the separate components and their ultimate strengths. This accumulation principle explains the high values of stiffness and ultimate plastic load of the composite cross sections. The plastic cross section capacity N_p leads to the column's buckling load N_{cr} by considering the influence of the reference slenderness ratio $\bar{\lambda}$.

The reference slenderness ratio defined by

$$\bar{\lambda} = \sqrt{N_p / N_E} \quad [2.2]$$

depends on the plastic capacity N_p of the cross section according to equation [2.1.1] and on Euler's buckling load N_E of the composite column. N_E is a function of the total cross section stiffness EI and of the column buckling length L_{cr}

$$N_E = (\pi^2 / L_{cr}^2) (E_s I_s + E_c I_c + E_r I_r). \quad [2.3]$$

The ultimate or buckling load N_{cr} of centrally loaded columns follows by

$$N_{cr} = \kappa \cdot N_p \quad [2.4]$$

where the reduction factor κ depends on $\bar{\lambda}$ and can be determined by using the European buckling curves (fig. 2.1). As a last step, the design load N follows by using the design safety factor γ_0 for composite columns

$$N = N_{cr} / \gamma_0. \quad [2.5]$$

This is the general basis for all design codes and results differ only slightly by differences in national regulations for material parameters and safety factors (2.1, 2.2). It can be seen that this approach is in complete agreement with EUROCODE N° 3 and EUROCODE N° 4 (2.3, 1.10).

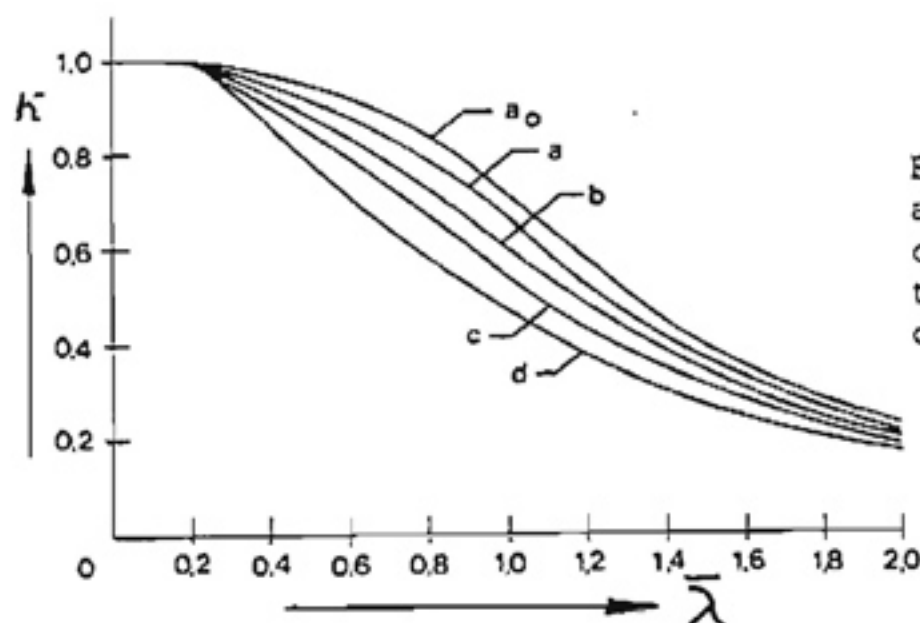


Fig. 2.1; EECS buckling curves at room temperature (2.3). For composite columns dealt with in this technical note the buckling curve c is proposed.

2.2. General load bearing behaviour under fire action

Two basic types of composite cross section for columns can be discerned (fig. 1.1)

- steel as an external casing and
- steel embedded in concrete.

Columns of such different types of cross section having the same load bearing capacity at normal temperature, will behave differently in fire.

Temperature gradients over the cross section of a composite structural member lead to different heating rates of its separate components. In connection with temperature dependent material properties, fire resistance will depend on cross section design. Differences in cross section type, involve differences in load bearing behaviour under fire action, independent of the load bearing capacity at normal temperature. In fig. 2.2 the change in the ultimate plastic load of the components is plotted against standard fire duration t (2.4). External steel directly exposed to a fire will lose its load bearing capacity rapidly, because of its loss of strength as a consequence of the high heating rate. That part of the plastic load bearing capacity, which comes from an external steel shell, must then be carried by the concrete core. Contrary to this characteristic, for cross sections with centrally embedded steel profiles the load will be transferred from the concrete to the profile. In this case, the average concrete strength over the cross section will decrease more rapidly than the strength of the embedded steel profile.

Cross sections of type a and d of figure 1.1 belong to the second basic type, whereas concrete filled hollow sections (type c) must be attributed to the first basic type. Rolled profiles with concrete between the flanges (type b) form a mixed type with directly exposed steel flanges and an insulated steel web.

From figure 2.2 and equation [2.1] it can be easily understood that different cross sections will each show a different loss of load bearing capacity under fire action. For a fire resistant design the main load bearing component should be the component exhibiting the least loss in strength. For concrete filled hollow sections, high strength concrete and a significant amount of

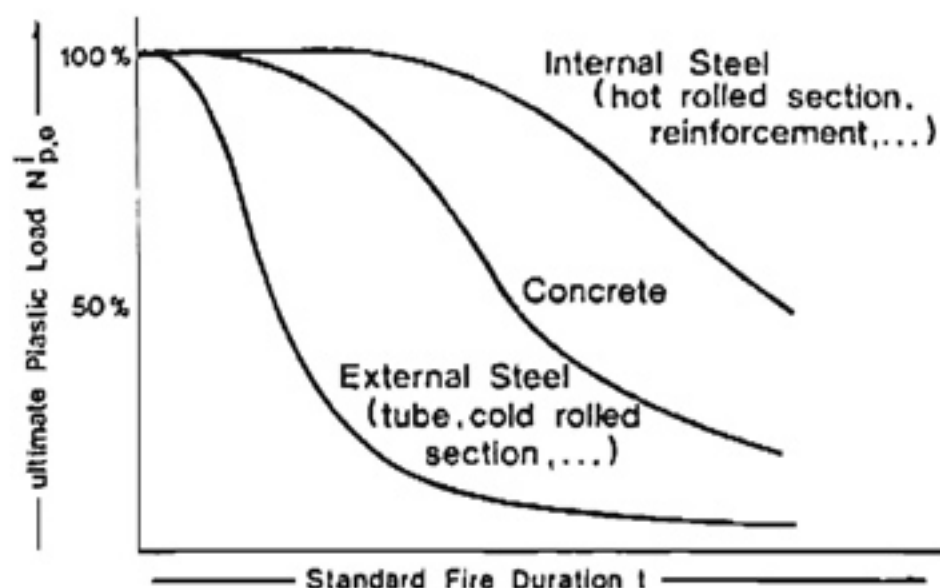


Fig. 2.2; Ultimate plastic load of the components of the cross-section as a function of fire duration time t .

internal reinforcement are both advantages, whereas the steel shell should be minimized in terms of thickness and strength. For columns of rolled section encased in concrete, the hot-rolled profile will be the main load bearing component and should be maximized in terms of steel cross section and strength.

Taking into account this interaction between the components of the cross section under fire action, the fire resistance of composite columns can be optimized by cross section design. For cross sections where the load bearing capacity decreases early with rising temperature (external unprotected steel), a reduction of the design load could be requested in order to reach higher fire resistant times.

2.3. The calculation of the temperature distribution

As the heat capacity of a composite column is usually important, the general two-dimensional differential equation of heat must be solved in order to get the accurate transient temperature field

$$c(\theta) \cdot \rho \cdot \frac{\partial \theta}{\partial t} = \frac{\partial}{\partial x} \left(\lambda(\theta) \cdot \frac{\partial \theta}{\partial x} \right) + \frac{\partial}{\partial y} \left(\lambda(\theta) \cdot \frac{\partial \theta}{\partial y} \right) \quad [2.6]$$

where

$\lambda(\theta)$	: Thermal conductivity	W/mK
$c(\theta)$	: Specific heat	J/kgK
ρ	: Density	kg/m ³
θ	: Temperature in a point of the composite section (K) the furnace temperature being a function of time t (in minutes) according for example ISO-standard 834, $\theta_F = 20 + 345 \log_{10} (8t + 1)$.	

The thermal properties of concrete and steel as a function of temperature could be chosen according to appendix D. The numerical methods using finite difference or finite elements are the most common and unfailing ways to solve this differential heat conduction problem. Typical networks with temperature distributions obtained by using numerical methods are presented in figures 2.3 and 2.4 for the same cross section.

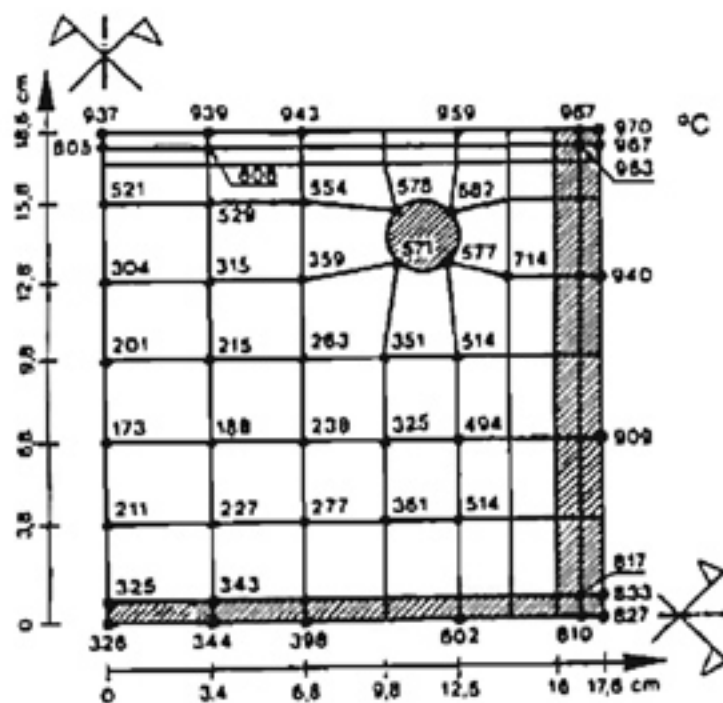


Fig. 2.3; Typical temperature distribution for one quarter of steel profile with concrete between the flanges. Steel sections (profile and reinforcing bar) have been hatched, whereas the numbers given represent the temperatures (°C) computed by a finite element method in nodes after 90 minutes of ISO-standard fire exposure (2.9).

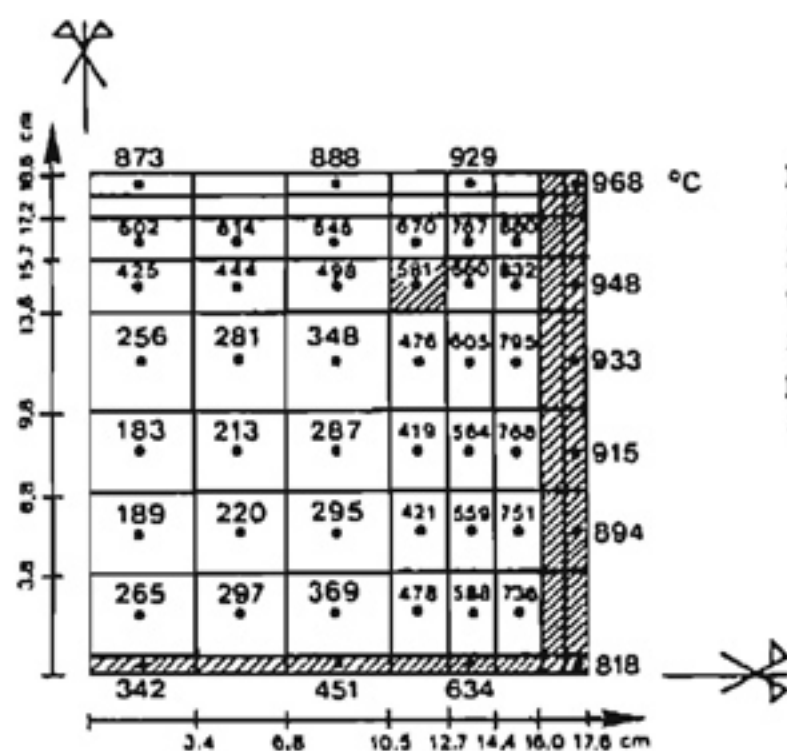
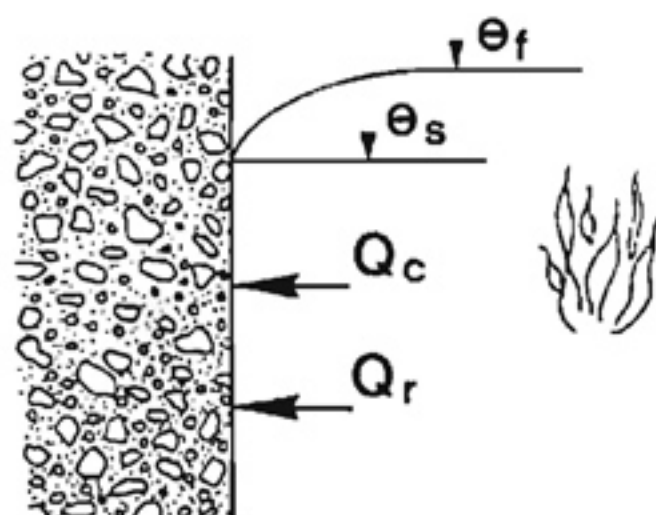


Fig. 2.4; Typical temperature distribution for the same composite section as in fig. 2.3. The numbers presented indicate the temperatures computed by a finite difference method, in the centre of network patches, after 90 minutes of ISO-standard fire exposure (2.14 to 2.18).

It is essential to know, that the heat Q is transferred (fig. 2.5) from the hot gases of the furnace or fire to the surfaces of a column by the two mechanisms of convection Q_c and radiation Q_r

$$Q = Q_c + Q_r \quad (\text{W/m}^2). \quad [2.7]$$



CONCRETE OR STEEL

Fig. 2.5: Heat transfer at the surface of exposed material

θ_f - furnace temperature

θ_s - column surface temperature

Q_c - convection heat flow

Q_r - radiation heat flow

Together with the furnace or fire temperature θ_f , the column surface temperature θ_s , the coefficient α of convection heat transfer and the resultant emissivity ε , determine the heating up of the column

$$Q = \alpha (\theta_f - \theta_s) + \sigma^* \cdot \varepsilon (\theta_f^4 - \theta_s^4) \quad [2.8]$$

where σ^* = Stefan Boltzmann Constant = $5,67 \cdot 10^{-8} \text{ W/m}^2 \text{ K}^4$,

and ε = resultant emissivity of the flames, combustion gases and exposed surfaces concrete or steel.

For practical use, values of $\alpha = 25 \text{ W/m}^2 \text{ K}$ and $\varepsilon = 0,5$ will be chosen (1.6). Computer programs, allowing the analysis of two or three dimensional transient temperature fields, are available worldwide (2.5, 2.6, 2.7, 2.8, 2.9).

2.4. Load bearing capacity of composite columns under the action of fire

2.4.1. Exact method

As seen before, the transient temperature field, created under the action of fire in a composite cross section, is highly differential (fig. 2.6.). Thus the simplified proceedings (1.6) used with sufficient precision in order to analyse protected or unprotected steel elements are no longer applicable.

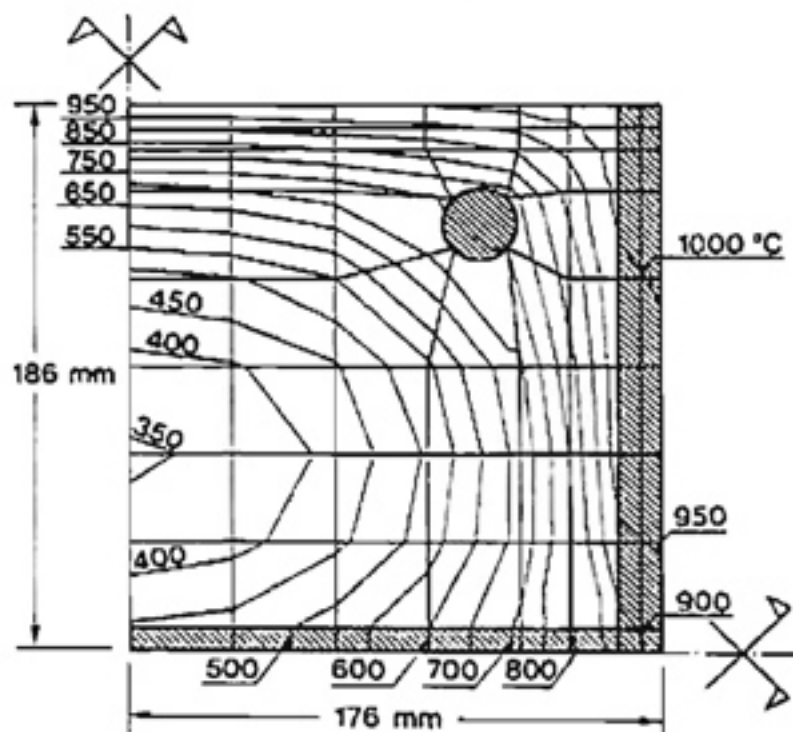


Fig. 2.6; Temperature field given by isothermal curves in a composite cross section with concrete between the steel flanges, after 120 minutes of ISO fire (2.9).

Indeed, the unequal temperature field creates a rather inhomogeneous distribution of material properties within the cross section due to the temperature depending highly non linear material properties. In addition the unequal temperature field leads to strong internal stresses by the assumption of a plane cross section (fig. 2.7), depending upon the type of composite column. These internal stresses, together with second order effects, such as thermal restraint etc., undoubtedly have an influence on the load bearing capacity of composite constructional elements.

Indeed the outer network patches of the composite cross section might be heavily deformed before the core patches, which are less thermally damaged, become fully active. For these reasons the calculation of the buckling load $N_{cr,\theta}$ of composite columns under the action of fire is only possible (without any restrictions as to the geometry of the cross sections, the building structural system, the load combination etc) if a numerical model is used allowing a precise thermal and mechanical system analysis (2.10 to 2.18).

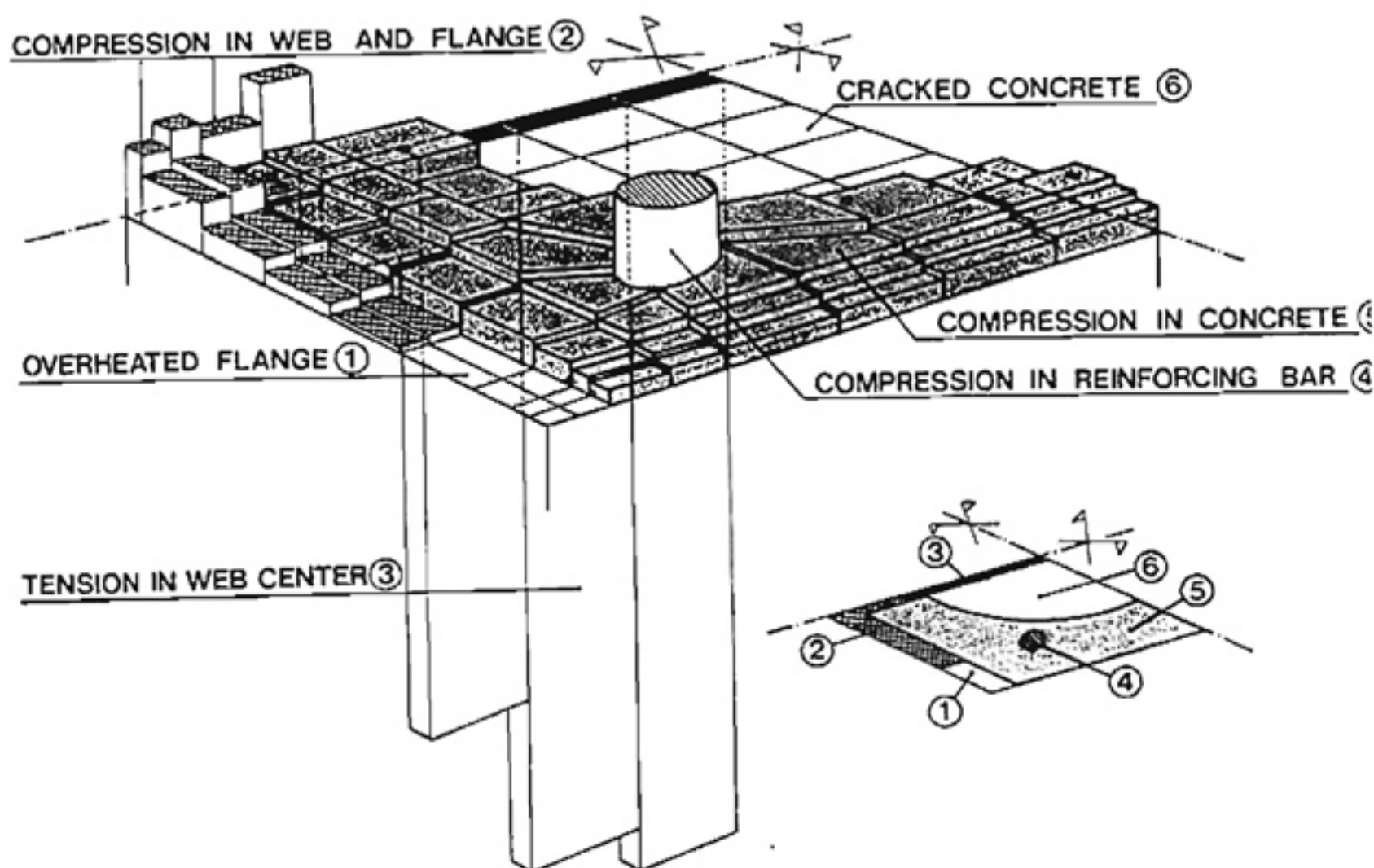


Fig. 2.7; Internal stress situation due to the unequal temperature field given in figure 2.6 for a composite cross section with concrete between the steel flanges, after 120 minutes of ISO fire (2.9).

2.4.2. Limited calculation models

The numerical complexity increases rapidly with the greater precision of exact thermal and mechanical system analysis. Therefore limited calculation models have been developed. These models allowing a readily usable method of calculating the buckling load of composite columns, for a given application field and a given cross section, are based generally on

- a transient thermal field analysis,
- the calculation of the ultimate plastic load $N_{p,\theta}$,
- and the calculation of the buckling load $N_{cr,\theta}$.

The transient thermal field analysis finally assigns, for a given fire class, a mean temperature to any mesh. This temperature gives the respective material parameters of the cross section for the chosen fire class. The selection of identical discretizations for both the thermal and the subsequent load bearing analysis is beneficial.

The calculation models currently available for evaluating the ultimate plastic load $N_{p,\theta}$ of composite cross sections under fire action may be divided into two groups

- the summation method

$$N_{p,\theta} = \sum_{i,j} (A_i \cdot \beta_{i,\theta} + A_j \cdot \sigma_{yj,\theta}) \quad [2.9]$$

- the balanced summation method

$$N_{p,\theta} = \sum_{i,j} (\psi_{i,\theta} \cdot A_i \cdot \beta_{i,\theta} + \psi_{j,\theta} \cdot A_j \cdot \sigma_{yj,\theta}) \quad [2.10]$$

with $0 \leq \psi_{i,\theta} \leq 1$.

The summation method adds the plastic loads of all meshes i,j of the discretized section without considering their interaction. This simple method overestimates the bearing capacity of the section, as the effects of the thermal stress fields are not covered.

The balanced summation method approximately takes care of these very important physical facts. Border elements are either not considered, or their contribution in the total plastic load is strongly reduced. The reduction factors $\psi_{i,\theta}$ of equation [2.10] must of course be available in a parametric form, obtained either by calibration on real fire resistance tests, or by calculation.

The calculation of the buckling load $N_{cr,\theta}$ for a given class, consists in using the ultimate plastic load previously calculated and following the guidelines for ambient temperature (1.10, 2.1 to 2.3)

$$N_{cr,\theta} = \kappa \cdot N_{p,\theta} \quad [2.11]$$

It should be mentioned, that the European buckling curves were developed for the normal temperature range. Therefore, their use at elevated temperatures must be regarded as an approximation.

Among those simplified calculation models, the summation method should only be used for short columns which are more or less centrally loaded. The balanced summation method may be applied to columns with moderate slenderness and small eccentricity of load, if based on real fire test calibration.

For the analysis according to these simplified calculation models, the European buckling curve c is proposed (see fig. 2.1), which is of course a conservative approach, also considered in (1.6).

2.4.3. Methods used in the different chapters

In the following main chapters and appendices dealing with the composite column types a, b and c of figure 1.1, the following calculation methods have been used:

- a precise thermomechanical analysis according to chapter 2.4.1 forms the basis of chapter 3 and appendix A,
- a balanced summation method according to chapter 2.4.2 has been used in chapter 4 and appendix B,
- the summation method given in chapter 2.4.2 has been used in chapter 5 and appendix C.

For all these models, the maximum admissible load under fire action is directly given by the ultimate load according to

$$N_{\theta, \max} = N_{cr, \theta} / \gamma_{\theta} = N_{cr, \theta} \quad [2.12]$$

with a fire related partial safety factor given by

$$\gamma_{\theta} = 1,0 \quad [2.13]$$

Furthermore when calculating the ultimate load under fire action $N_{cr, \theta}$, no special material related partial safety factor is applied in order to reduce the various material design strengths. Therefore, the concrete design strength β as well as the yield point σ_y of the steel components, are considered in exactly the same way as when comparing the tested and computed failure times of the composite columns of chapters 3, 4 and 5 (see figures 3.3, 4.5 and 5.4).

2.4.4. Practical design

Normally fire resistance design consists of two alternatively used tasks

- design of a construction element or
- checking an existing construction element for a requested fire class.

In case of DESIGN the diagrams give at once an adequate composite cross section including the geometry and the quality of all steel and concrete components, for the imposed fire class, the axial design load N to be supported and the buckling length $L_{cr, \theta}$.

In case of CHECKING two approaches can be chosen

- either the existing composite cross section and design load give the maximum buckling length $L_{cr, \theta}$ to be compared to the existing column end conditions,
- or the existing composite cross section and the real buckling length $L_{cr, \theta}$ give the critical load $N_{cr, \theta}$ to be compared to the actual design load N .

As in fire resistance design the safety factor is 1,0, the guaranteed resistance time is equal to the failure time t_u . As a consequence of this, the ultimate load $N_{cr, \theta}$ could be greater than the design load N , for which reason a parallel analysis for the room temperature situation must always be done (1.10, 2.2)

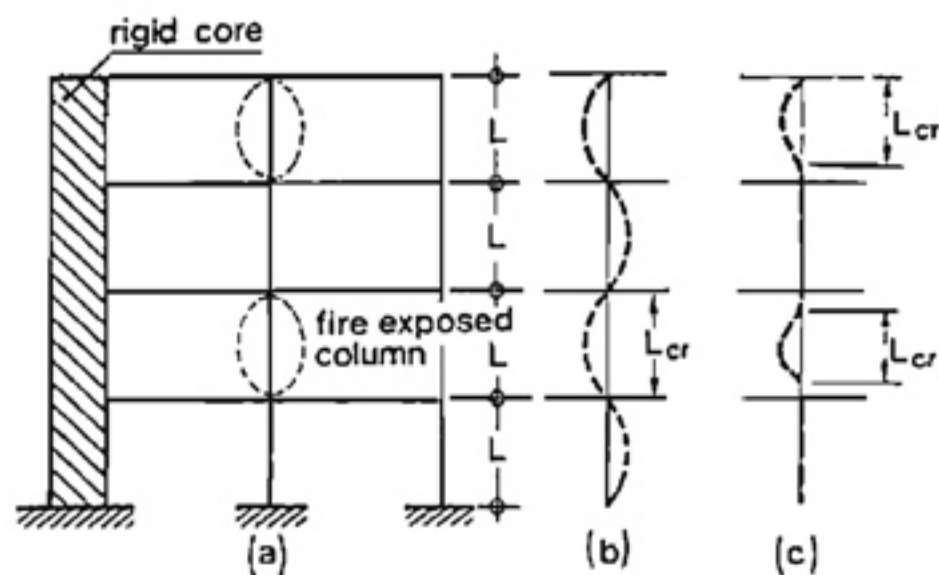


Fig. 2.8; Structural behaviour of columns in braced frames (1.6)

- a) section through the building
- b) deformation mode at room temperature
- c) deformation mode at elevated temperature

The curves of the appendices A to C can be only used for ISO FIRE RESISTANCE ANALYSIS! The column buckling loads $N_{cr,\theta}$ have been computed for the different standard fire classes as a function of the column buckling length $L_{cr,\theta}$. This buckling length $L_{cr,\theta}$ should be equal to the value used in the design at room temperature L_{cr} , unless it can be shown that the conditions are more favourable under fire exposure (2.16).

However, it should be underlined that, initially, the fire will occupy one room and in general, is likely to be limited to one storey. Consequently the structural members adjacent to the column under consideration will remain relatively cold. If the column is rigidly connected to the adjacent members, calculations show that the assumption of built-in end conditions can be assumed (fig. 2.8 c). This cannot be assumed at room temperature for which the buckling length L_{cr} is normally taken equal to the column length L from floor to floor, thus covering the most unfavourable sinusoidal instability mode over several levels (fig 2.8 b). Under fire conditions a smaller buckling length may be taken into account. If the column heated on one level continues rigidly to the upper and lower levels, the minimum buckling length should be $L_{cr,\theta} = 0,5 L$. This value covers the majority of practical situations in multi-storey buildings.

For a column on the top floor, this condition will not generally be met as far as the connection of the column with the roof girders is concerned. At the top, the column itself and all the adjacent members are normally affected by the fire in a similar way. Consequently, at the top of the column, a hinge must be assumed. If this column continues rigidly to the lower level, the minimum buckling length should be $L_{cr,\theta} = 0,7 L$.

The practical design diagrams given in the appendixes A, B and C have been established in order to enable architects and engineers a quick, safe and economic calculation of the buckling loads for composite columns. Thus in practice three dimensional, non linear, transient finite element programs have not to be used. They remain research tools which are needed to develop the knowledge on composite construction elements and to assure a firm basis for practical design methods (2.19).

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Chapter 3: FIRE RESISTANCE OF COMPOSITE COLUMNS WITH ROLLED PROFILES ENCASED IN CONCRETE

3.1. Description

Columns of hot rolled sections totally encased in concrete have been used in practical engineering for a long time. Recent developments on this type of composite column, allow to take advantage of the load bearing capacity of the concrete in a more scientific way. Thus, besides the well-known fire resistance, a considerable improvement in load bearing capacity is achieved. Columns of this type attain a much better load bearing capacity than columns of reinforced concrete or pure steel (1.1).

The cross section of this composite column consists of an H-shaped rolled profile arranged centrally within a reinforced concrete section. According to the design regulations for a reinforced cross section, the latter is provided with longitudinal reinforcement at least in the section corners and with stirrups (fig. 3.1).

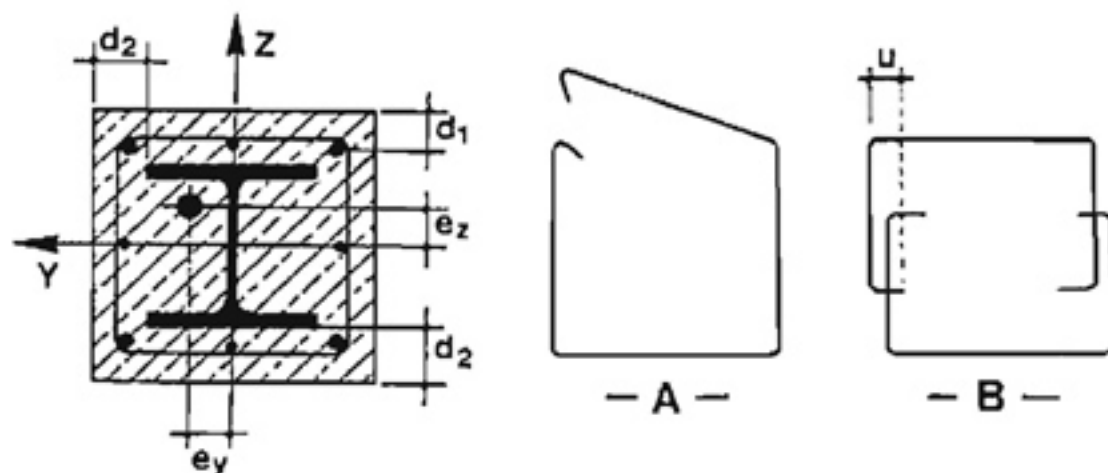


Fig. 3.1; Composite column section with H-profile encased in concrete, including rebars and stirrups.

The construction of closed stirrups of form A can be replaced by two half-stirrups of form B, provided that the hook "u" is long enough to tie the two half-stirrups. The reinforcement can either be concentrated in the corners, distributed proportionally along the section sides or be symmetrically arranged on the tension side and the compression side in case of additional bending moments.

3.2. Method of calculation and experimental basis

Within a European research project, extensive experimental and theoretical work has been done in order to ascertain the fire behaviour of these columns (3.1).

The different heating behaviours of the individual component parts, i.e. rebars, concrete and hot rolled section are shown in fig. 3.2. As a consequence of the more rapid heating of the reinforcing bars in comparison to the steel section, the increase of the percentage of reinforcement, used to improve considerably the load bearing capacity at normal temperature, gives way, in case of fire, to an earlier failure of columns. Indeed, the total load of the failing outer component must be transferred to the efficient inner one. When the influence of the steel section prevails, the load bearing capacity of the column is improved under the action of fire due to the much slower heating of this component.

The knowledge by experimental investigations of the dependence on parameters has been generalized by numerical analysis (2.10, 2.11, 3.2, 3.3). More detailed test conditions and test results are given in figures 3.3 and 3.4 (3.1). The comparison between failure times ascertained numerically and by

experiment, given in figure 3.3, shows a divergence of less than 10 %, which may partially be a result of geometrical imperfections.

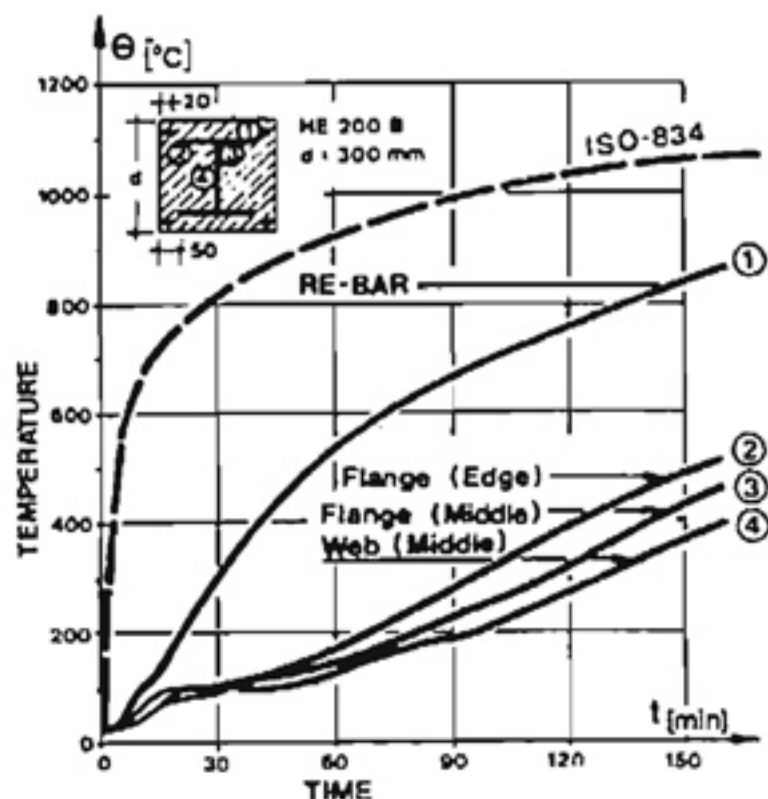


Fig. 3.2; The different heating behaviours of individual component parts.

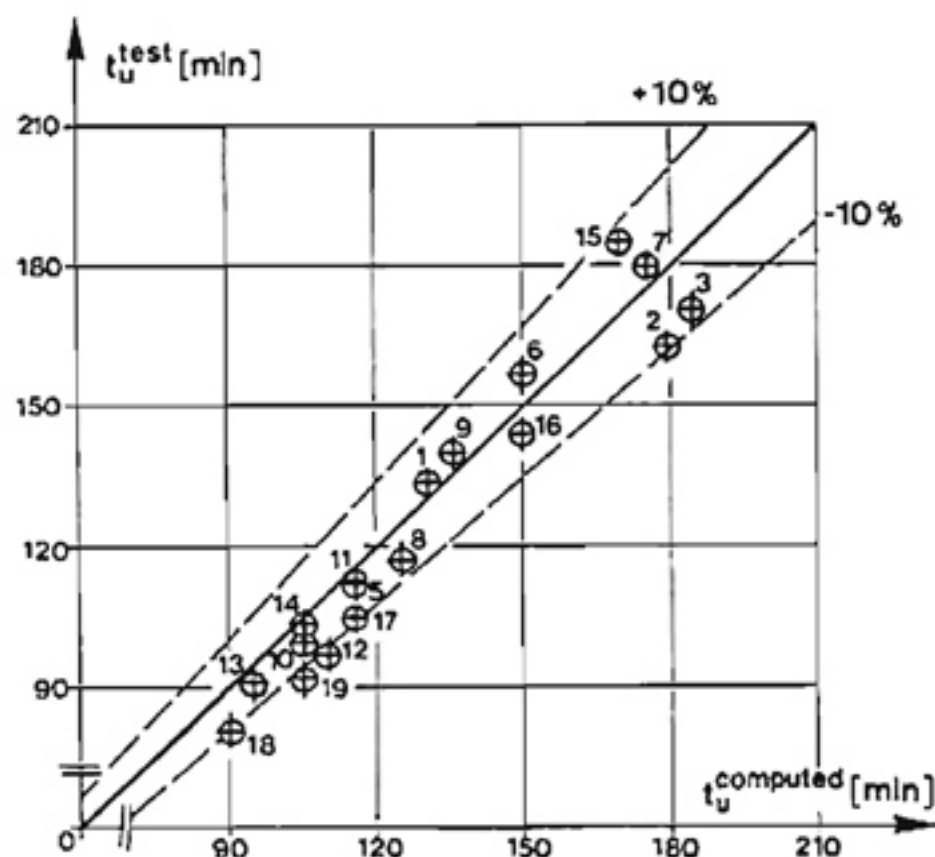


Fig. 3.3; Failure times of experiments (t_u^{test}) compared to numerically (exact calculation method) predicted failure times t_u^{computed} , for 18 tested composite columns with encased rolled profiles (3.1, 3.5, 3.9).

Concrete should not be the dominant component, because the outer concrete part heats up first and therefore loses strength earlier than the rolled profile. An increase in the profile cover d_2 intensifies the effect of insulation and thus improves failure time under fire action. This increase in profile cover means however, an increase in the amount of the concrete as the load bearing component and, at the same time, a decrease in the proportion of the rolled profile in relation to the load bearing capacity at service conditions. Since concrete has a higher heating rate than the steel profile, an increasing portion of load has to be redistributed from concrete to profile. This procedure cannot be recommended to obtain optimum fire resistant qualities of cross sections of this type of composite column. With equal loading

TEST NUMBER	SECTION TYPE	COVER		REINFOR- CEMENT	COLUMN		TEST LOADING NORMAL FORCE	TEST LOADING ECCEN- TRICITY 1)	TEST DATE	FAILURE TIME	
		d ₁	d ₂		length L mm	support				min.	min.
1	HE 200 B	20	50	4 Ø 12	4200	E3	1157	50	1979	134	130
2	HE 200 B	20	75	4 Ø 12	4200	E3	1477	58,3		163	180
3	HE 200 B	20	100	4 Ø 12	4200	E3	1887	66,7		171	185
5	HE 200 B	20	50	4 Ø 12	4200	E3	2303	5		112	115
6	HE 200 B	20	50	4 Ø 12	4200	E3	646	150		157	150
7	HE 180 M	20	50	4 Ø 12	4200	E3	1263	50		180	175
8	HE 120 M	20	50	4 Ø 12	4200	E3	642	40 } 2)		117	125
9	HE 120 M	20	50	4 Ø 12	4200	E3	963	40 }		140	135
10	HE 200 B	20	40	4 Ø 12	4200	E3	2036	5		100	105
11	HE 200 B	20	50	4 Ø 20	4200	E3	2313	5		113	115
12	HE 200 B	20	50	4 Ø 28	4200	E3	2625	5		97	110
13	HE 200 B	20	50	4 Ø 12	5700	E3	1691	5		91	95
14	HE 200 B	20	50	4 Ø 12	4200	E3	2540	5		1981	103
15	HE 200 B	20	50	4 Ø 12	4200	E4	2035	0	185		170
16	HE 280 M	20	40	4 Ø 12	5700	E3	3690	5	144		150
17	HE 100 B	20	50	4 Ø 12	4200	E3	340	5	105		115
18	HE 100 M	20	40	4 Ø 12	4200	E3	632	5	91		90
19	HE 200 B	20	40	4 Ø 12	4200	E3	1780	5	92		105

1) eccentricity e_z around strong axis (y-y bending)

2) eccentricity e_y around weak axis (z-z bending)

3)

E3

E4

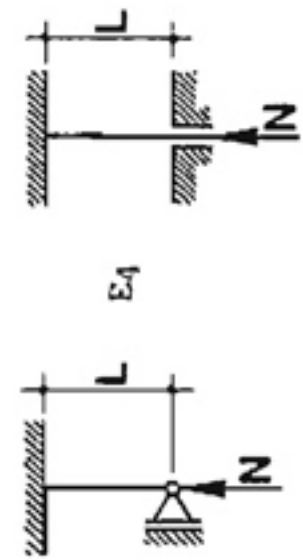


Fig. 3.4; Composite columns with rolled sections encased in concrete, tested in Braunschweig (F.R.G.), (3.1).

and equal failure time, the outer column dimensions can be considerably reduced, if the quantity of reinforced concrete is reduced and the profile becomes the predominant component.

Composite columns with hot rolled steel sections totally embedded in concrete, which have been optimized in this way, not only attain a considerably higher load bearing capacity than columns of reinforced concrete with comparable dimensions (1.1, 3.4) but additionally have higher fire resistance times. In figure 3.5 it is proved that, in comparison to reinforced concrete columns, the required fire resistance can be obtained with much smaller cross section dimensions.

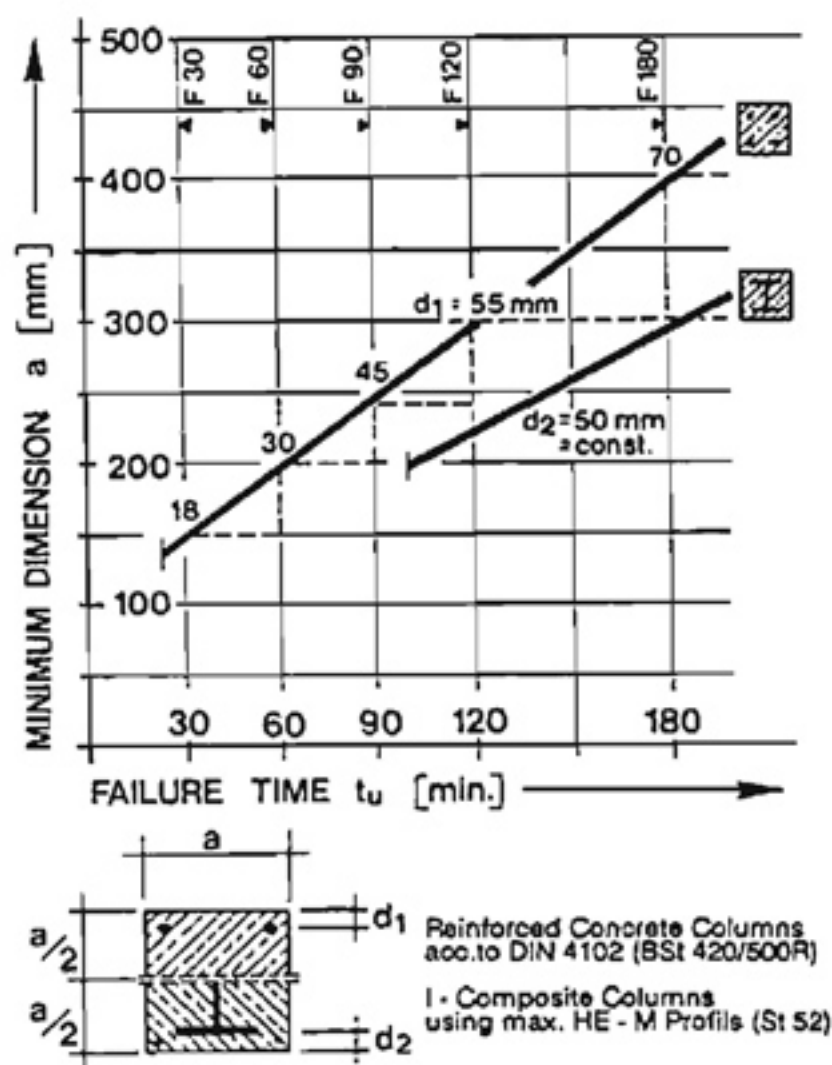


Fig. 3.5; Minimum dimension "a" of concrete columns and composite columns with encased rolled profiles, as a function of the required fire class F30 to F180 (DIN 4102, 4; (3.1)) and $\eta=100\%$. For composite columns with encased profiles the given curve is based on tests (3.1), with a column buckling length of 2940 mm.

Increase in column length involves a decrease of failure time due to slenderness. Figure 3.6 shows however, that even in the case of full design load ($\eta = 100\%$), with column slenderness typical of normal multi-storey buildings (both ends hinged), where columns can be supposed to have at least one fixed end under fire action ($L_{cr,\theta} = 0,7 L$), a minimum of 90 minutes failure time will still be attained. Furthermore, figure 3.6 proves the validity of this statement for instance by using HEM-profiles. Indeed in spite of smaller outer dimensions, a failure time of 120 minutes is obtained in case of these cross sections with high fire resistant qualities. The heavier HD profiles will of course be even more efficient (3.5, 1.9).

Systematic investigations have shown that load eccentricities have no negative effects on column fire behaviour, if the design load is also calculated in function of the bending moments caused by this eccentricity. This favourable behaviour can be assumed up to an eccentricity of $e \leq 0,5a$, i.e. normal force acting at the cross section edge. In case of a normal force applied outside the cross section, the eccentricity $e > 0,5a$ should act according to the main axis of inertia of the hot rolled steel profile.

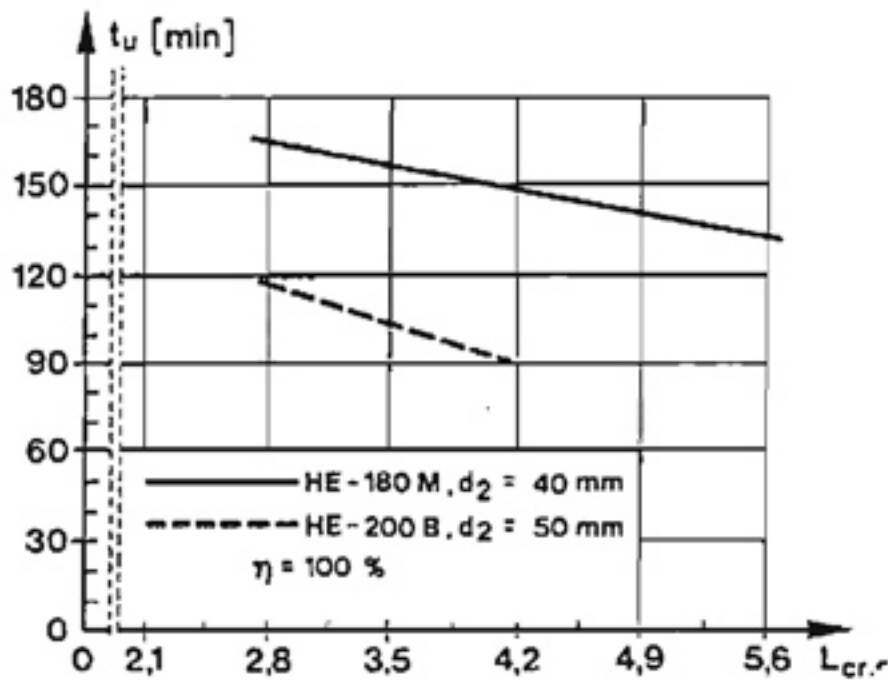
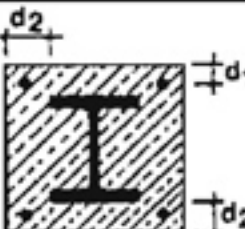


Fig. 3.6; Failure times of composite columns with encased profiles, as a function of the buckling length $L_{cr,\theta}$ (3.5).

3.3. Practical design

The following recommendations (3.5, 3.6) guarantee, besides optimum behaviour in case of fire, advantages under service conditions and for construction (like prevention of spalling etc).

t_u	$L_{cr,\theta}$	Profile				remarks
		HE	d_2	HD	d_2	
min	mm					
30	≤ 4200	≥ 100 A	≥ 50	$\geq 210 \cdot 210 \cdot 46$		
		≥ 100 B	≥ 40	$\geq 260 \cdot 260 \cdot 73$		
	≤ 5600	≥ 100 B	≥ 40	$\geq 310 \cdot 310 \cdot 97$		
60	≤ 4200	≥ 100 A	≥ 50	$\geq 360 \cdot 360 \cdot 134$		
		≥ 100 B	≥ 40	$\geq 400 \cdot 400 \cdot 187$		
	≤ 5600	≥ 100 B	≥ 40			
90	≤ 3150	≥ 100 B	≥ 50	$\geq 210 \cdot 210 \cdot 87$		≥ 40
		≥ 200 B	≥ 40	$\geq 260 \cdot 260 \cdot 101$		
		≥ 100 M	≥ 40	$\geq 310 \cdot 310 \cdot 130$		
	≤ 4200	≥ 200 B	≥ 50	$\geq 360 \cdot 360 \cdot 148$		
		≥ 120 M	≥ 50	$\geq 400 \cdot 400 \cdot 187$		
	≤ 5600	≥ 180 M	≥ 40	$\geq 210 \cdot 210 \cdot 100$		
120	≤ 2800	≥ 120 M	≥ 50	$\geq 260 \cdot 260 \cdot 149$		
	≤ 5600	≥ 180 M	≥ 40	$\geq 310 \cdot 310 \cdot 158$		
				$\geq 360 \cdot 360 \cdot 196$		
				$\geq 400 \cdot 400 \cdot 187$		

cover d_1 :
according to
reinforced
concrete columns

Reinforcement :
 $\mu \leq 3 \%$
for $t_u \leq 60$

for $t_u > 60$
 μ neglected

Fig.3.7; Minimum steel section sizes and concrete cover for a given fire resistance class without design load reduction, as a function of the buckling length $L_{cr,\theta}$. If d_1 does not fulfill the conditions of concrete specifications, (f.i. d_1 too small) reinforcing bars are not considered for the calculation of the load capacity. This table is based on numerical calculations and test results (2.1, 2.2, 3.1, 3.5).

For steel sections of figure 3.7 and within the geometric limitations indicated there, full design load can be applied to reach the indicated fire resistance times. For smaller profiles load reduction is necessary, because of accelerated heating. Values of ultimate loads for this situation are given in the diagrams of appendix A.

In the case of thicker profile cover d_2 than indicated in figure 3.7, fire resistance will increase remarkably. For profile cover $d_2 > 80$ mm and up to a buckling length in the fire case of $L_{cr,\theta} \leq 4,00$ m, the following design criteria can be used :

- profile types HE-AA and HE-A correspond to the fire class F 120,
- and profile types HE-B and HE-M to the fire class F 180 without any design load reduction.

For stickability of such thick concrete cover stirrup reinforcement near the column surface is necessary ($d_1 \leq 40$ mm).

Design of cross section:

- the minimum cover d_2 of the hot rolled steel section has to be 40 mm,
- in order to obtain optimum fire resistant qualities with minimized cross section dimensions, heavy hot rolled steel sections are of advantage i.e. HEM, HD profiles and the like,
- this design leads to an economic solution, when high strength steel is used for rolled shapes,
- reinforcement ($A_r \times \sigma_{y,r}$) should be minimized,
- concrete of higher quality is not advantageous (if possible, limit β_{cy} to 30 N/mm²),
- if composite columns of this type are built in normal multi-storey buildings with a load design for both ends hinged under service conditions, they will have under fire load effective supporting conditions for $L_{cr,\theta} = 0,7 L$. Without reduction of design load ($\eta = 100$ %) these columns will reach a minimum fire resistance of 90 minutes; the fire resistance of 120 minutes is given when the rolled profile is clearly the predominant load bearing component (see fig. 3.7).

3.4. Constructional details (3.5, 3.7, 3.8)

- * The hot rolled steel section has to be embedded evenly into the concrete; vertical concreting is advantageous. The concrete has to be compacted by vibration. The maximum grain size should be limited to 16 mm.
- * Spacers should be arranged in a sufficient number and size in order to guarantee the central position of the hot rolled steel profile within the cross section, during concreting.
- * The cross section has to be strengthened by stirrups. If high bending moments are introduced in the beam-column connection, the number of stirrups has to be increased.
- * Distribution of load between the hot rolled steel section and reinforced concrete component, has to be ensured for instance by using top and base plates. Load introduction, between these plates into the steel section of the column, is possible by means of a welded butt strap, further transfer of the load to the reinforced concrete can be obtained through shear studs welded to the web of the profile or the like.
- * Composite columns with encased rolled profiles are suitable for prefabricated single storey as well as multi-storey elements.

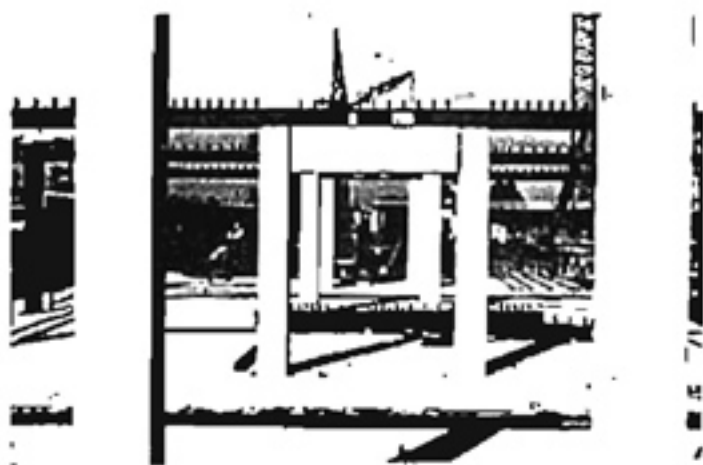


Fig. 3.8: Columns with rolled profiles encased in concrete - LUNEBURG, Administrative building (3.7), W. GERMANY.

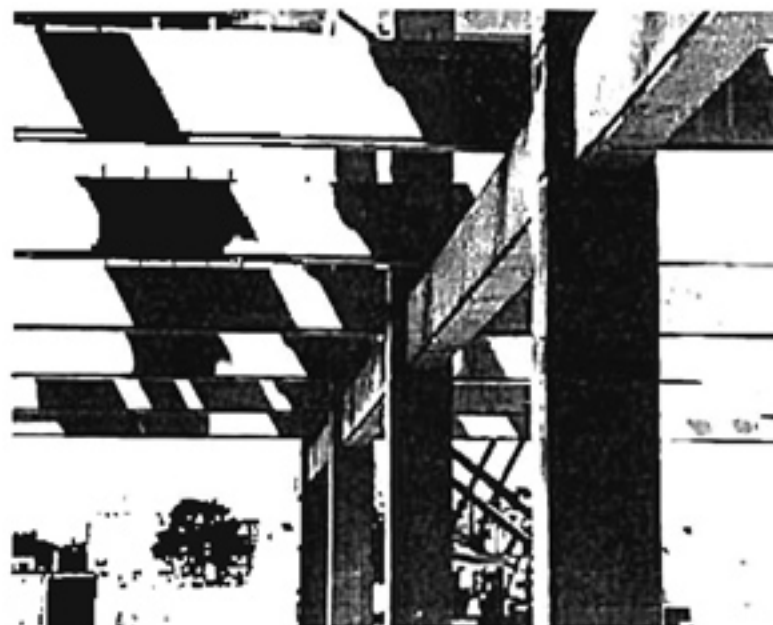


Fig. 3.9: Columns with rolled profiles encased in concrete used in connection with composite beams and slabs - MUNICH, Industrial building (3.10, 3.11), W. GERMANY.



Fig. 3.10 and 3.11: Composite columns before and after concreting, used in connection with flat slabs - Office building LE FOYER (1.5), LUXEMBOURG.

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Chapter 4: FIRE RESISTANCE OF COMPOSITE COLUMNS WITH ROLLED PROFILES CONCRETED BETWEEN THE FLANGES

4.1. Description

This column type developed by the Steel Industry of Luxembourg in collaboration with several European Universities (1.2, 4.1 to 4.8), is a component of the composite AF-construction system currently available for columns, beams and the corresponding connections. It can be designed for any fire resistance time of 30, 60, 90 or 120 minutes (4.9 to 4.21) and has been installed at numerous buildings in several European countries (4.22 to 4.24, 1.5).

The main characteristic of this column type consists in using rolled H-profiles and filling the spaces between the flanges with concrete. The exterior faces of the steel flanges remain visible. The concrete contains longitudinal reinforcing bars which contribute to support loads. Stirrups or shear studs are welded to the web of the section in order to ensure solidarity of the reinforced concrete with the steel section at normal service conditions (1.10, 2.2) and under the action of fire (Fig. 4.1).

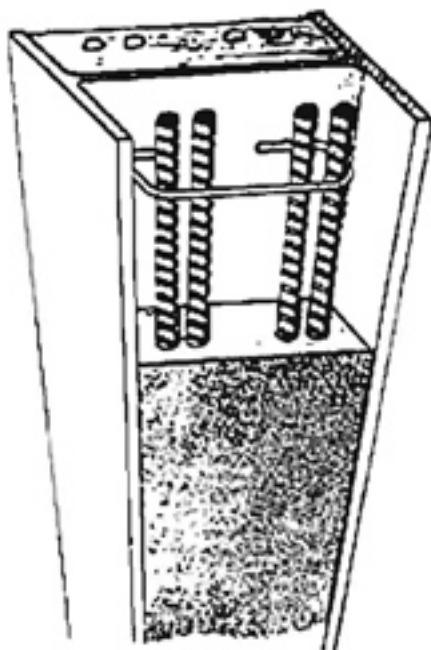


Fig. 4.1: Composite column section with main components: H-profile, re-bars, stirrups welded to web and concrete between the flanges.

4.2. Method of calculation and experimental basis

The exact method of calculation is based on the principles outlined in chapter 2. The transient thermal field analysis of these composite cross sections and the numerical calculation of the buckling loads $N_{cr,\theta}$ under fire conditions of the same columns, have shown a very good agreement with the measured temperatures and the buckling loads during the corresponding fire tests. For the calculation of the temperature distribution during the fire, the SAMCEF-Program (2.9) and the recently developed CEFIOSS-Thermomechanical Numerical Computer Code (2.14 to 2.18) have been used. Typical results of such calculations are presented in Fig. 2.3 and 2.4.

For everyday use, a balanced summation method (see chapter 2.4.2) using large cross-section parts has been developed for centrally loaded columns composed of rolled H-profiles concreted between the flanges (4.9, 4.10). This model has been calibrated on a series of 8 column tests performed before 1980 and the $\Psi_{i,\theta}$ -factors were determined from these tests (4.1, 4.5, 4.25, 4.26). This procedure was validated by 10 more column tests, executed between 1983 and 1987 (see figure 4.4).

According to that method, the cross section of this composite column type is divided into four parts (see figure 4.2). It is appreciated that the mechanical properties of these parts vary according to their average temperatures, which are known as a function of the fire exposure time t and of the section size. When calculating the ultimate load bearing capacity, the whole section is assumed to yield according to the changes in properties as a function of the average attained temperatures.

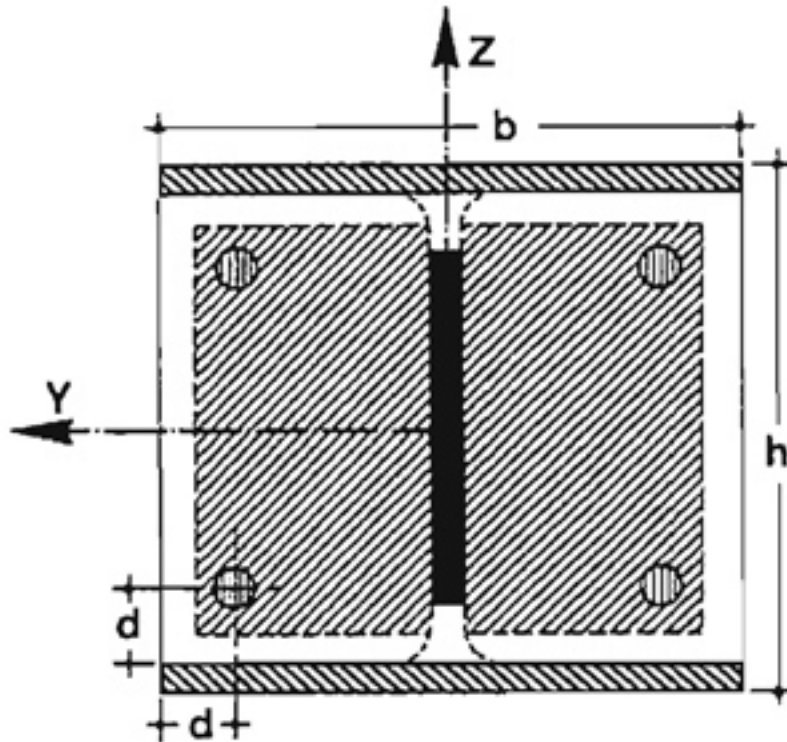


Fig. 4.2; Reduced composite cross section depending on the fire exposure time. For the buckling around the weak axis Z , the static properties of the four individual parts are given by:

- I_w , moment of inertia of reduced web area $\psi_{w,\theta} \cdot A_w$
- I_{fl} , moment of inertia of flanges with area A_{fl}
- I_r , moment of inertia of reinforcing bars with area A_r
- I_c , moment of inertia of reduced concrete area $\psi_{c,\theta} \cdot A_c$

By comparing the various test results, showing the temperature evolution as a function of time and cross sectional shape, to the well known variation of material properties as a function of temperature (4.2, 4.27, 4.28) the following relationships could be set up:

The ultimate plastic load is given by (see [2.10])

$$N_{p,\theta} = \psi_{w,\theta} \cdot A_w \cdot \sigma_{yw,\theta} + A_{fl} \cdot \sigma_{yfl,\theta} + A_r \cdot \sigma_{yr,\theta} + \psi_{c,\theta} \cdot A_c \cdot \beta_{c,\theta} \quad [4.1]$$

and the effective rigidity is calculated as

$$EI_{eff,\theta} = (E_{w,\theta} \cdot I_w) + (E_{fl,\theta} \cdot I_{fl}) + (E_{r,\theta} \cdot I_r) + (E_{c,\theta} \cdot I_c) \quad [4.2]$$

The Euler buckling load follows by

$$N_{E,\theta} = EI_{eff,\theta} \cdot (\pi^2 / L_{cr,\theta}^2) \quad [4.3]$$

and the reference slenderness ratio is

$$\bar{\lambda} = \sqrt{N_{p,\theta} / N_{E,\theta}} \quad [4.4]$$

This slenderness ratio $\bar{\lambda}$ used with the European buckling curve c (2.3) gives a reduction factor κ (2.1) to be applied to the total ultimate plastic load [4.1] in order to obtain the ultimate, or buckling load.

$$N_{cr,\theta} = \kappa \cdot N_{p,\theta} \quad [4.5]$$

More than twenty practical tests of fire endurance, on composite columns with H-sections concreted between the flanges have been carried out in several European Fire Testing Laboratories (4.5, 4.6, 4.15, 4.19, 2.14).

These tests consisted of heating the ambient gas temperature, surrounding the previously loaded column, according to the ISO standard curve. The moment at which the loaded column collapses under these fire conditions gives the respective ultimate failure time.

The temperatures measured during the column test described in the test report Nr 5093 of GENT UNIVERSITY (4.6), is given on figure 4.3 up to the failure time of 116 minutes.

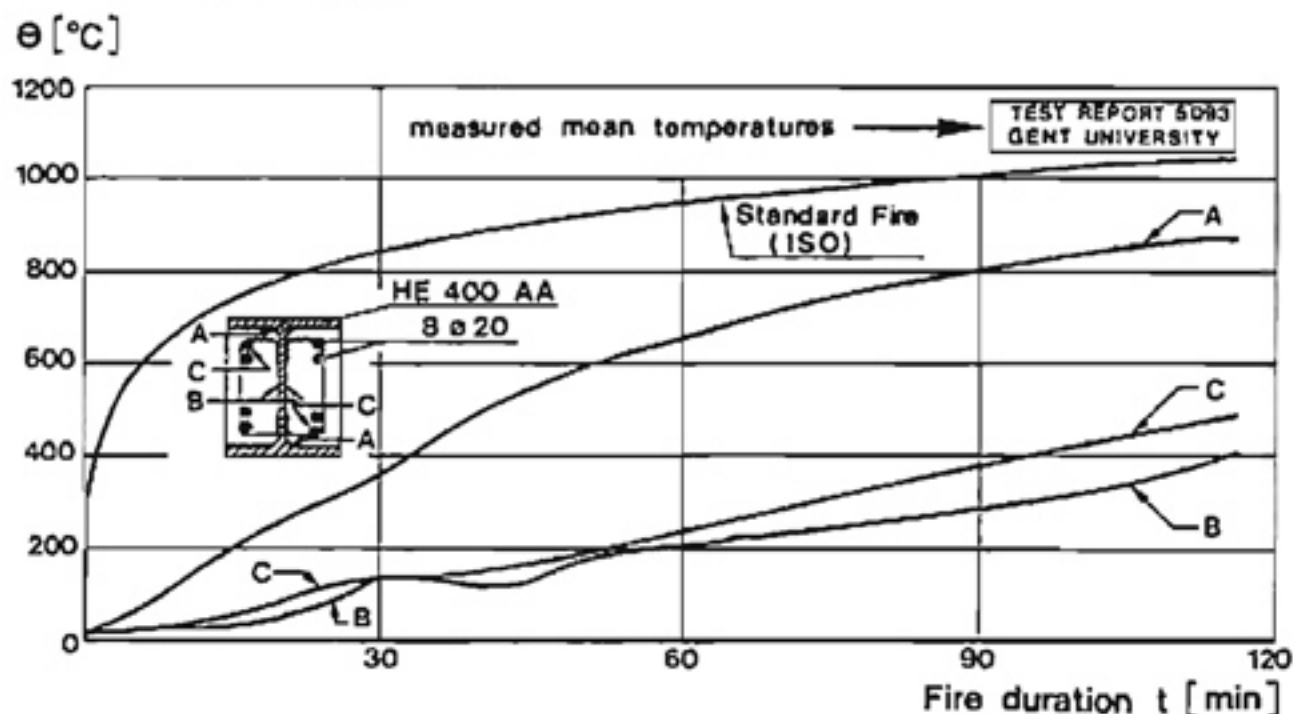


Fig. 4.3; The heating behaviour of individual component parts measured in test column N° 15 (see figure 4.4):

- A, profile web-flange junction
- B, centre part of profile web
- C, outer re-bars

More detailed test conditions and test results are given in figures 4.4 and 4.5 and in the respective available technical certificates. It can be seen from these figures that the ultimate failure times predicted by the limited calculation method are very close to the measured failure times, except for test numbers 7, 17 and 18.

Test number 7 gave a shorter time of failure because concrete spalling occurred right at the beginning of the test (23^d minute), exposing one reinforcing bar. This failure occurred because the test was carried out too early, before the concrete was sufficiently dried out. For this reason composite column tests should be conditioned for 6 months at least. By that time the expected water content will represent 2 to 4 % of the weight of concrete.

Test numbers 17 and 18 gave higher times of failure compared to the values computed by the limited calculation model. This is due to the low thermal characteristic values α and ϵ of the FIRTO-Furnace in Borehamwood, in comparison to the corresponding higher values of the BRAUNSCHWEIG-Furnace in which the column tests, used for the calibration of the limited calculation model, were performed.

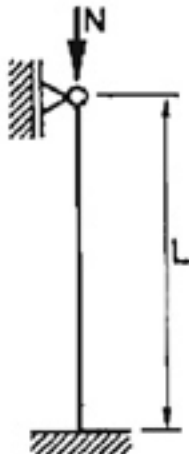
Test number	Column profile	Column length L (m)	Column end conditions	Axial test load N (kN)	Test date	Ultimate failure time (min)		Technical certificates
						tested	computed	
1 2	HP360x132 HP360x132	3,8 3,8		2200 3000	1977	141 112	123 95	77150 R FRG
3 4 5 6 7 8	HP240x57 HP240x57 HP300x86 HP300x86 HP360x174 HP360x174	3,8 3,8 3,8 3,8 3,8 3,8		700 600 1600 1250 3250 2800	1979	81 99 68 87 50 91	91 97 79 96 73 87	86044 FRG
9 10	HE 320 AA HE 320 AA	3,68 3,68		2090 1600	1983	86 102	89 102	831009 FRG
11 12	HP310x79 HP310x79	5,71 5,71		700 700	1983	115 116	129 127	831016 FRG
13 14	HP300x122 HP300x202	3,73 3,73		1500 3420	1984	107 39	95 50	SFB 148 FRG
15	HE 400 AA	4,19		2400	1984	116	102	5093 B
16	HE 500 AA	3,66		2200	1986	117	119	86-U-052 FR
17 18	UC 356x202 HP 305x79	3,30 3,30		1900 1700	1987	179 130	130 99	TE6143/44 UK

Fig. 4.4: Centrally loaded columns tested under fire conditions in Braunschweig (FRG), Gent (B), Maizières-les-Metz (FR) and in Borehamwood (UK) and calculated by the limited calculation model

- the load eccentricity e_y was 0 mm for tests 1 to 8, but equal to 5 mm for tests 9 to 18,
- the buckling length $L_{cr,\theta}$ considered in the calculation was taken equal to $0,7 \cdot L$, except for tests 11, 12, 16, 17 and 18 where $L_{cr,\theta} = L$,
- in all the tests, buckling occurred around the weak axis 2 (see fig. 4.2),
- the fire resistances have been computed on the basis of the measured real material qualities.

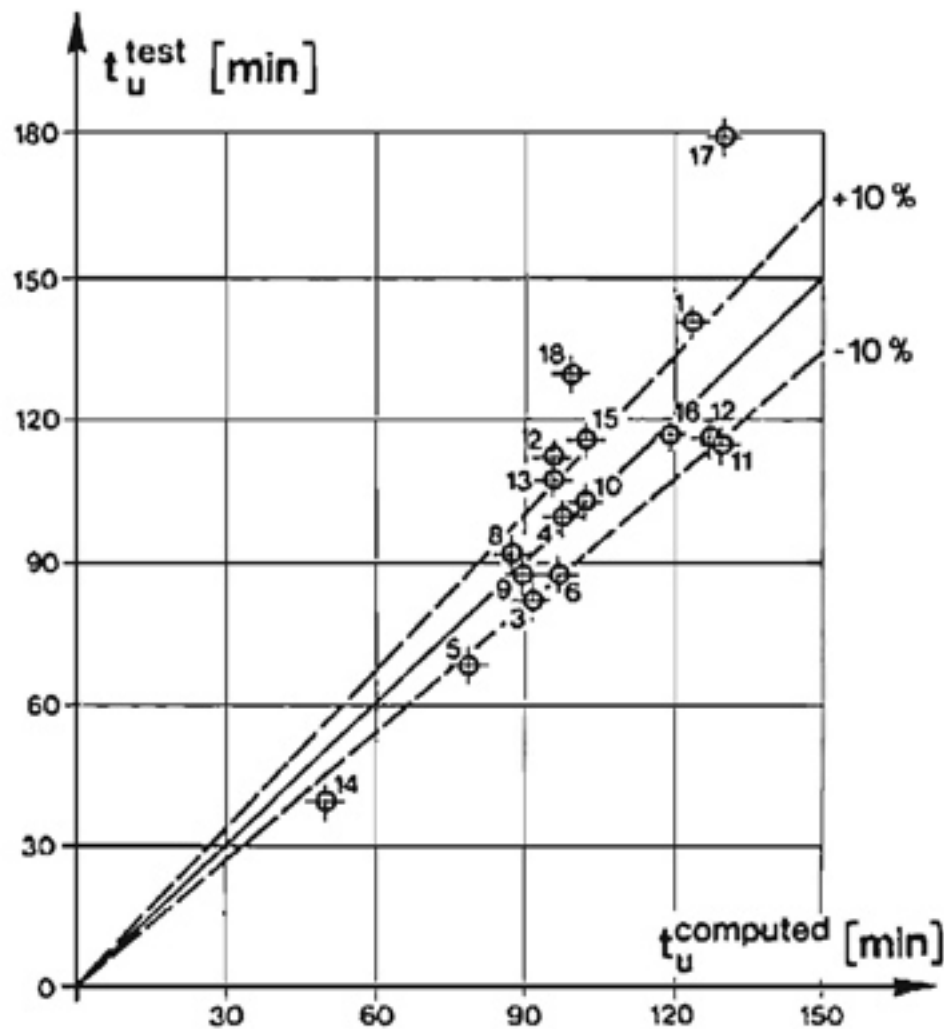


Fig. 4.5; Comparison of fire endurance for centrally loaded composite columns based on H-sections with concrete between the flanges, measured from tests and computed according to the limited calculation model of chapter 4.2 (see also fig. 4.4).

4.3. Practical design

- * A set of diagrams for nearly all series of European and American rolled H-profiles (1.9) has been drawn up on the basis of the calculation method described in chapter 4.2. These diagrams allow a very fast design. Indeed each diagram contains the whole series of one profile-type from 240 to 1000 mm in height. For a given fire class, design load and column buckling length, each diagram presents an adequate composite cross section including geometry and qualities of all the steel and concrete components. Loads are supposed to be practically axial, an accidental eccentricity of 5 mm being included.

The following parameters have been considered:

- the column buckling lengths $L_{cr,\theta}$ up to 5 m,
- the fire classes F60, F90, F120,
- the concrete qualities C20, C30, C40 according to Eurocode 2,
- the steel grades Fe 360 and Fe 510 according to Euronorm 25,
- the longitudinal reinforcing bars of quality S500 according to Euronorm 80 up to 3 % of the concrete area.

A selection of these diagrams is given in appendix B. In this appendix, the diagrams for the F30 fire class are not given because a proper design for normal temperature also ensures a minimum fire resistance time of 30 minutes.

Buckling lengths should be chosen according to the proposals of chapter 2.4.4. In this chapter 4 as well as in appendix B, buckling is presumed to occur around the weak axis Z.

- * Furthermore computer programs for the design of this type of composite column are available on different hardware configurations. These programs allow to consider any steel or concrete qualities given by national standards and to establish different catalogues according to material grades of national standards, provided the ranges of parameters covered by the initial catalogue (4.9, 4.10) is not exceeded.
- * It should be mentioned that, apart from the previously explained balanced summation method applicable to centrally loaded AF-columns, a general numerical, thermal and mechanical system analysis is also available (2.14 to 2.18, 4.29). In that respect general N-M interaction diagrams, buckling included, will be established for different types of composite columns (2.19).

4.4. Constructional details

It should be noted that a construction designed according to this process still has the characteristics of a steel structure. The elements are prefabricated in the shop, or on site, and are erected and assembled according to methods similar to those of traditional steel construction. Steel or composite beams can be bolted to vertical steel plates welded to the composite columns (Fig. 4.6 and 4.7). Another method consists in using block shear connectors welded to the column flanges, the steel or composite beams simply laying on these connectors (Fig. 4.8).

But these composite columns are also used in connection with flat slabs. The column is delivered with the steel shear head already welded (Fig. 4.9).

Also this type of column can be prefabricated in one piece over several levels, re-bars being adapted to the changing axial loads, so that the speed of building construction will be accelerated (Fig. 4.7 and 4.8).

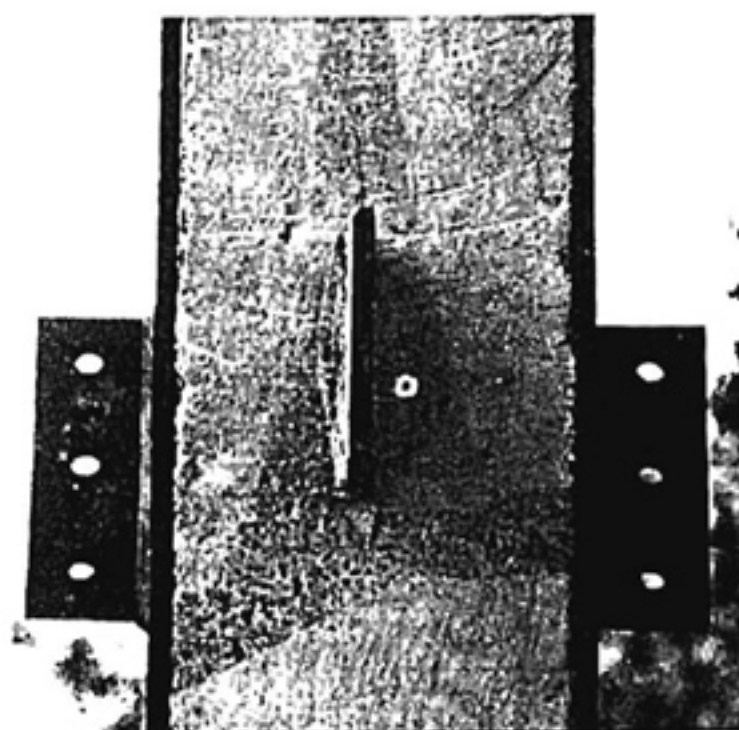


Fig. 4.6; Composite F90 column with gussets for floor beam connection - COLOGNE, Office building TradeARBED, W. GERMANY

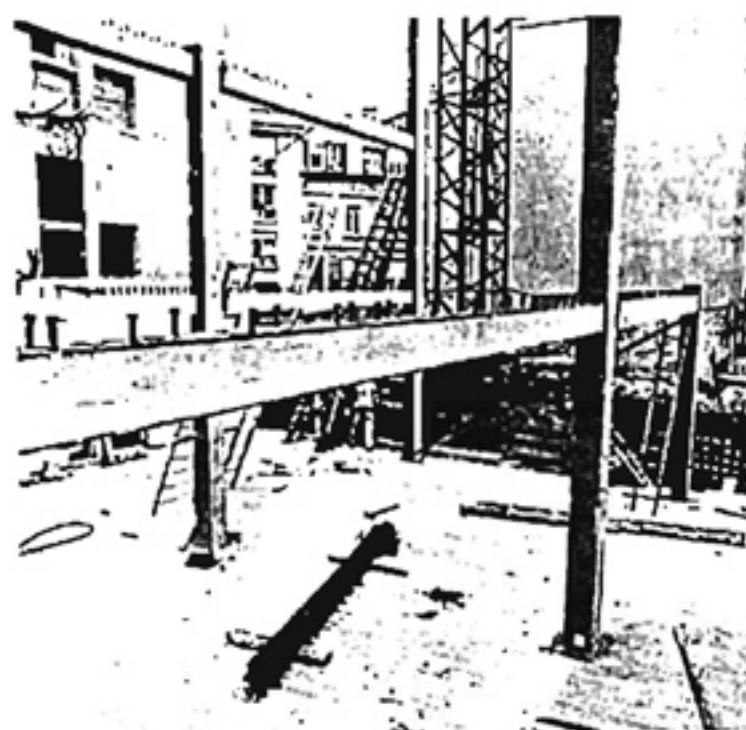


Fig. 4.7; Composite F90 beams bolted to composite F90 columns - Eight level office building "Val de la Pétrusse", LUXEMBOURG.

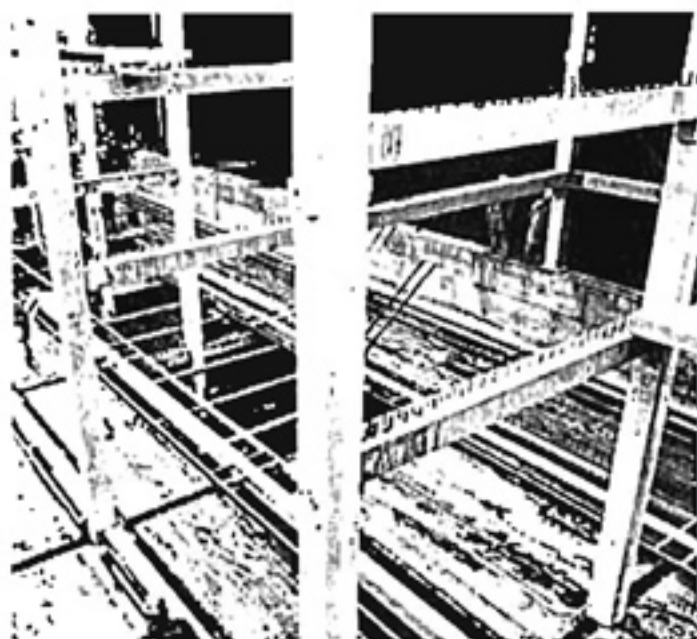


Fig. 4.8; Composite F90 beams assembled to F90 columns by block shear connectors - Industrial building ADU, LUXEMBOURG.

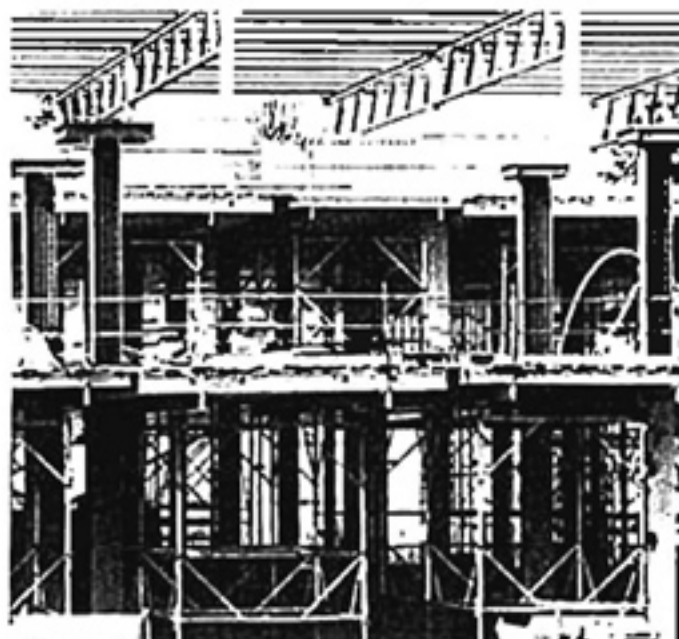


Fig. 4.9; Composite F 90 columns with steel shear heads after erection - GENEVA, Commercial Centre Jumbo Vernier, SWITZERLAND.

The casting of concrete in the horizontal column position before erection, requires no shuttering (Fig. 4.10); for an AF column in vertical position after erection, shuttering will be simplified, due to the straight edges of the profile flanges.

The proportional transfer of load between the two components of hot rolled steel section and reinforced concrete must be guaranteed, either, by using top and base plates (Fig. 4.11), or by welding locally shear studs to the web of the profile or the like.

The outer dimensions of these composite profiles are significantly reduced compared to the dimensions of a traditional steel or concrete column. In addition, their high resistance to impact, due to the exposed steel flanges, is a big advantage in the case of industrial and public buildings.



Fig. 4.10; Composite F120 column during horizontal concreting before erection - Social Assurance Building EVI, LUXEMBOURG.

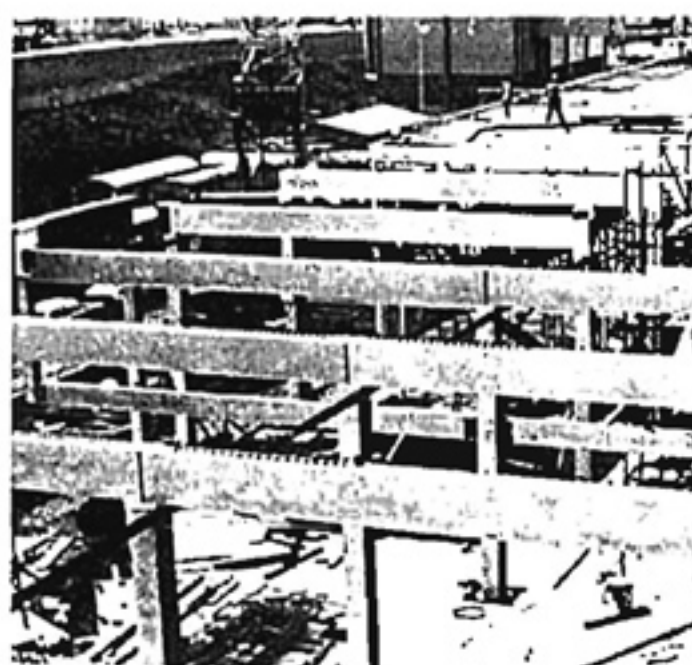


Fig. 4.11; Composite F90 beams directly settled on the top plates of composite F90 columns - NORDERSTEDT, Chocolate works PEA, W. GERMANY.

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Chapter 5: FIRE RESISTANCE OF COMPOSITE COLUMNS WITH CONCRETE FILLED HOLLOW STRUCTURAL SECTIONS

5.1. Description

Concrete filling of Hollow Structural Section (HSS), has been initially developed to increase the load bearing capacity of the steel section, without increasing its external dimensions. Another feature of this column type is that the steel remains visible, which is often considered as an advantage from an architectural point of view.

Concrete filling has, however, also been shown to improve the fire resistance. One reason is that the concrete increases the heat capacity of the column. More important however is that, when exposed to fire, a redistribution of loads will occur in the column from the hot steel section to the relatively cold concrete core.

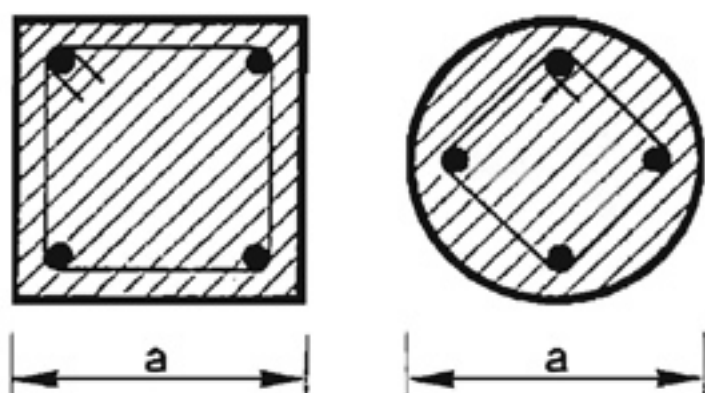


Fig. 5.1: Examples of concrete filled HSS with optional reinforcement.

It follows that for a given buckling length and loading, the fire resistance will depend mainly on the cross sectional dimensions, the concrete quality and the reinforcement. By a proper choice of these parameters practically any fire resistance requirement can be met. If the column is under its maximum design load at room temperature conditions, a fire resistance of 30 minutes may be achieved for sections wider than 140 mm. If the loading is reduced, an even higher fire resistance is possible (5.1).

5.2. Method of calculation and experimental basis

Both in France and Germany and with financial aid from the Commission of the European Community (C.E.C.) and the "Comité International pour le Développement et l'Etude de la Construction Tubulaire" (CIDECT), experimental and theoretical work has been carried out into the behaviour of fire exposed concrete filled HSS-columns. On the basis of this work, calculation models for the fire resistance were established (5.2, 5.3, 2.11).

The German model (5.3, 2.11) starts with a thermal analysis, leading to temperature fields calculated for a certain time of fire exposure. The ultimate load is calculated by means of an incremental elasto-plastic analysis, taking into account geometrical and physical non-linear effects. Stress-strain relationships at elevated temperatures are specified and the effect of thermal stresses is taken into account explicitly. This model can therefore be identified to the "exact method" reviewed in chapter 2.4.1. In the French model (5.2), the temperature distribution over the cross section of the column is first determined as a function of time. For a given temperature state, the ultimate buckling load is determined by means of a procedure

similar to the Guyaux/Janss calculation method developed for composite columns under room temperature conditions (5.4). This procedure is adapted by using temperature-dependent, idealized stress-strain relationships for the steel section, reinforcement and concrete. Effective values for the bending stiffness of the composite cross section are derived from these stress-strain relationships, in combination with the column geometry. The starting point for the calculation is the c-buckling curve according to the ECSC (5.5). A curve fitting factor is introduced in order to obtain reasonable agreement between theoretical and experimental results. The effect of the thermal stresses is ignored. Thus, the French model is an approximate method and can be identified with the summation method, reviewed in chapter 2.4.2.

Both the French and German models show reasonable agreement with available test data. However, the calculated fire resistance according to the two models differs quite significantly for conditions outside the range of buckling lengths used in the tests. The reasons for this discrepancy are analysed in (5.7) and appear to be mainly due to

- the different idealisation of the mechanical properties of concrete at elevated temperatures and
- the different appreciation of the effect of thermal stresses.

From the point of view of consistency, this situation is unsatisfactory. Therefore, a set of harmonized properties of concrete at elevated temperatures is proposed in (5.7). Using these input data produces almost identical results, when applying the French and German calculation models. This is illustrated for instance in figure 5.2 which refers to one type of reinforced concrete filled HSS-column having a fire resistance of 90 minutes. The harmonized input data are given in Appendix D.

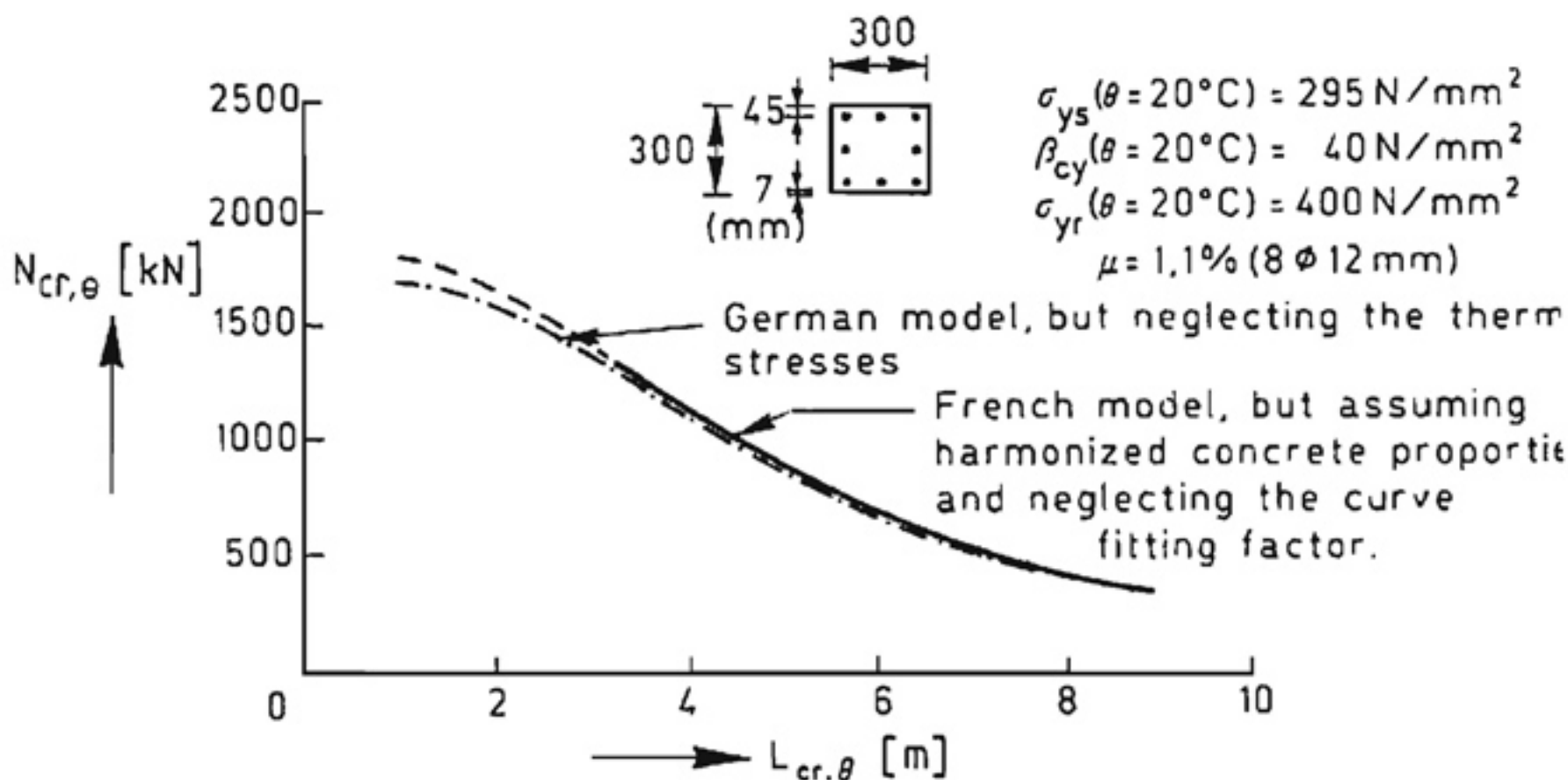


Fig. 5.2; Buckling curves at elevated temperature, based on the French and German models for a reinforced concrete filled HSS-column and the fire class F90.

The original French calculation model with the harmonized input data is denoted as the "harmonized calculation model". The advantage of this model, when compared to the original, more "exact" German work, is that a simpler numerical procedure can be used.

The design information on concrete filled HSS-columns given in this Technical Note is based on the harmonized calculation model. Since the harmonized calculation model is closely related to the original French model, it must also be identified with the summation method as reviewed in chapter 2.4.2.

Test no.	Column profile (mm)	Column length (m)	Reinforcement μ (%)	Column end conditions	Test load (kN)	Eccentricity (mm)	Ultimate failure time (min)		Technical certificates	
							tested	comput.	no	ref
1	140x140x3.6	3.60	1.1	C	410	0.	46	39	74.8205	(5.2)
2	160x160x6.3	3.60	3.7	C	830	0.	35	38	74.8223	"
3	200x200x5.0	3.60	1.7	B	500	0.	83	68	77.U68/T68	"
4	225x225x3.6	3.60	1.6	C	560	0.	137	101	76.U35/T34	"
5	260x260x6.3	3.60	2.0	C	1500	0.	109	71	77.U49/T49	"
6	300x300x7.0	3.60	3.1	C	2570	0.	57	68	76.U33/T32	"
7	350x350x10	3.60	2.1	C	4560	0.	51	46	FI.3403	"
8	200x200x6.3	4.20	2.9	B	430	20	63	56	Br.79/1/1	(5.3)
9	260x260x7.1	4.20	1.7	B	1017	26	37	51	BR.80/1/9	"
10	300x300x7.0	4.20	1.2	B	634	30	92	93	BR.80/1/14	"
11	200x200x6.3	4.20	0.9	B	400	20	23	29	BR.80.1/16	"
12	200x200x6.3	4.20	2.9	A	650	0.	79	66	BR.80.1/19	"
13	220x220x6.3	4.20	4.4	B	421	22	88	74	BR.82/1/24	"
14	260x260x7.1	4.20	3.8	B	869	26	64	67	BR.82.1/25	"
15	300x300x7.0	4.20	3.6	B	1507	30	56	57	BR.82.1/26	"

Fig. 5.3; Review of the French and German tests, selected for verification of the harmonized calculation model; test and calculation results are also presented (5.2, 5.3, 5.7).

To validate the harmonized calculation model and set proper limits to its application, the correlation between a representative series of French and German test results and the corresponding calculation results have been analysed. A review of some major test conditions as well as the measured and calculated fire resistances is given in figure 5.3. The calculation results are based on the actual mechanical properties of steel and concrete as determined at the time of the tests and are based on the harmonized calculation model (5.2, 5.3, 5.7).

A graphical display of the correlation between calculated and measured results is presented in figure 5.4. This shows reasonable agreement.

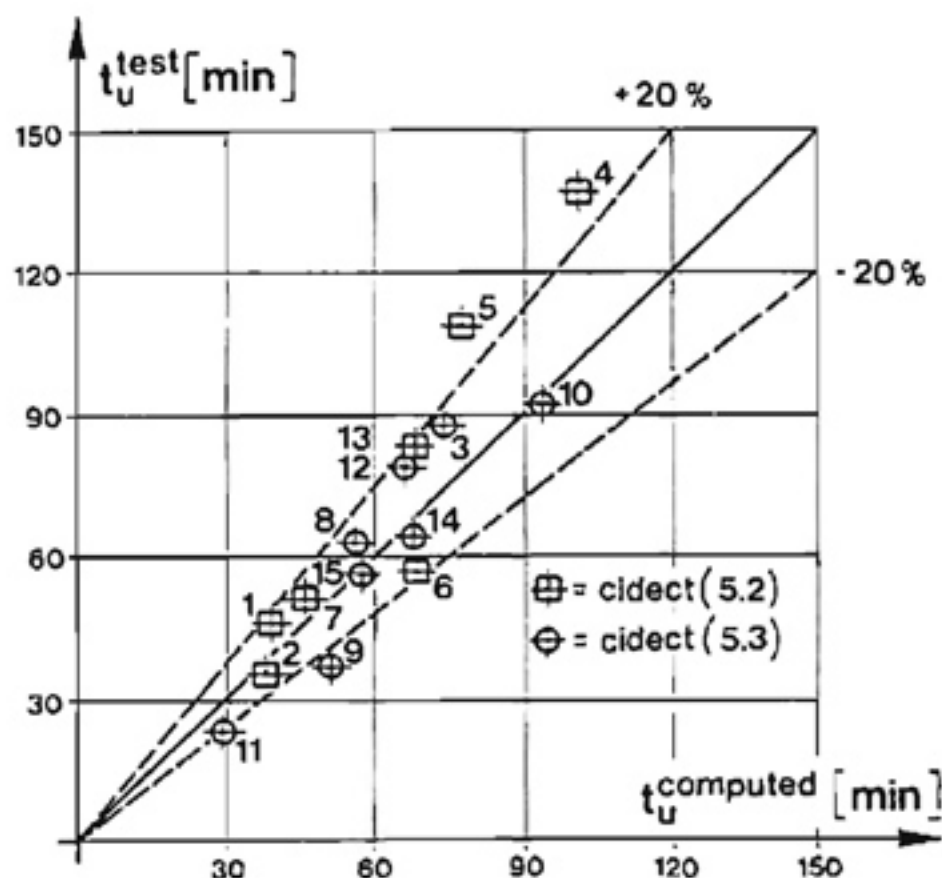


Fig. 5.4; Calculated fire resistances according to the harmonized model (5.7) vs. measured values (5.2, 5.3).

For a statistical analysis of the effect of the various test parameters (buckling length, size of cross section and reinforcement) on the discrepancy between measured and calculated fire resistance, see reference (5.7). On the basis of this analysis, the following limitations for a proper use of the harmonized calculation model are suggested:

1 m	≤ buckling length $L_{cr,\theta}$	≤ 4.5 m
140 mm	≤ size cross section a	≤ 400 mm
0 %	≤ reinforcement μ	≤ 5 %
30 min.	≤ fire resistance t_u	≤ 120 min.

If any of the four above conditions is not fulfilled, the harmonized calculation model should not be used and a more detailed analysis is necessary (2.11, 5.6). It is felt however that most practical situations are within the given range.

5.3 Practical design

For practical design purposes, a set of diagrams is given in appendix C, in which, for a standard fire exposure of 60, 90 or 120 minutes, the critical load $N_{cr,\theta}$ of concrete filled HSS-columns is given as a function of the buckling length $L_{cr,\theta}$. A practical range of square and circular sections, reinforcement, and concrete qualities is chosen, taking due account of the limitations suggested in chapter 5.2. Material qualities at room temperature are in accordance with Eurocode 4.

All the diagrams are calculated for a structural steel quality Fe 360 and HSS-profiles with minimum wall thickness. Using the diagrams for structural steel with a higher grade and/or for sections with a larger wall thickness, will render conservative results.

5.4 Constructional details (5.2, 5.8, 5.9)

It is essential to ensure that the concrete is of correct quality. To achieve this, the concrete should be designed so as to be sufficiently workable, i.e. the slump measured using 'Abrams' cone (5.9) should be between 70 and 200 mm. The minimum amount of water needed to obtain good workability should be used.

Filling can be accomplished in situ by gravity, with a concrete pump or plunge pipe, and by injection from the bottom.

Gravity filling is only recommended if the length of the column does not exceed 3 m in height. When a pump is used for filling, the end of the discharge pipe must be placed close to the surface of the concrete. Both for filling by gravity and with a pump or plunge pipe, the concrete should be vibrated. Size of the gravel aggregate has to be chosen accordingly.

The quality of the filling process can be checked by tapping the column, so as to detect by ear possible filling defects. When a defect is located, it is necessary to correct the fault by drilling holes, injecting grout and resealing the holes.

For the concrete cover, d_1 on the reinforcement bars (Fig. 5.5), the largest of the following values must be taken:

$$d_1 = 30 \text{ mm} \quad \text{where}$$

$$d_1 = a/8 \text{ mm} \quad \left. \begin{array}{l} \\ \end{array} \right\} \begin{array}{l} a = \text{side of square section or diameter of circular} \\ \text{section in mm.} \end{array}$$

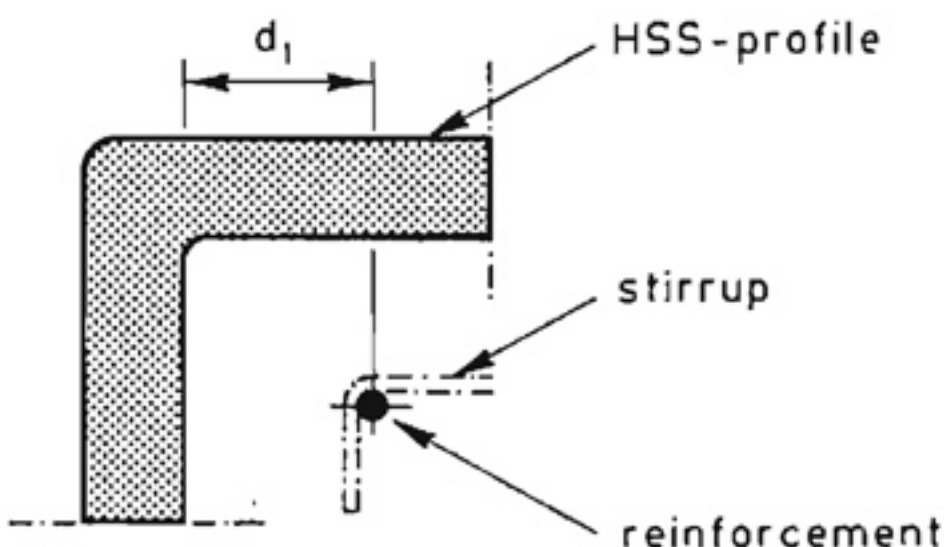


Fig.5.5;Concrete cover d_1

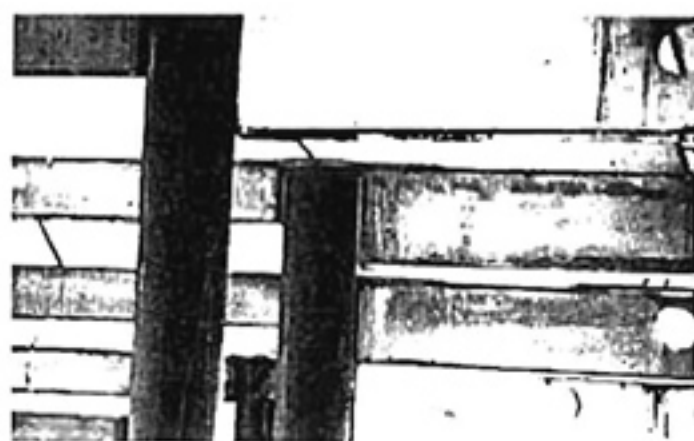
Stirrups must be provided over the full length of the (reinforced) column at distances of not more than $15 \times$ the diameter of the smallest reinforcement bar.

Small drain holes ($\phi = 10$ to 15 mm) are required in the walls of the HSS, usually in pairs. Such holes must be provided for each single length and at each floor level, with a maximum distance of 5.00 m. They must be placed between 100 and 120 mm from each column end. These holes are intended to prevent the bursting of the column under steam pressure from the heating of entrapped water in the enclosed concrete.

Examples of buildings using the concrete filling HSS-column concept for improving the fire resistance are given in figures 5.6 to figures 5.12.



Figures 5.6, 5.7 and 5.8; Concrete filled circular HSS-columns with bolted beams and concreting by injection of superplasticized concrete - DÜSSELDORF, Forschungsinstitut VdEH, W. GERMANY.



Figures 5.9 and 5.10; Concrete filled circular HSS-columns and column-beam connections - BERLIN, Research building of the Fraunhofer Gesellschaft, W. GERMANY.

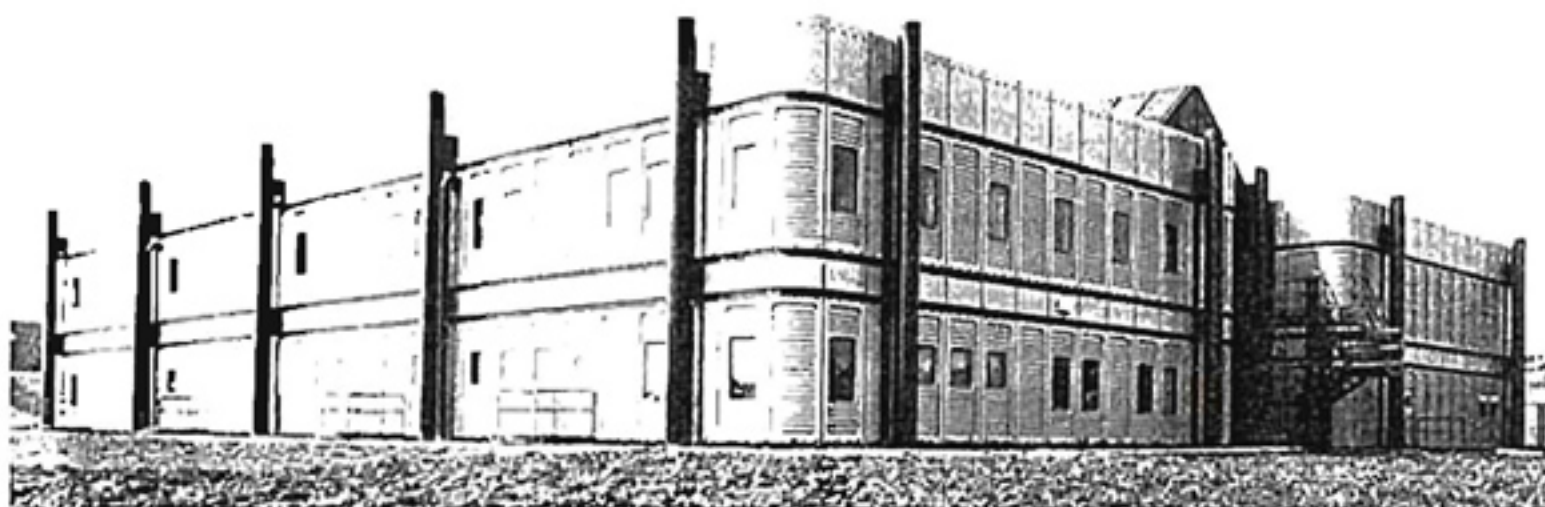


Fig. 5.11; Concrete filled HSS-columns for F60 resistance - SWINDEN, Water Research Centre, UNITED KINGDOM.



Fig. 5.12; Concrete filled HSS-columns for F90 resistance - BORDEAUX, Apartment building "Résidence les Mille pertuis", FRANCE.

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- (5.5) Rondal J., Maquoi R. - Formulation d'Ayrton / Perry pour le flambement des barres métalliques - Construction Métallique No. 4, 1979.
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- (5.8) Klingsch W., Würker K.G., Martin-Bullmann R. - Brandverhalten von Stahlprofil - Verbundstützen - Stahlbau 53, Oktober 1984.
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GENERAL NOTE ON PRACTICAL FIRE DESIGN DIAGRAMMS

The following appendices A, B and C give fire design diagrams for the column types a, b and c (see fig. 1.1) allowing to determine the AXIAL BUCKLING LOAD $N_{cr,\theta}$ in function of the BUCKLING LENGTH $L_{cr,\theta}$ and of the FIRE CLASSES F60, F90 or F120.

This means that column failure by buckling will occur immediately after 60, 90 or 120 minutes of ISO-834 fire exposure.

Furthermore it has to be noted that all diagrams of appendices A, B and C are based on profile sections corresponding to the well known Section Standards (EURONORM or ASTM). Of course these diagrams could be used in connection to similar rolled sections.

The column buckling length should be chosen according to the recommendations of chapter 2.4.4.

Appendix A: FIRE DESIGN DIAGRAMS FOR ROLLED PROFILES ENCASED IN CONCRETE

The design graphs given on pages A.4 to A.9 are limited to the following material and geometrical properties:

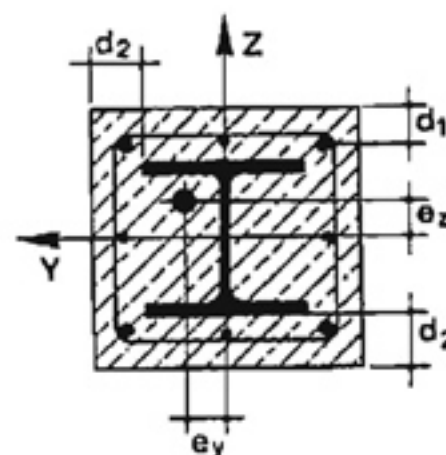
Material properties:

Combinations: C20/Fe360/S400
C40/Fe510/S400

Geometrical properties:

Reinforcement: 4 $\varnothing$ 12 mm

Covers: $d_1 = 30$ mm
 $d_2 = \begin{cases} 40 \text{ mm for F 90} \\ 50 \text{ mm for F 120} \end{cases}$



It has to be noted that the buckling loads given for the fire class F90 with $d_2 = 40$ mm are more or less similar to those given for the fire class F120 with $d_2 = 50$ mm.

Axial force eccentricity: $e_y \leq L_{cr,\theta} / 500$

Buckling length : $1,5 \text{ m} \leq L_{cr,\theta} \leq 5,0 \text{ m}$

Profiles (1.9):

HE-B: 100/120/140/160/180/200/220/240/260

HD : 210.210.46/210.210.59
260.260.73/260.260.89/310.310.97

W : 130.130.23,8/150.150.22,5
200.200.46,1/200.200.71/200.200.100
250.250.115/250.250.167

The following 12 design graphs give interactions, for buckling around the weak axis Z, between failure load $N_{cr,\theta}$ and buckling length $L_{cr,\theta}$ for fire classes F90 and F120 respectively and different combinations of material properties of profile and concrete. In order to reach the F-classification, design load must be smaller or equal to the $N_{cr,\theta}$ values of the graphs. The single plots are assigned to profile types, each curve is appointed to a profile section. The graphs show only profile types and sections which cannot be loaded with full design load to reach the assigned fire resistance. All HEA-profiles and heavier profile types and sections than shown here reach F90/F120 within this slenderness area without design load reduction (see figure 3.7)

The fire classes F30 and F60 are guaranteed automatically for full design loads at normal temperature range if a minimum concrete cover d_1 is foreseen.

Example 1:

- $L_{cr,\theta} = 4,00$ m; F90; $N = 5000$ kN,
- $N > N_{cr,\theta}$ in all design graphs,
- no one of the small profiles is possible.

Result: Column must be designed according to the standard for normal temperature range with minimum profiles fulfilling the conditions of figure 3.7 in order to reach F90.

Example 2:

- $L_{cr,\theta} = 5,00$ m; F120; HE 180 M by design for normal temperature range
- no graph for this profile type is given.

Result: By figure 3.7 this profile can be fully loaded for F120.

Example 3:

INPUT : $L_{cr,\theta} = 0,7 \cdot 7,00 = 4,90$ m,
 $N = 650$ kN,
 F90.

DESIGN : Using the design-graphs the following minimum cross-sections can be chosen for the input-data given above.

PROFILE	MATERIAL	$N_{cr,\theta}$ [kN]	percentage η of design load
HE 220 B	C20 - Fe360	700	93 %
HD260/260/73	C20 - Fe360	750	87 %
W200/200/71	C20 - Fe360	700	93 %
HE 200 B	C40 - Fe510	800	81 %
HD210/210/46	C40 - Fe510	650	100 %
W200/200/46,1	C40 - Fe510	650	100 %

All cross sections : profile-cover $d_2 = 40$ mm,
 reinforcement 4 $\varnothing 12$ - S 400, cover $d_1 = 30$ mm.

The definitive cross section can be selected for

- minimum total cross section size,
- minimum profile weight,
- optimized combination of profile type and material quality by economic reasons.

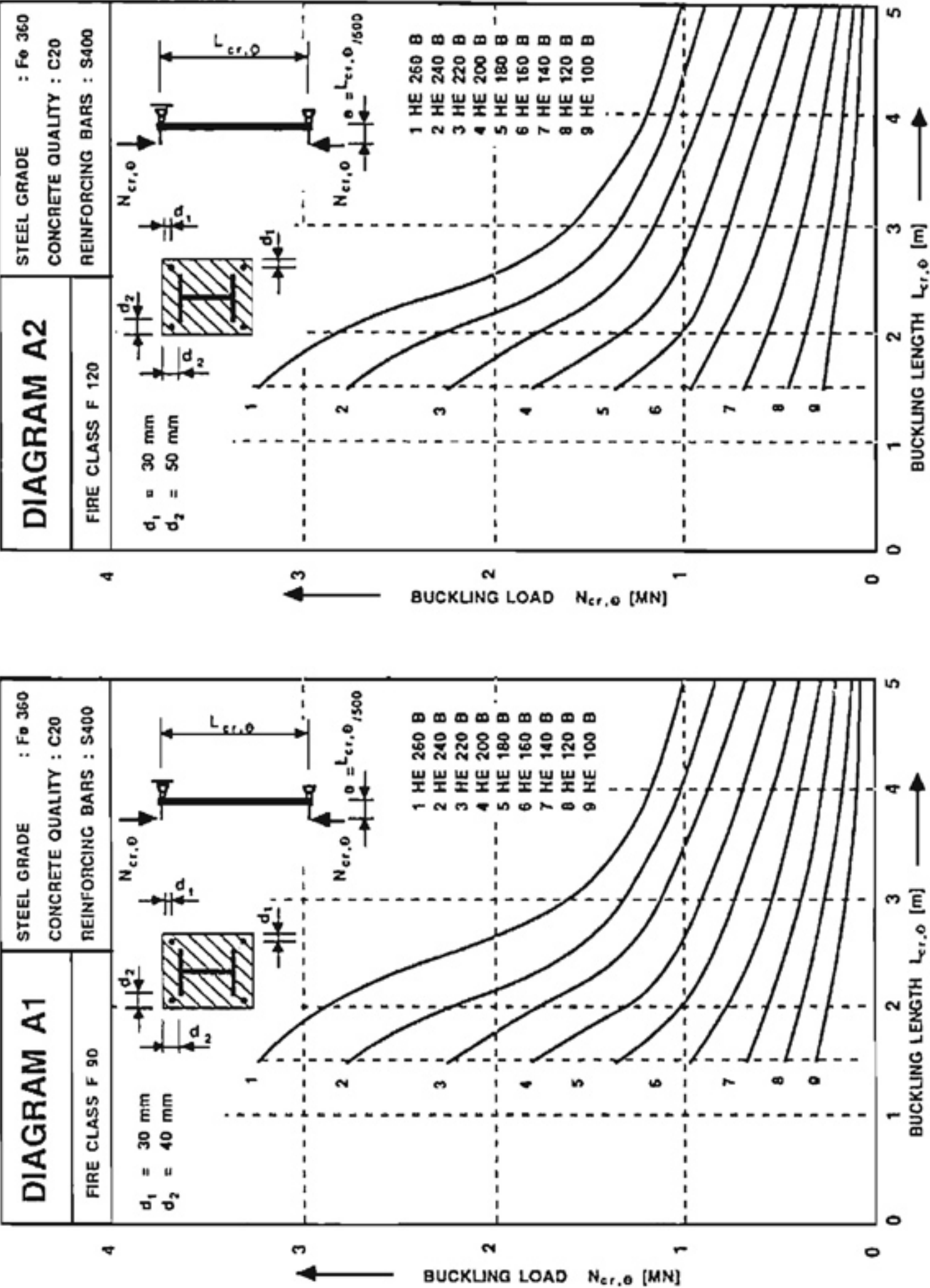
EXAMPLE 4:

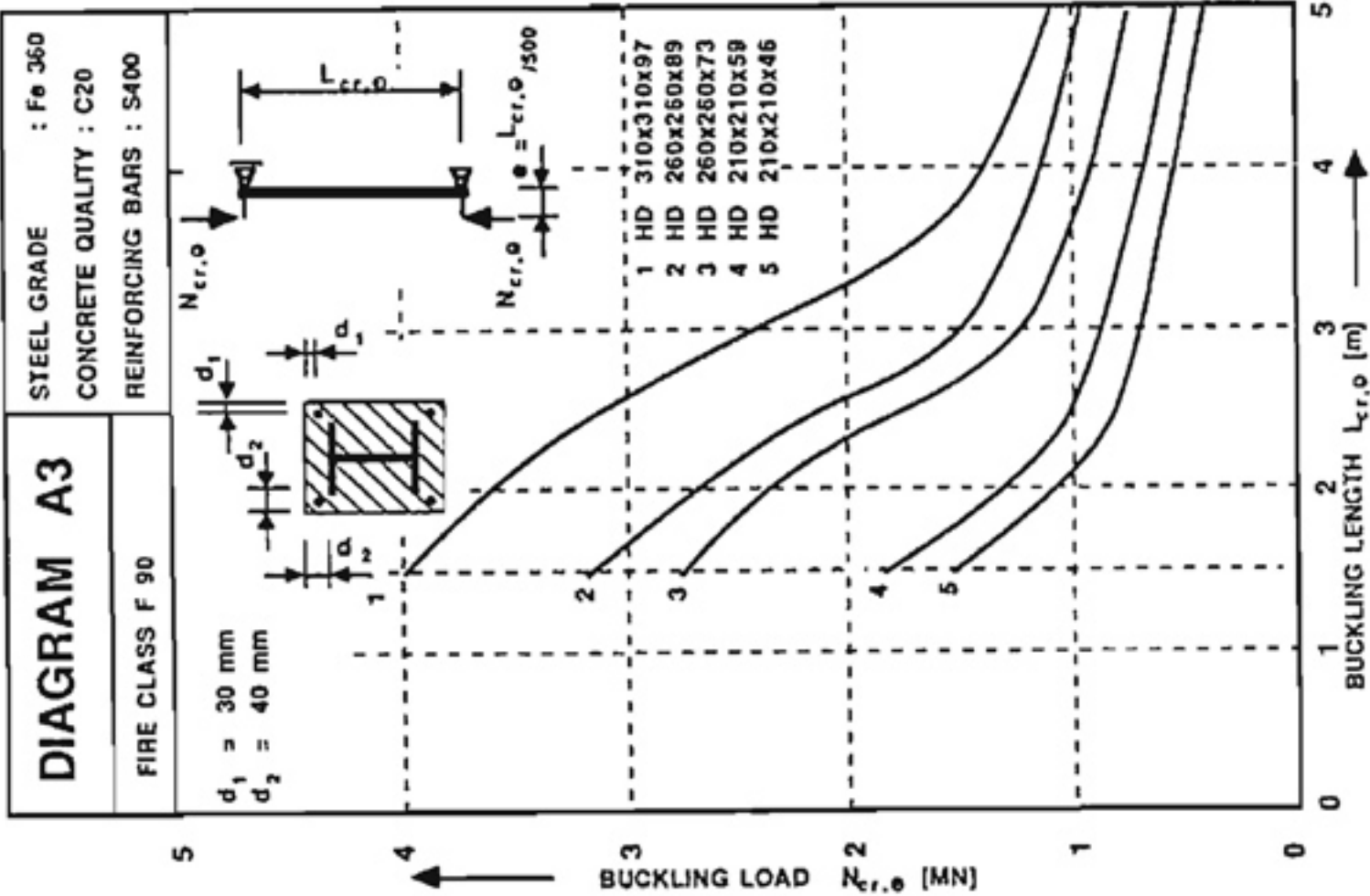
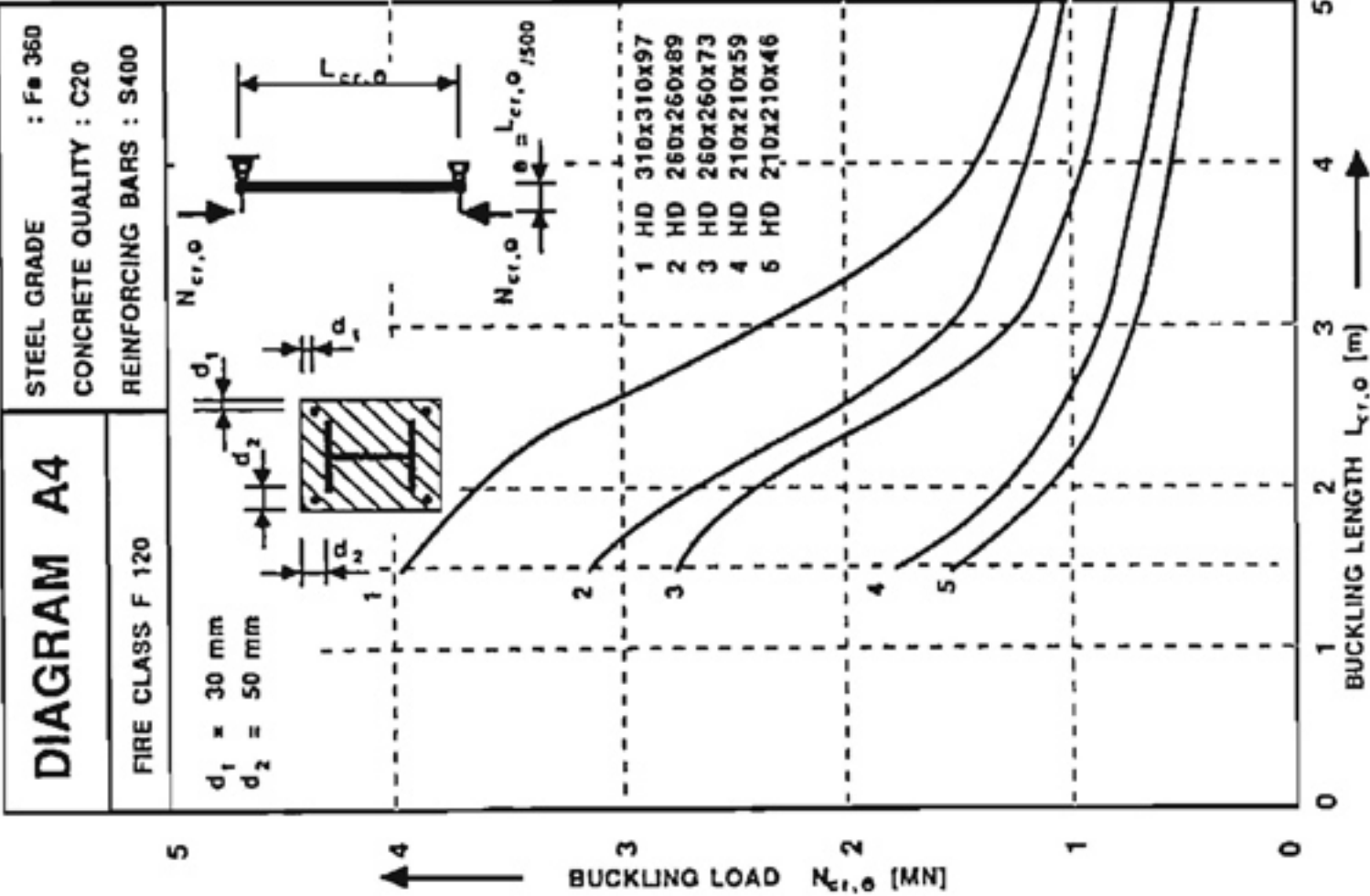
INPUT : $L_{cr,\theta} = 3,50$ m,
 $N = 8000$ kN,
 F120.

DESIGN : All design tables $N_{cr,\theta} \ll N$, that means heavier sections than the plotted ones are needed.
 Consequently the design table of figure 3.7 for fully loaded columns is valid. HE 180 M and heavier or HD210.210.100 and heavier can be designed without design load reduction. Profile type and its section can be determined by design for normal temperature range without restrictions for the required fire resistance.

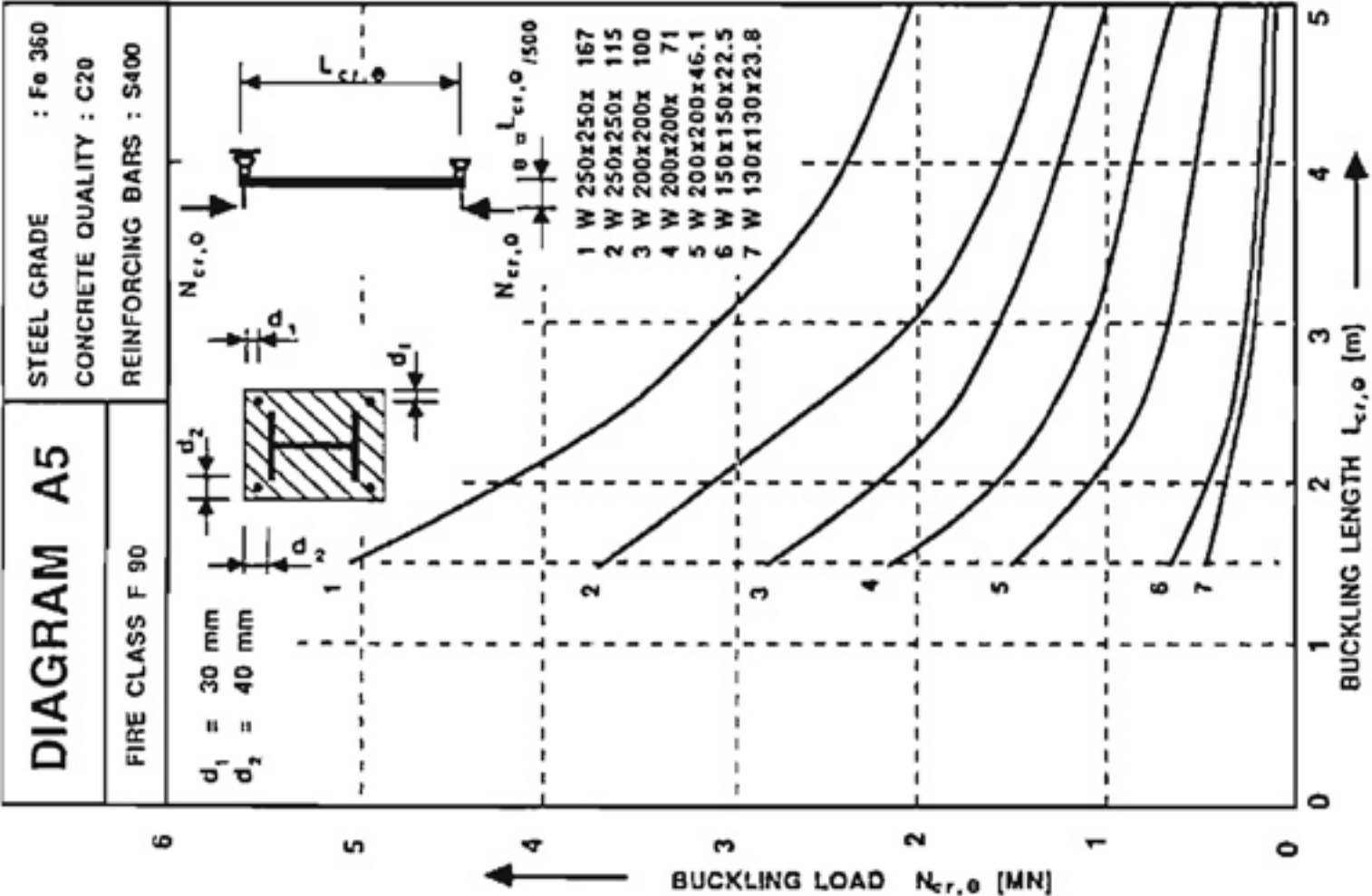
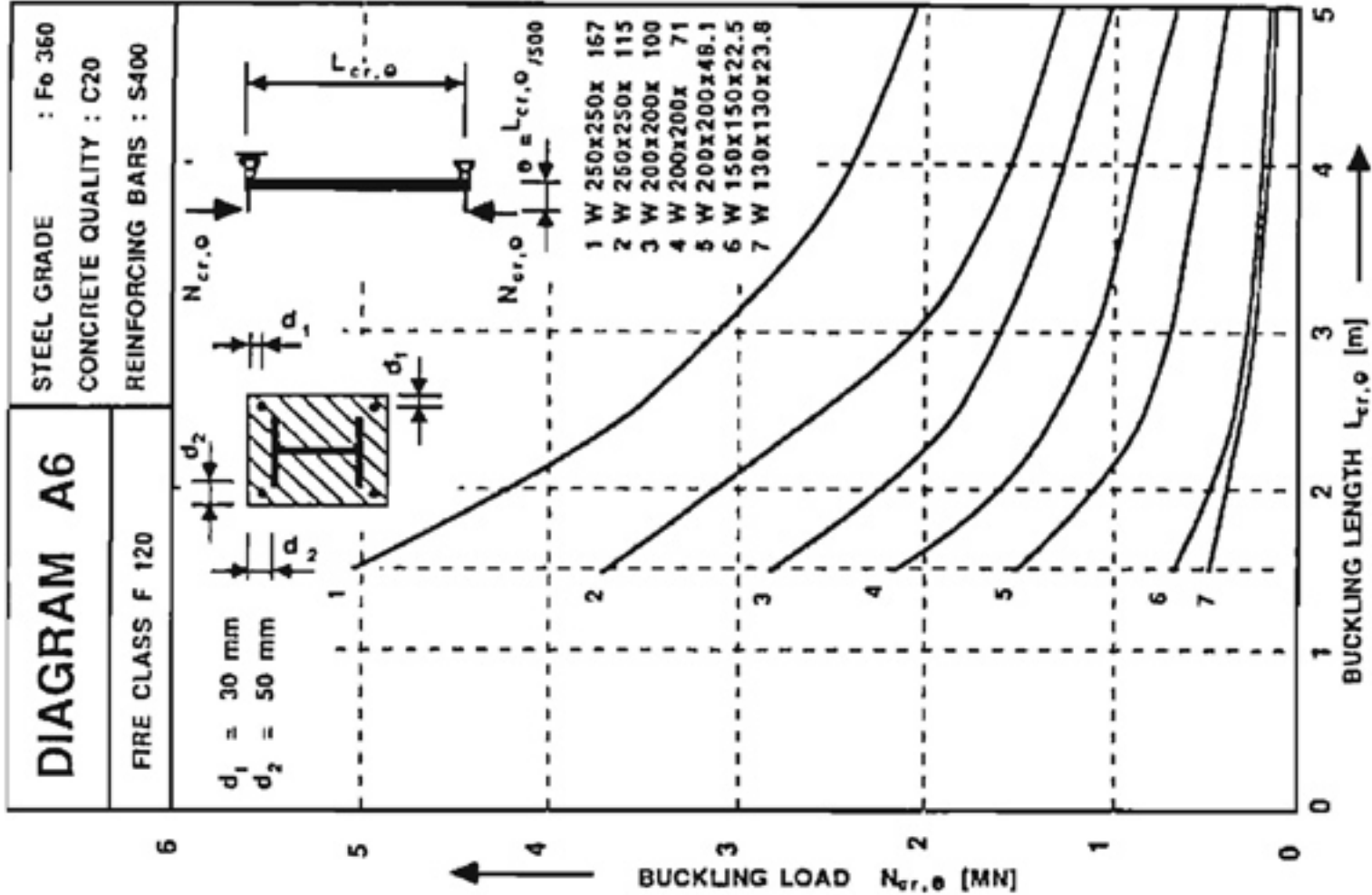
Survey on diagrams

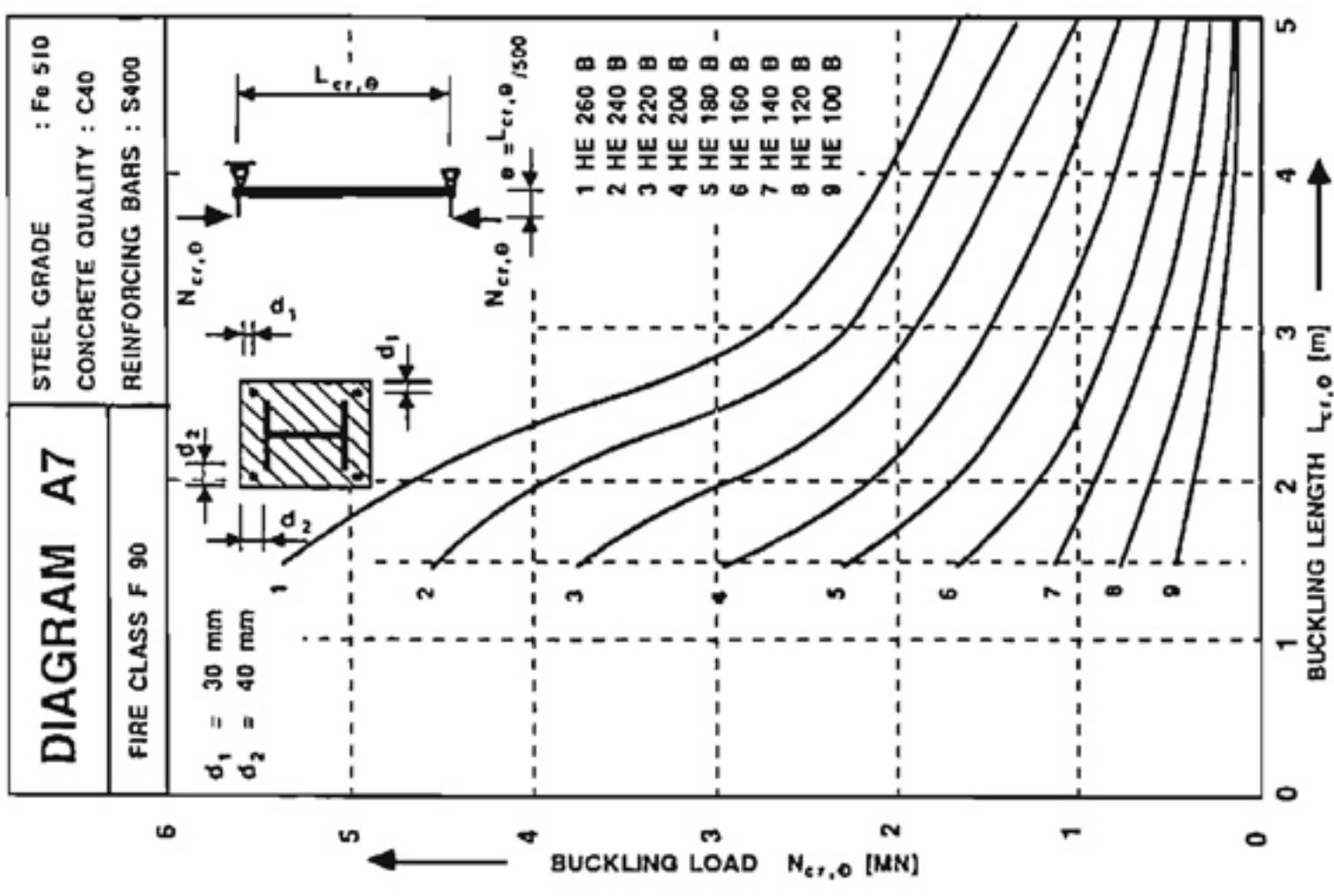
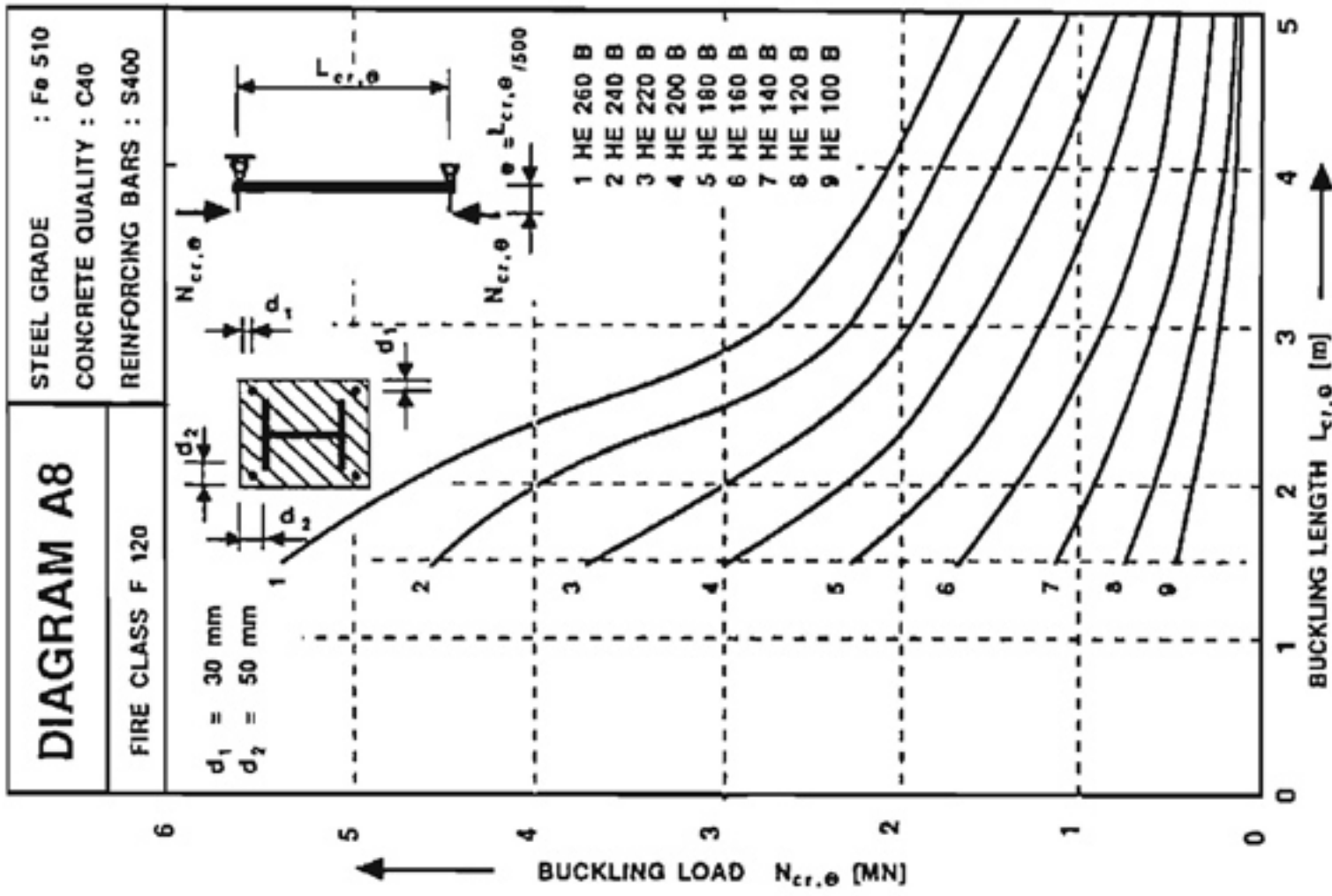
Fire class	Concrete quality	Profile type	Profile steel quality	Reinforcement	d ₁ [mm]	d ₂ [mm]	diagram No.
F90 F120	C20	HE-B	Fe 360	4 ø 12 mm S 400	30	40 50	A1 A2
F90 F120		HD				40 50	A3 A4
F90 F120		W				40 50	A5 A6
F90 F120	C40	HE-B	Fe 510			40 50	A7 A8
F90 F120		HD				40 50	A9 A10
F90 F120		W				40 50	A11 A12



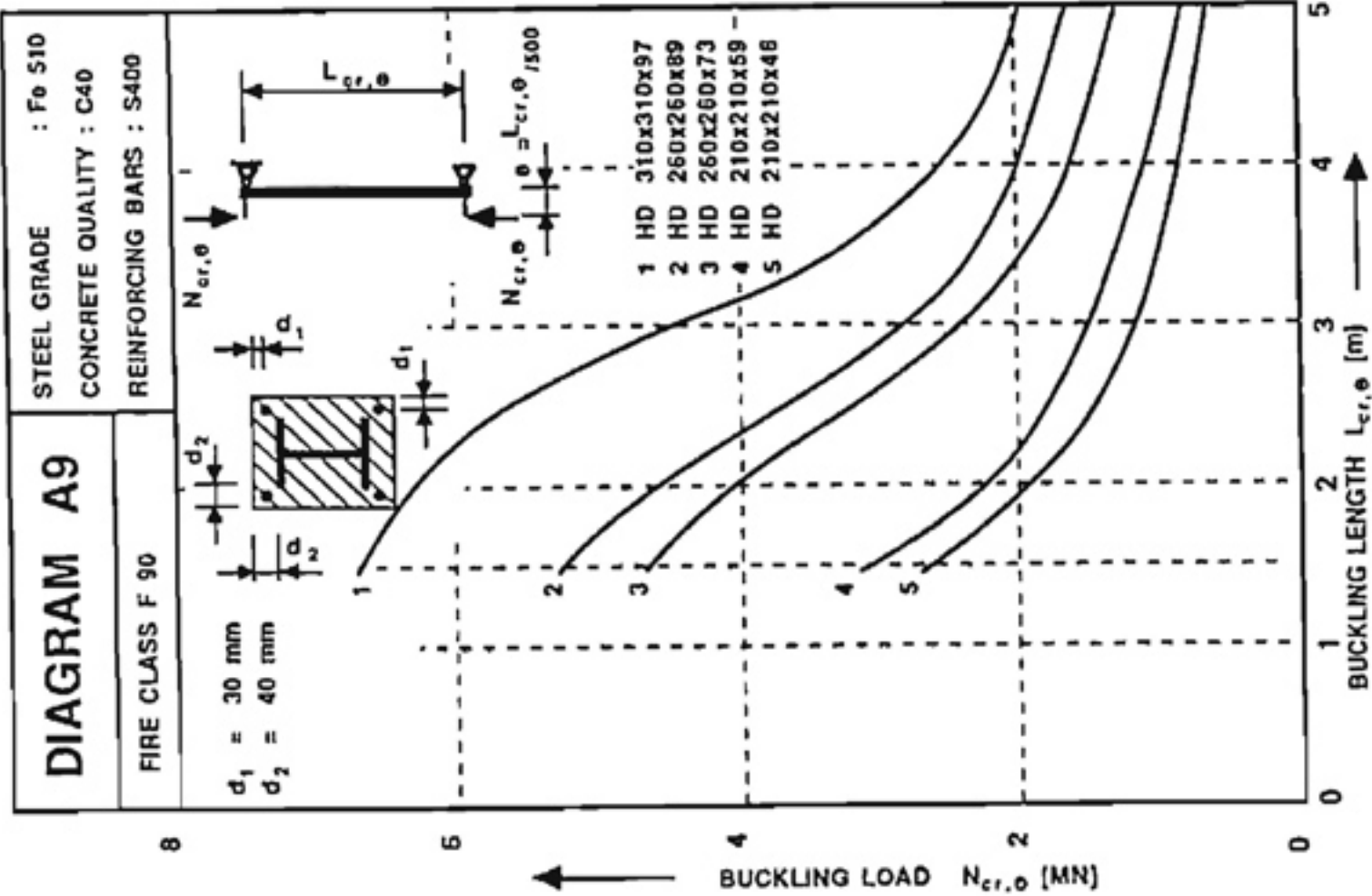
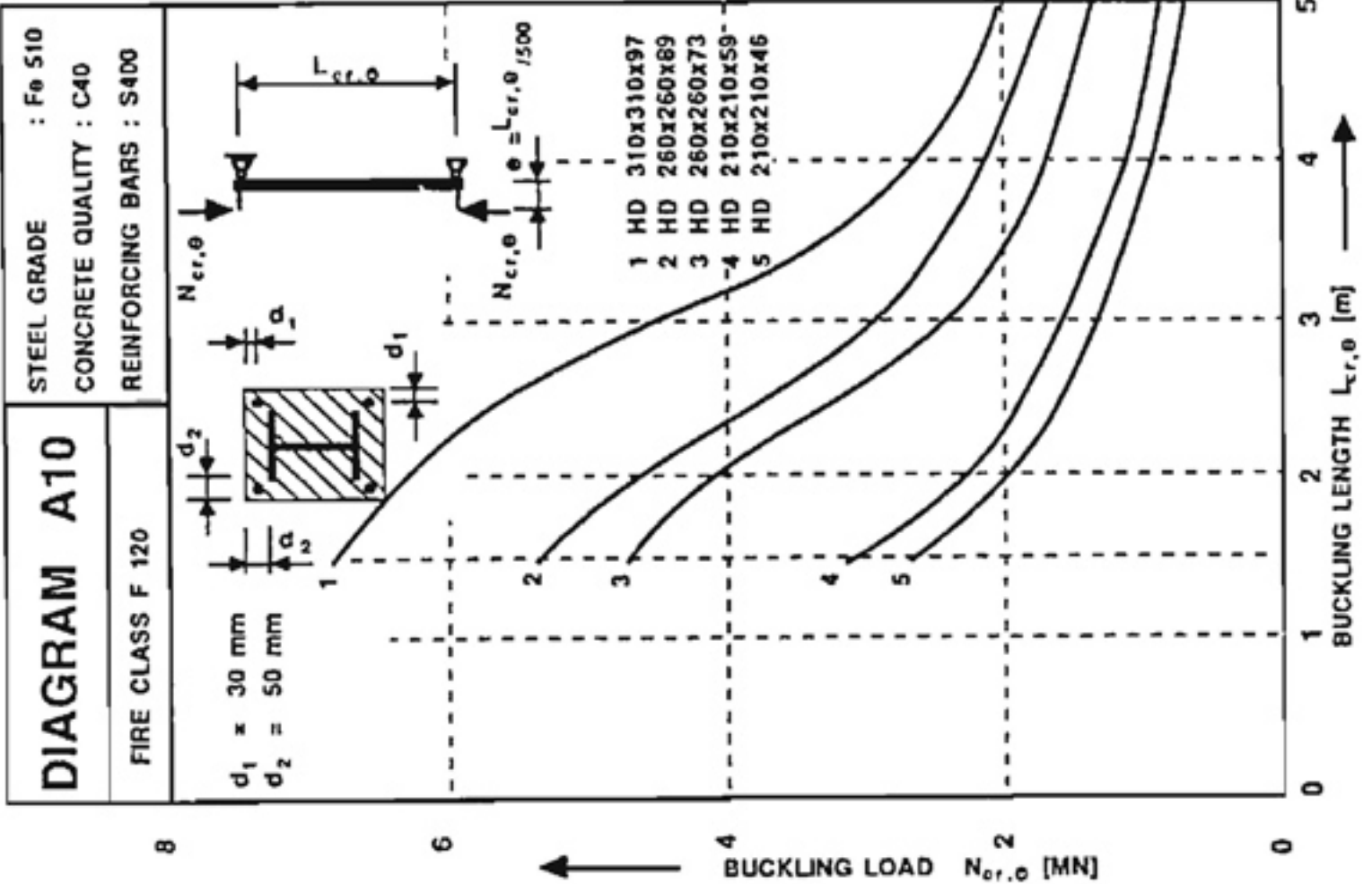


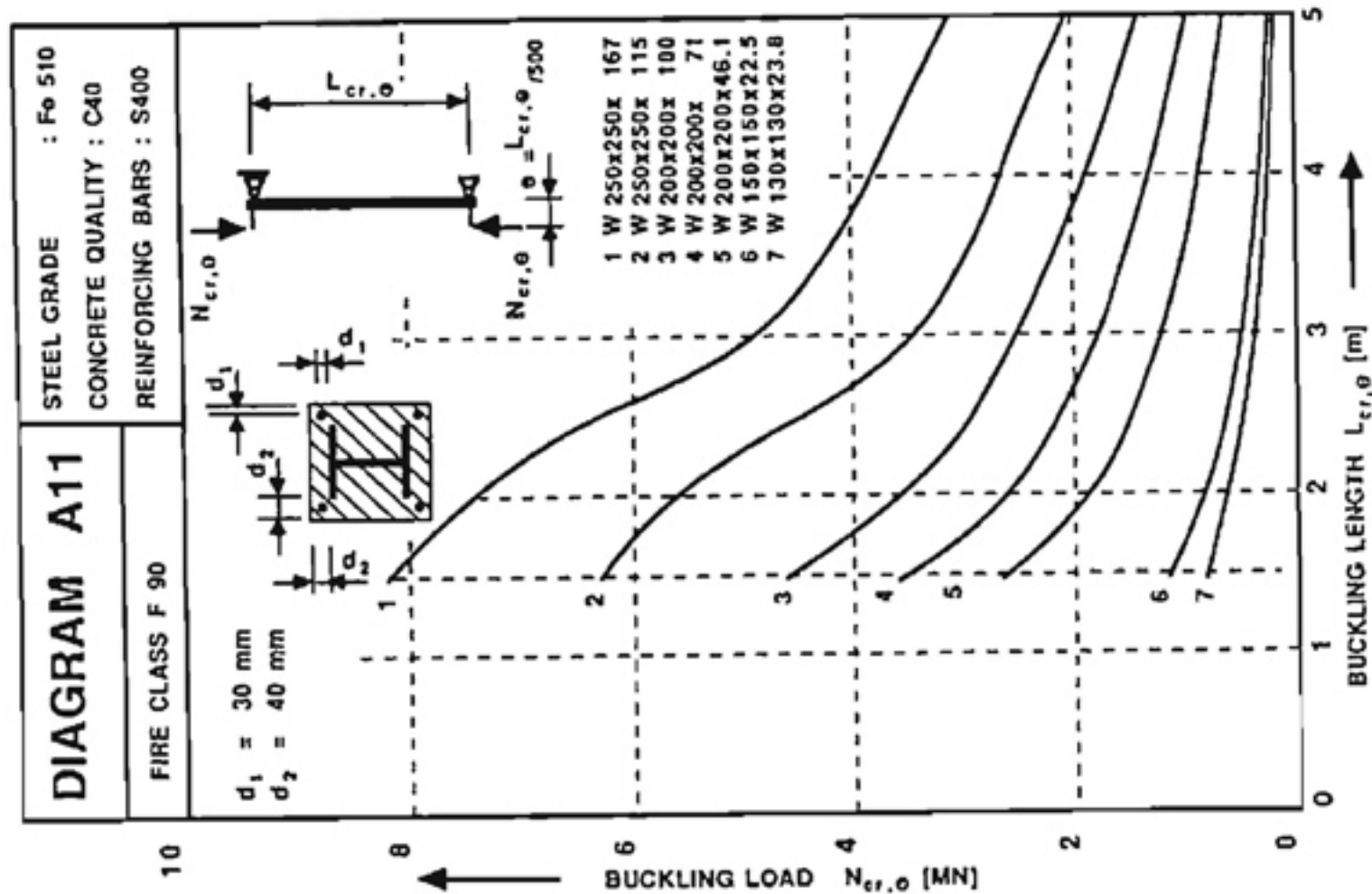
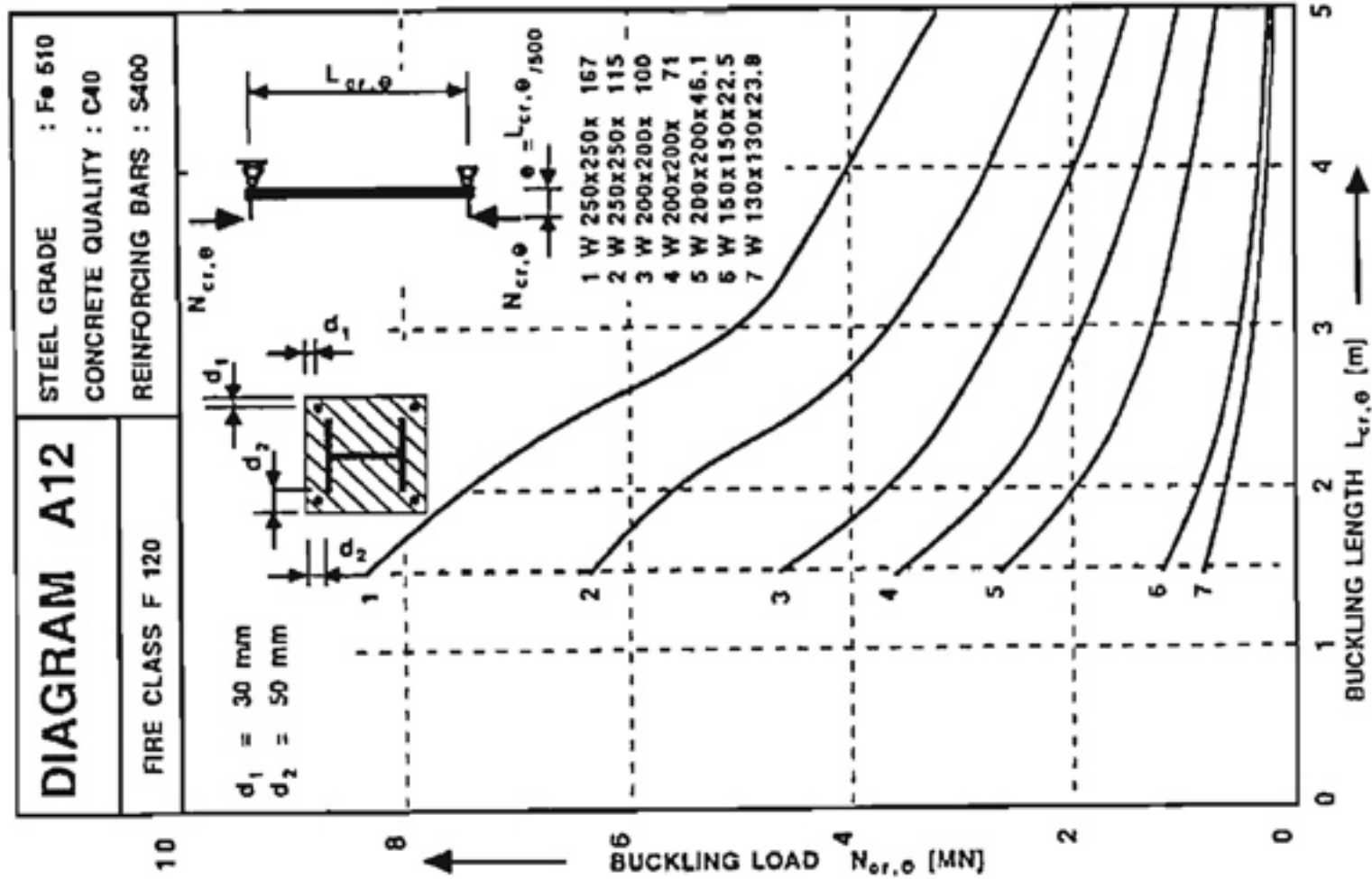
AXIAL LOAD DIAGRAMS FOR HD-PROFILES ENCASED IN CONCRETE





AXIAL LOAD DIAGRAMS FOR HE-PROFILES ENCASED IN CONCRETE



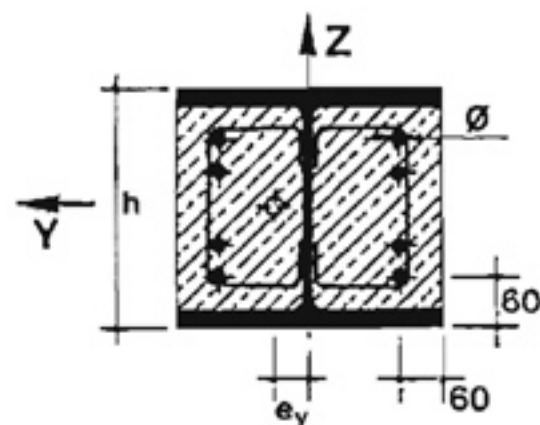


Appendix B: FIRE DESIGN DIAGRAMS FOR ROLLED PROFILES CONCRETED
BETWEEN THE FLANGES

The design graphs given on pages B.4 to B.35 are limited to the following material and geometrical properties:

Material properties:

Combinations: C20/Fe360/S500
 C30/Fe360/S500
 C30/Fe510/S500
 C40/Fe510/S500



Geometrical properties:

Reinforcement: $20 \text{ mm} \leq \varnothing \leq 25 \text{ mm}$,
 diameters and numbers chosen to achieve $\mu \sim 3 \%$.

Covers : see diagrams

Axial force eccentricity: $e_y = 5 \text{ mm}$

Buckling length: $0,5 \text{ m} \leq L_{cr,\theta} \leq 5,0 \text{ m}$

Profiles (1.9):

HE-AA } 240/280/320/360/400/450/500/550/600
 - A }
 - B } 650/700/800/900/1000

HP : 250.250.62/310.310.79/360.370.108/360.410.176

W : 530.315.150/610.325.155/610.325.217/690.360.217/690.360.265/
 760.380.257/760.380.314/840.400.299/840.400.359/920.420.342/
 920.420.417/1000.450.286/1000.450.399/1000.450.444/1000.450.488

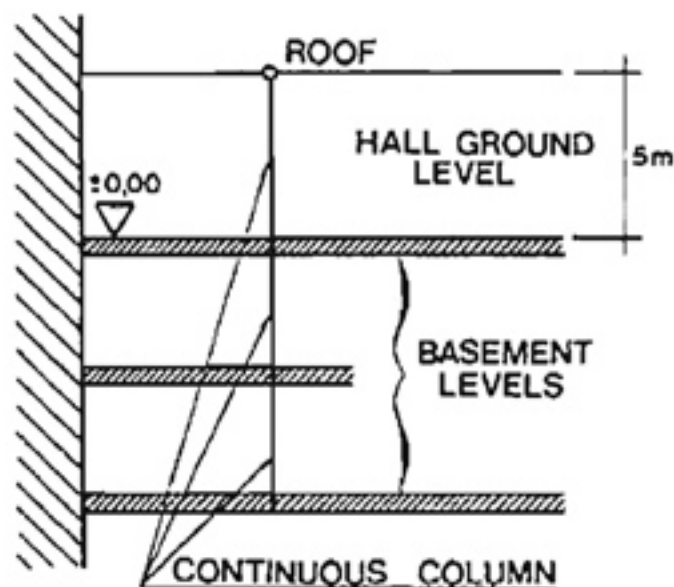
The rolled profiles considered in the diagrams correspond to the European Standard Euronorm 53-62 and American Standard ASTM A6-88 (1.9).

All loads have been established on the basis of the limited calculation model according to chapter 4.2, for buckling around the weak axis Z, including a maximum eccentricity of 5 mm, which corresponds either to practical load applications or to real column geometrical imperfections.

The diagrams given on pages B.4 to B.35 correspond to the FIRE CLASSES F60, F90 and F120 for steel and concrete qualities according to EURONORM 25 and 80 respectively to EUROCODE 2. The water content is expected to represent 2 to 4 % of the weight of concrete.

Example 1:

CHECKING FOR STORAGE HALL COLUMNS; F90



EXISTING CROSS SECTION:

HE 240 AA	Fe510
4 ø 20	S 500
concrete	C 40

EXISTING AXIAL DESIGN LOAD: $N = 650 \text{ kN}$

HALL COLUMNS $\equiv$ TOP FLOOR COLUMNS,
(see chapter 2.4.4):

$$L_{cr,\theta} = 0,7 \cdot L = 3,5 \text{ m}$$

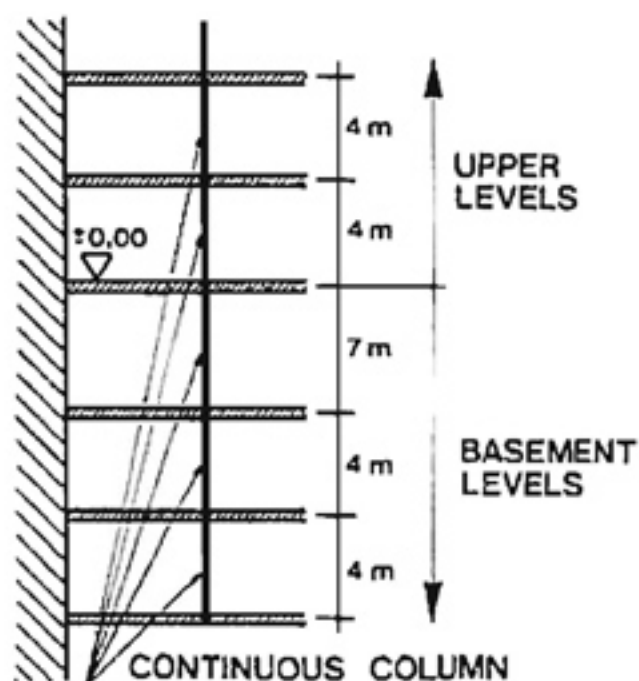
This composite section together with the buckling length of 3,5 m allows one to find the axial design load for the fire class F90 in diagram B.38:

$$N_{cr,90} = 0,72 \text{ MN} = 720 \text{ kN} > \text{existing axial design load} = 650 \text{ kN}.$$

Conclusion: For the design load of 650 kN, hall columns can be classified F90

Example 2:

DESIGN OF HIGHLY LOADED BUILDING BASEMENT LEVEL; F120



EXISTING CONDITIONS: $L = 7 \text{ m}$

$$N = 8000 \text{ kN}$$

CONTINUOUS COLUMN (see chapter 2.4.4):

$$L_{cr,\theta} = 0,5 \cdot L \\ = 3,5 \text{ m}$$

Knowing this buckling length as well as the design load, the following F120 diagrams give a possible cross section:

- in diagram B12, with a Fe360 profile steel grade and a C20 concrete quality, the steel profile to be chosen is W 1000x450x399 reinforced by 20 rebars of 25 mm diameter (S 500).

- in diagram B48, with a Fe510 profile steel grade and a C40 concrete quality, the steel profile to be chosen is W840x400x299 reinforced by 16 rebars of 25 mm diameter (S 500).

Normally the last solution with the lighter steel profile will be chosen, provided the Fe510 profile steel grade ($\sigma_y = 355 \text{ N/mm}^2$) is available in due time and the C40 concrete quality ($\beta_{cy} = 40 \text{ N/mm}^2$) can be guaranteed.

Survey on diagrams

Fire class	Concrete quality	Profile type	Profile steel quality	Reinforcement	diagram no.
F60 F90 F120	C20	HE-AA	Fe 360	S500; ø see diagrams	B1
F60 F90 F120		HE-A			B2
F60 F90 F120		HE-B			B3
F60 F90 F120		HP/W			B4
F60 F90 F120		HE-AA			B5
F60 F90 F120		HE-A			B6
F60 F90 F120		HE-B			B7
F60 F90 F120		HP/W			B8
F60 F90 F120		HE-AA			B9
F60 F90 F120		HE-A			B10
F60 F90 F120		HE-B			B11
F60 F90 F120		HP/W			B12
F60 F90 F120	C30	HE-AA			B13
F60 F90 F120		HE-A			B14
F60 F90 F120		HE-B			B15
F60 F90 F120		HP/W			B16
F60 F90 F120		HE-AA			B17
F60 F90 F120		HE-A			B18
F60 F90 F120		HE-B			B19
F60 F90 F120		HP/W			B20
F60 F90 F120		HE-AA			B21
F60 F90 F120		HE-A			B22
F60 F90 F120		HE-B			B23
F60 F90 F120		HP/W			B24
F60 F90 F120	C30	HE-AA	Fe 510	S500; ø see diagrams	B25
F60 F90 F120		HE-A			B26
F60 F90 F120		HE-B			B27
F60 F90 F120		HP/W			B28
F60 F90 F120		HE-AA			B29
F60 F90 F120		HE-A			B30
F60 F90 F120		HE-B			B31
F60 F90 F120		HP/W			B32
F60 F90 F120		HE-AA			B33
F60 F90 F120		HE-A			B34
F60 F90 F120		HE-B			B35
F60 F90 F120		HP/W			B36
F60 F90 F120	C40	HE-AA			B37
F60 F90 F120		HE-A			B38
F60 F90 F120		HE-B			B39
F60 F90 F120		HP/W			B40
F60 F90 F120		HE-AA			B41
F60 F90 F120		HE-A			B42
F60 F90 F120		HE-B			B43
F60 F90 F120		HP/W			B44
F60 F90 F120		HE-AA			B45
F60 F90 F120		HE-A			B46
F60 F90 F120		HE-B			B47
F60 F90 F120		HP/W			B48

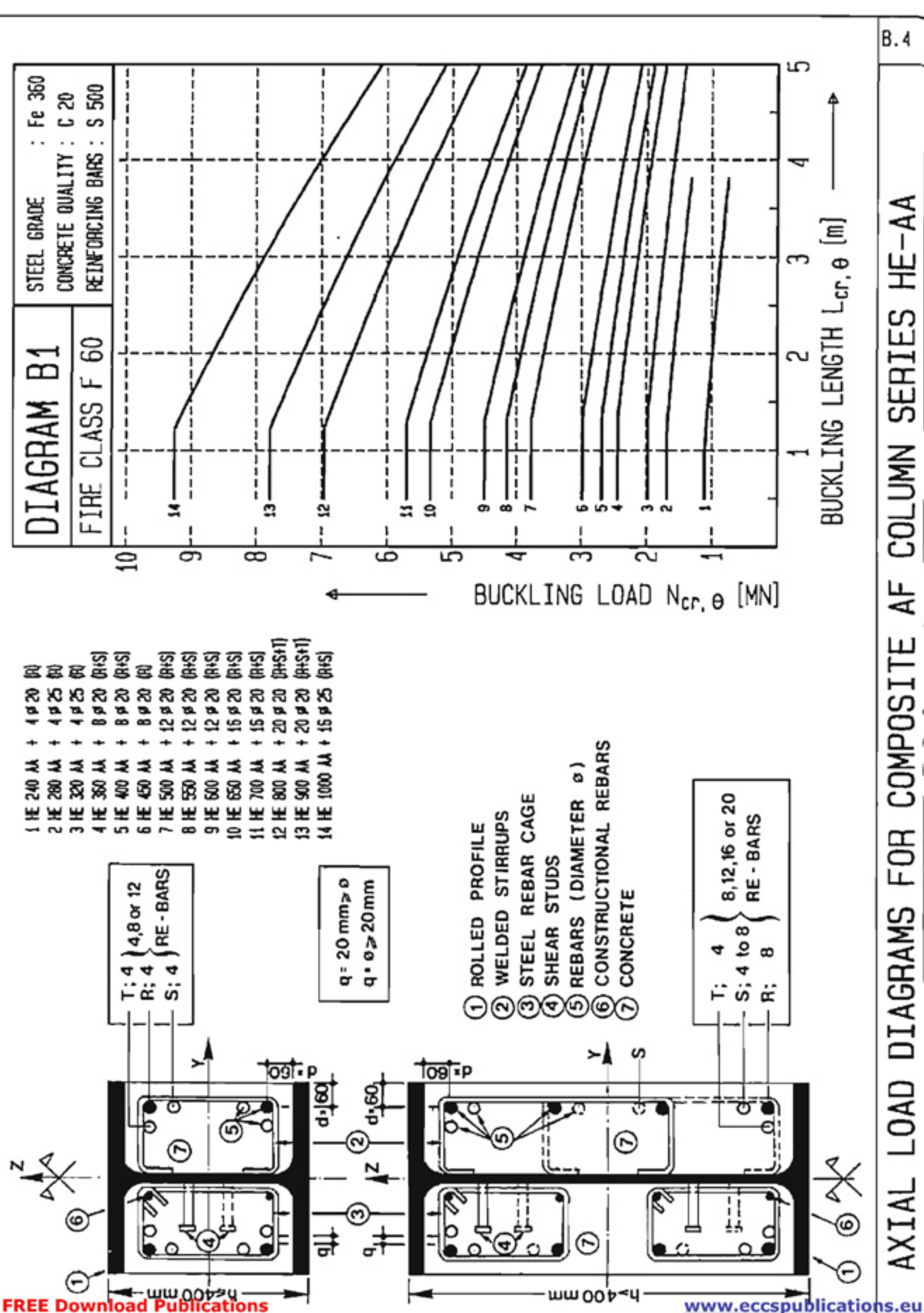


DIAGRAM B3
FIRE CLASS F 120

STEEL GRADE : Fe 360
CONCRETE QUALITY : C 20
REINFORCING BARS : S 500

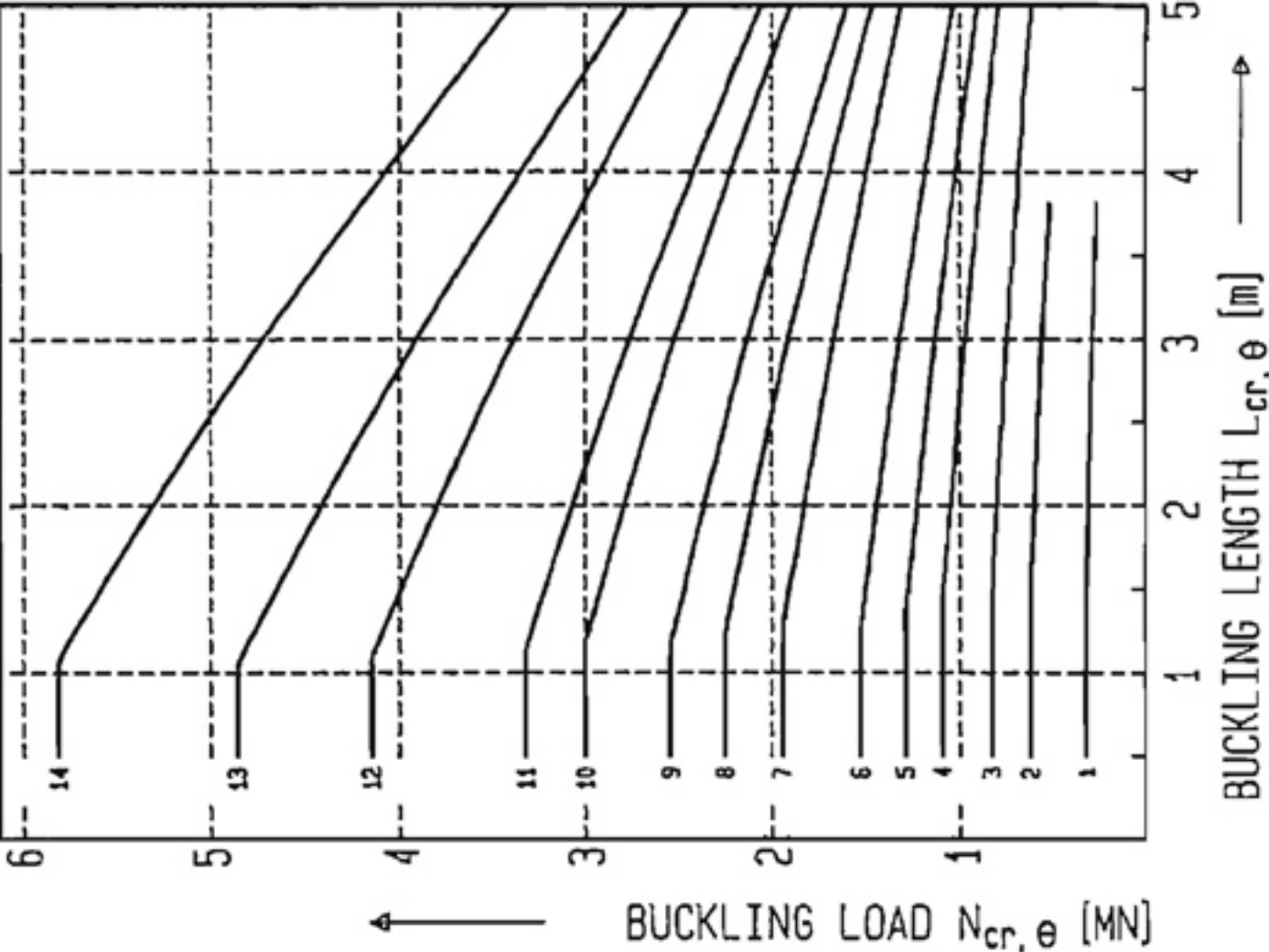
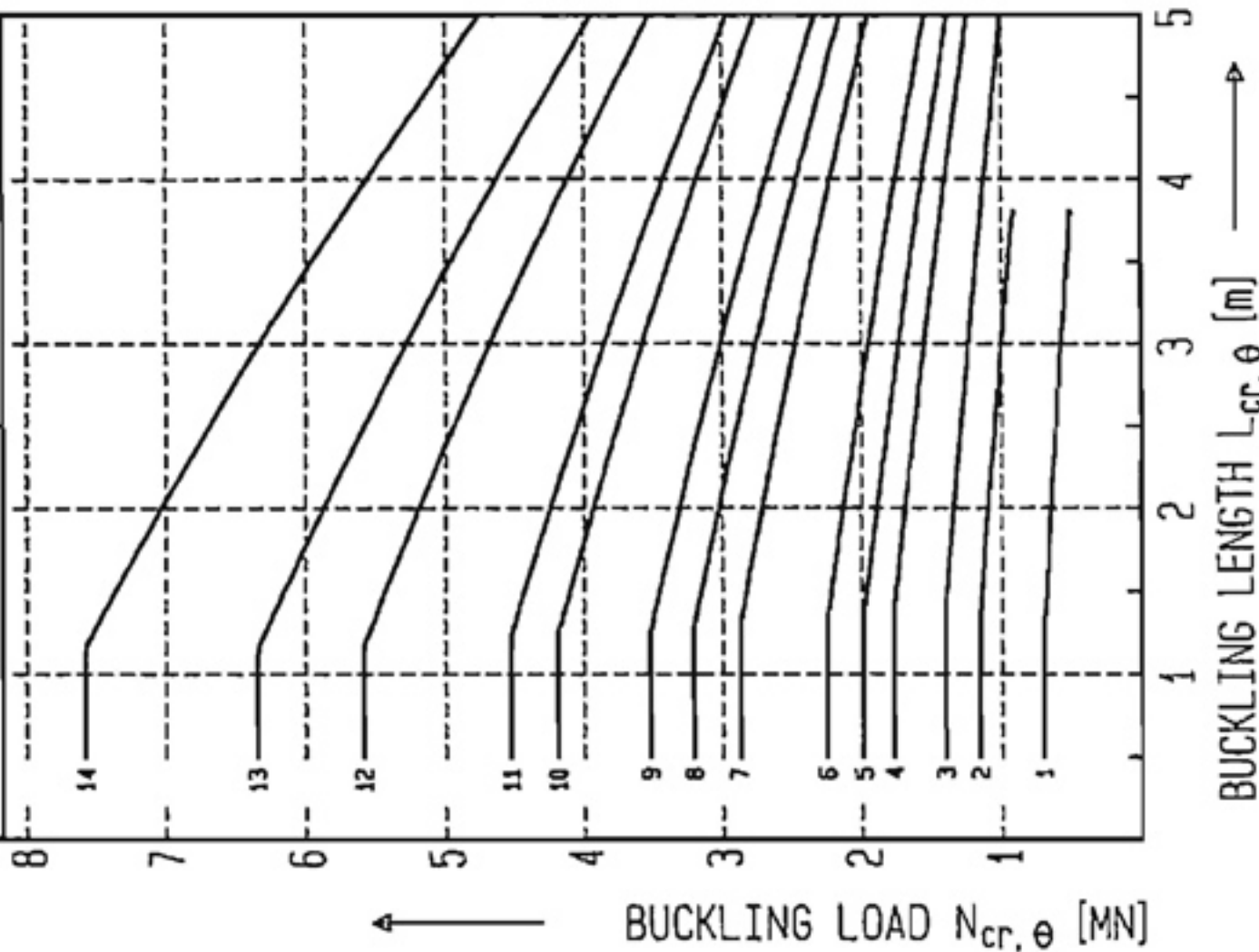
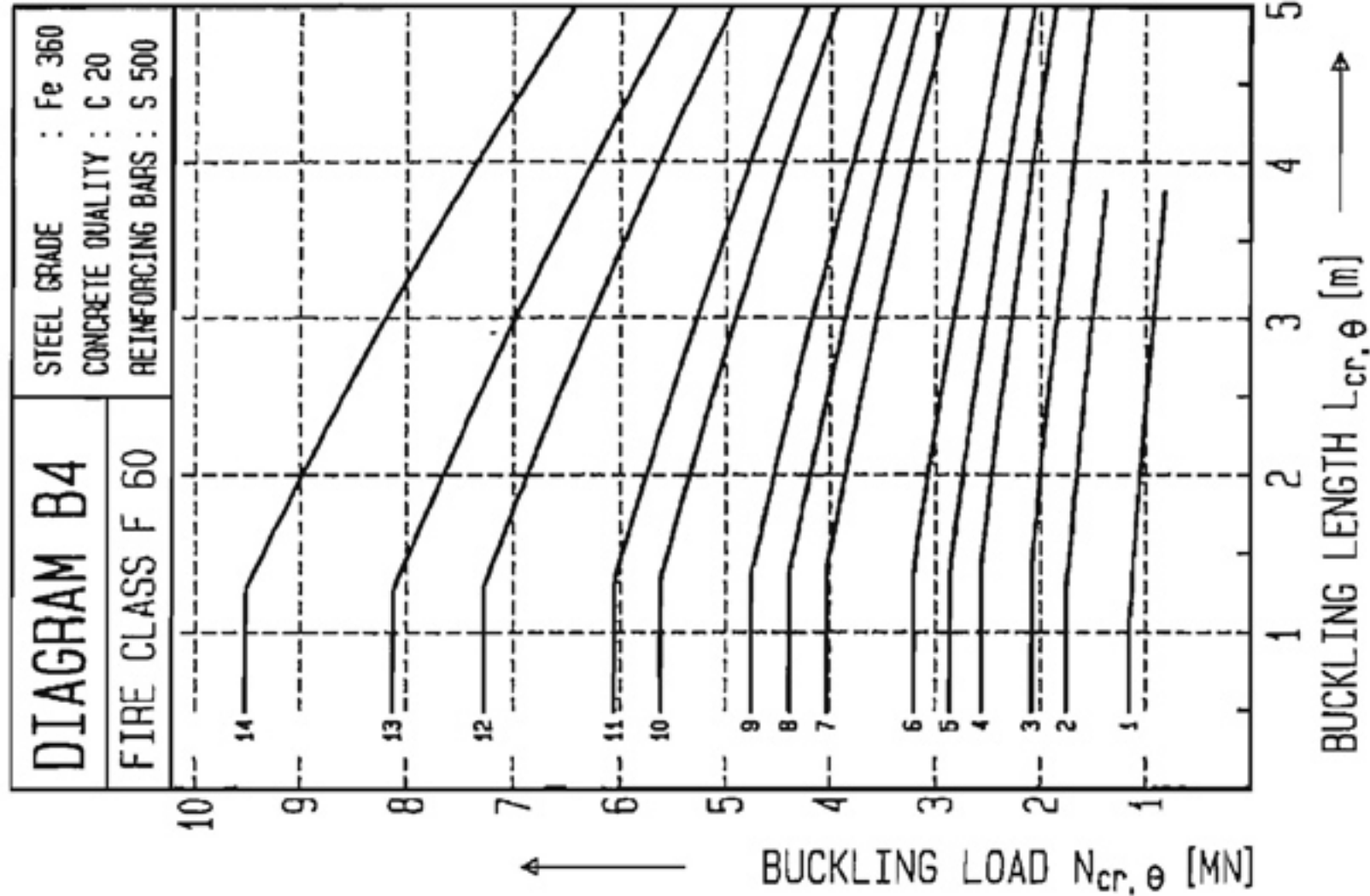
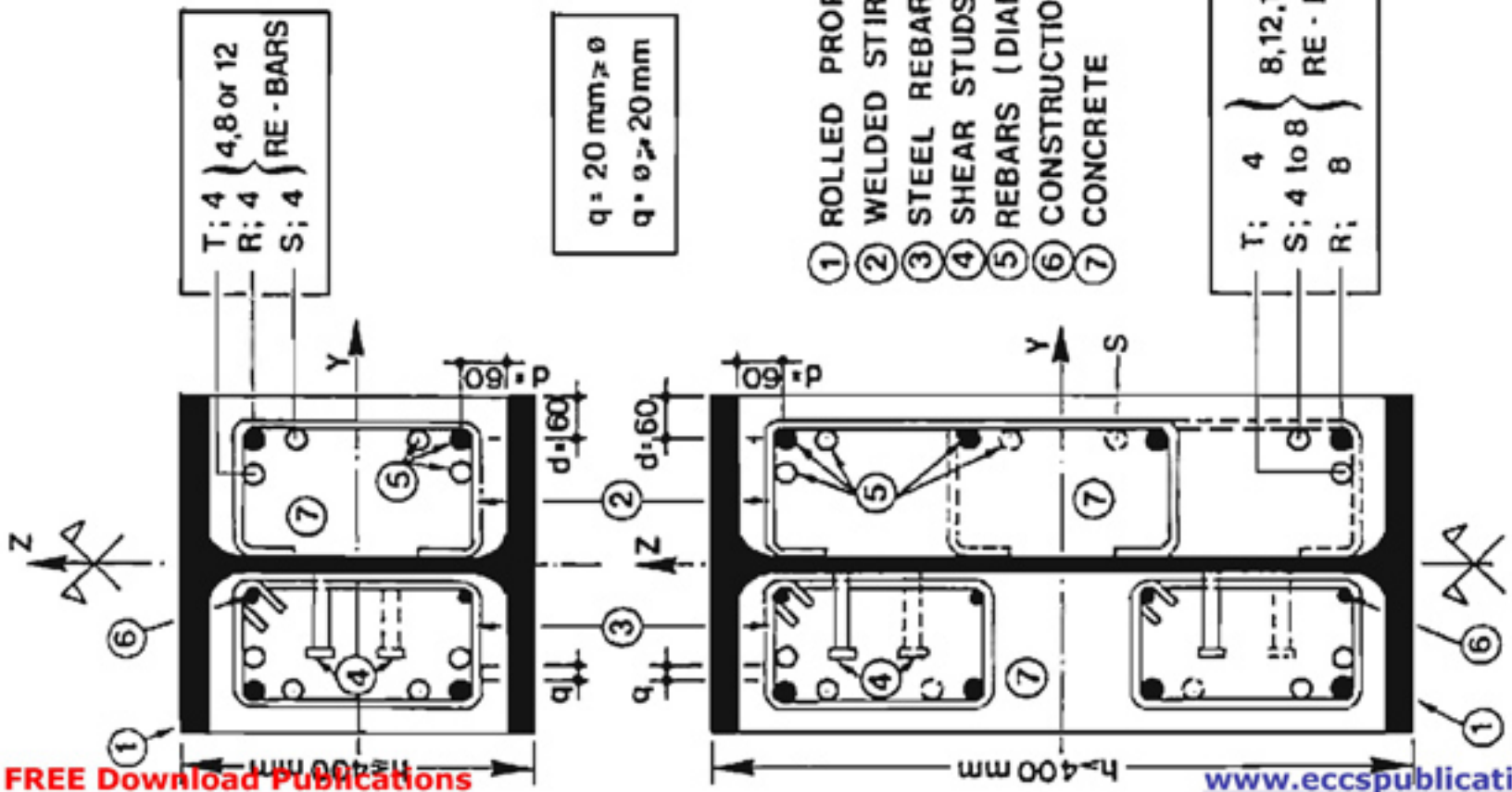


DIAGRAM B2
FIRE CLASS F 90

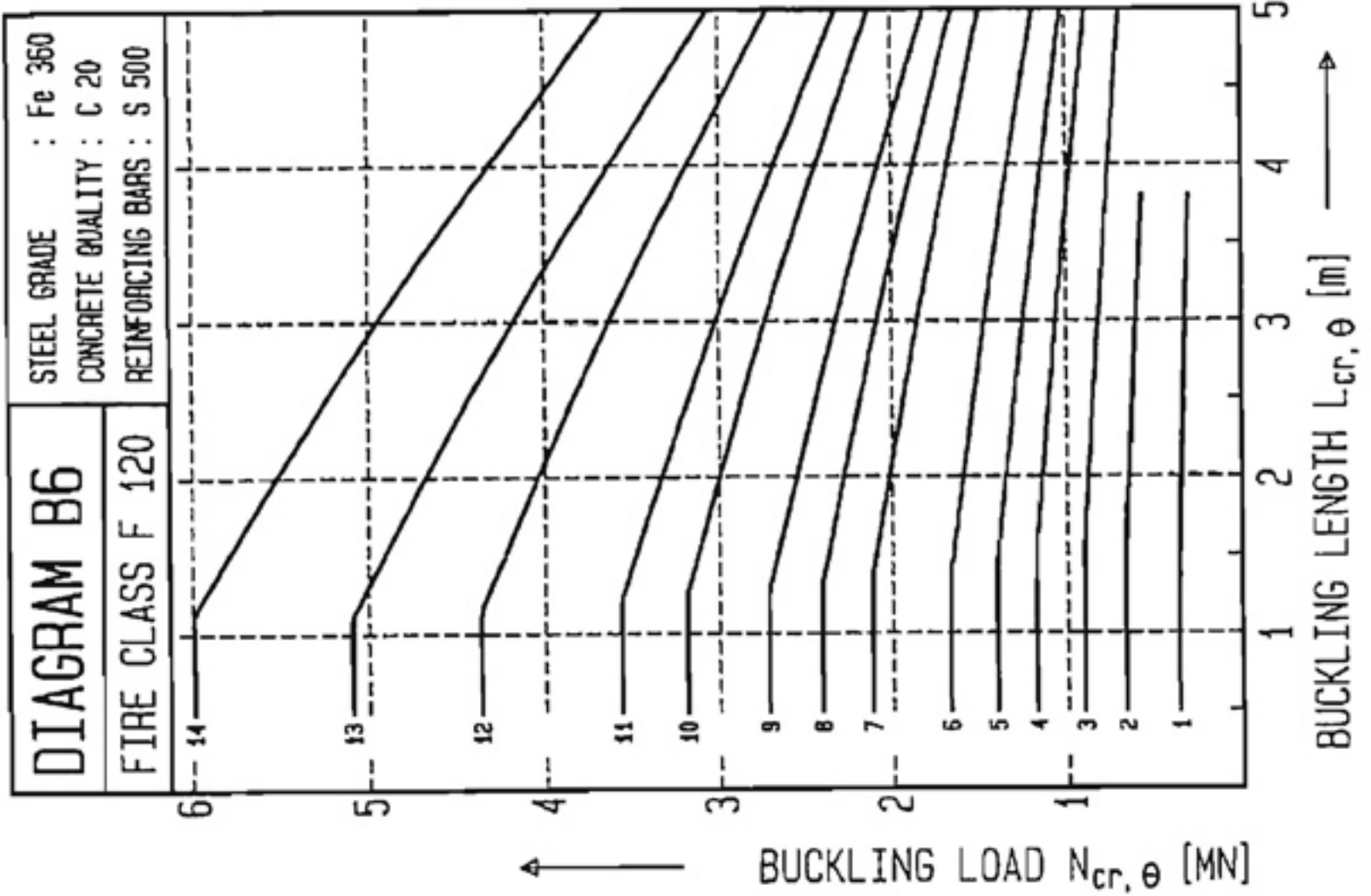
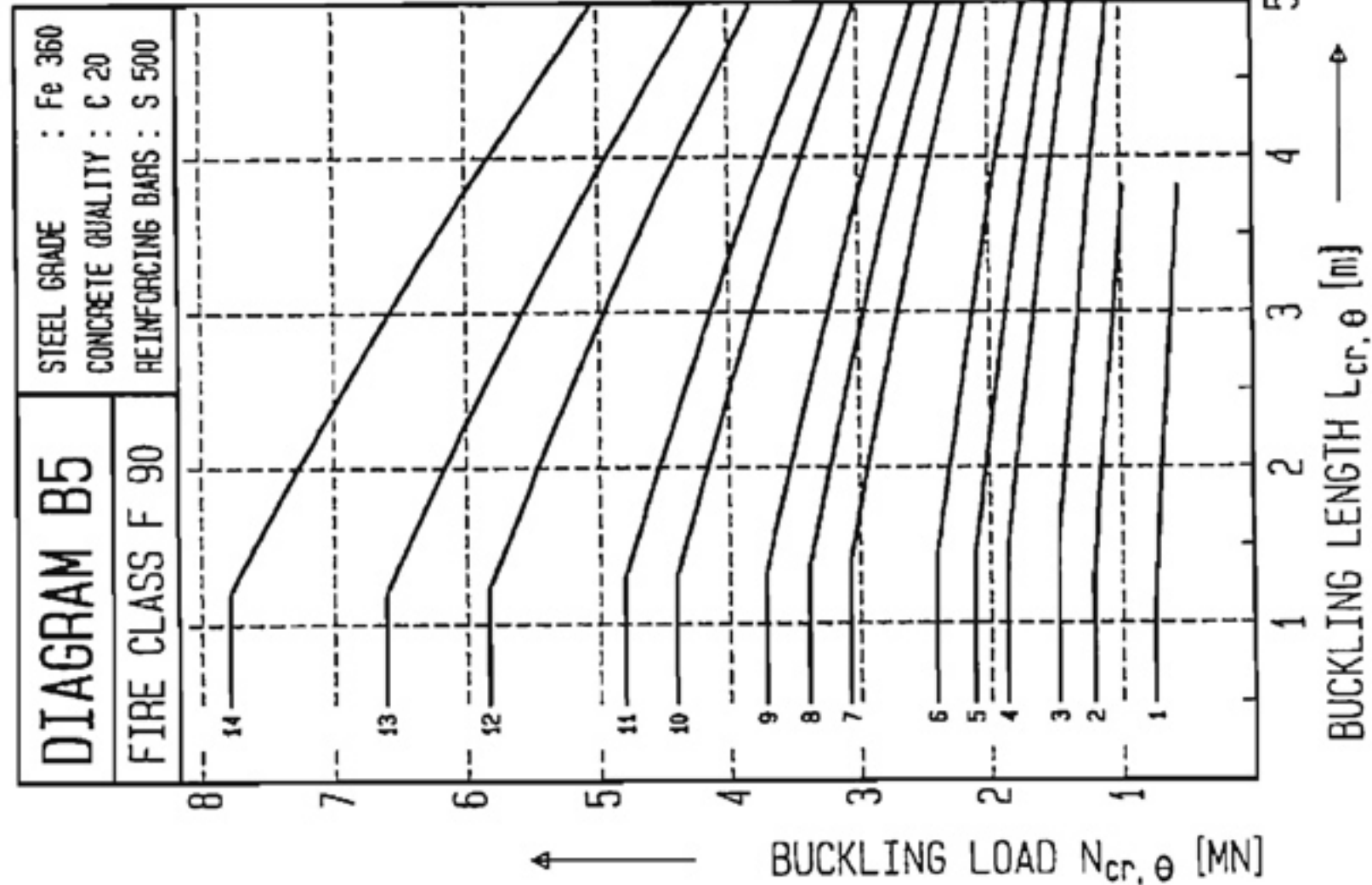
STEEL GRADE : Fe 360
CONCRETE QUALITY : C 20
REINFORCING BARS : S 500



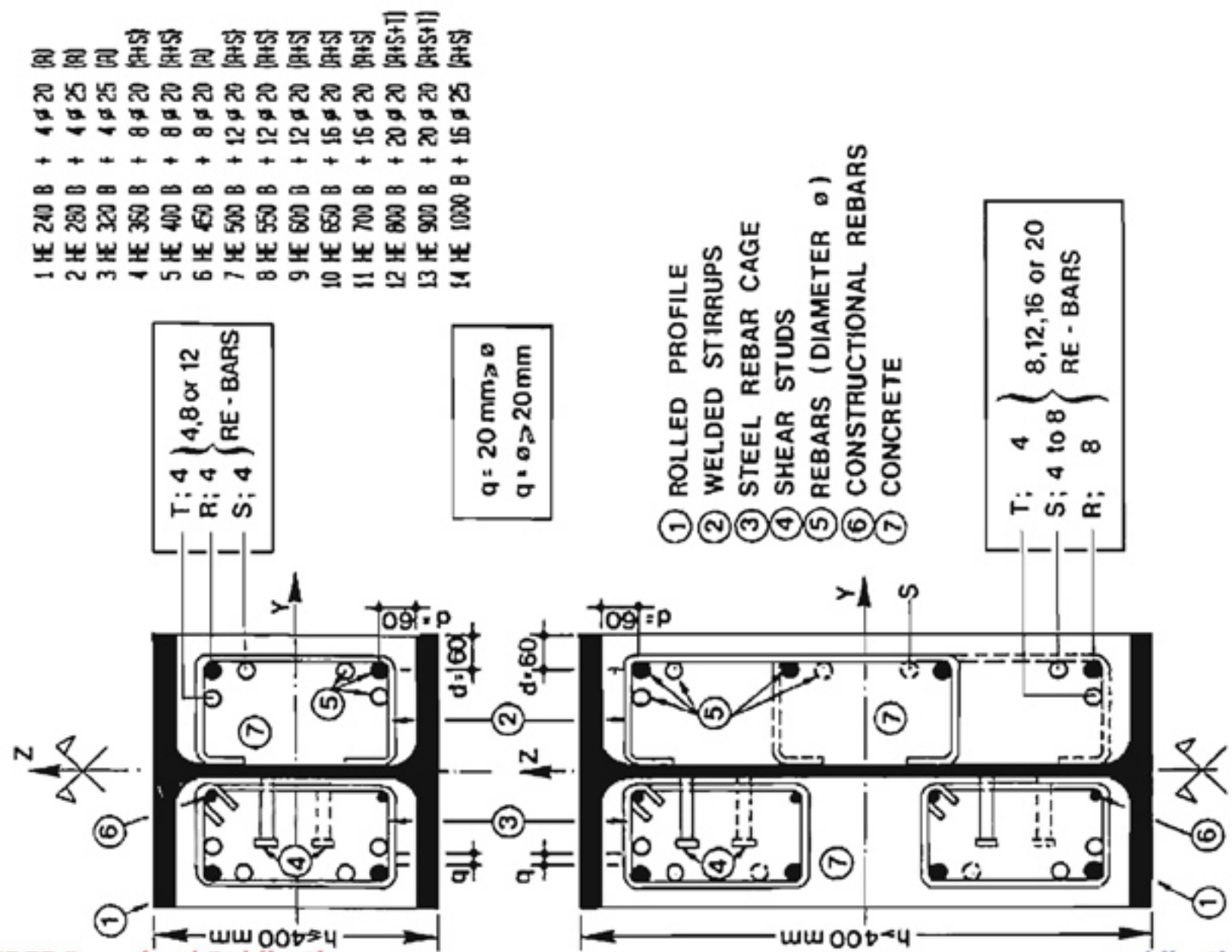
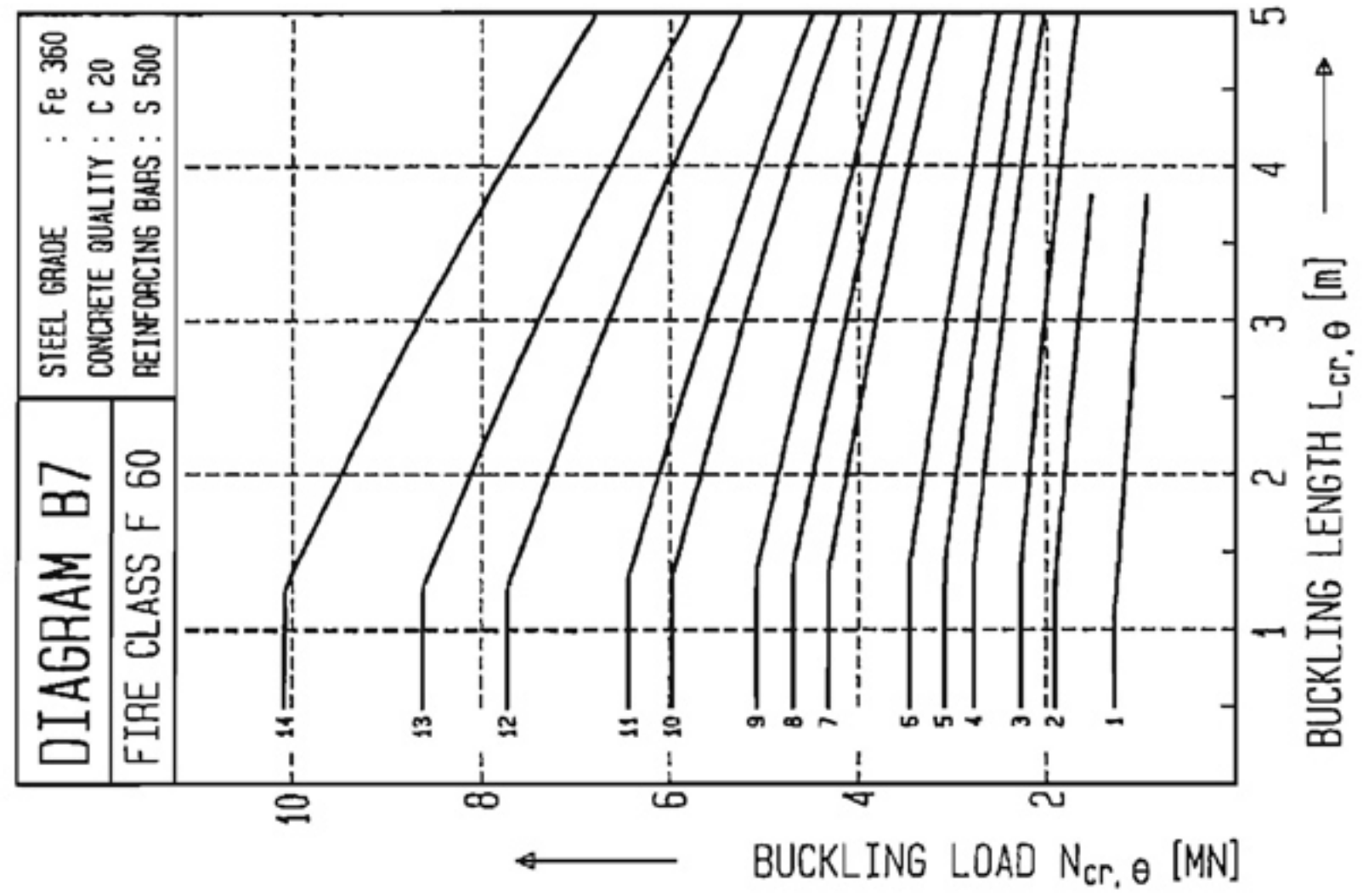
- 1 HE 240 A + 4 ϕ 20 (R)
- 2 HE 280 A + 4 ϕ 25 (R)
- 3 HE 320 A + 4 ϕ 25 (R)
- 4 HE 360 A + 8 ϕ 20 (R+S)
- 5 HE 400 A + 8 ϕ 20 (R+S)
- 6 HE 450 A + 8 ϕ 20 (R)
- 7 HE 500 A + 12 ϕ 20 (R+S)
- 8 HE 550 A + 12 ϕ 20 (R+S)
- 9 HE 600 A + 12 ϕ 20 (R+S)
- 10 HE 650 A + 16 ϕ 20 (R+S)
- 11 HE 700 A + 16 ϕ 20 (R+S)
- 12 HE 800 A + 20 ϕ 20 (R+S+T)
- 13 HE 900 A + 20 ϕ 20 (R+S+T)
- 14 HE 1000 A + 16 ϕ 25 (R+S)



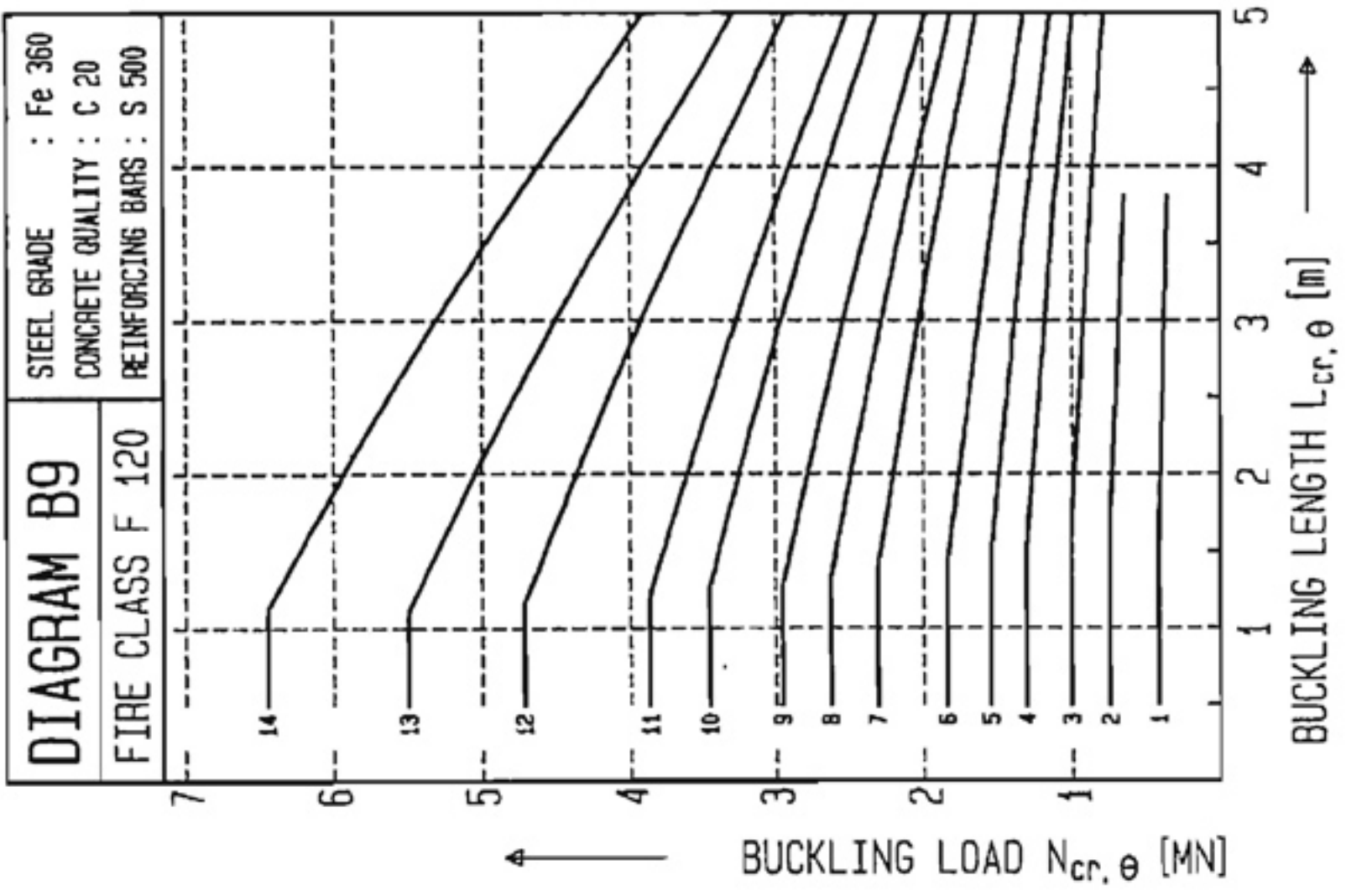
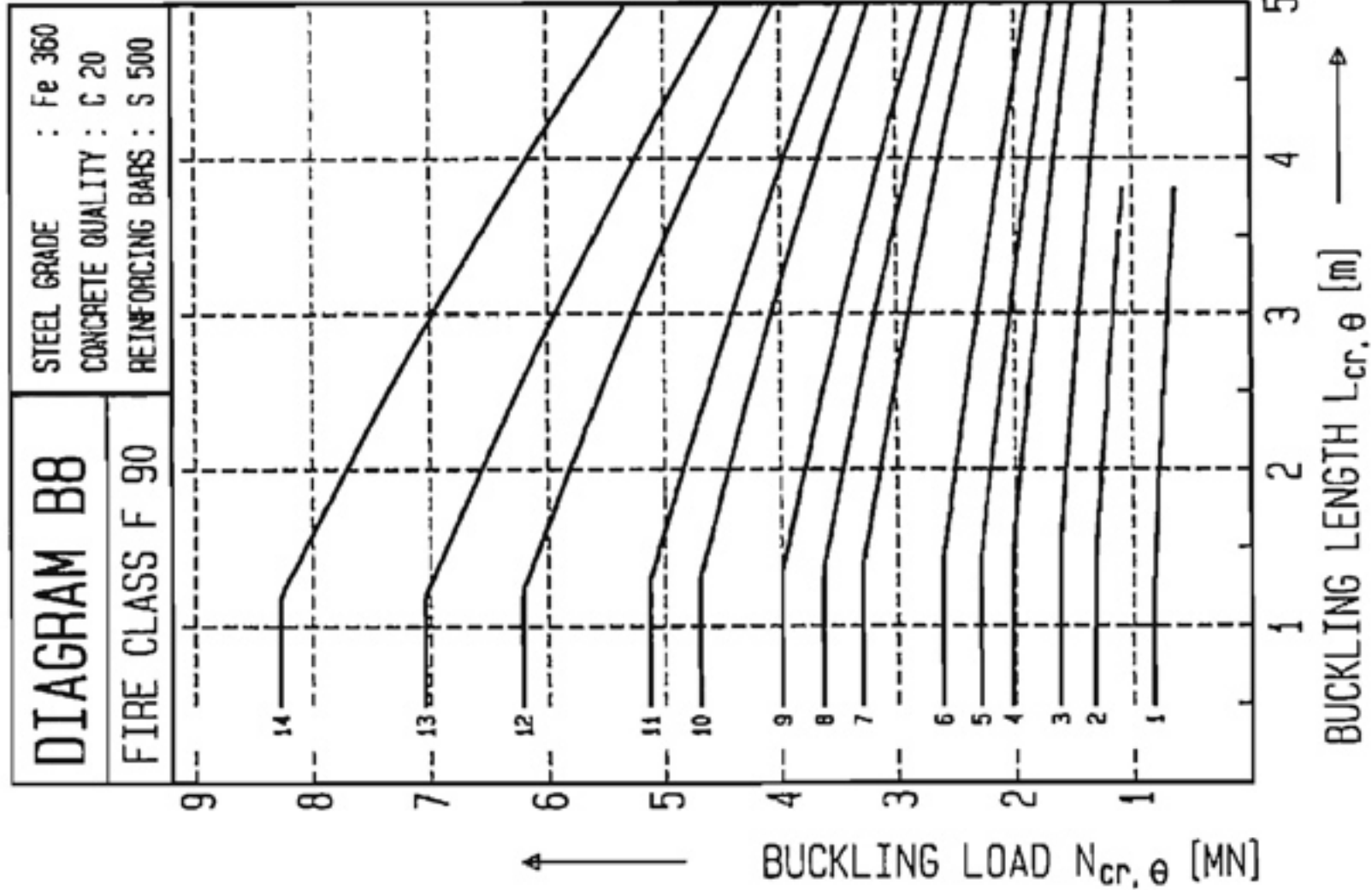
AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-A



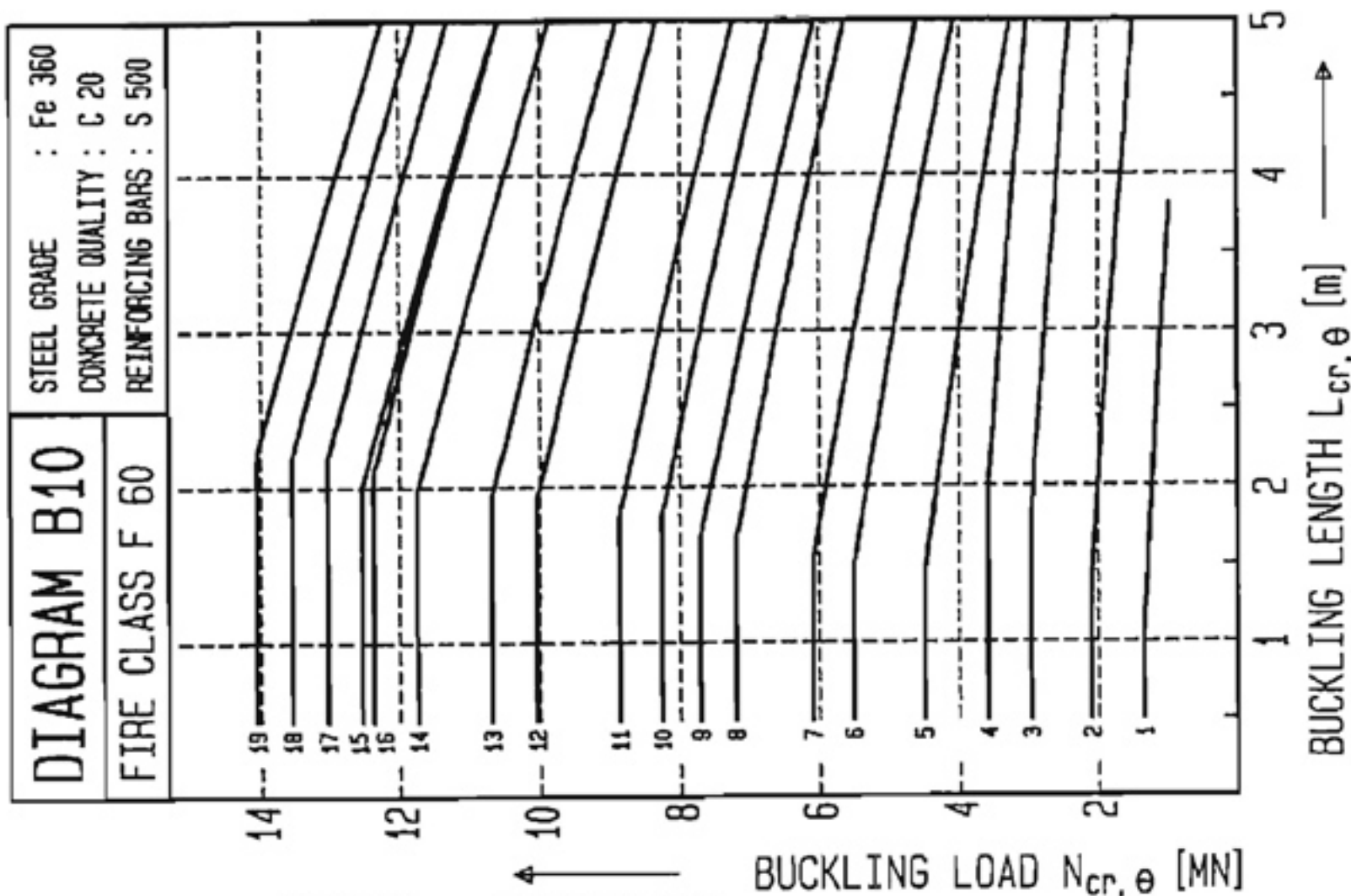
AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-A



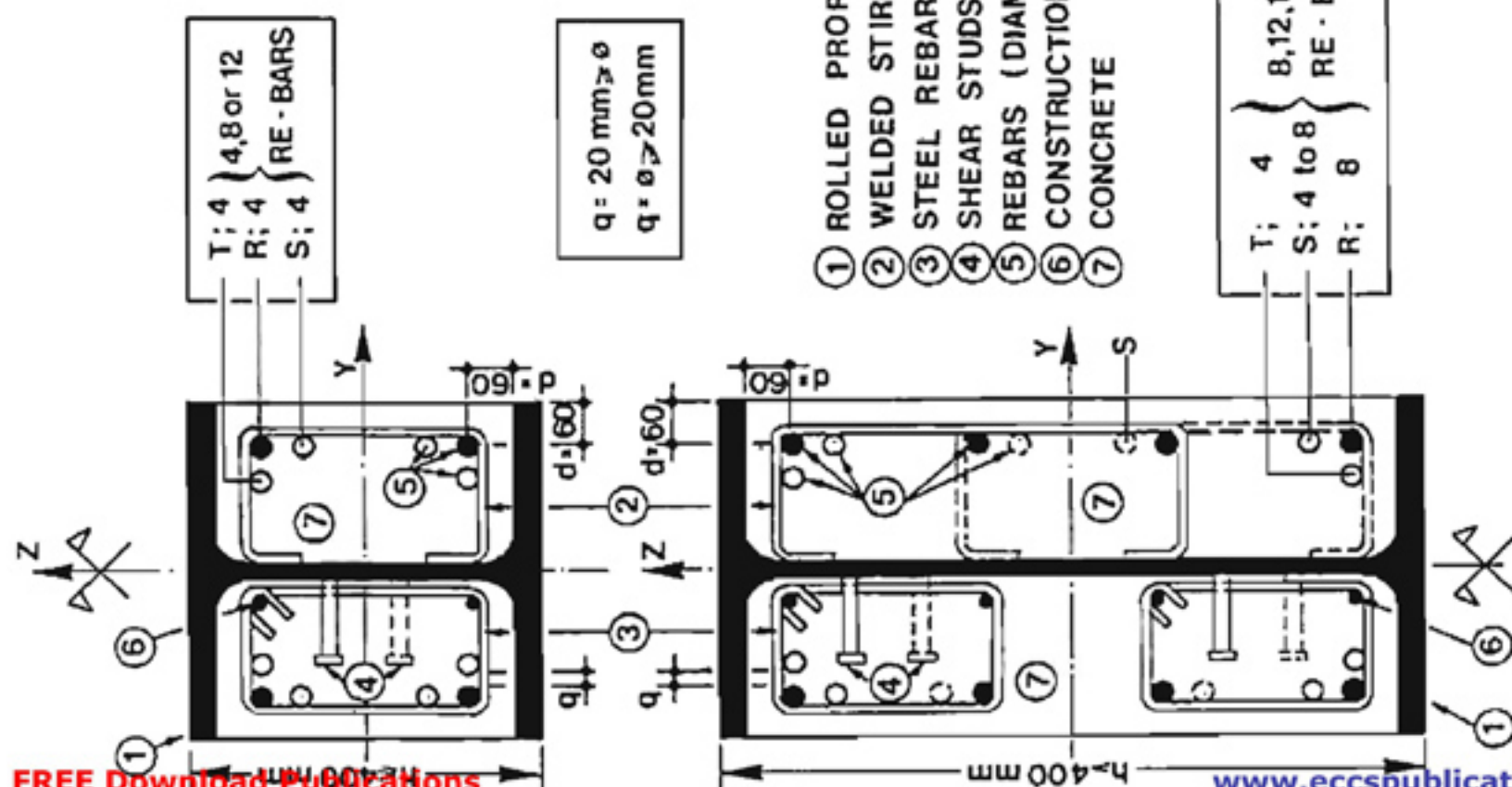
AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-B

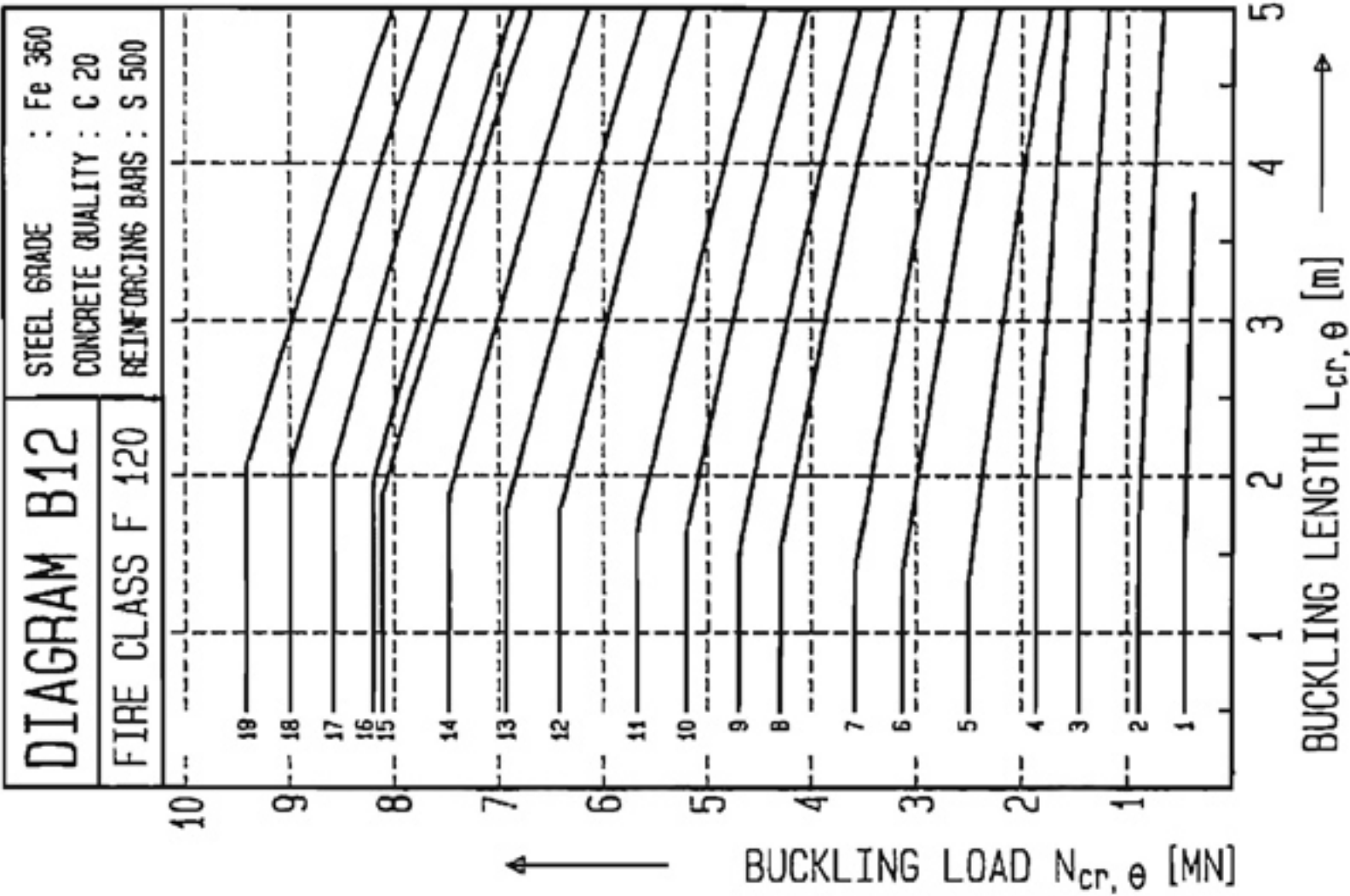
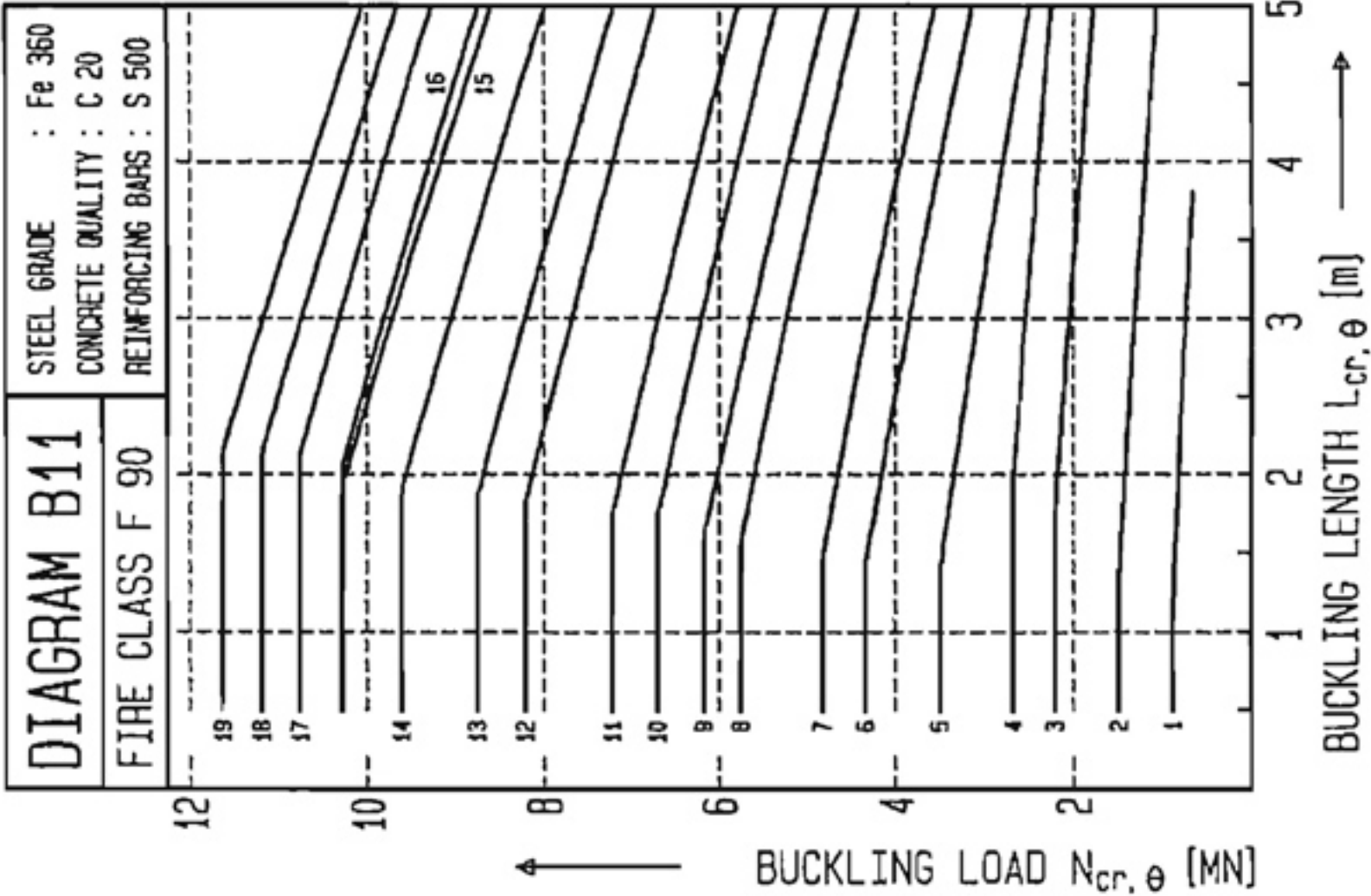


AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-B



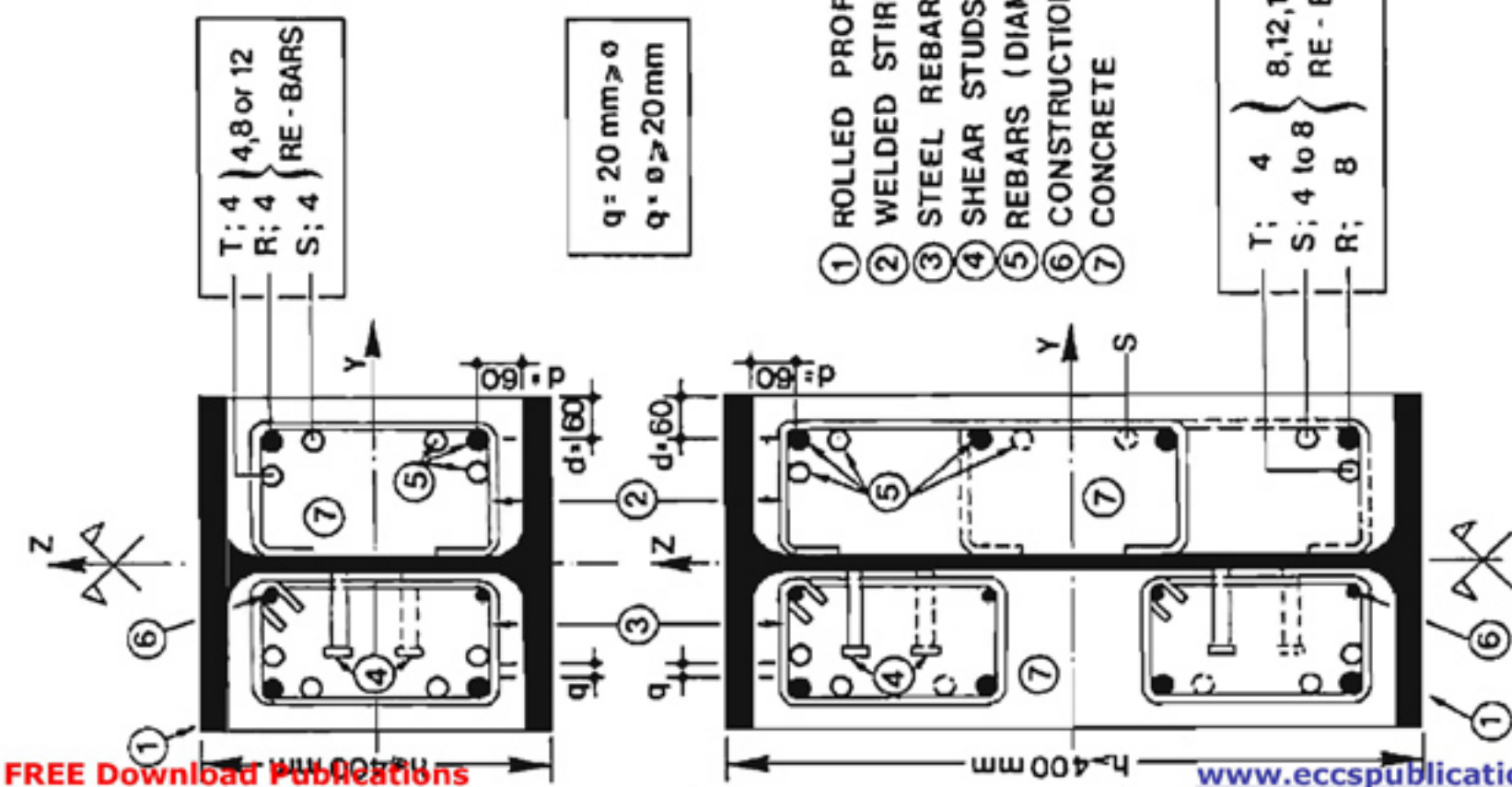
- 1 HP 250x250x62 + 4 ϕ 20 (R)
 2 HP 310x310x79 + 4 ϕ 25 (R)
 3 HP 360x370x108 + 8 ϕ 20 (R+S)
 4 HP 360x410x176 + 8 ϕ 20 (R+S)
 5 M 530x315x150 + 12 ϕ 20 (R+S)
 6 M 510x325x155 + 16 ϕ 20 (R+S)
 7 M 510x325x217 + 16 ϕ 20 (R+S)
 8 M 630x360x217 + 20 ϕ 20 (R+S+T)
 9 M 630x360x255 + 20 ϕ 20 (R+S+T)
 10 M 760x380x257 + 20 ϕ 20 (R+S+T)
 11 M 760x380x314 + 20 ϕ 20 (R+S+T)
 12 M 840x400x299 + 16 ϕ 25 (R+S)
 13 M 840x400x359 + 16 ϕ 25 (R+S)
 14 M 920x420x342 + 20 ϕ 25 (R+S+T)
 15 M 920x420x417 + 20 ϕ 25 (R+S+T)
 16 M 1000x450x286 + 20 ϕ 25 (R+S+T)
 17 M 1000x450x399 + 20 ϕ 25 (R+S+T)
 18 M 1000x450x444 + 20 ϕ 25 (R+S+T)
 19 M 1000x450x488 + 20 ϕ 25 (R+S+T)





AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HP/W

- 1 HE 240 AA + 4 ϕ 20 (R)
2 HE 280 AA + 4 ϕ 25 (R)
3 HE 320 AA + 4 ϕ 25 (R)
4 HE 360 AA + 8 ϕ 20 (R+S)
5 HE 400 AA + 8 ϕ 20 (R+S)
6 HE 450 AA + 8 ϕ 20 (R)
7 HE 500 AA + 12 ϕ 20 (R+S)
8 HE 550 AA + 12 ϕ 20 (R+S)
9 HE 600 AA + 12 ϕ 20 (R+S)
10 HE 650 AA + 16 ϕ 20 (R+S)
11 HE 700 AA + 16 ϕ 20 (R+S)
12 HE 800 AA + 20 ϕ 20 (R+S+T)
13 HE 900 AA + 20 ϕ 20 (R+S+T)
14 HE 1000 AA + 16 ϕ 25 (R+S)



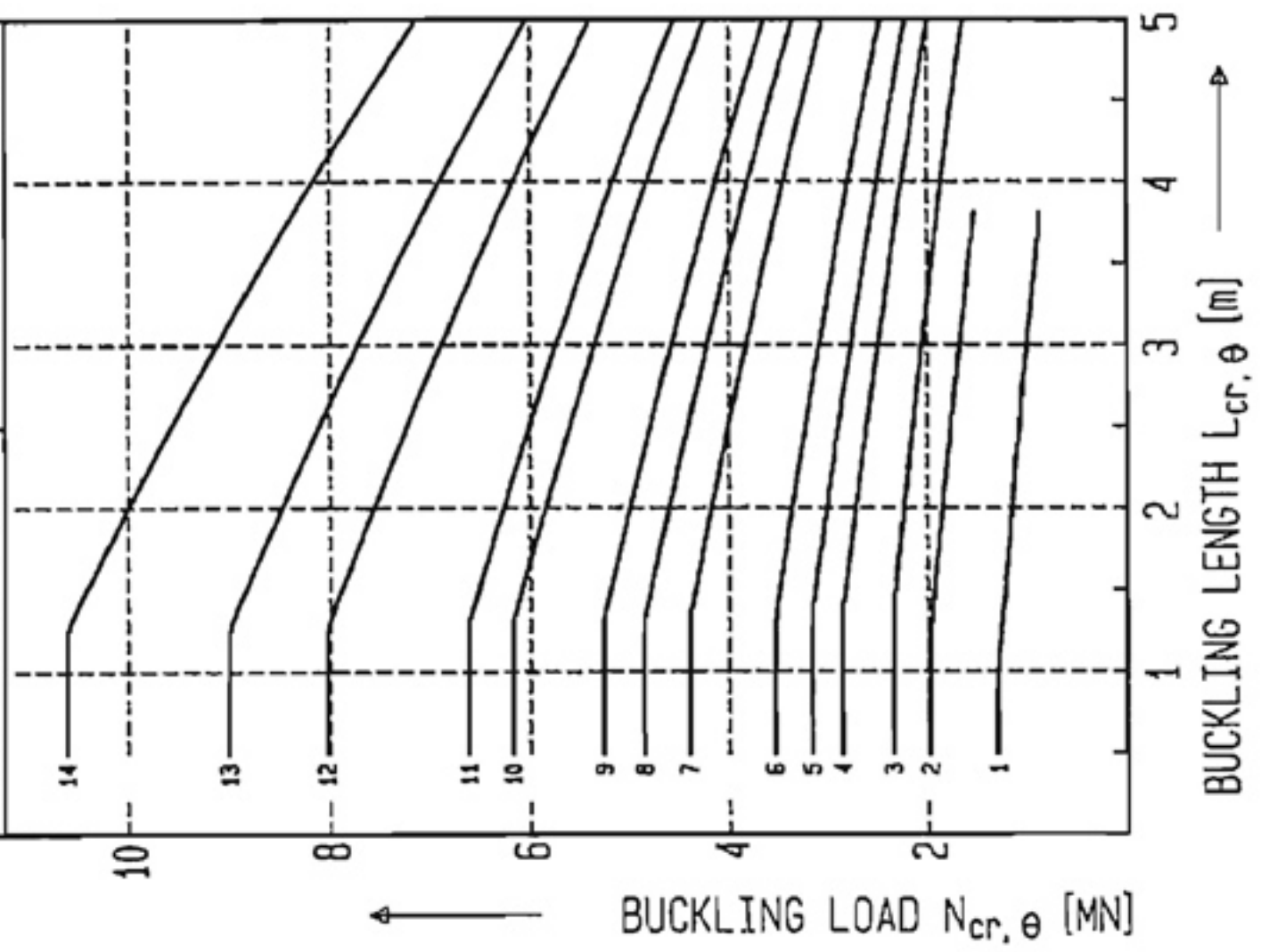
- ① ROLLED PROFILE
② WELDED STIRRUPS
③ STEEL REBAR CAGE
④ SHEAR STUDS
⑤ REBARS (DIAMETER ϕ)
⑥ CONSTRUCTIONAL REBARS
⑦ CONCRETE

T; 4
S; 4 to 8
R; 8
8, 12, 16 or 20 RE - BARS

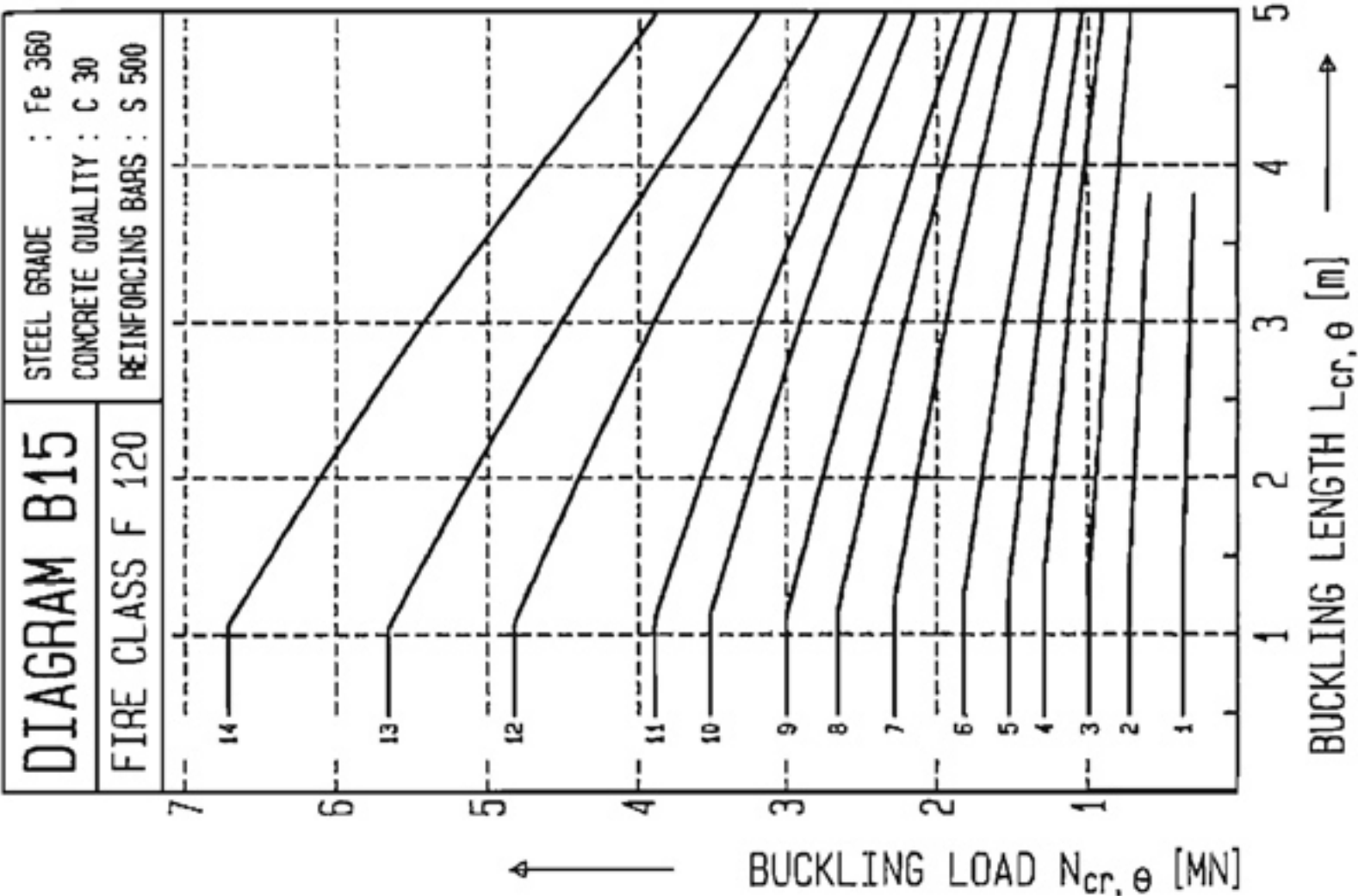
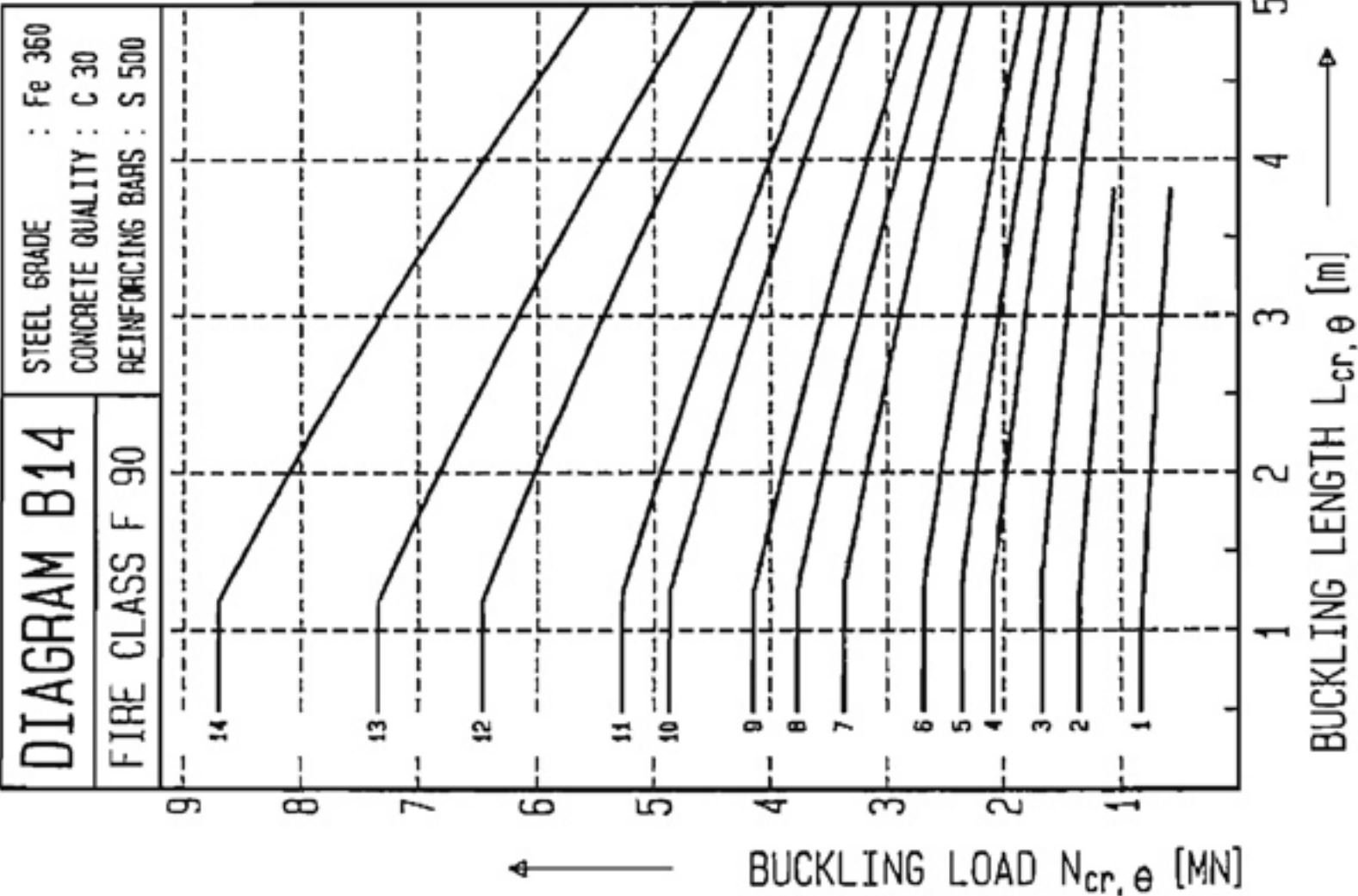
DIAGRAM B13

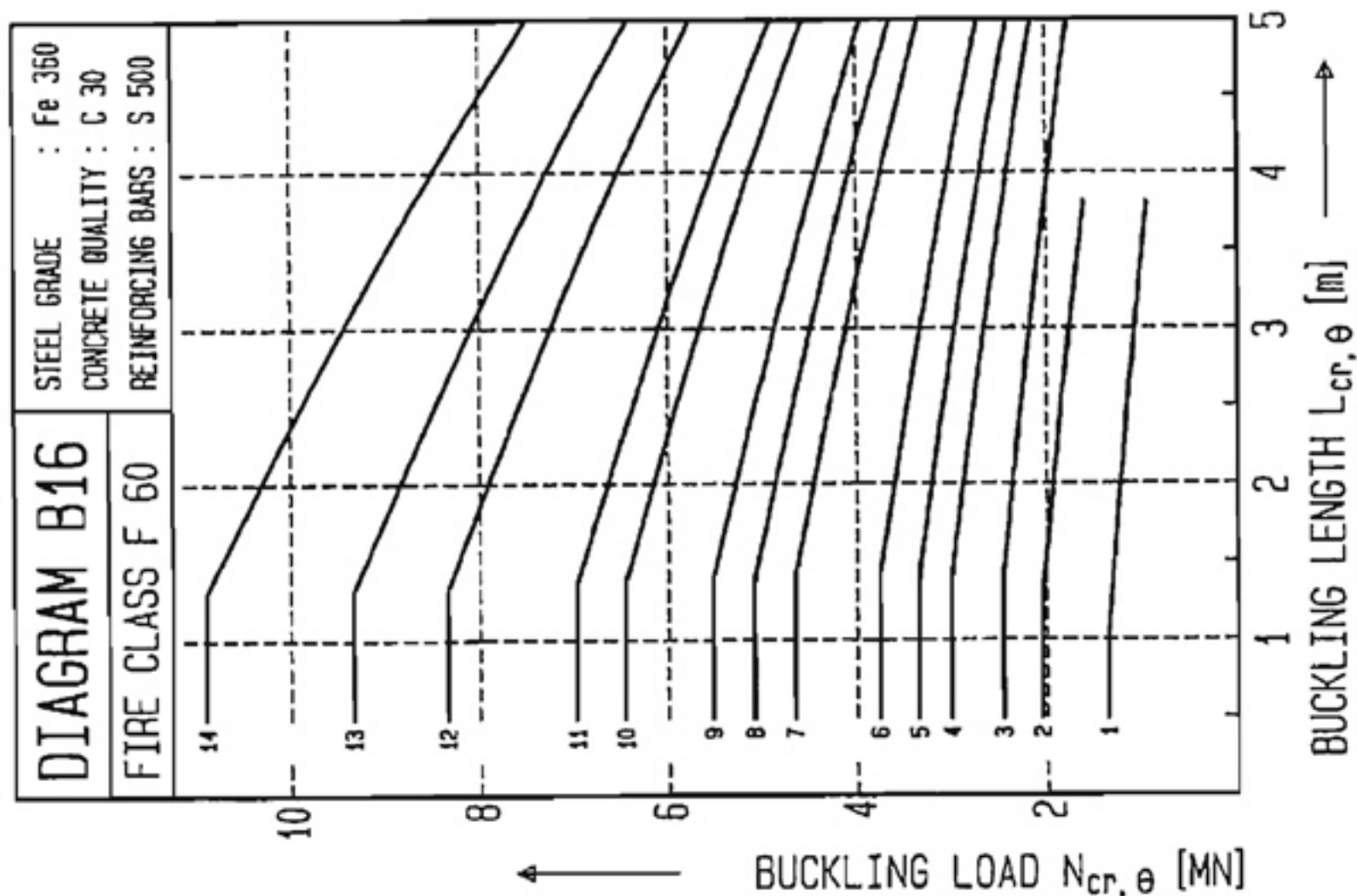
FIRE CLASS F 60

STEEL GRADE : Fe 360
CONCRETE QUALITY : C 30
REINFORCING BARS : S 500

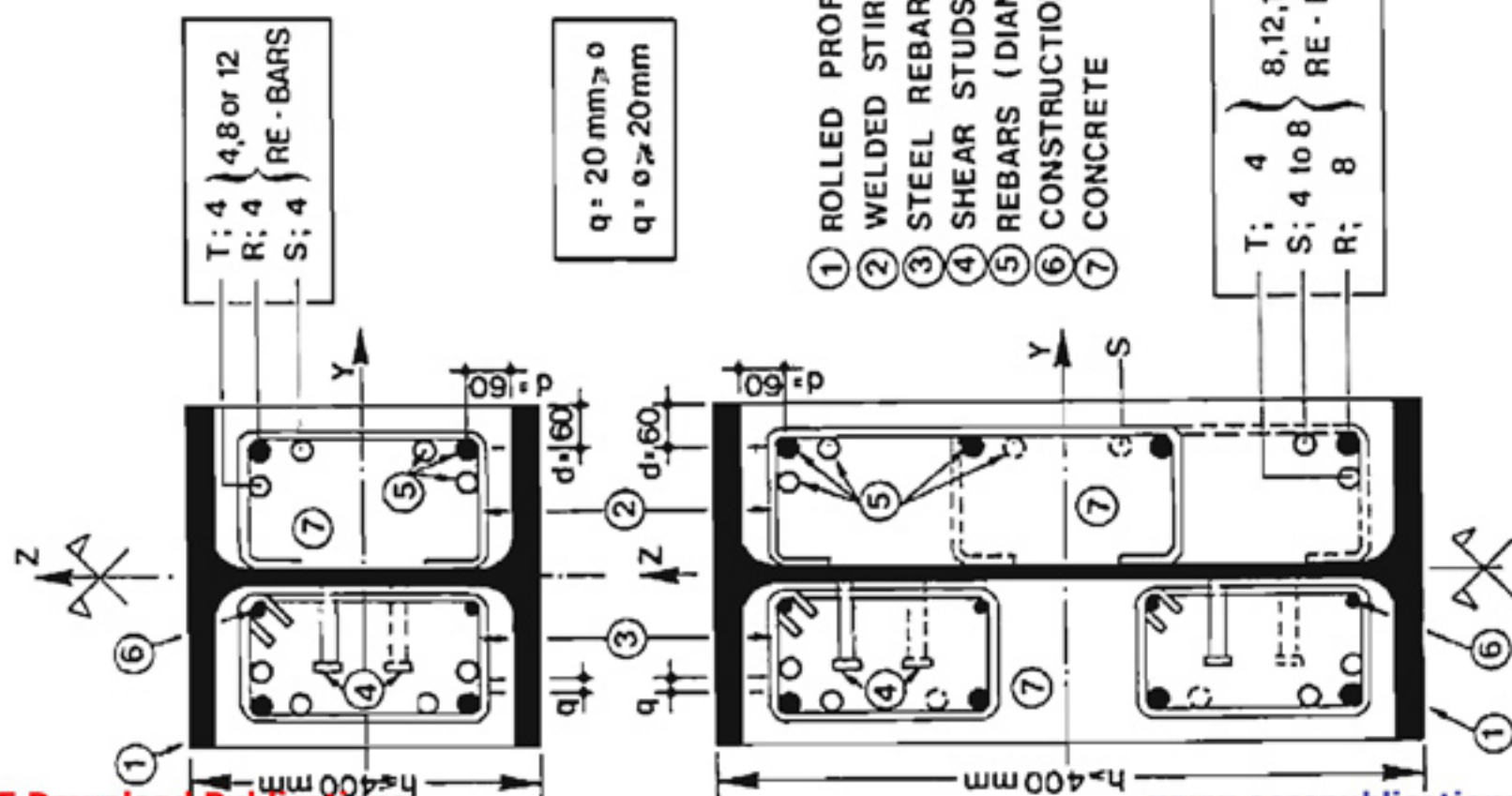


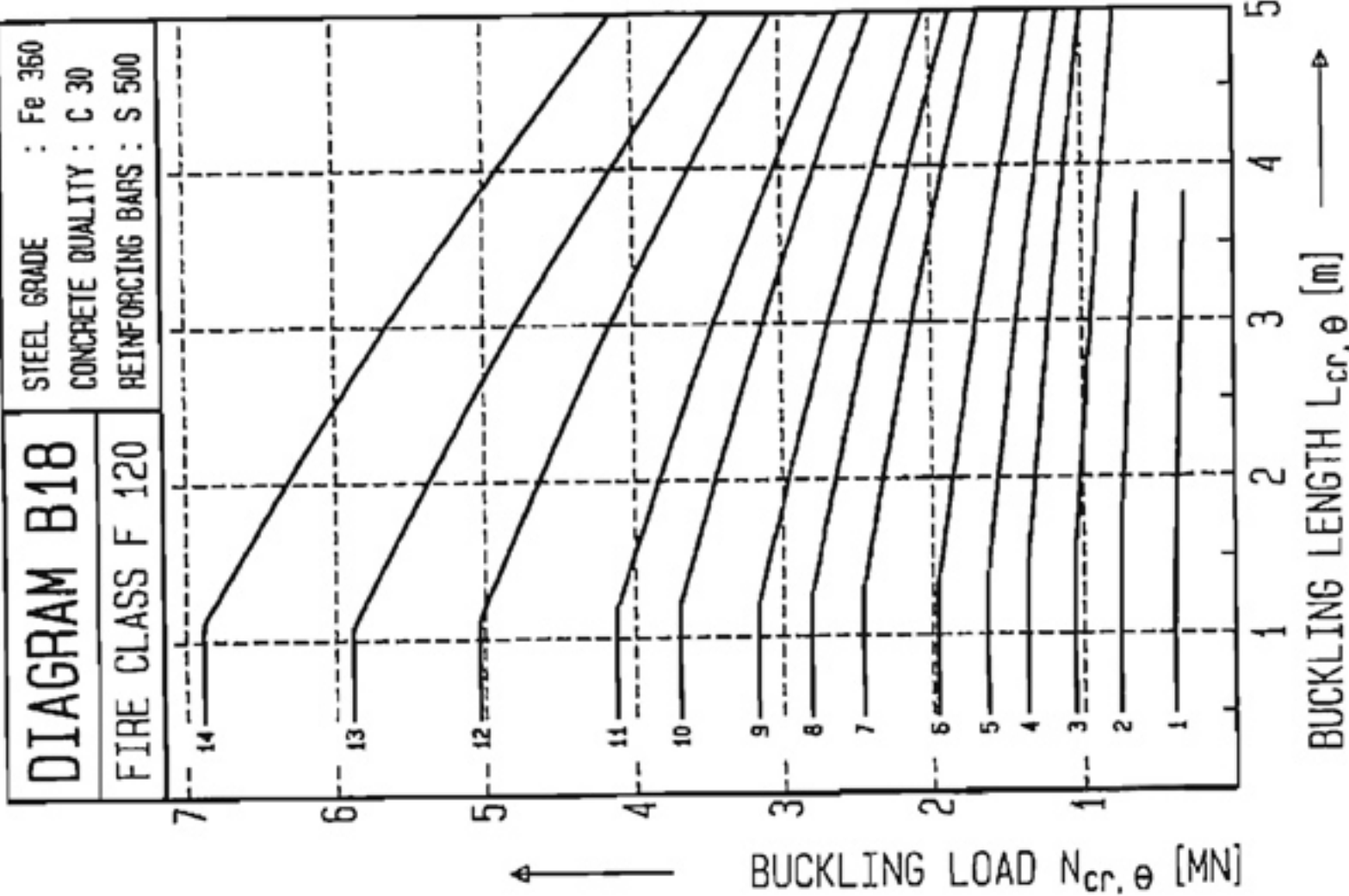
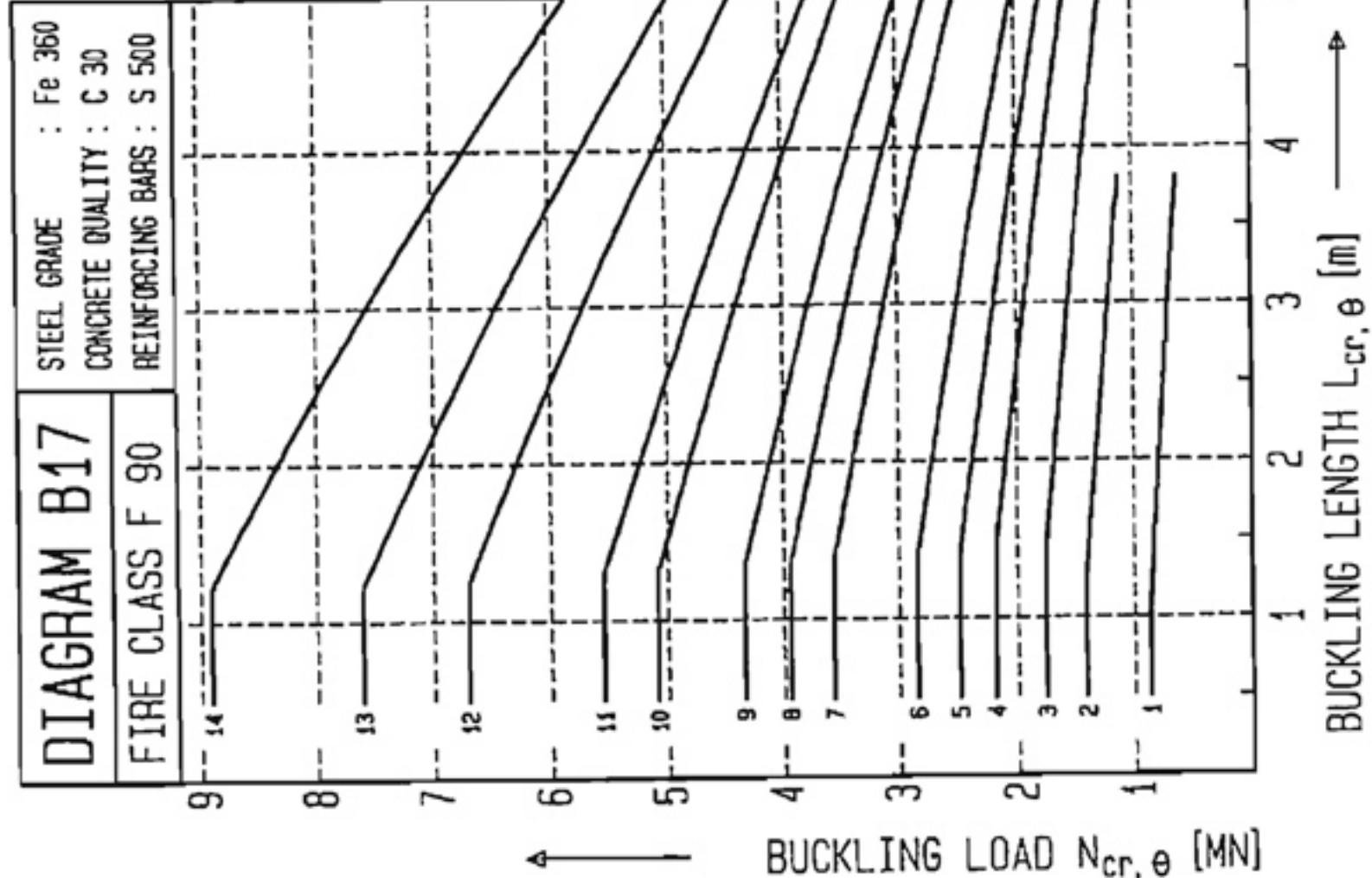
AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-AA



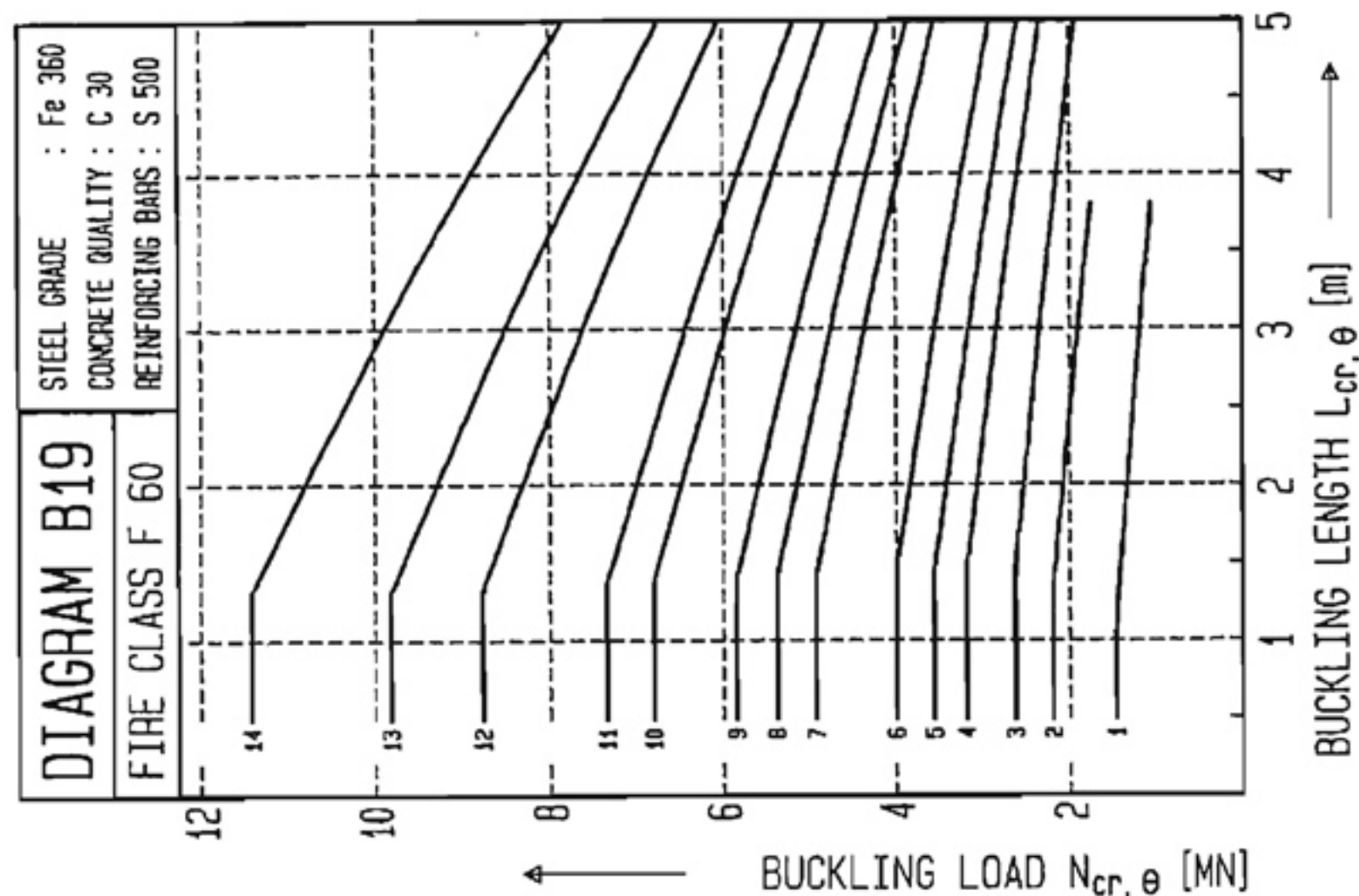


1	HE	240	A	+	4	20	(R)
2	HE	280	A	+	4	25	(R)
3	HE	320	A	+	4	25	(R)
4	HE	360	A	+	8	20	(R+S)
5	HE	400	A	+	8	20	(R+S)
6	HE	450	A	+	8	20	(R)
7	HE	500	A	+	12	20	(R+S)
8	HE	550	A	+	12	20	(R+S)
9	HE	600	A	+	12	20	(R+S)
10	HE	650	A	+	16	20	(R+S)
11	HE	700	A	+	16	20	(R+S)
12	HE	800	A	+	20	20	(R+S+T)
13	HE	900	A	+	20	20	(R+S+T)
14	HE	1000	A	+	16	25	(R+S)

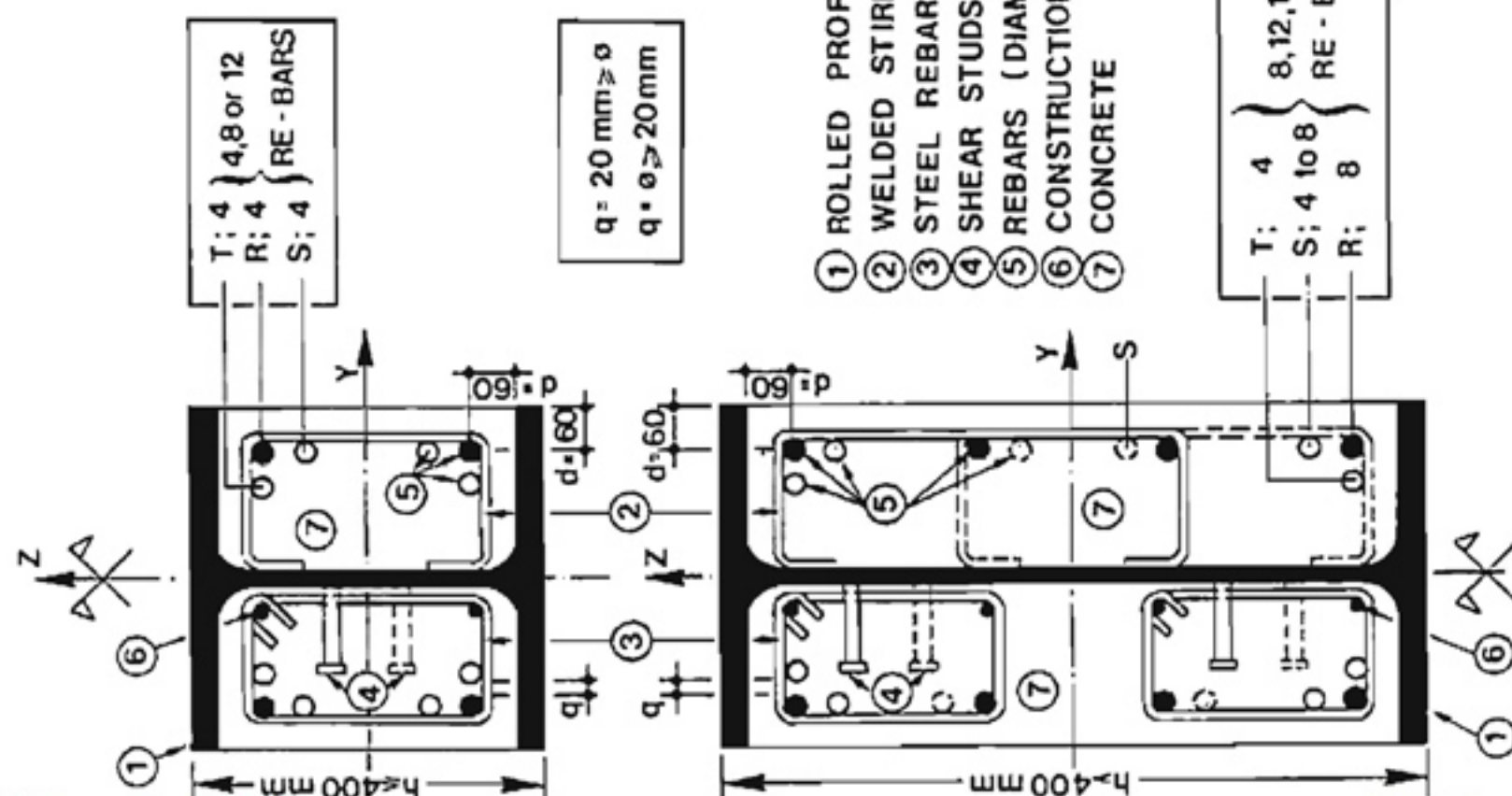


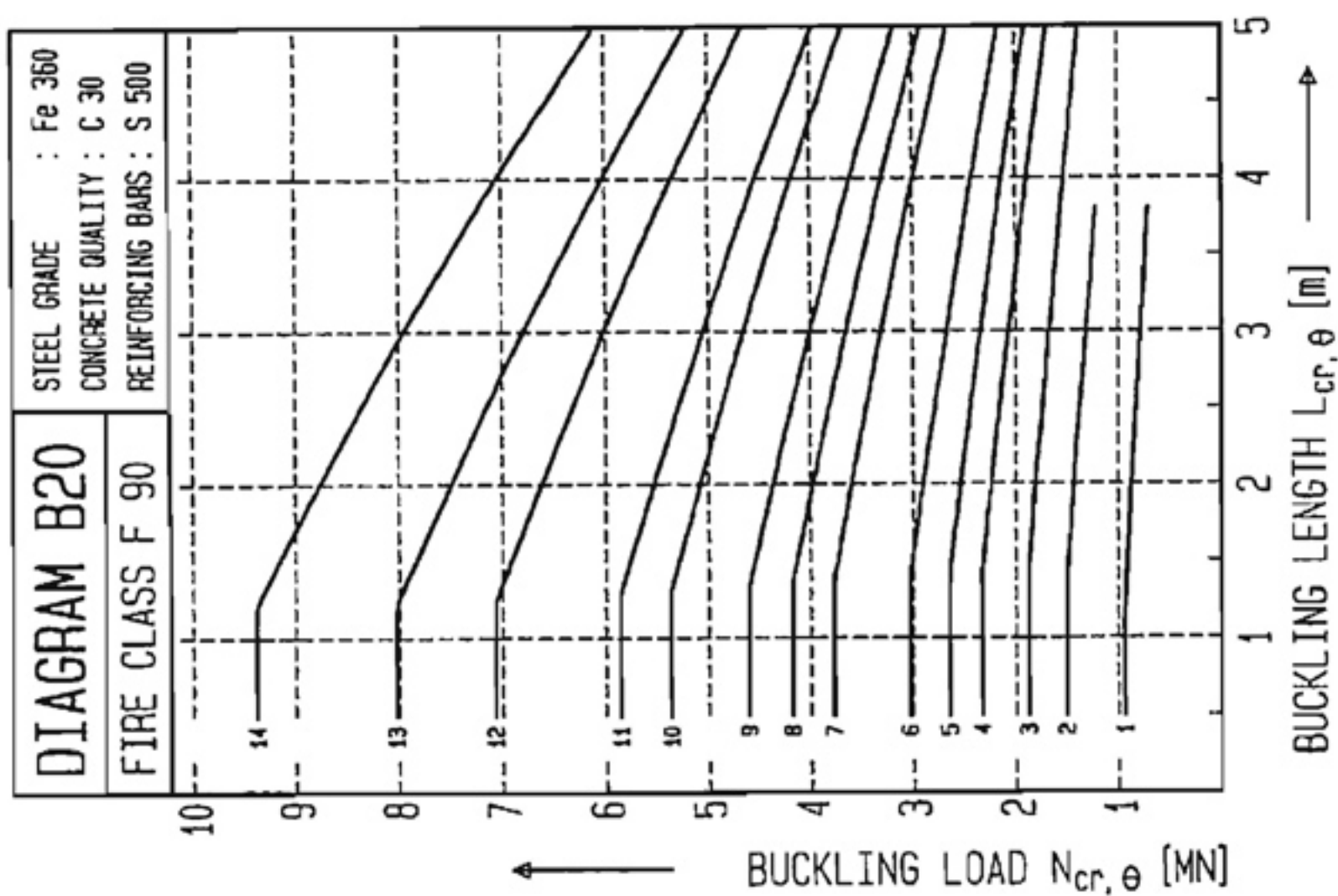
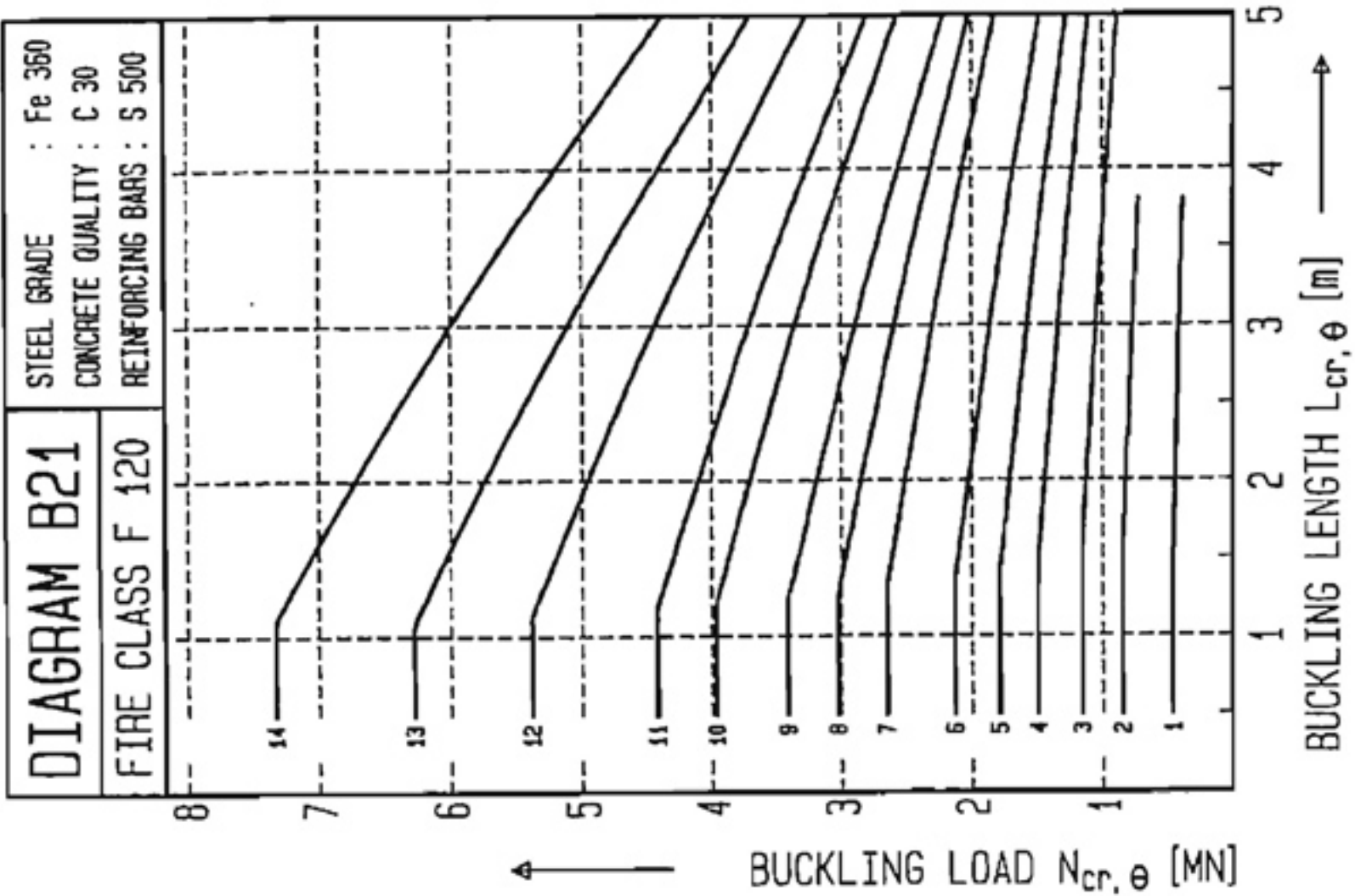


AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-A



1	HE 240 B	+ 4 p 20	(R)
2	HE 280 B	+ 4 p 25	(R)
3	HE 320 B	+ 4 p 25	(R)
4	HE 360 B	+ 8 p 20	(R+S)
5	HE 400 B	+ 8 p 20	(R+S)
6	HE 450 B	+ 8 p 20	(R)
7	HE 500 B	+ 12 p 20	(R+S)
8	HE 550 B	+ 12 p 20	(R+S)
9	HE 600 B	+ 12 p 20	(R+S)
10	HE 650 B	+ 16 p 20	(R+S)
11	HE 700 B	+ 16 p 20	(R+S)
12	HE 800 B	+ 20 p 20	(R+S+T)
13	HE 900 B	+ 20 p 20	(R+S+T)
14	HE 1000 B	+ 16 p 25	(R+S)



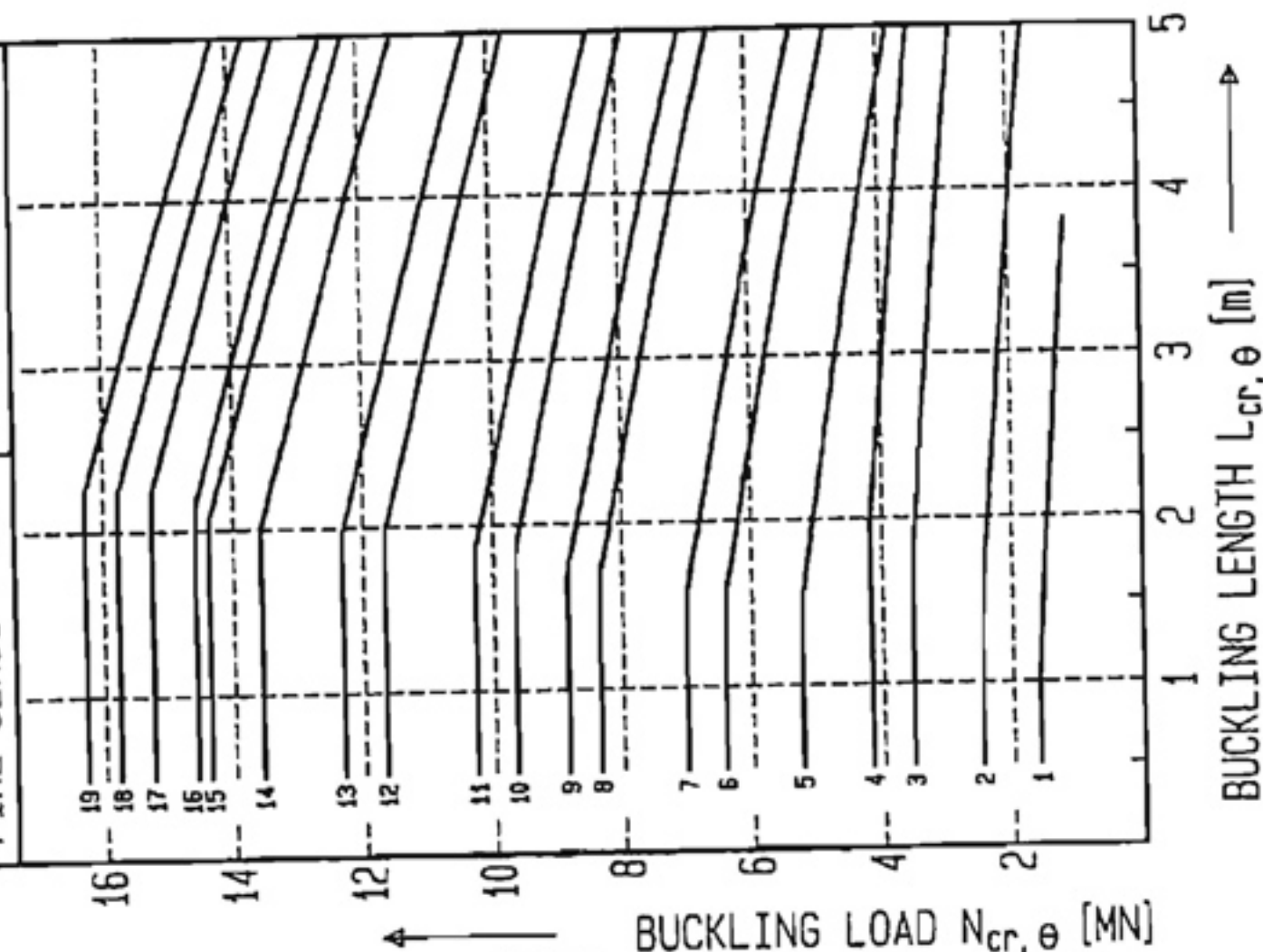


AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-B

DIAGRAM B22

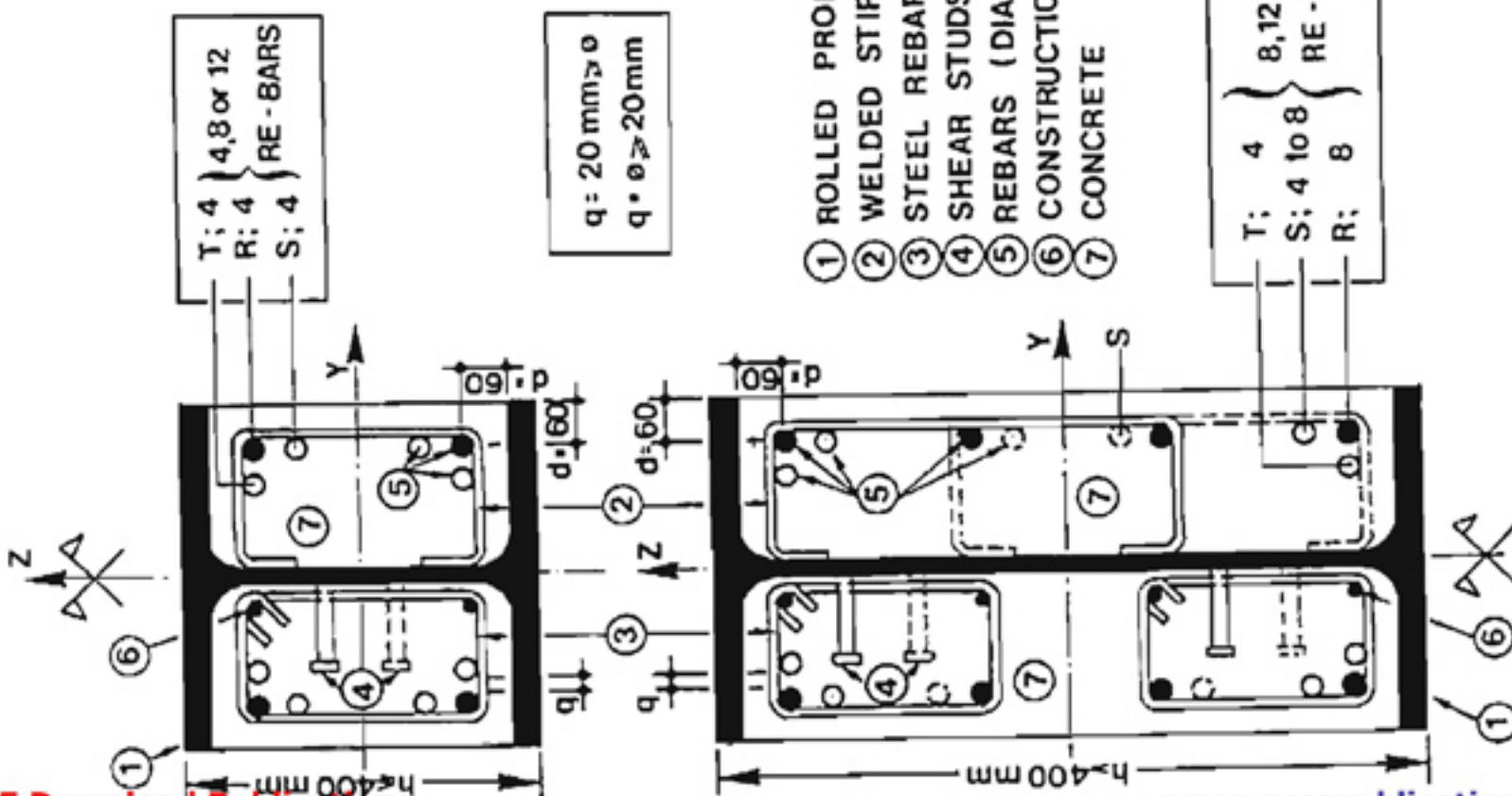
FIRE CLASS F 60

STEEL GRADE : Fe 360
CONCRETE QUALITY : C 30
REINFORCING BARS : S 500

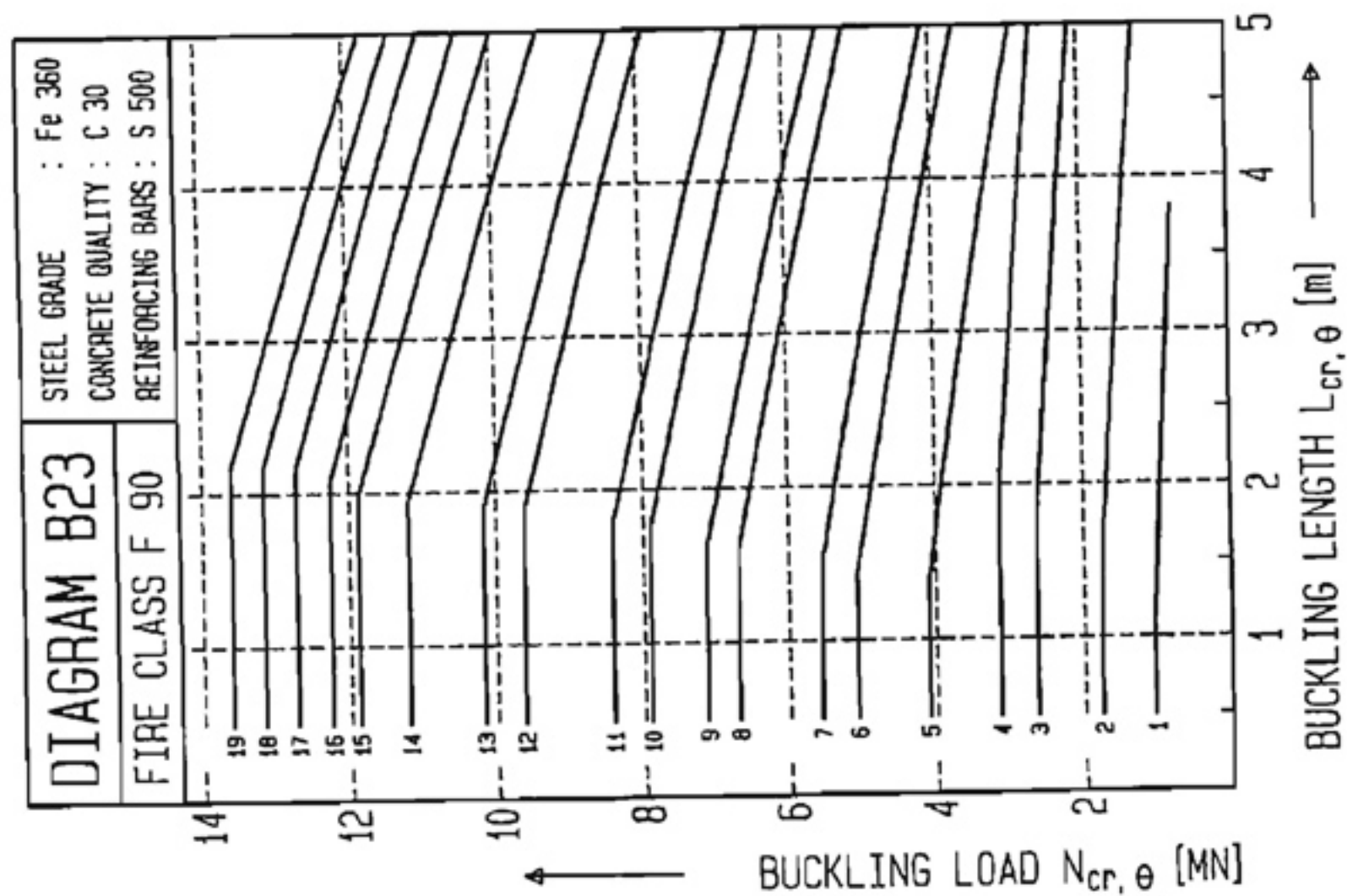
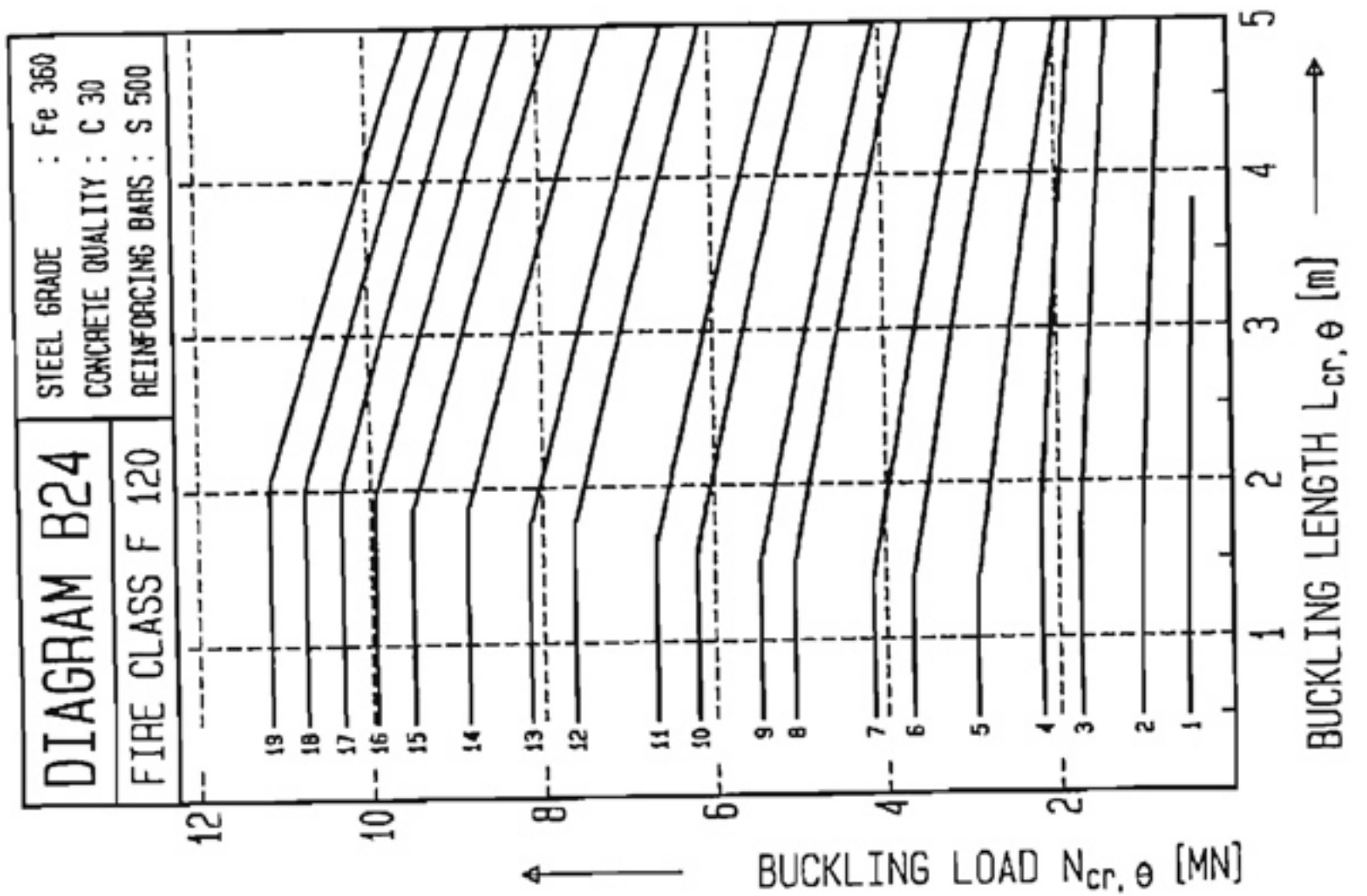


1 HP 250x250x162	+ 4 ϕ 20 (R)
2 HP 310x310x179	+ 4 ϕ 25 (R)
3 HP 360x370x108	+ 8 ϕ 20 (R+S)
4 HP 360x410x176	+ 8 ϕ 20 (R+S)
5 M 530x315x150	+ 12 ϕ 20 (R+S)
6 M 610x325x155	+ 16 ϕ 20 (R+S)
7 M 610x325x217	+ 16 ϕ 20 (R+S)
8 M 690x360x217	+ 20 ϕ 20 (R+S+T)
9 M 690x360x265	+ 20 ϕ 20 (R+S+T)
10 M 760x380x257	+ 20 ϕ 20 (R+S+T)
11 M 760x380x314	+ 20 ϕ 20 (R+S+T)
12 M 840x400x299	+ 16 ϕ 25 (R+S)
13 M 840x400x359	+ 16 ϕ 25 (R+S)
14 M 920x420x342	+ 20 ϕ 25 (R+S+T)
15 M 920x420x417	+ 20 ϕ 25 (R+S+T)
16 M 1000x450x286	+ 20 ϕ 25 (R+S+T)
17 M 1000x450x399	+ 20 ϕ 25 (R+S+T)
18 M 1000x450x444	+ 20 ϕ 25 (R+S+T)
19 M 1000x450x488	+ 20 ϕ 25 (R+S+T)

- 1 ROLLED PROFILE
- 2 WELDED STIRRUPS
- 3 STEEL REBAR CAGE
- 4 SHEAR STUDS
- 5 REBARS (DIAMETER ϕ)
- 6 CONSTRUCTIONAL REBARS
- 7 CONCRETE



AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HP/W



AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HP/W

DIAGRAM B25

FIRE CLASS F 60

STEEL GRADE : Fe 510
CONCRETE QUALITY : C 30
REINFORCING BARS : S 500

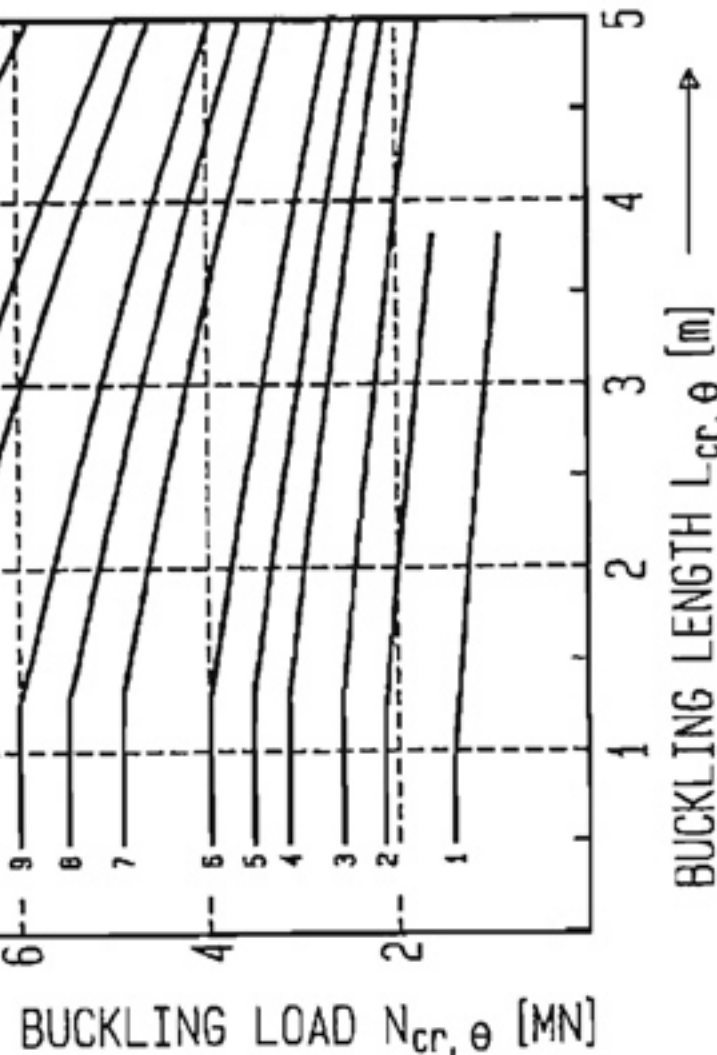
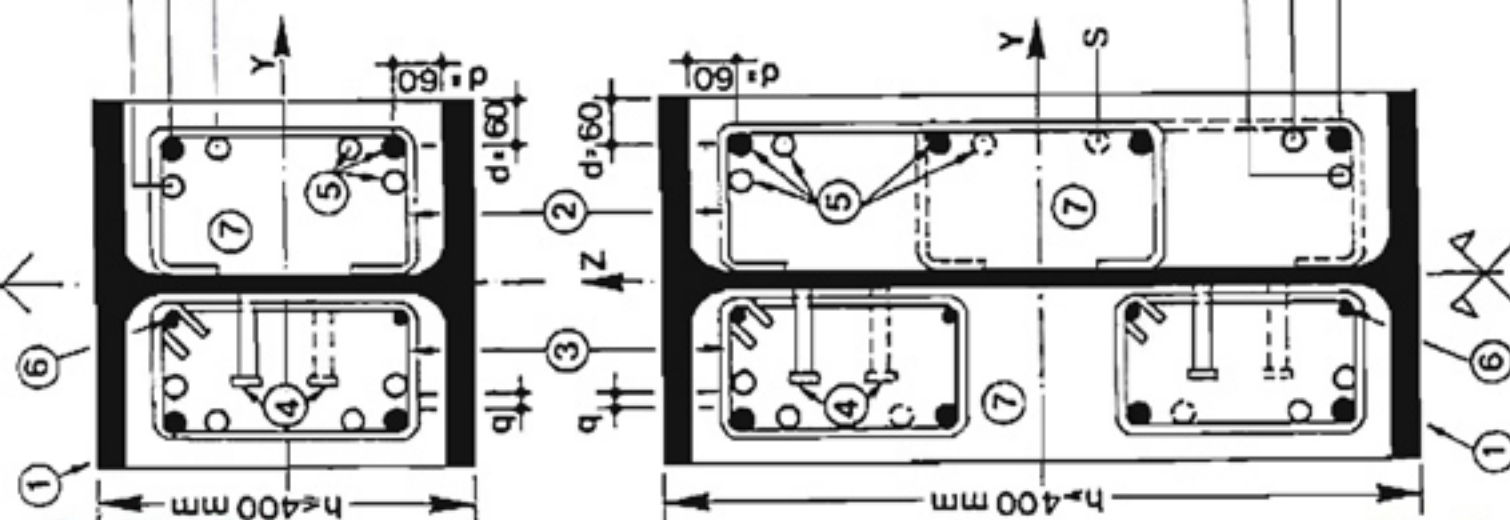
1 HE 240 AA	+ 4 ϕ 20 (R)
2 HE 280 AA	+ 4 ϕ 25 (R)
3 HE 320 AA	+ 4 ϕ 25 (R)
4 HE 360 AA	+ 8 ϕ 20 (R+S)
5 HE 400 AA	+ 8 ϕ 20 (R+S)
6 HE 450 AA	+ 8 ϕ 20 (R)
7 HE 500 AA	+ 12 ϕ 20 (R+S)
8 HE 550 AA	+ 12 ϕ 20 (R+S)
9 HE 600 AA	+ 12 ϕ 20 (R+S)
10 HE 650 AA	+ 16 ϕ 20 (R+S)
11 HE 700 AA	+ 16 ϕ 20 (R+S)
12 HE 800 AA	+ 20 ϕ 20 (R+S+T)
13 HE 900 AA	+ 20 ϕ 20 (R+S+T)
14 HE 1000 AA	+ 16 ϕ 25 (R+S)

T: 4 } 4, 8 or 12
R: 4 } RE - BARS
S: 4 }

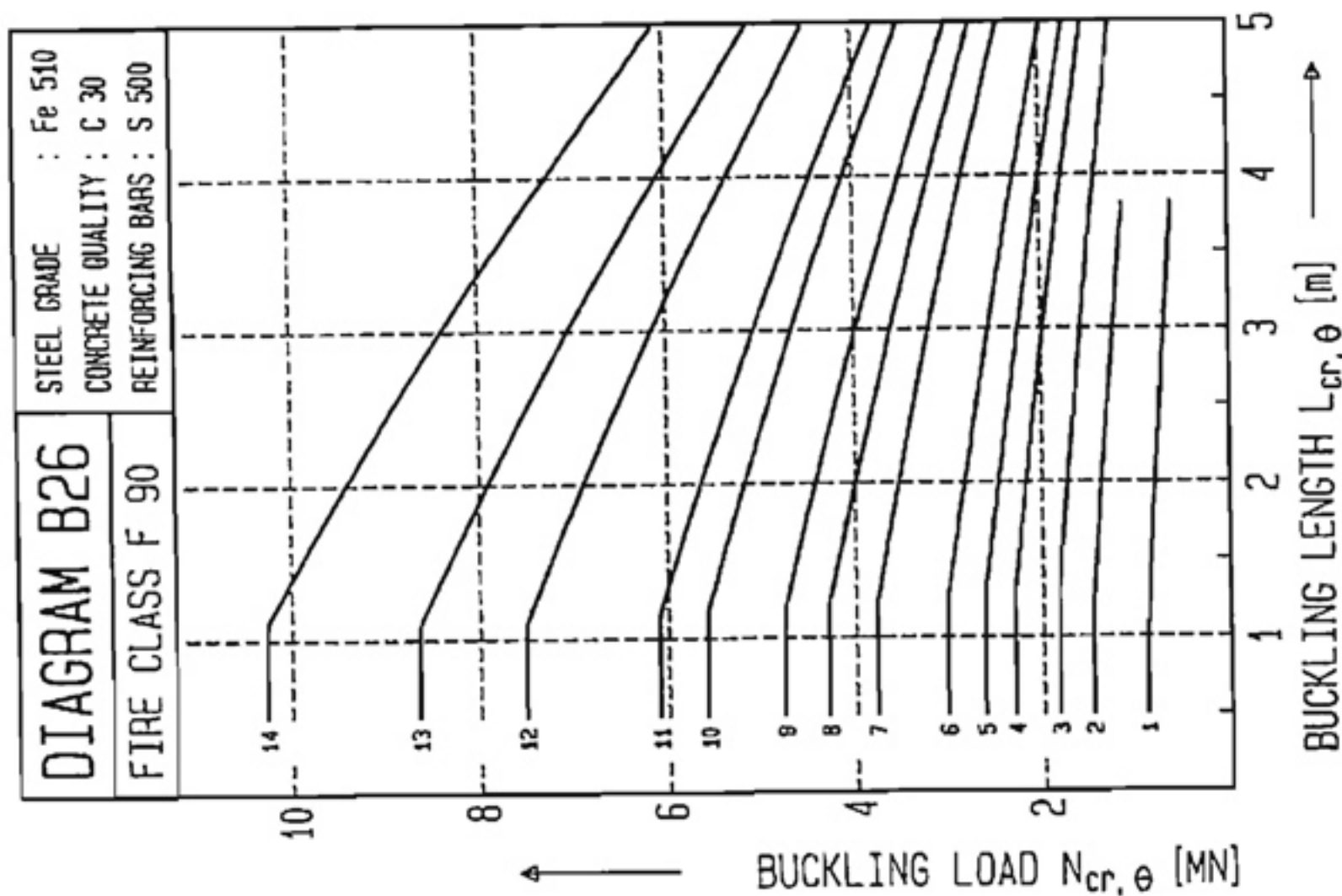
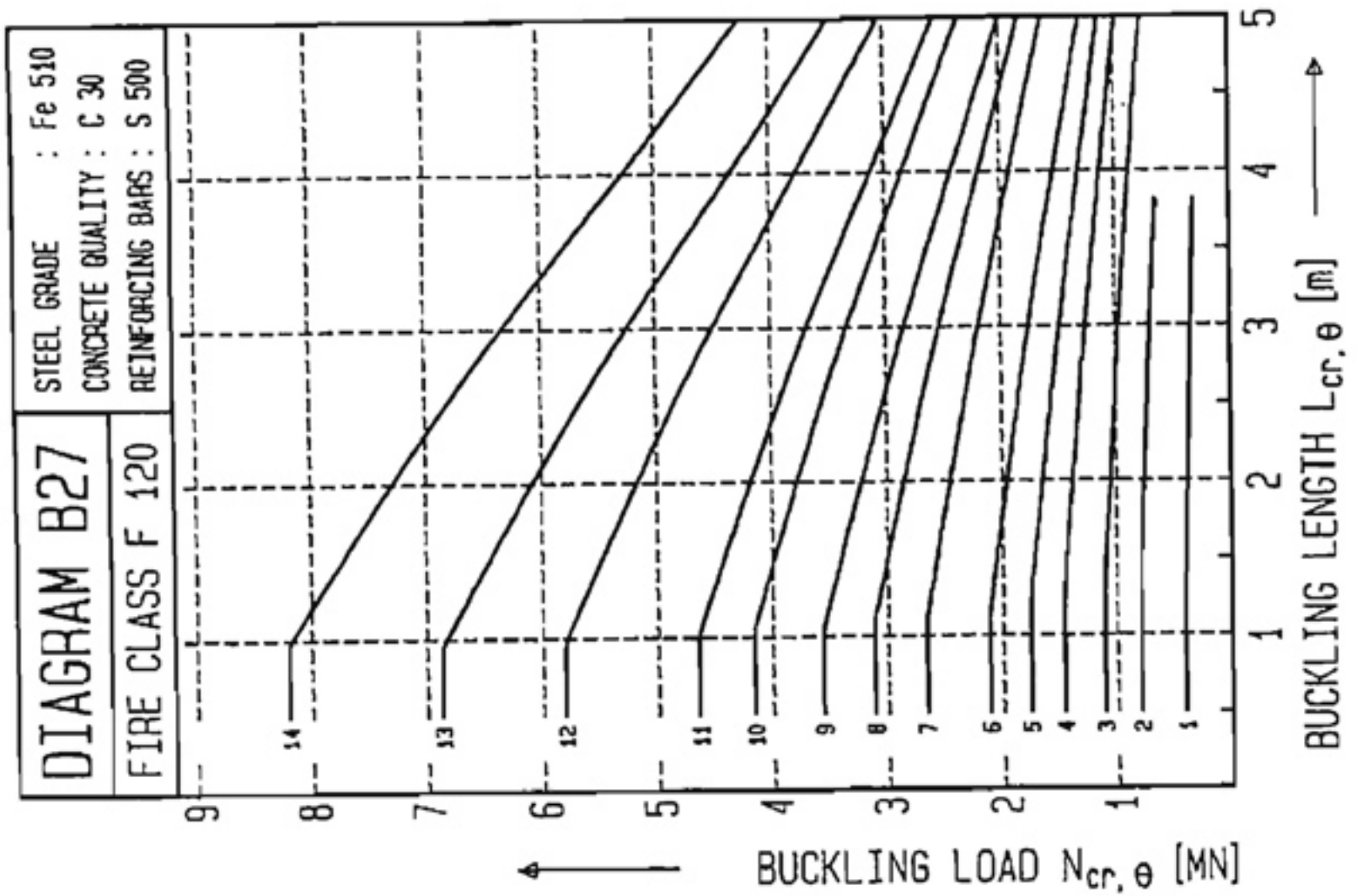
$q = 20 \text{ mm} \geq \phi$
 $q \cdot \phi \geq 20 \text{ mm}$

- 1 ROLLED PROFILE
- 2 WELDED STIRRUPS
- 3 STEEL REBAR CAGE
- 4 SHEAR STUDS
- 5 REBARS (DIAMETER ϕ)
- 6 CONSTRUCTIONAL REBARS
- 7 CONCRETE

T: 4 } 8, 12, 16 or 20
S: 4 to 8 } RE - BARS
R: 8 }

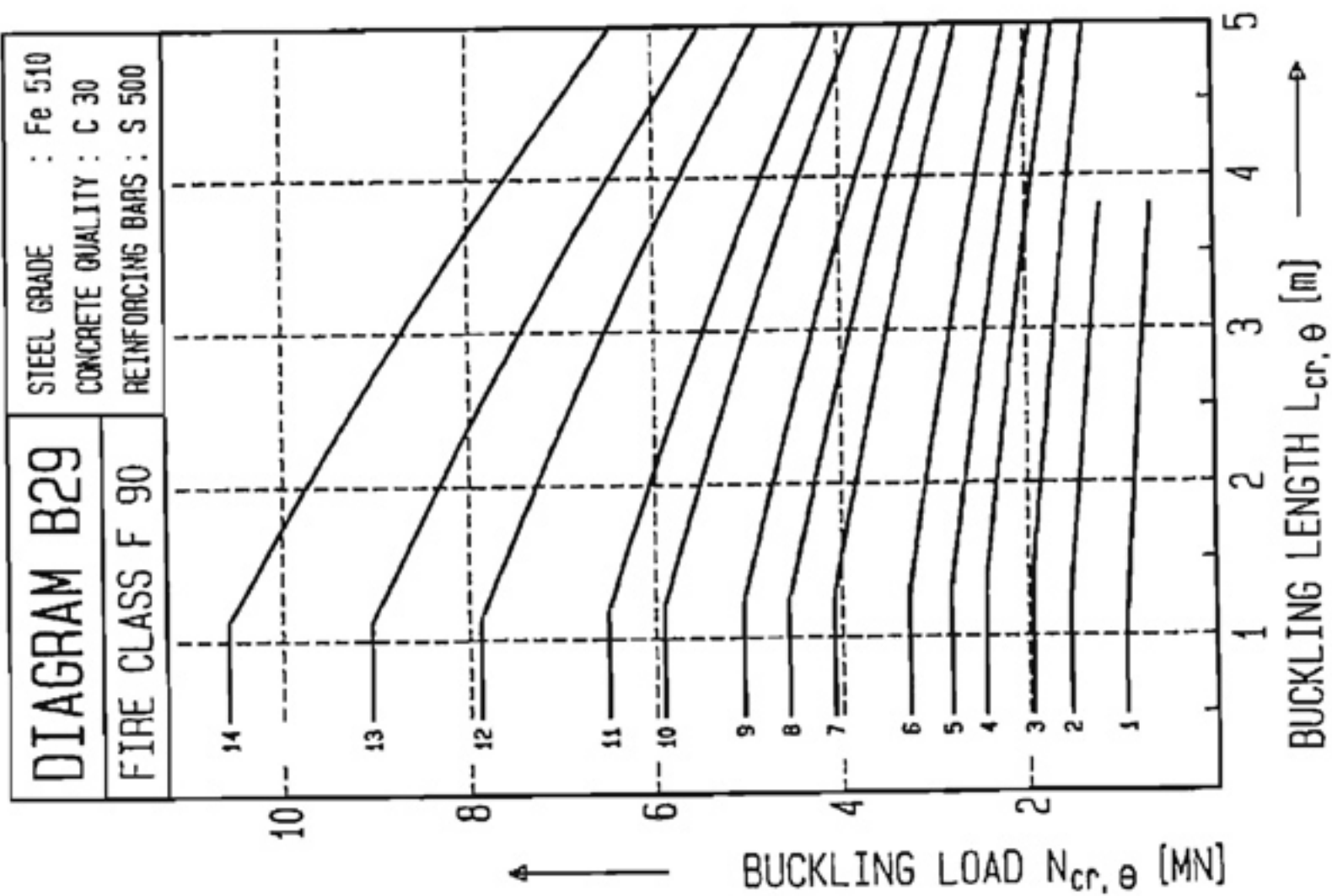
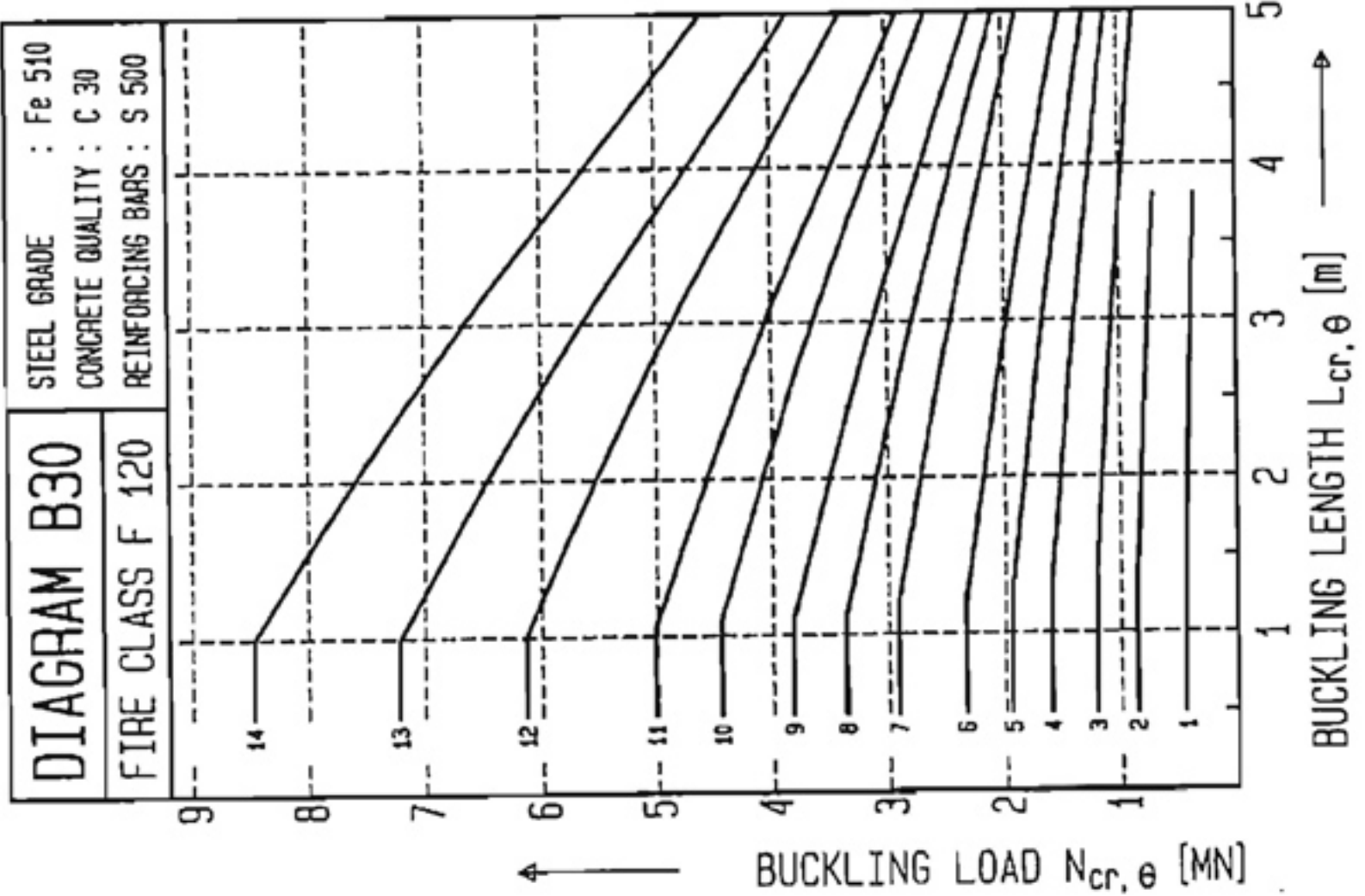


AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-AA

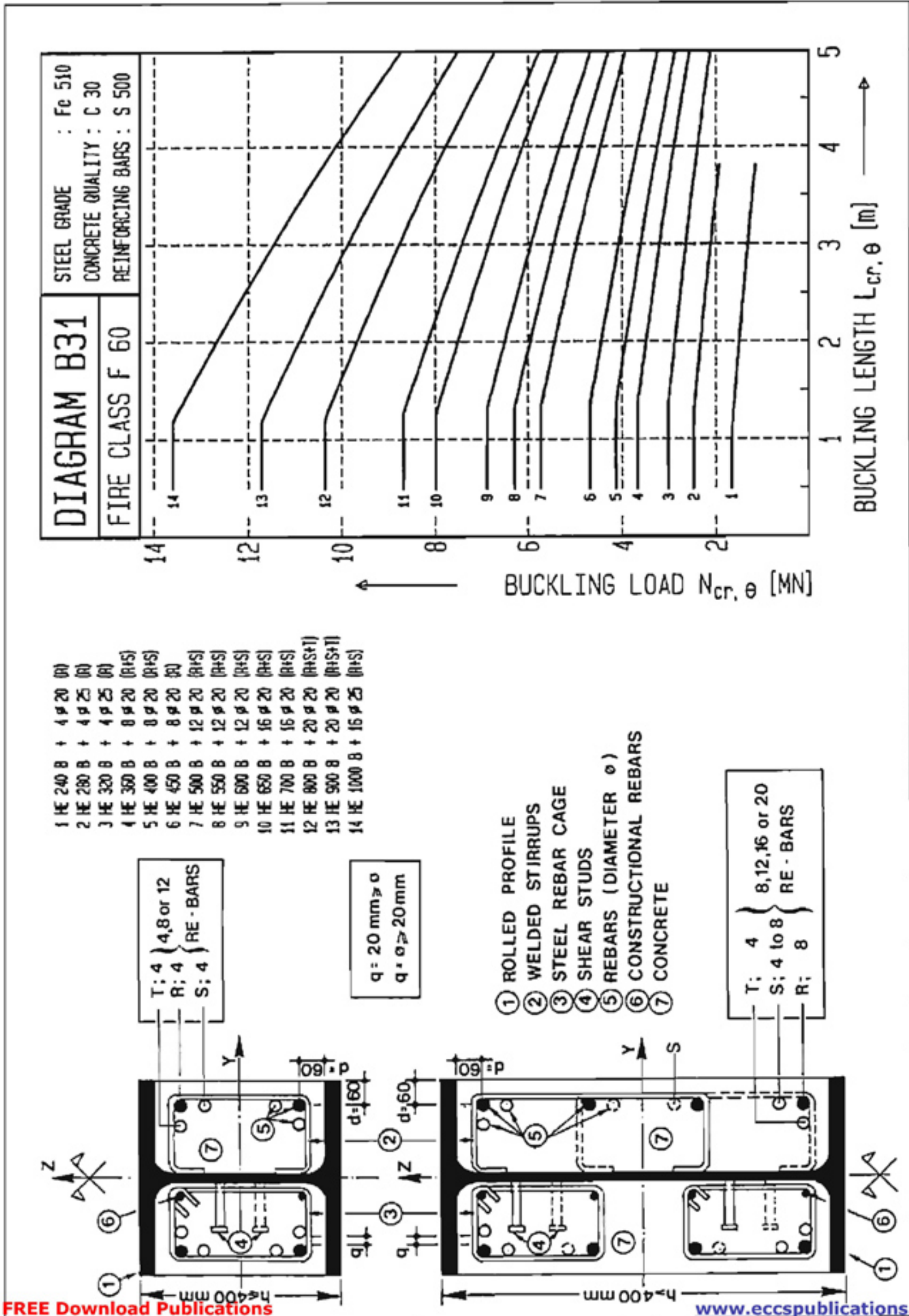


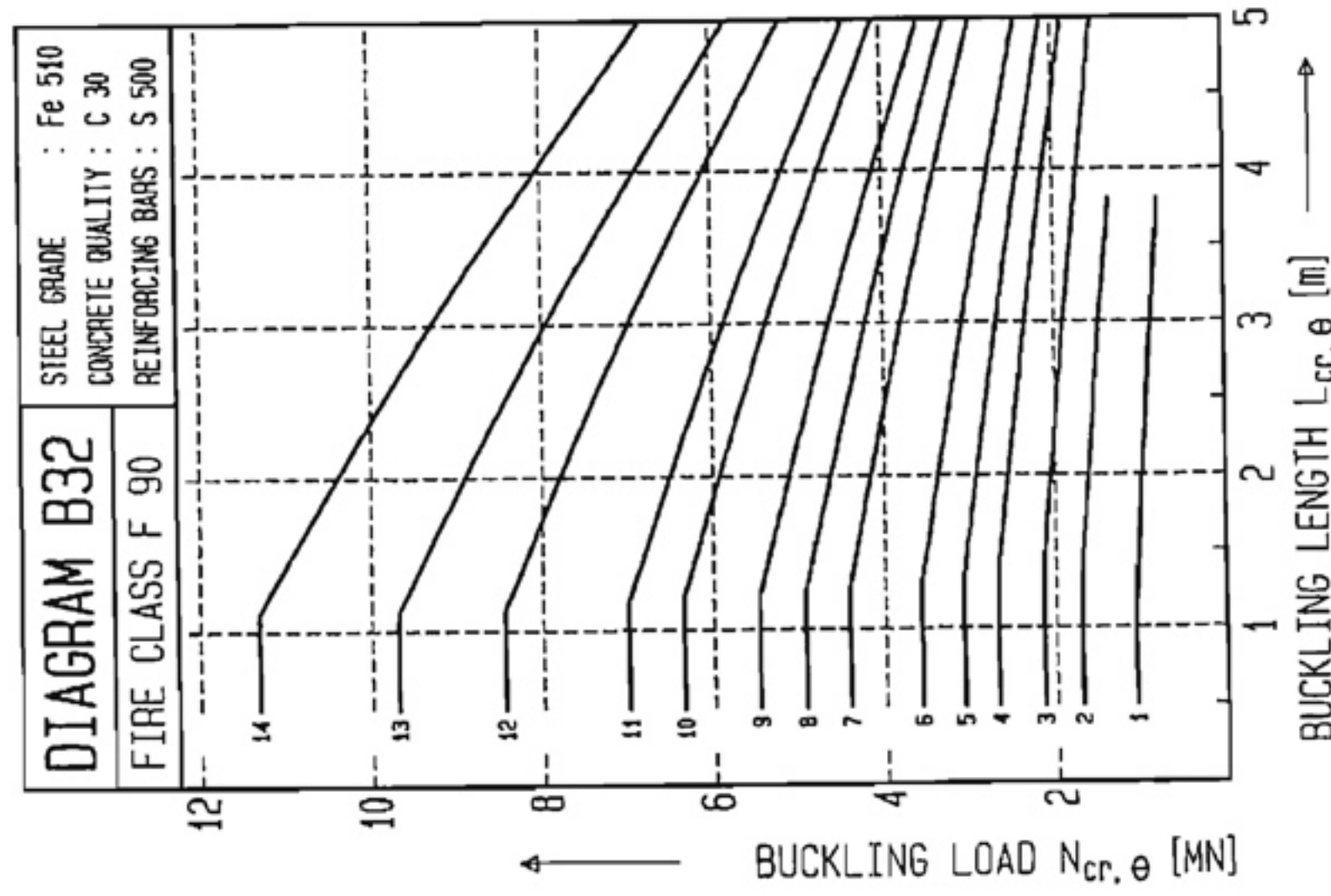
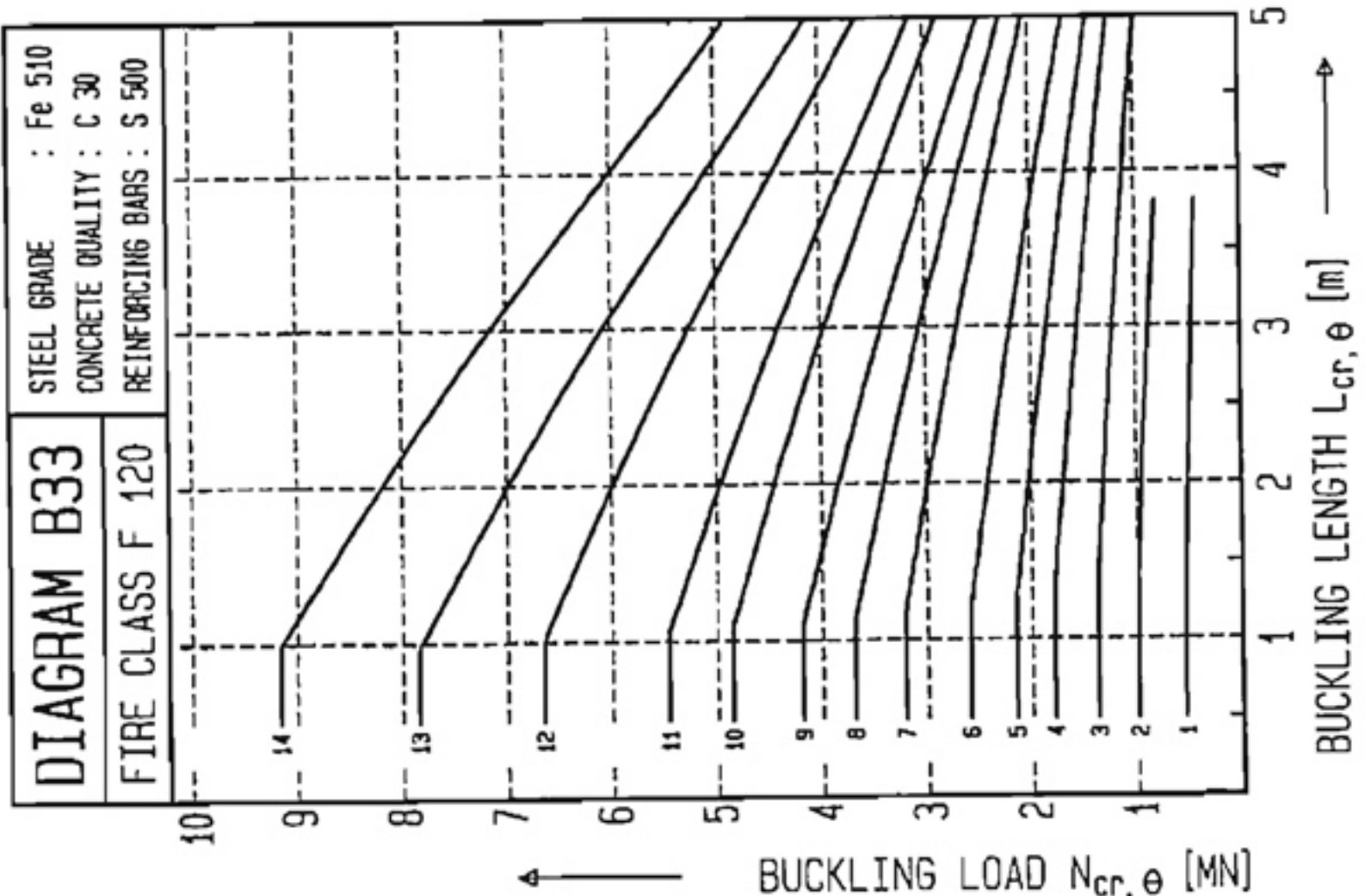
AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-AA



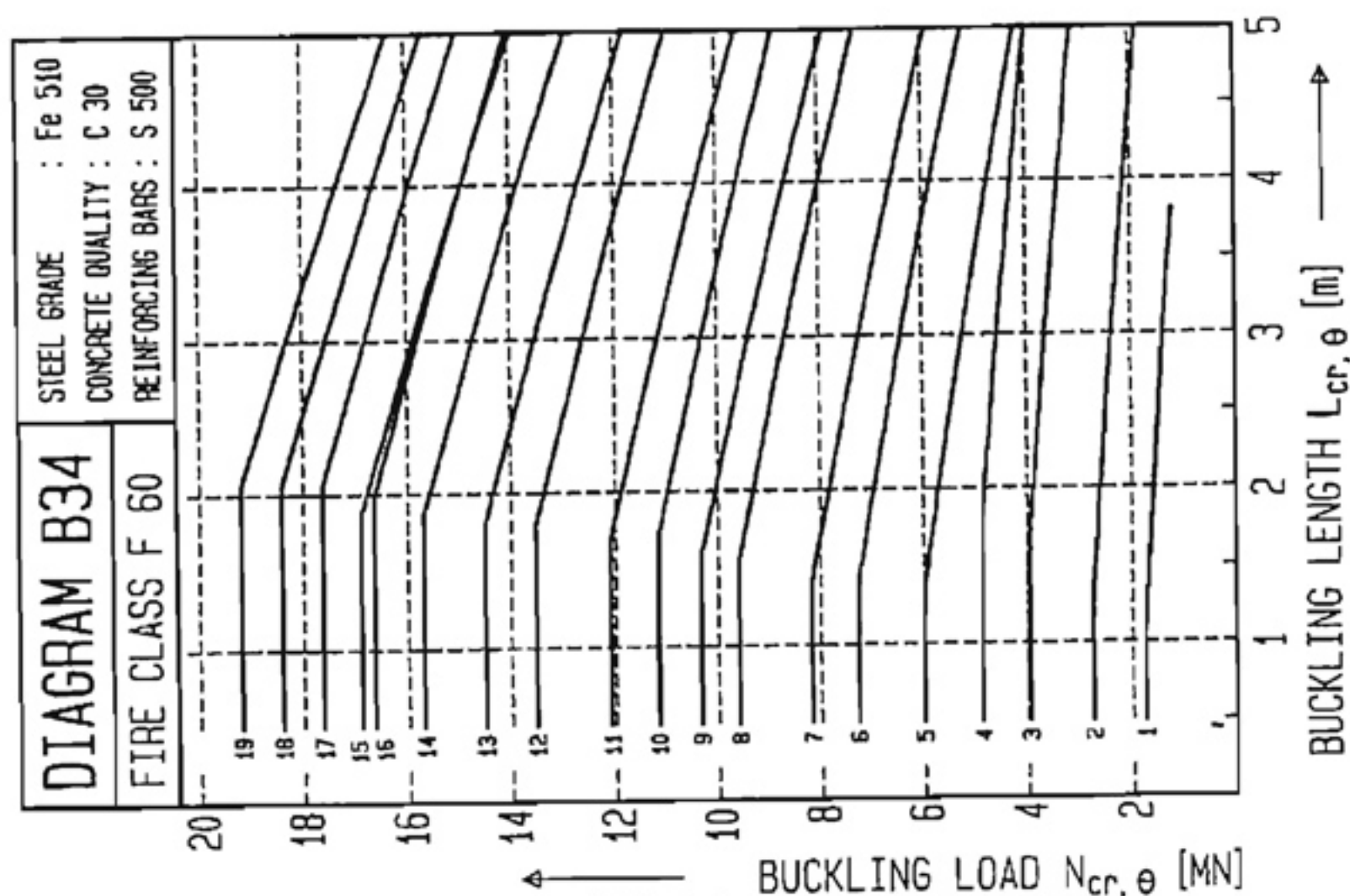


AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-A



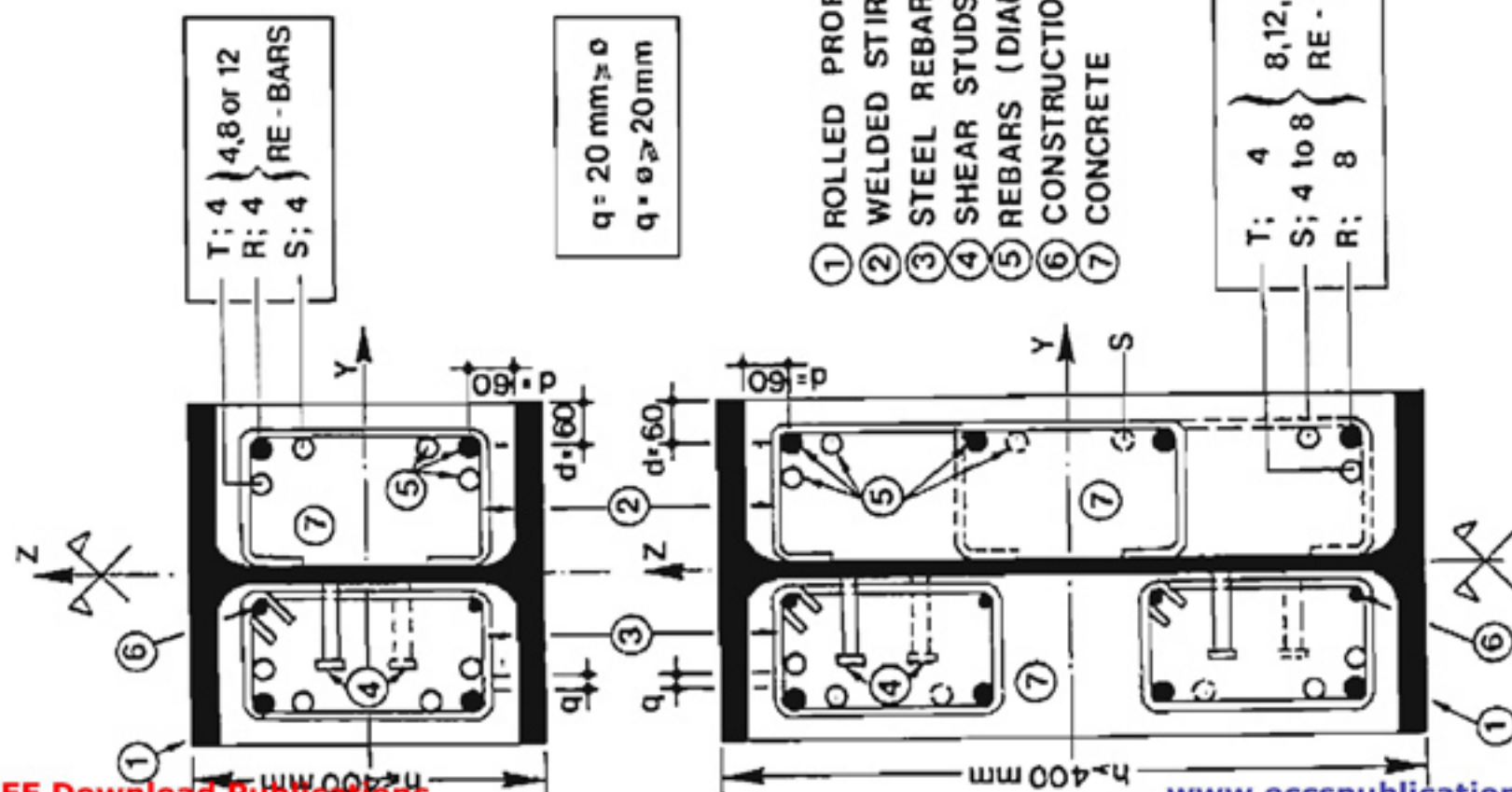


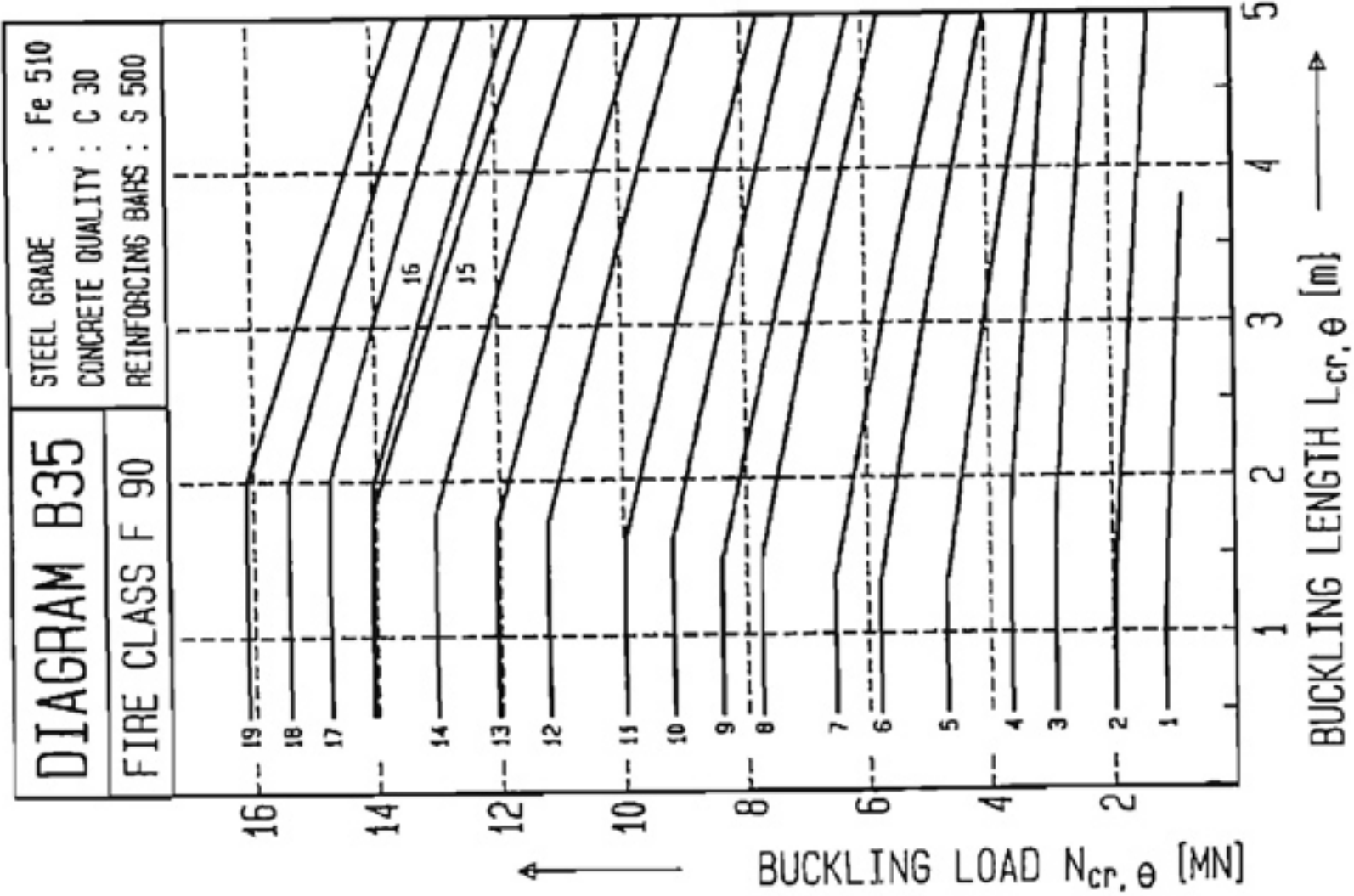
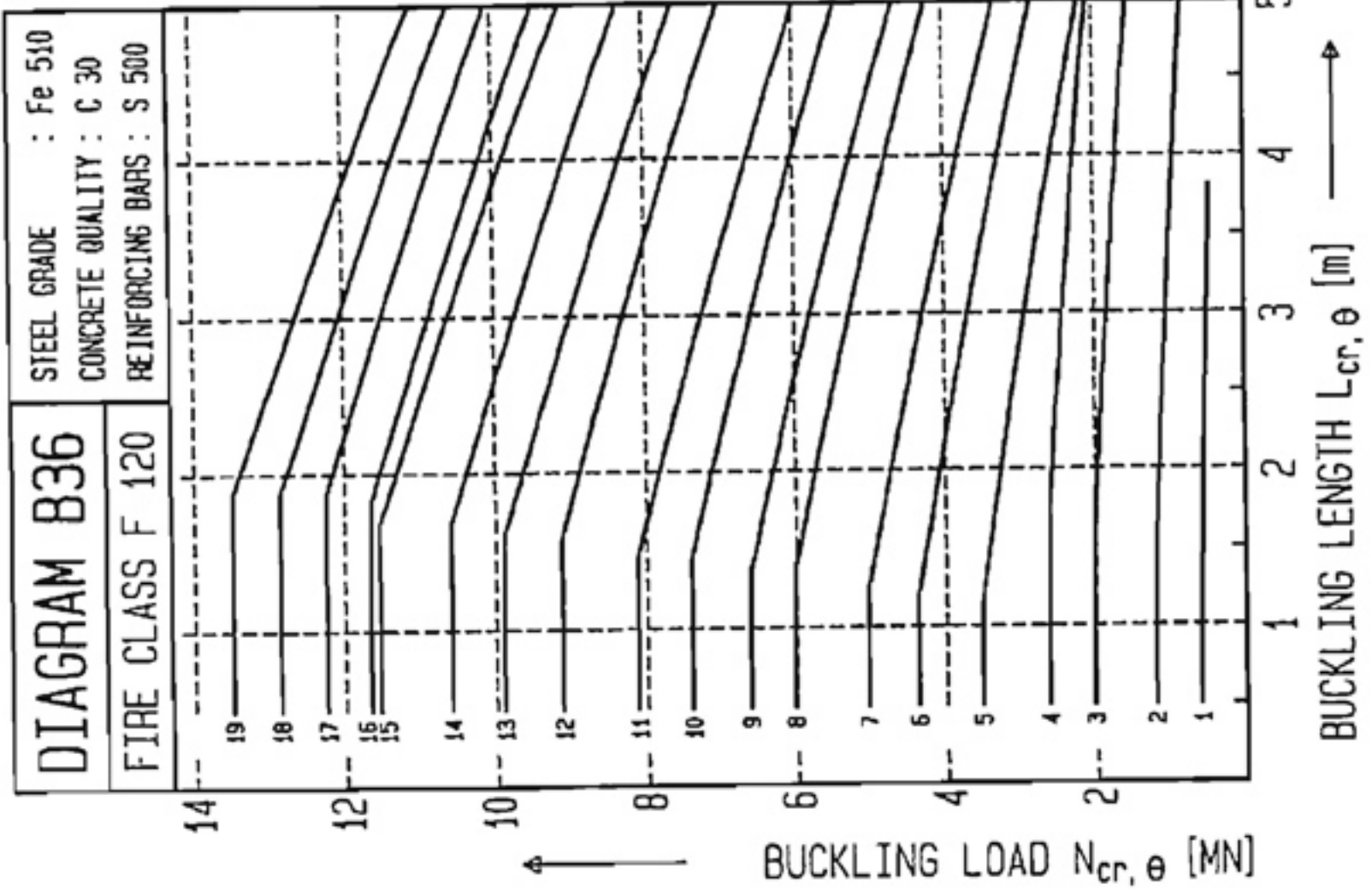
AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-B



- 1 HP 250x250x62 + 4 ϕ 20 (R)
2 HP 310x310x79 + 4 ϕ 25 (R)
3 HP 360x370x108 + 8 ϕ 20 (R+S)
4 HP 360x410x176 + 8 ϕ 20 (R+S)
5 M 530x315x150 + 12 ϕ 20 (R+S)
6 M 610x325x155 + 16 ϕ 20 (R+S)
7 M 610x325x217 + 16 ϕ 20 (R+S)
8 M 690x360x217 + 20 ϕ 20 (R+S+T)
9 M 690x360x265 + 20 ϕ 20 (R+S+T)
10 M 750x380x257 + 20 ϕ 20 (R+S+T)
11 M 750x380x314 + 20 ϕ 20 (R+S+T)
12 M 840x400x299 + 16 ϕ 25 (R+S)
13 M 840x400x359 + 16 ϕ 25 (R+S)
14 M 920x420x342 + 20 ϕ 25 (R+S+T)
15 M 920x420x417 + 20 ϕ 25 (R+S+T)
16 M 1000x450x286 + 20 ϕ 25 (R+S+T)
17 M 1000x450x399 + 20 ϕ 25 (R+S+T)
18 M 1000x450x444 + 20 ϕ 25 (R+S+T)
19 M 1000x450x488 + 20 ϕ 25 (R+S+T)

- ① ROLLED PROFILE
② WELDED STIRRUPS
③ STEEL REBAR CAGE
④ SHEAR STUDS
⑤ REBARS (DIAMETER ϕ)
⑥ CONSTRUCTIONAL REBARS
⑦ CONCRETE





AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HP/W

1	HE 240 AA	+ 4 ϕ 20	(R)
2	HE 280 AA	+ 4 ϕ 25	(R)
3	HE 320 AA	+ 4 ϕ 25	(R)
4	HE 360 AA	+ 8 ϕ 20	(R+S)
5	HE 400 AA	+ 8 ϕ 20	(R+S)
6	HE 450 AA	+ 8 ϕ 20	(R)
7	HE 500 AA	+ 12 ϕ 20	(R+S)
8	HE 550 AA	+ 12 ϕ 20	(R+S)
9	HE 600 AA	+ 12 ϕ 20	(R+S)
10	HE 650 AA	+ 16 ϕ 20	(R+S)
11	HE 700 AA	+ 16 ϕ 20	(R+S)
12	HE 800 AA	+ 20 ϕ 20	(R+S+T)
13	HE 900 AA	+ 20 ϕ 20	(R+S+T)
14	HE 1000 AA	+ 16 ϕ 25	(R+S)

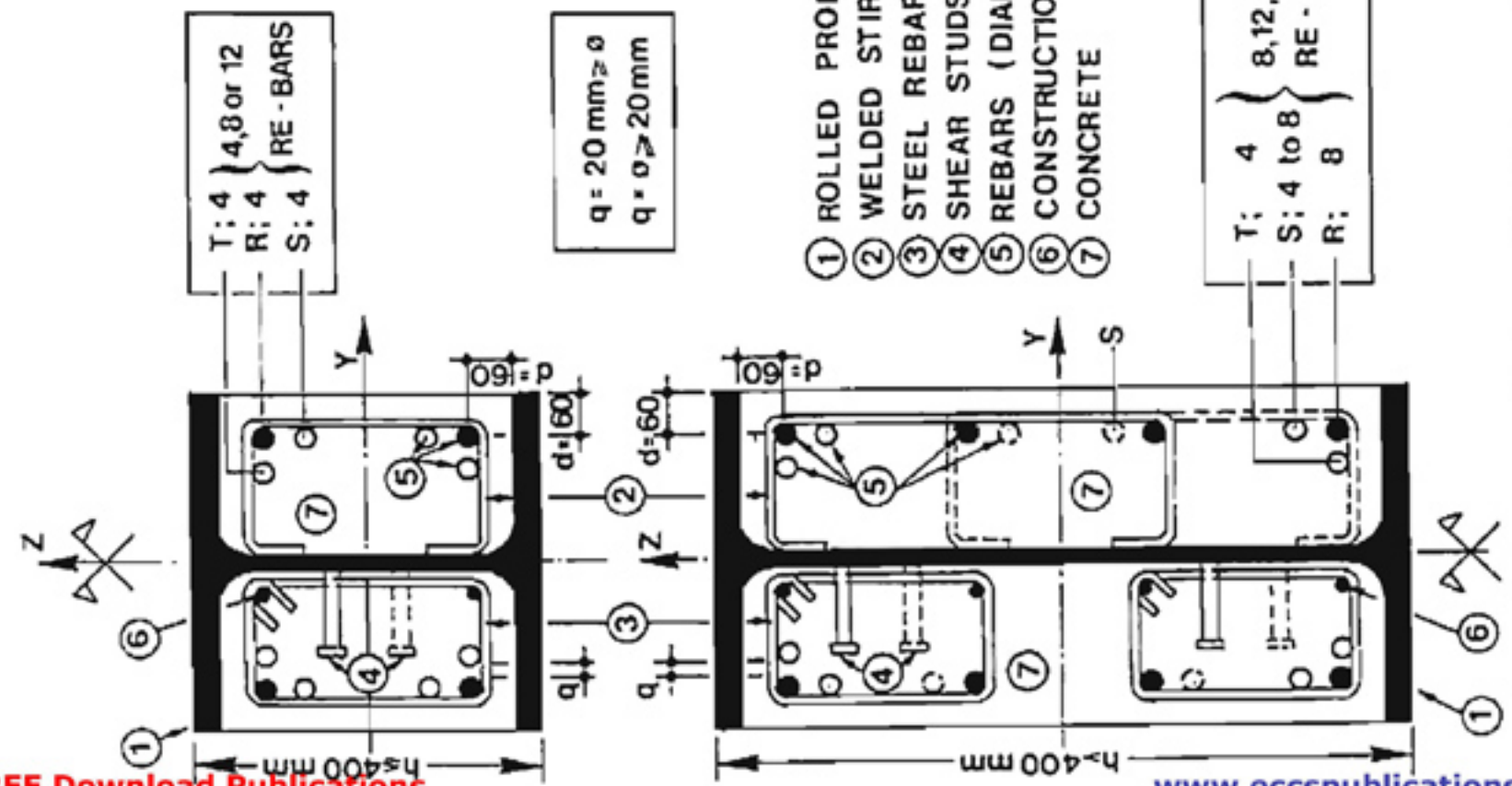
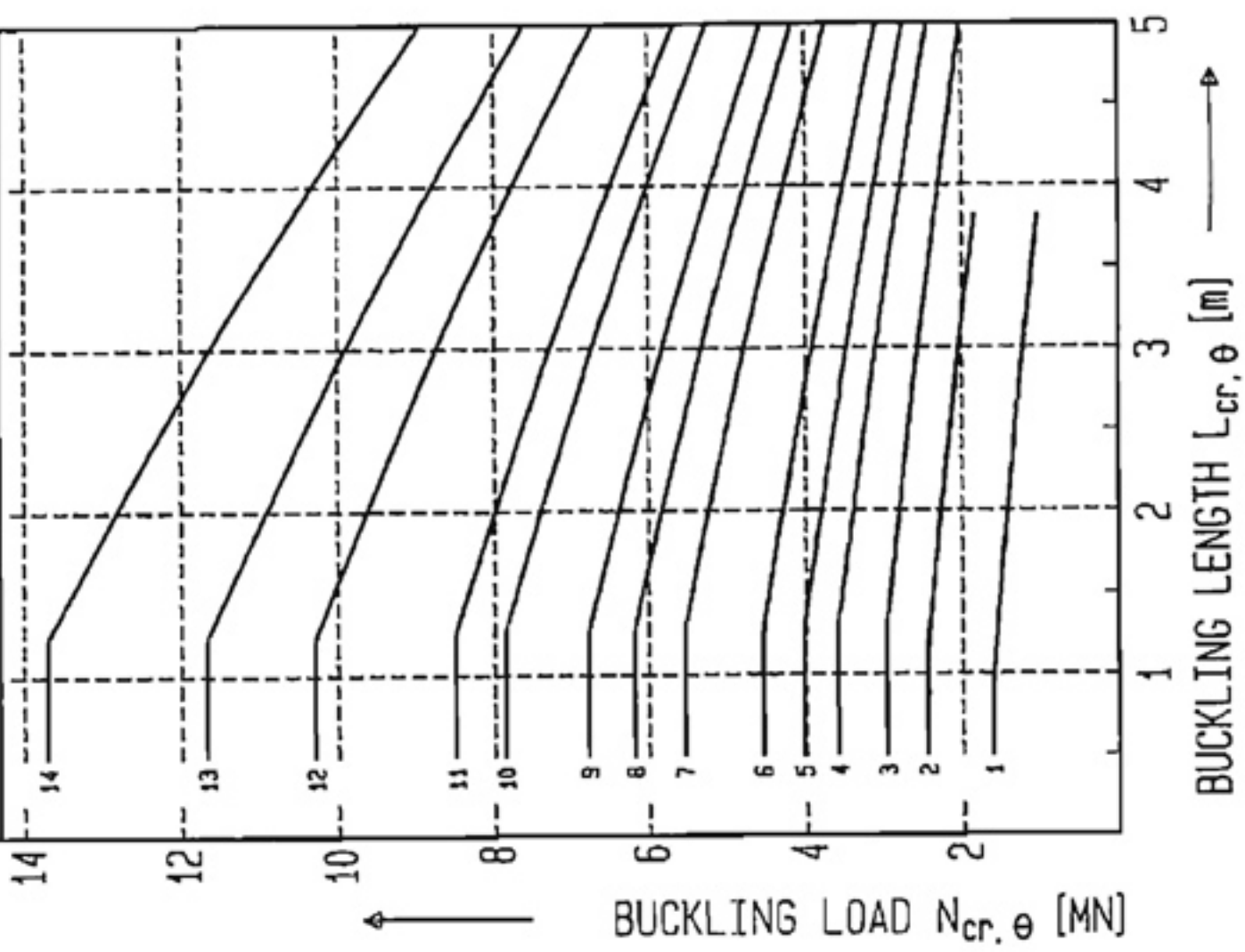


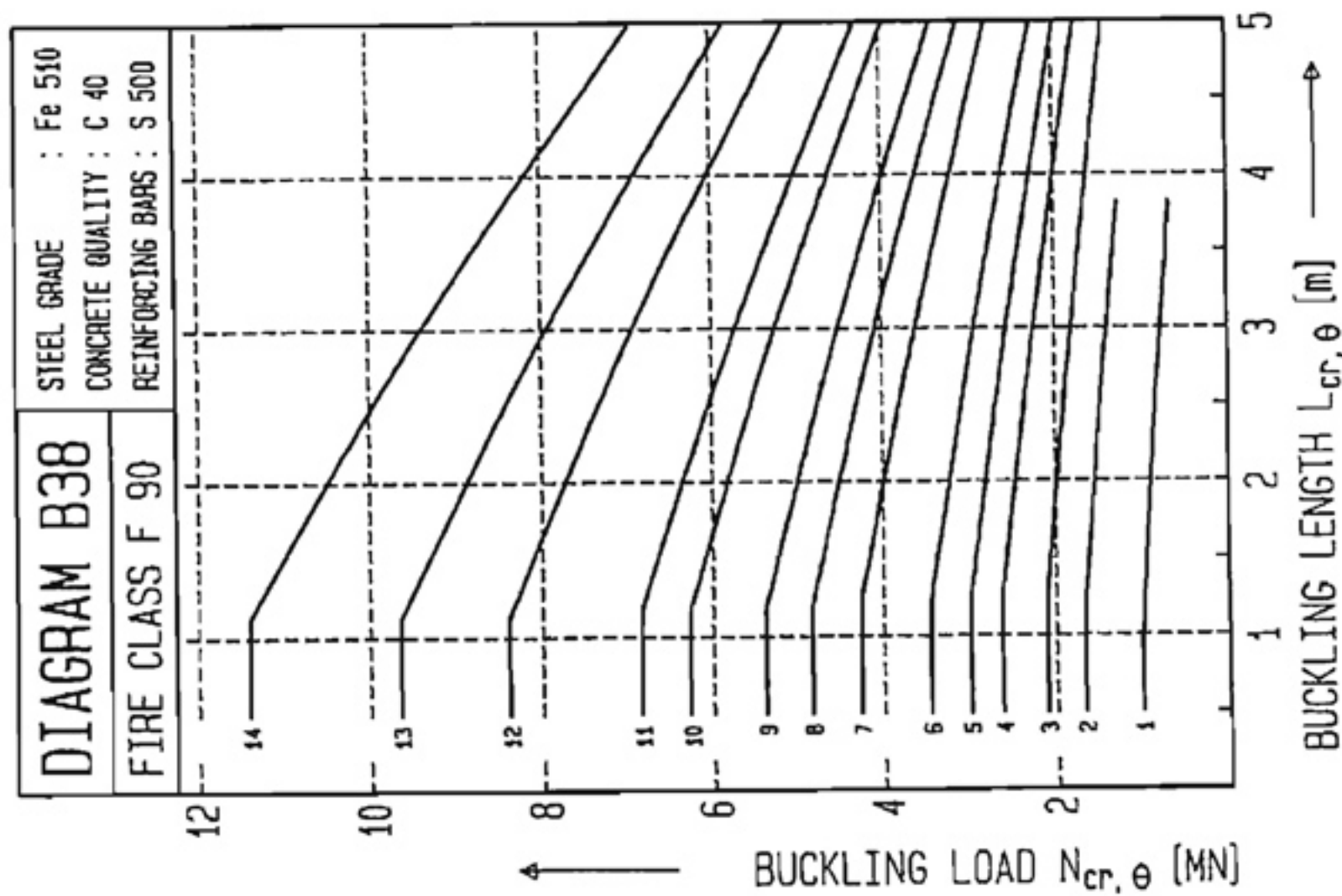
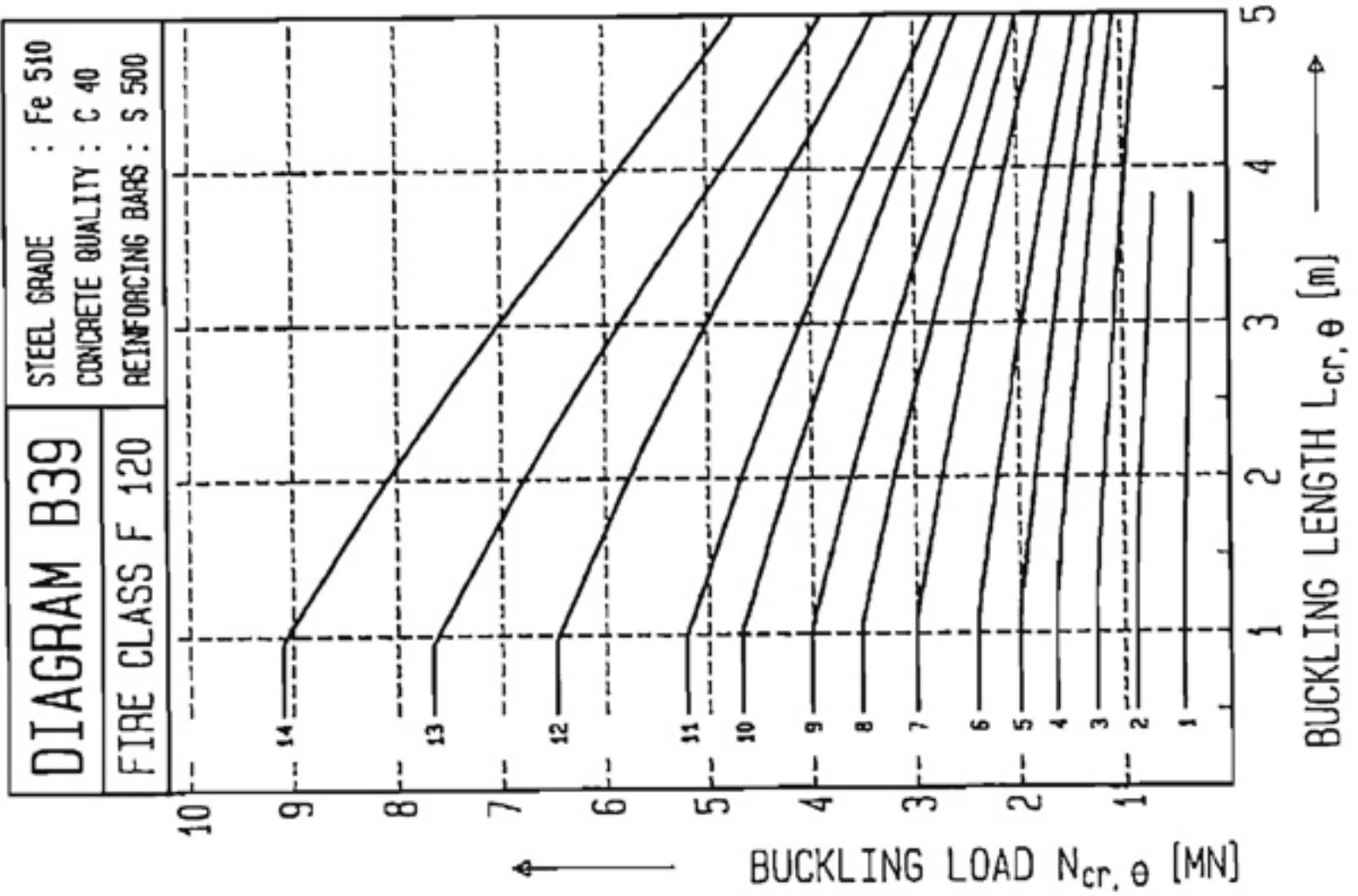
DIAGRAM B37

FIRE CLASS F 60

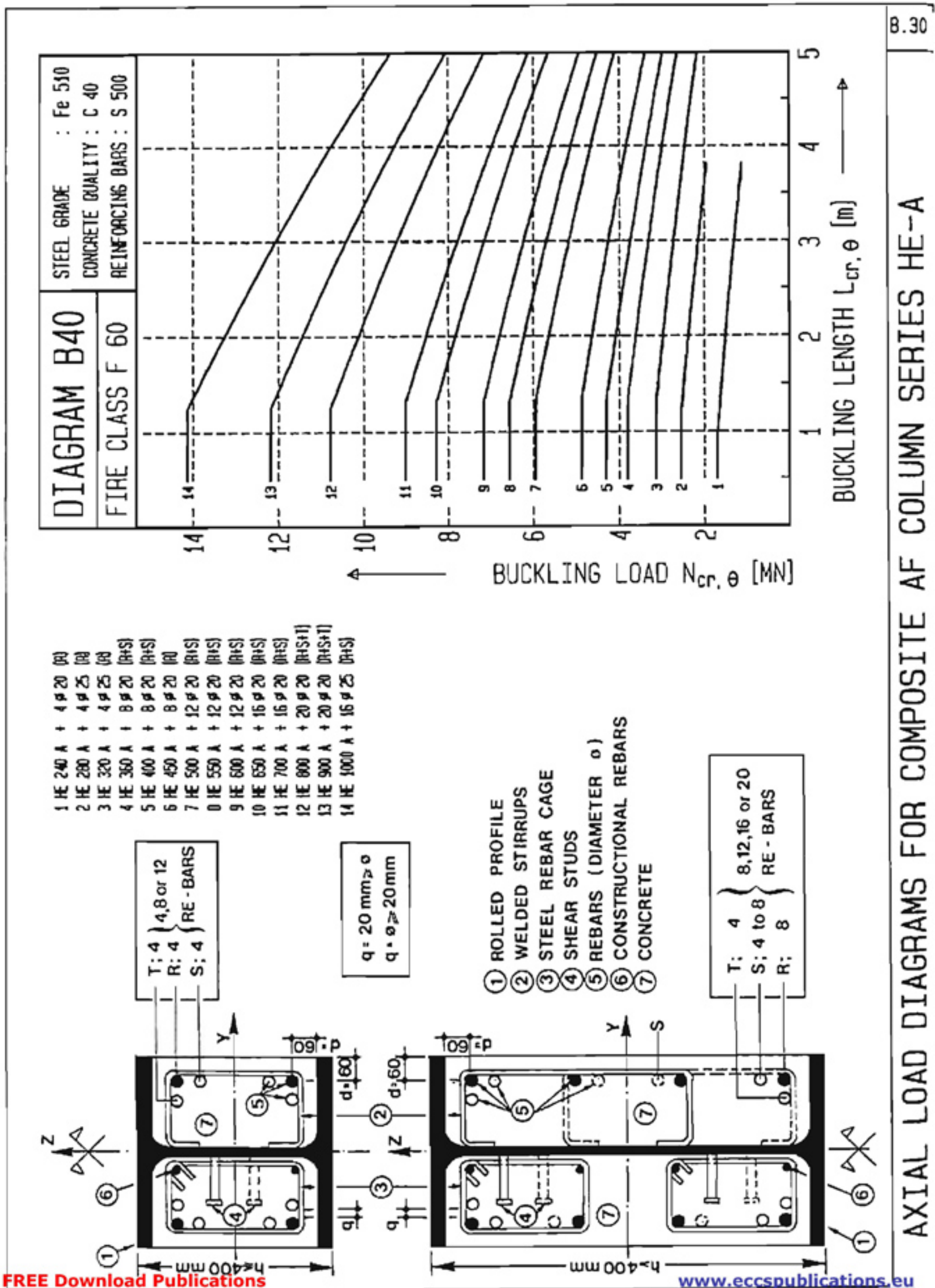
STEEL GRADE : Fe 510
CONCRETE QUALITY : C 40
REINFORCING BARS : S 500

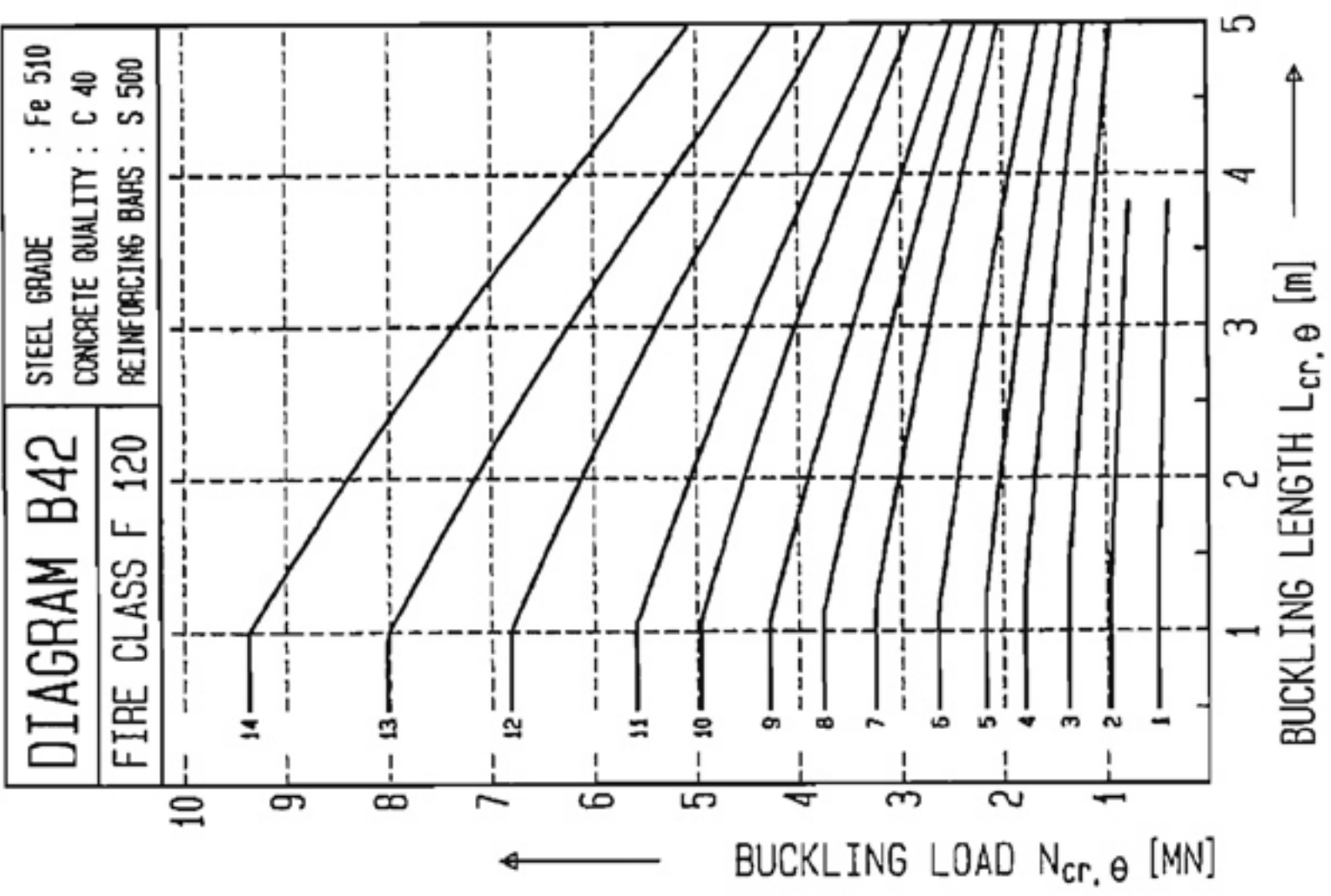
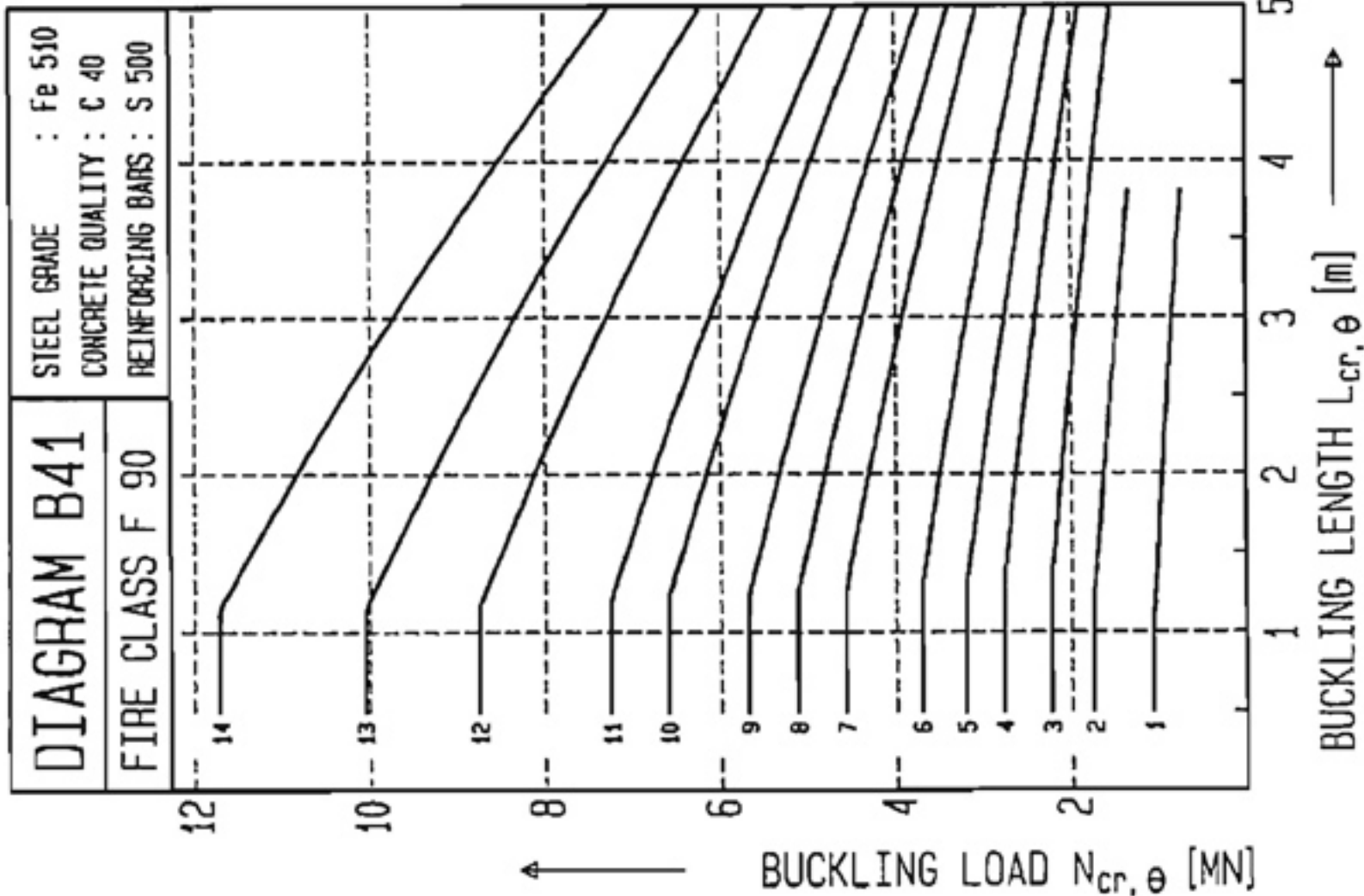


AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-AA



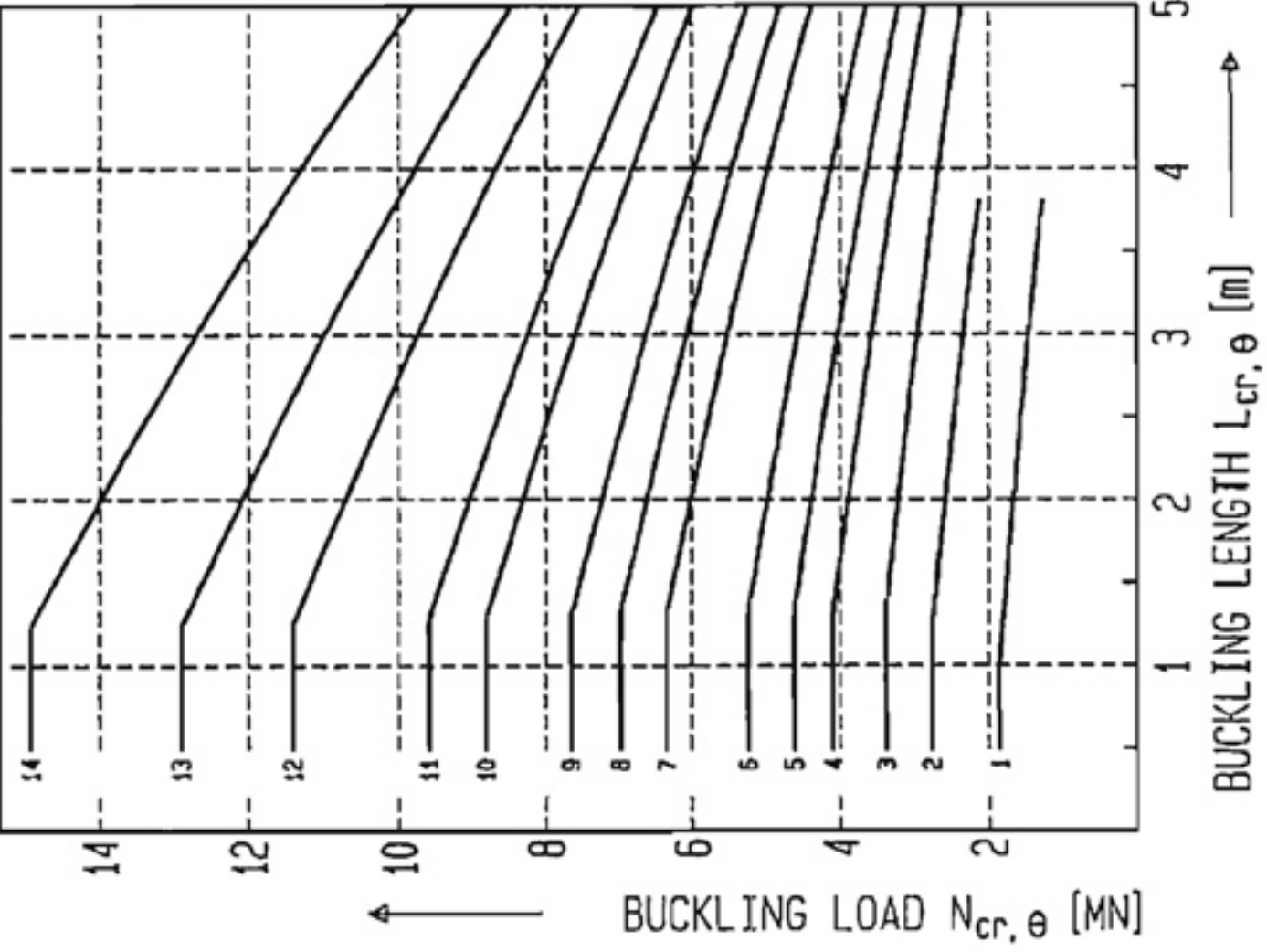
AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-AA



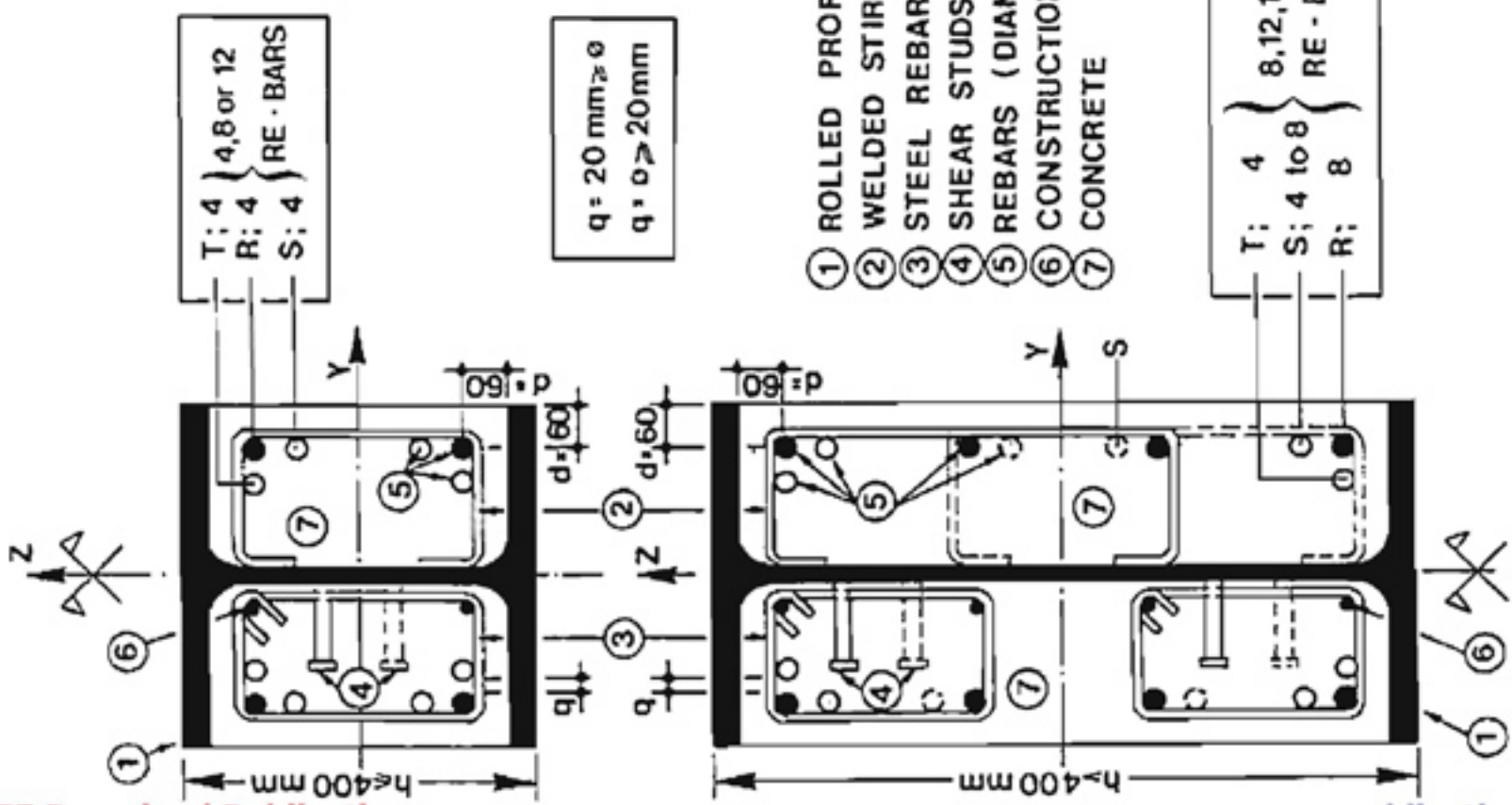


AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-A

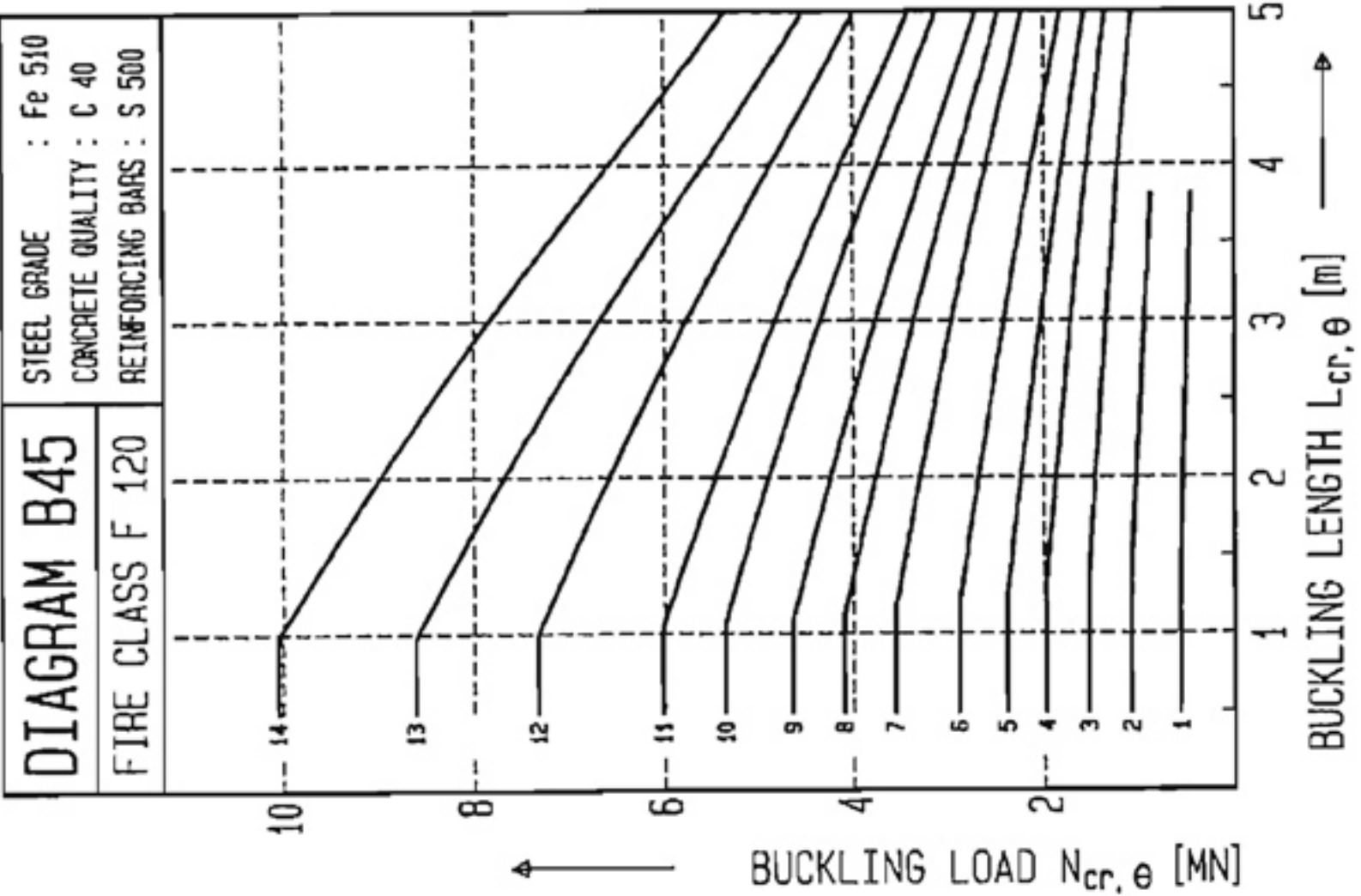
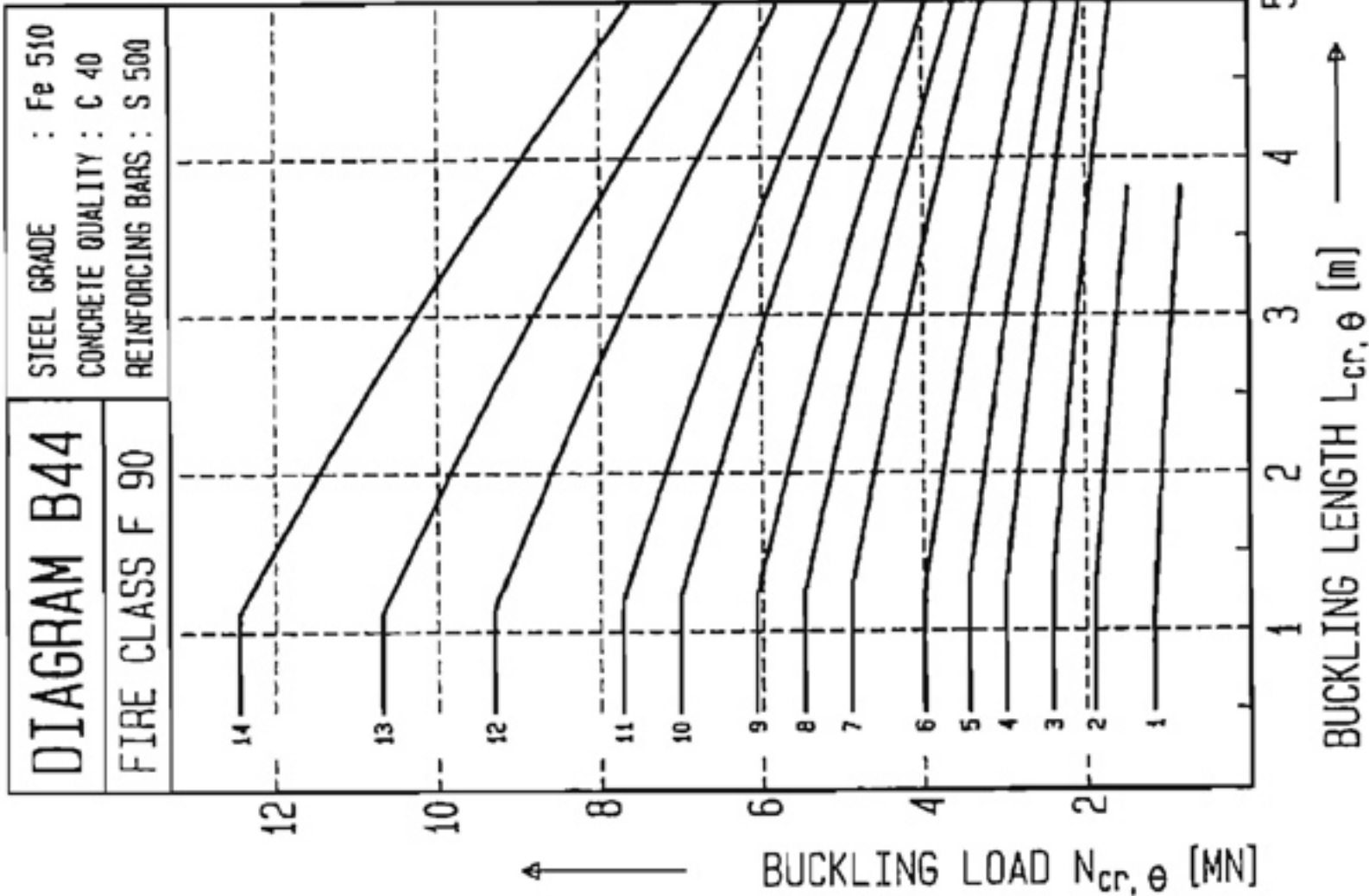
DIAGRAM B43
FIRE CLASS F 60
STEEL GRADE : Fe 510
CONCRETE QUALITY : C 40
REINFORCING BARS : S 500

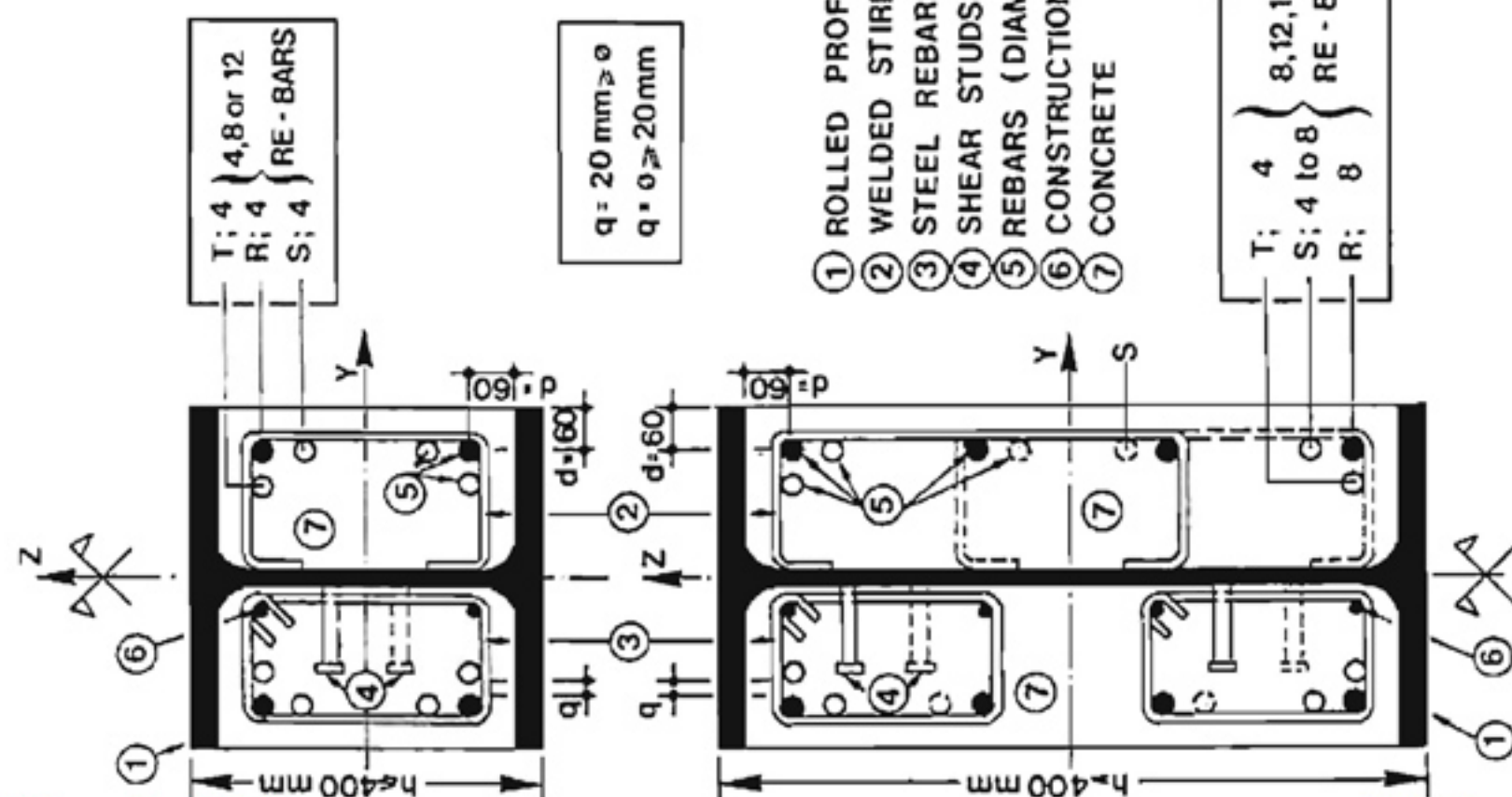
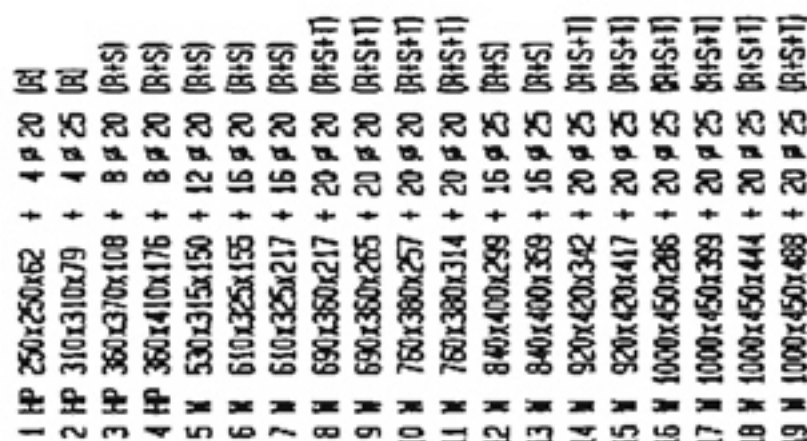


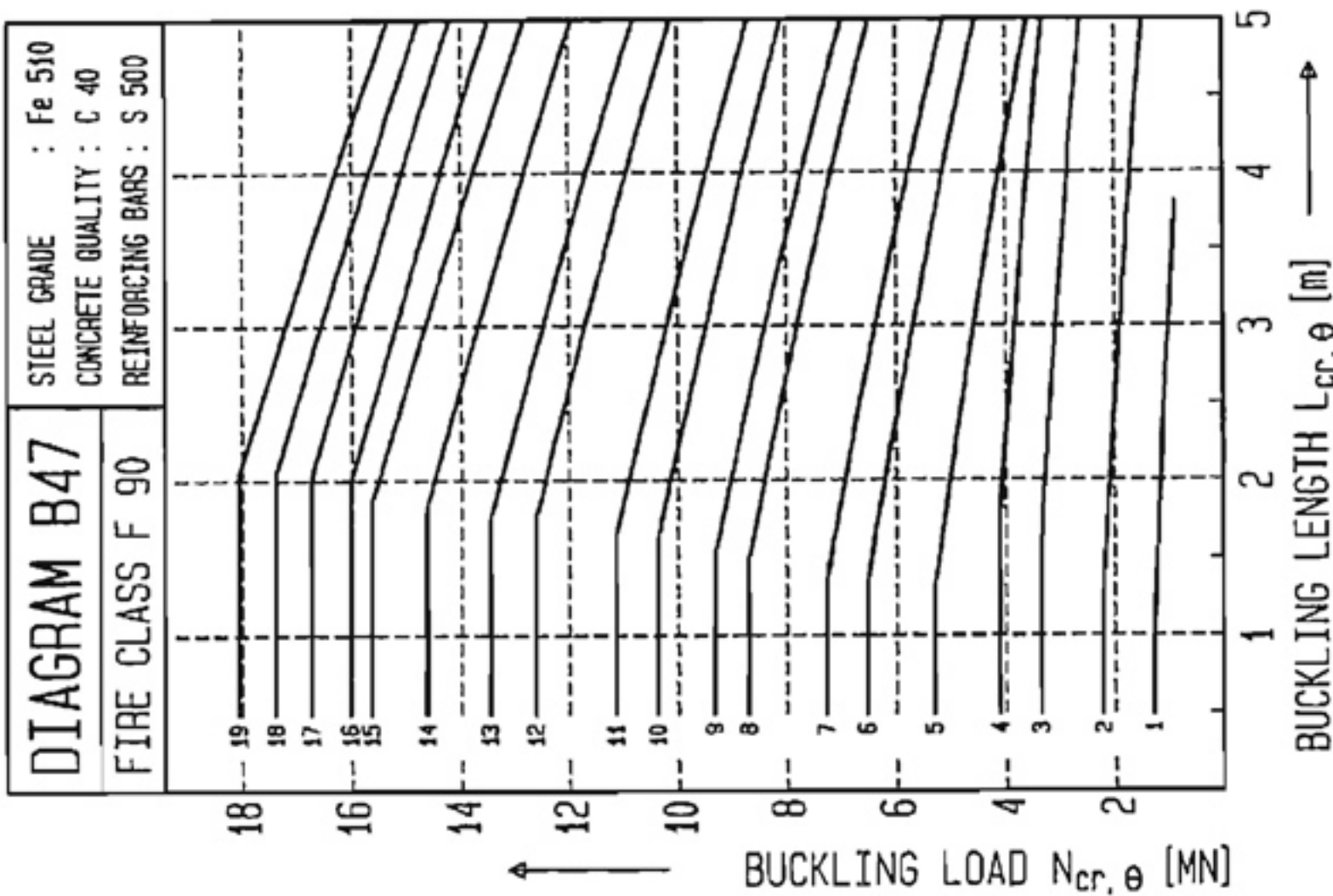
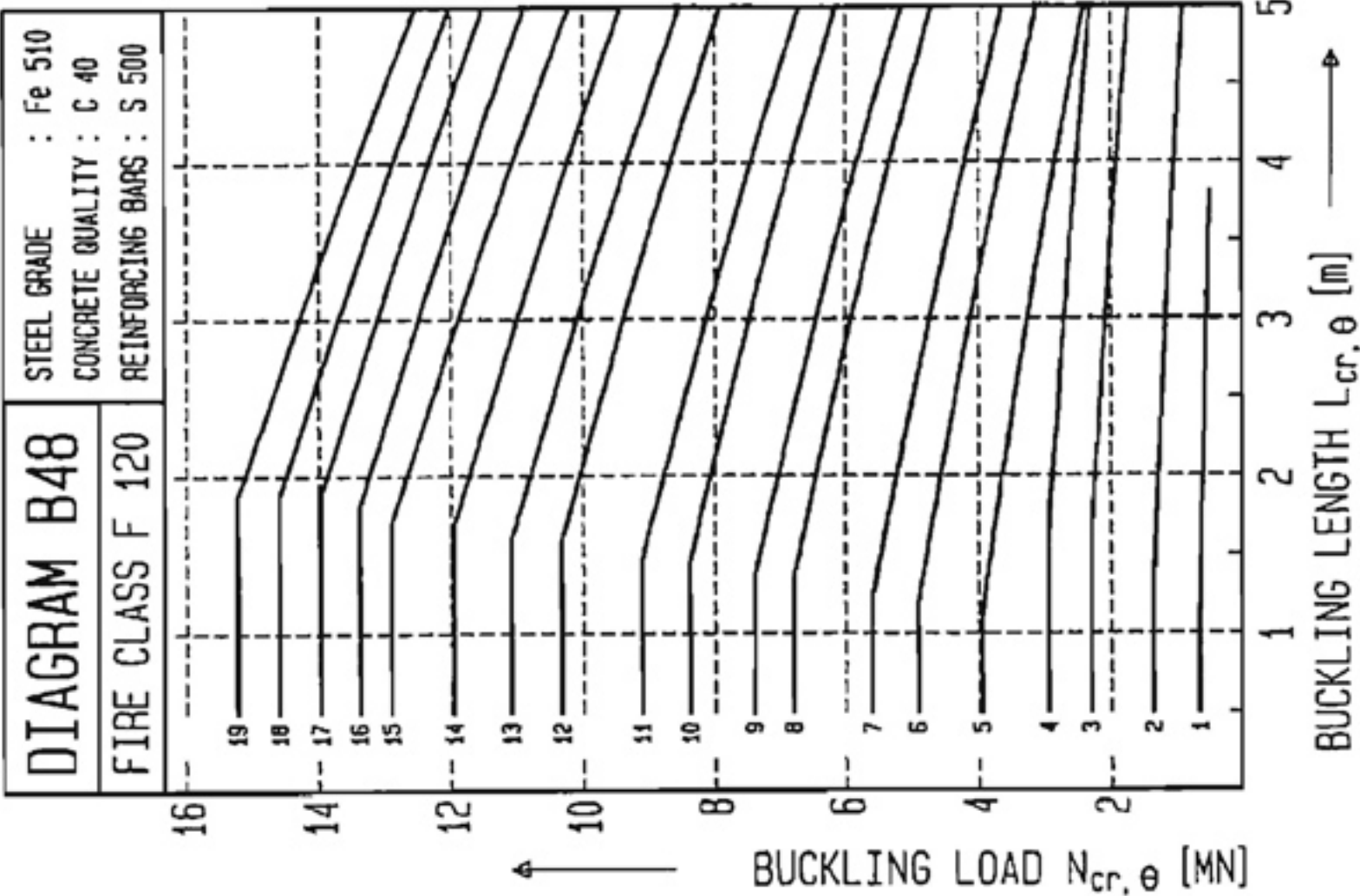
- 1 HE 240 B + 4 ϕ 20 (R)
- 2 HE 280 B + 4 ϕ 25 (R)
- 3 HE 320 B + 4 ϕ 25 (R)
- 4 HE 360 B + 8 ϕ 20 (R+S)
- 5 HE 400 B + 8 ϕ 20 (R+S)
- 6 HE 450 B + 8 ϕ 20 (R)
- 7 HE 500 B + 12 ϕ 20 (R+S)
- 8 HE 550 B + 12 ϕ 20 (R+S)
- 9 HE 600 B + 12 ϕ 20 (R+S)
- 10 HE 650 B + 16 ϕ 20 (R+S)
- 11 HE 700 B + 16 ϕ 20 (R+S)
- 12 HE 800 B + 20 ϕ 20 (R+S+T)
- 13 HE 900 B + 20 ϕ 20 (R+S+T)
- 14 HE 1000 B + 16 ϕ 25 (R+S)



AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HE-B







AXIAL LOAD DIAGRAMS FOR COMPOSITE AF COLUMN SERIES HP/W

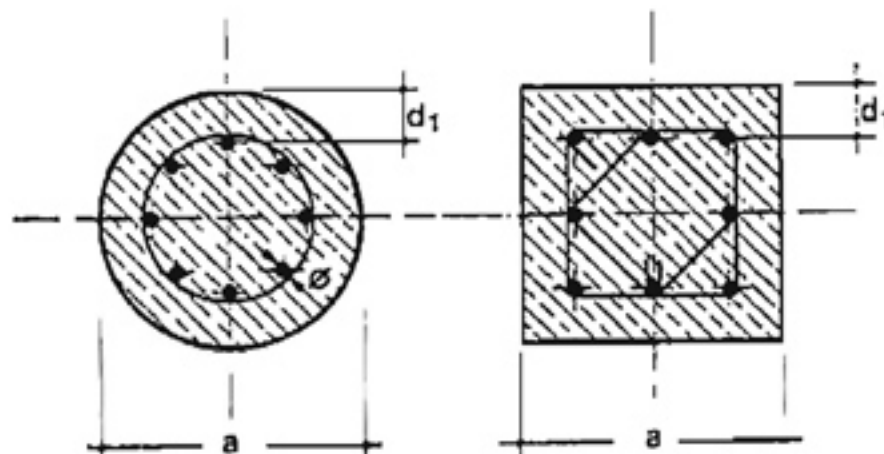
Appendix C: FIRE DESIGN DIAGRAMS FOR CONCRETE FILLED HOLLOW

STRUCTURAL SECTIONS

The design graphs given on pages C.4 to C.31 are limited to the following material and geometrical properties:

Material properties:

Combinations: C20/Fe360/S400
C30/Fe360/S400
C40/Fe360/S400



Geometrical properties:

Reinforcement: $\varnothing = 8, 10, 12, 14, 16, 20$ mm
diameter chosen to achieve $\mu = 1$ or 2,5 or 4 ‰

- circular hollow section: systematically 8 bars.
- square hollow section:
 - * 1st case: $a \geq 300$ mm, 8 bars
 - * 2nd case: $160 \leq a < 300$ mm
 - 4 bars if $\varnothing \leq 0,05 a$
 - otherwise 8 identical bars

Covers: $d_1 = \max (30 \text{ mm}; a/8 \text{ mm})$

Axial force eccentricity 0 mm

Buckling length: $1 \text{ m} \leq L_{cr,\theta} \leq 4,5 \text{ m}$

Profiles:

- Circular hollow section: 219,1 x 4,5 / 244,5 x 5 / 273 x 5
323,9 x 5,6 / 355,6 x 5,6 / 406,4 x 6,3
- Square hollow section : 180 x 6,3 / 200 x 6,3 / 220 x 6,3
250 x 6,3 / 260 x 6,3 / 300 x 7,1
350 x 8 / 400 x 10

The following 39 design graphs give interaction between buckling load $N_{cr,\theta}$ and buckling length $L_{cr,\theta}$ for the FIRE CLASSES F60, F90 and F120 and for different steel and concrete qualities.

The results have been established on the basis of the harmonized calculation model according to the chapter 5.2.

The results do practically not change and are on the safe side, whether a higher steel grade or a larger wall thickness is chosen for the hollow section.

Example 1:

CHECKING FOR TOP FLOOR COLUMNS: F90

EXISTING CROSS SECTION

Square hollow section: 220 x 6,3/Fe 360
Reinforcement: $\mu = 2,5 \%$, S 400
Concrete: C 40

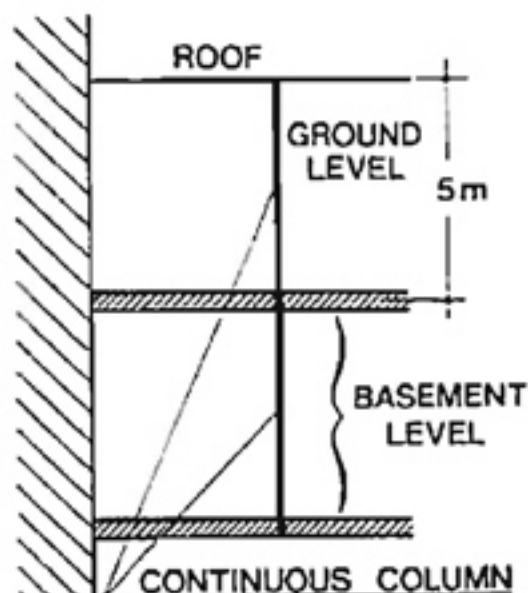
EXISTING AXIAL DESIGN LOAD: $N = 280 \text{ kN}$

Top floor columns (see chapter 2.4.4)
 $L_{cr,\theta} = 0,7 \times L = 3,5 \text{ m}$

On diagram C24, we have

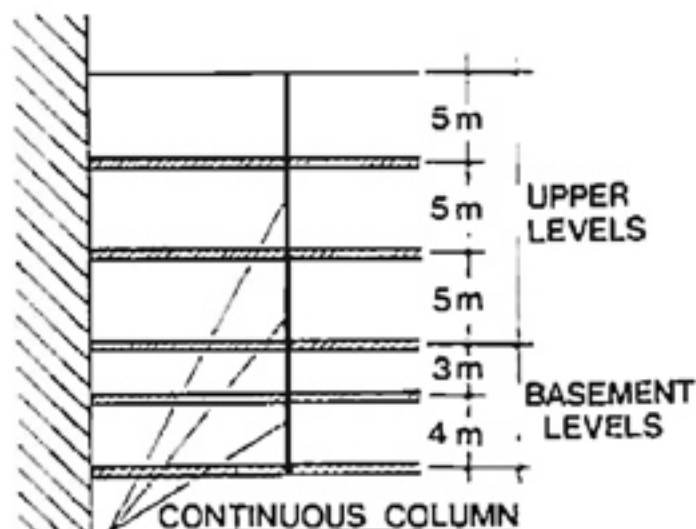
Curve 8
 $L_{cr,\theta} = 3,5 \text{ m}$ } $N_{cr,90} = 316 \text{ kN} > \text{existing axial design load} = 280 \text{ kN}.$

For the design load of 280 kN, these top floor columns can be classified F90.



Example 2:

DESIGN OF CENTRALLY LOADED BASEMENT COLUMNS: F60



EXISTING CONDITIONS: $L = 3 \text{ m}$

$N = 1330 \text{ kN}$

The designer wants that the dimensions of hollow section is less than 280 mm.

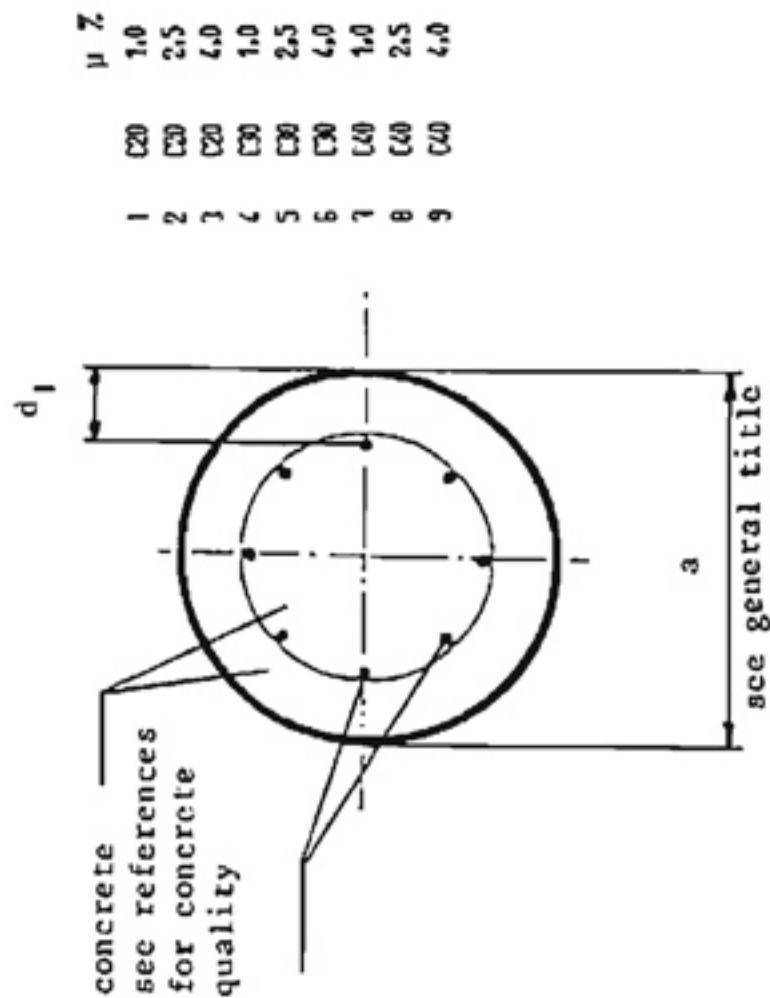
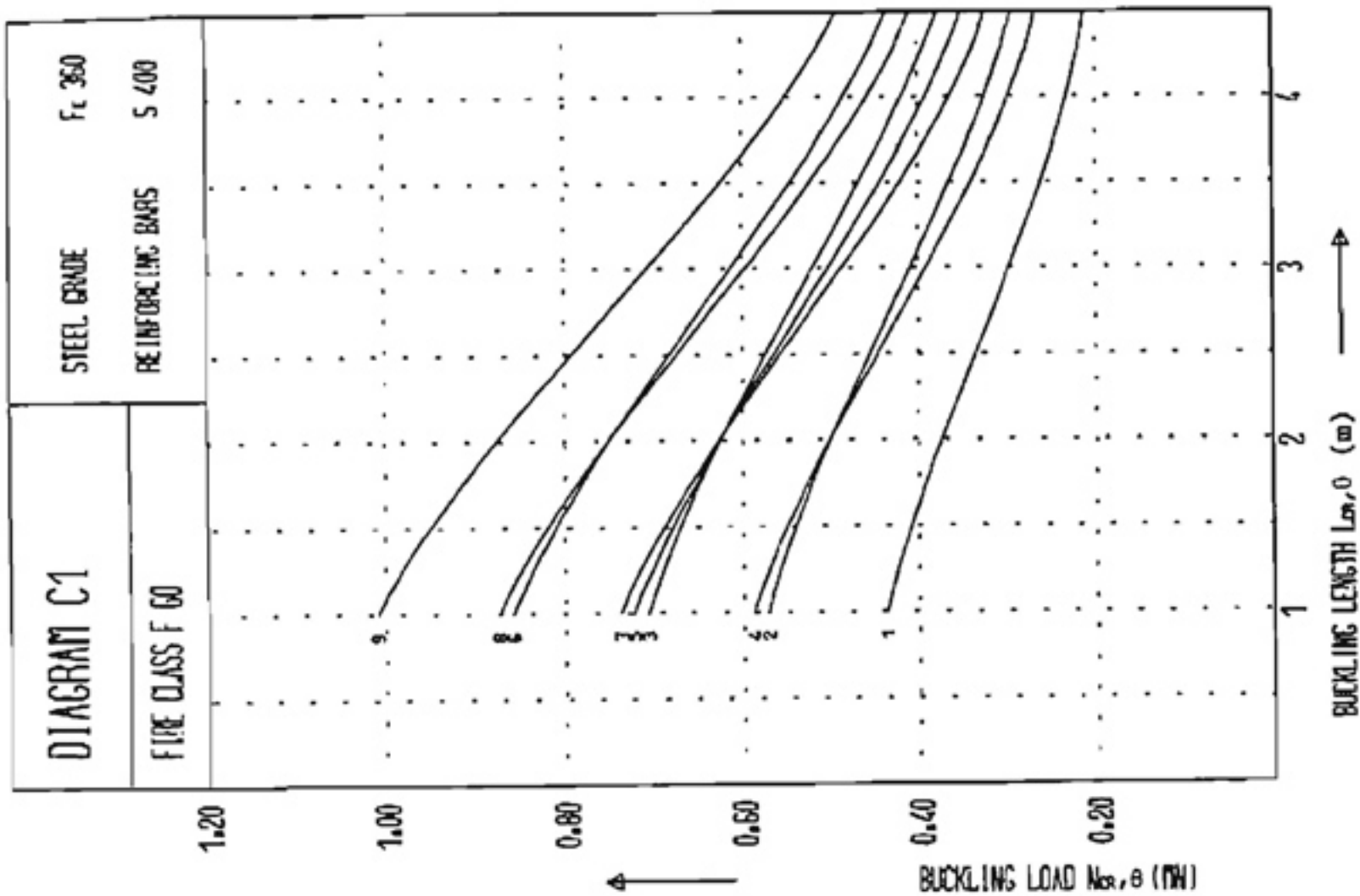
CONTINUOUS COLUMNS (see chapter 2.4.4) allow to consider that
 $L_{cr,\theta} = 0,5 \times L = 1,5 \text{ m}.$

Knowing the buckling length $L_{cr,\theta}$ and the design load it is possible to choose the requested hollow section:

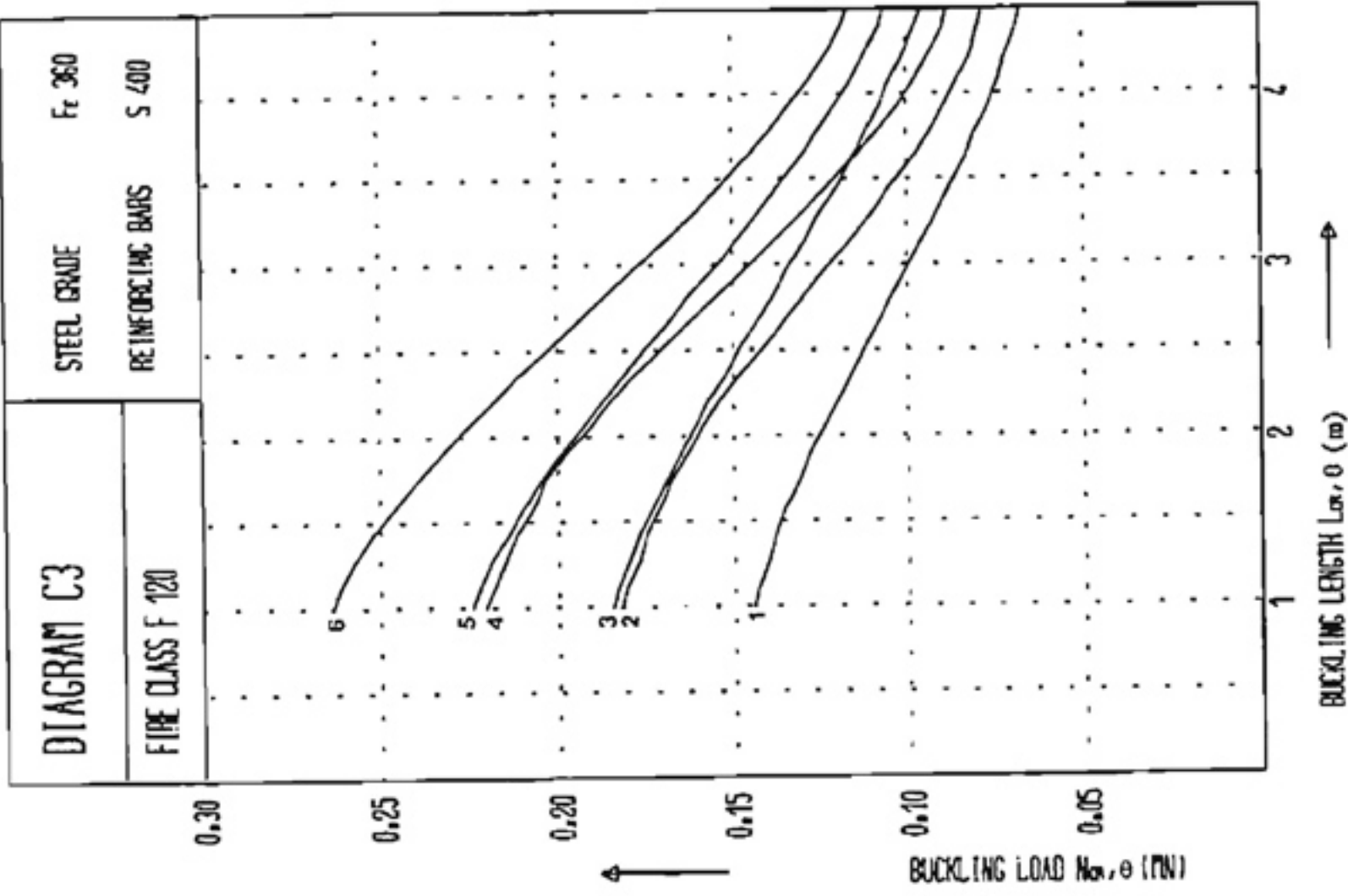
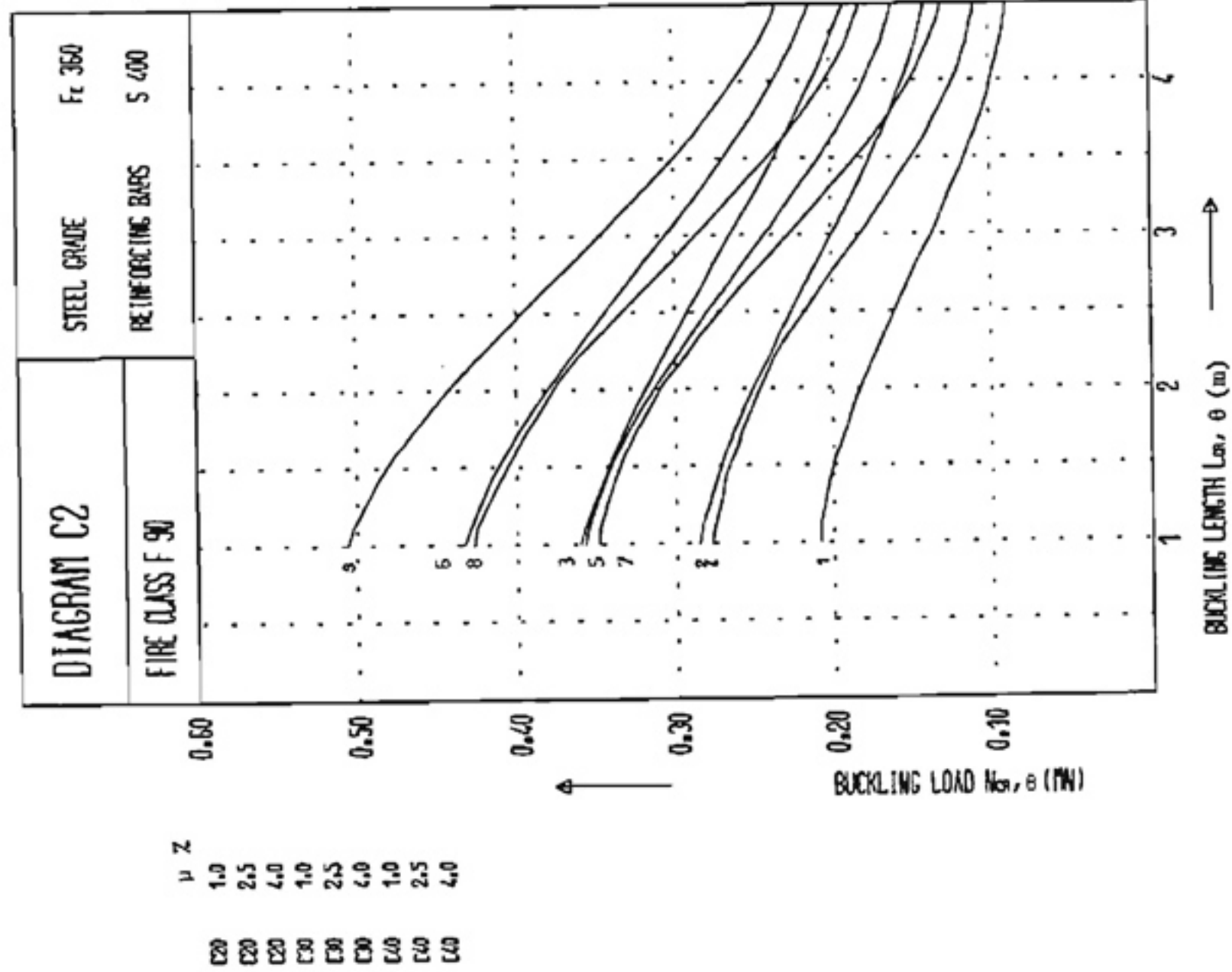
- Diagram C7 - curves 6 to 8 - circular hollow section 273 x 5
curve 8 : concrete quality C40; reinforcement: $\mu = 2,5 \%$
- Diagram C25 - curves 5 to 9 - square hollow section 250 x 6,3
curve 6 : concrete quality C30; reinforcement: $\mu = 4 \%$
- Diagram C29 - curves 5 to 9 - square hollow section 260 x 6,3
curve 7 : concrete quality C40; reinforcement: $\mu = 1 \%$.

Survey on diagrams

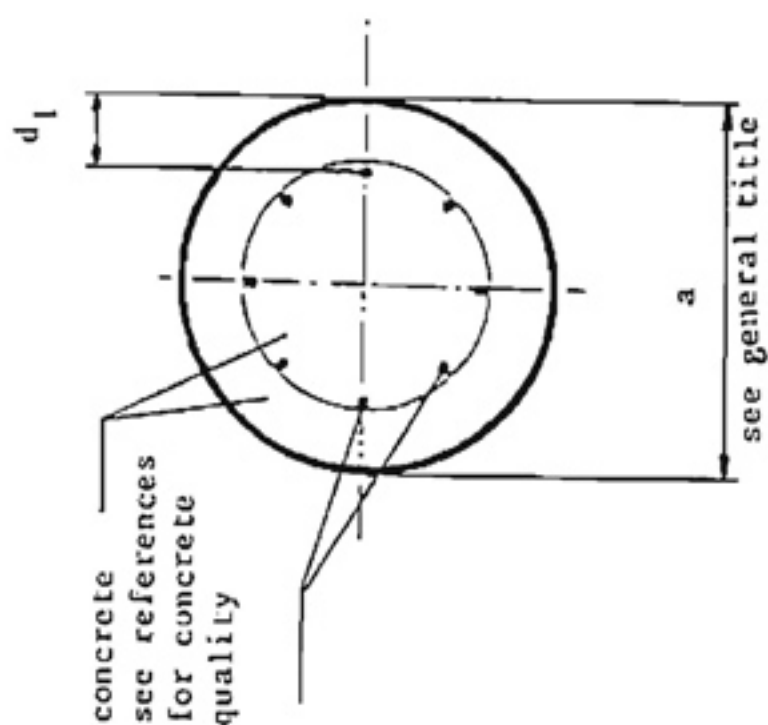
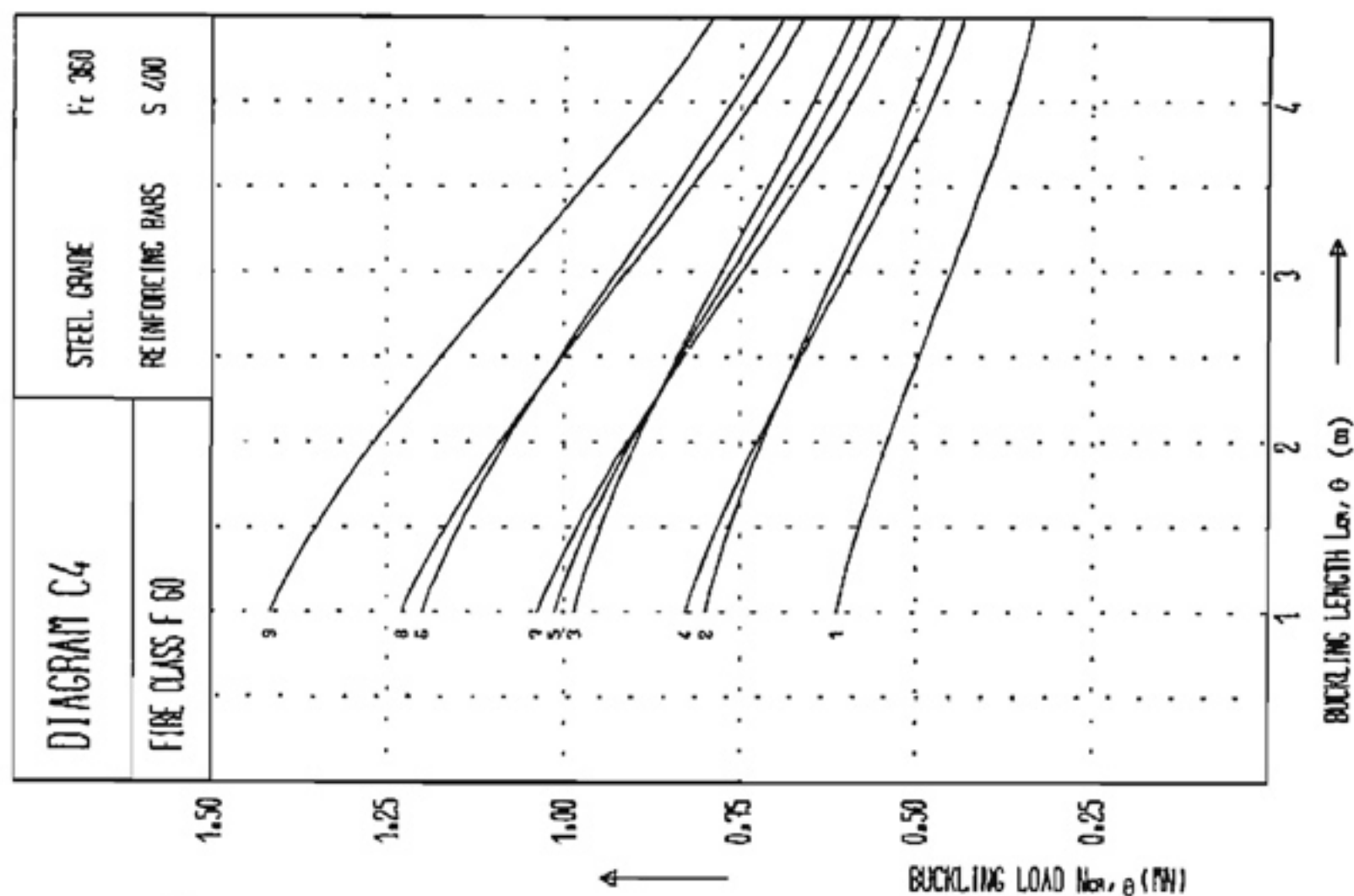
Fire class	Concrete quality	Section type	Section steel quality	Reinforcement	Section	Diagrams N°
F60 F90 F120	C20 C30 C40	219,1 . 4,5	Fe 360	S 400 (see diagrams)	CIRCULAR	C1 C2 C3
F60 F90 F120		244,5 . 5				C4 C5 C6
F60 F90 F120		273 . 5				C7 C8 C9
F60 F90 F120		323,9 . 5,6				C10 C11 C12
F60 F90 F120		355,6 . 5,6				C13 C14 C15
F60 F90 F120		406,4 . 6,3				C16 C17 C18
F60 F90	C20 C30 C40	180 . 6,3	Fe 360	S 400 (see diagrams)	SQUARE	C19 C20
F60 F90		200 . 6,3				C21 C22
F60 F90		220 . 6,3				C23 C24
F60 F90 F120		250 . 6,3				C25 C26 C27
F60 F90 F120		260 . 6,3				C28 C29 C30
F60 F90 F120		300 . 7,1				C31 C32 C33
F60 F90 F120		350 . 8				C34 C35 C36
F60 F90 F120		400 . 10				C37 C38 C39



AXIAL LOAD DIAGRAMS FOR COLUMN WITH CIRCULAR HOLLOW SECTION 219₀1₀4₀5



AXIAL LOAD DIAGRAMS FOR COLUMN WITH CIRCULAR HOLLOW SECTION 219.1*4.5

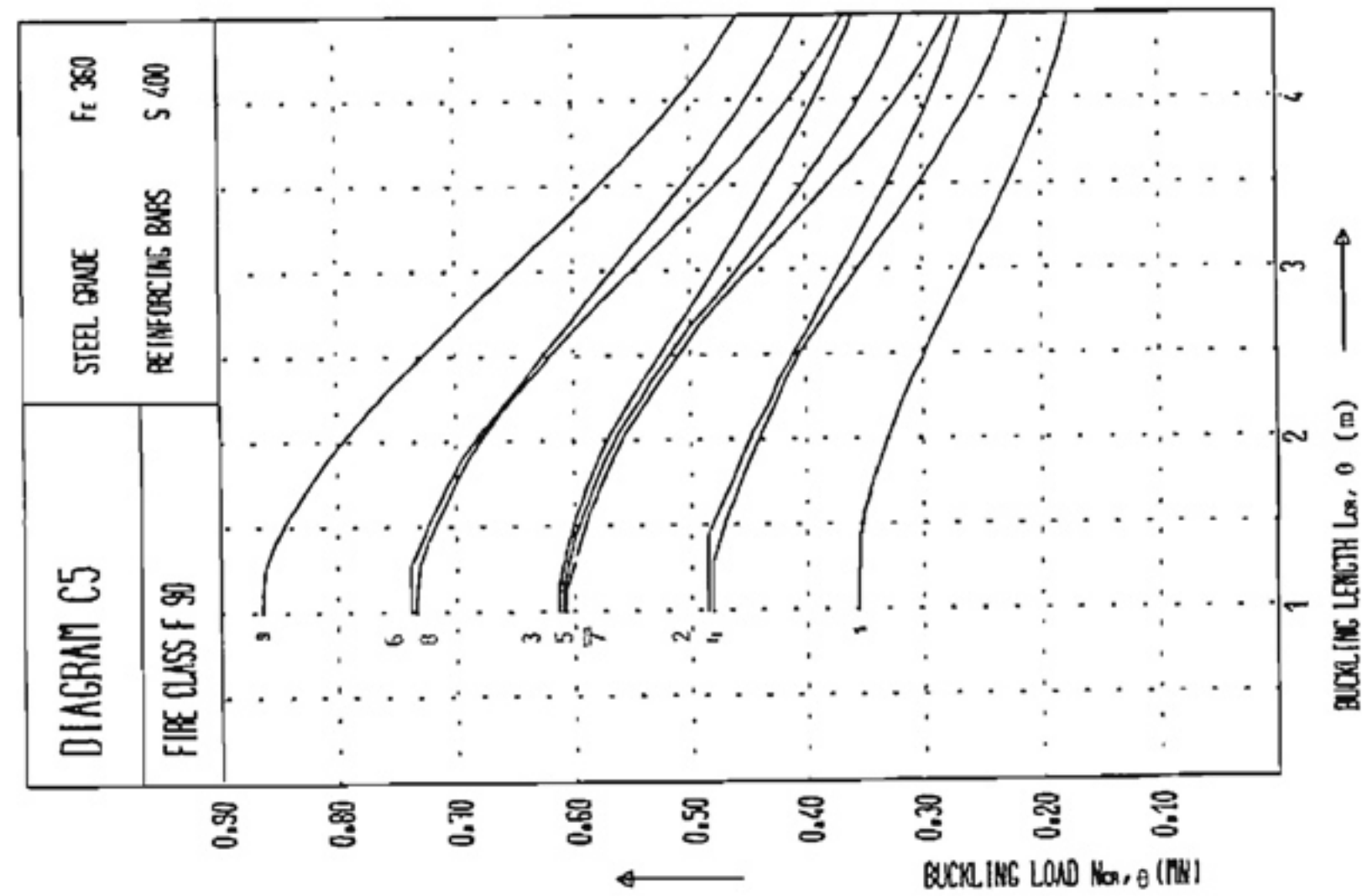
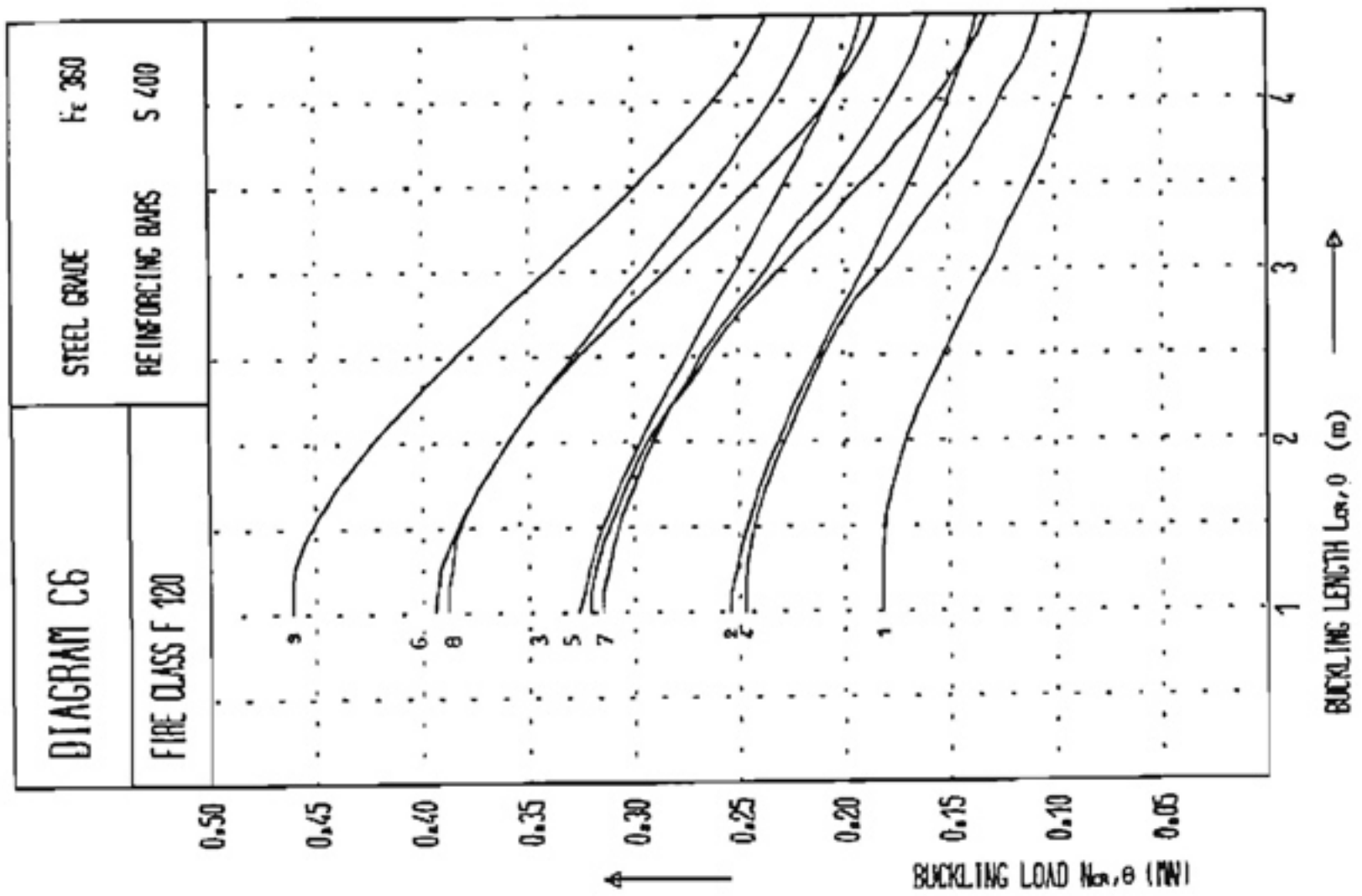


reinforcing bars
see references
or percentage
of reinforcement

concrete —
see references
for concrete
quality

see general title

AXIAL LOAD DIAGRAMS FOR COLUMN WITH CIRCULAR HOLLOW SECTION 244.5*5.0

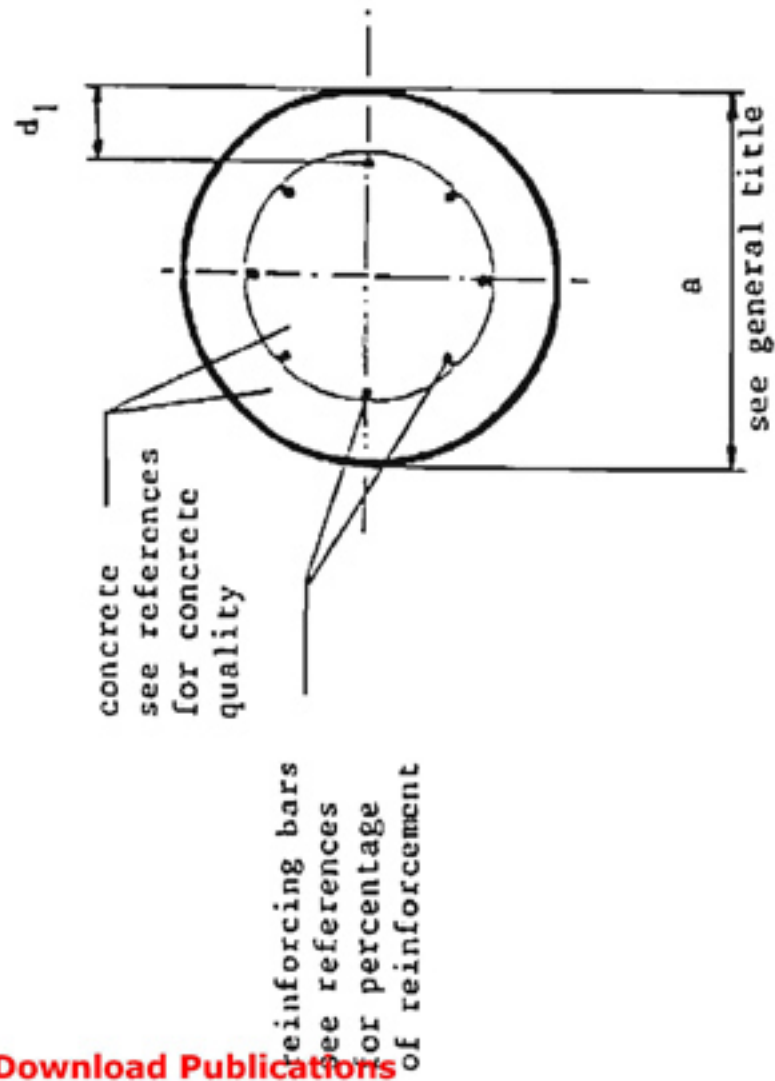
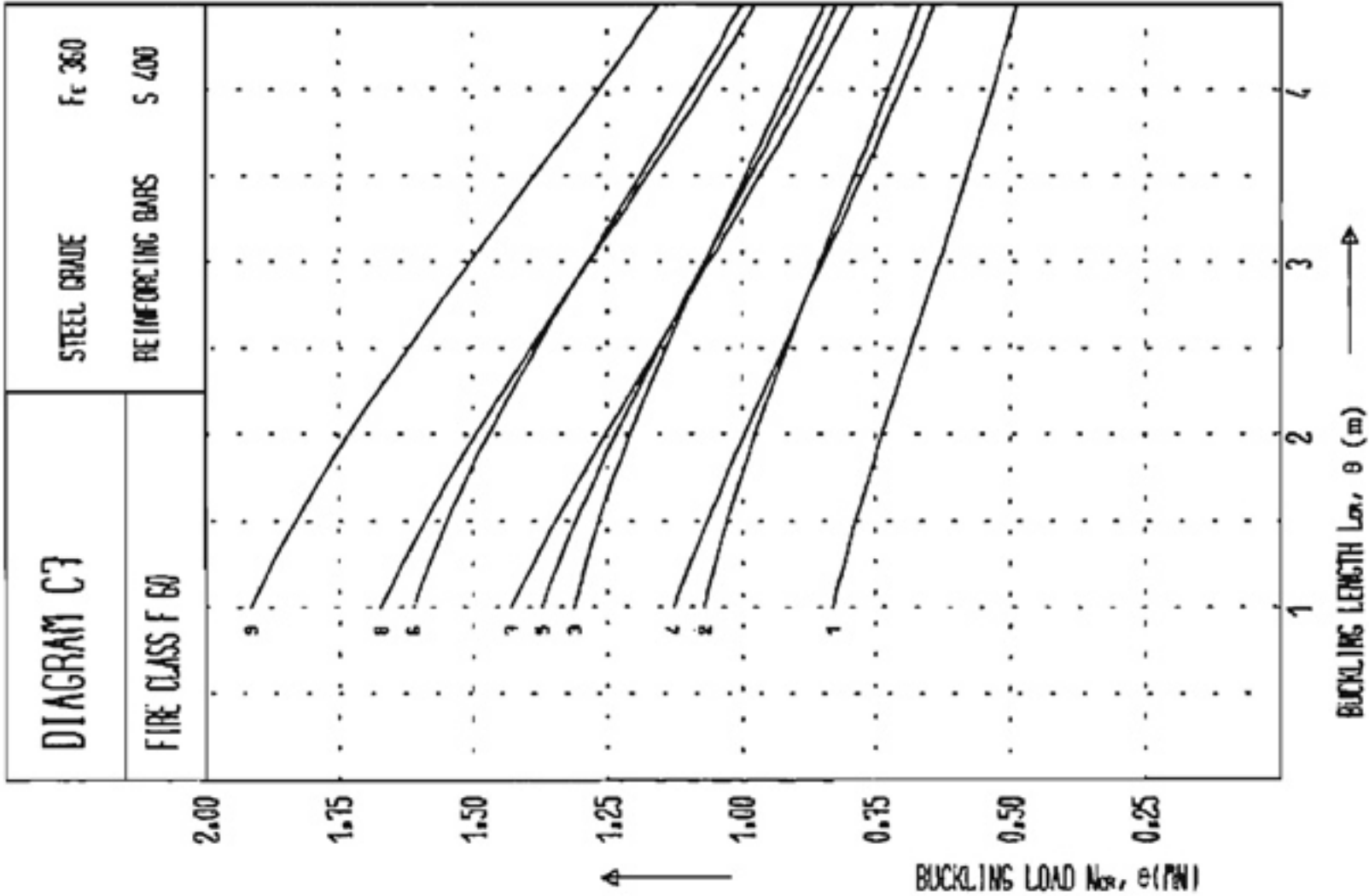


ν z

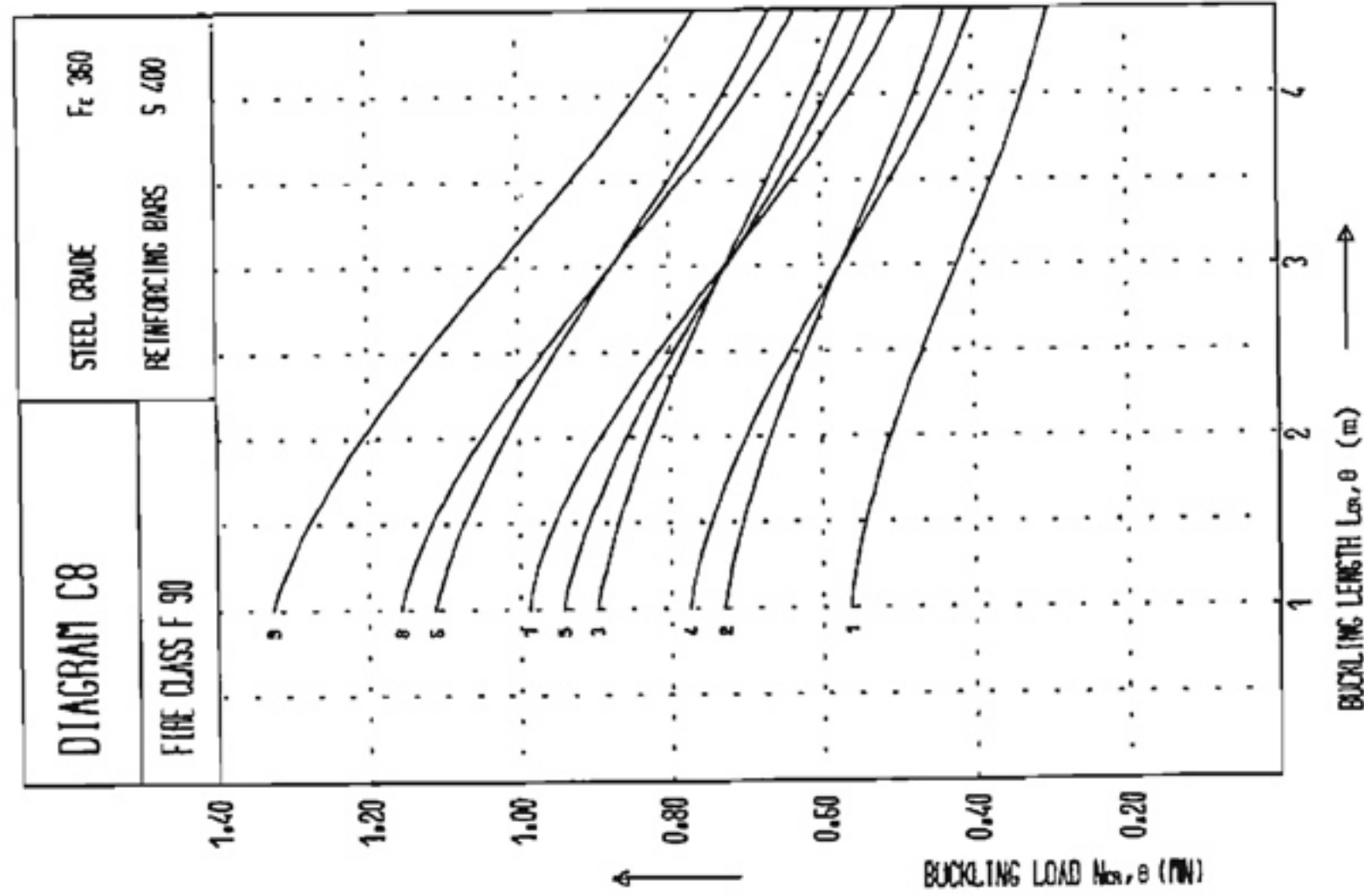
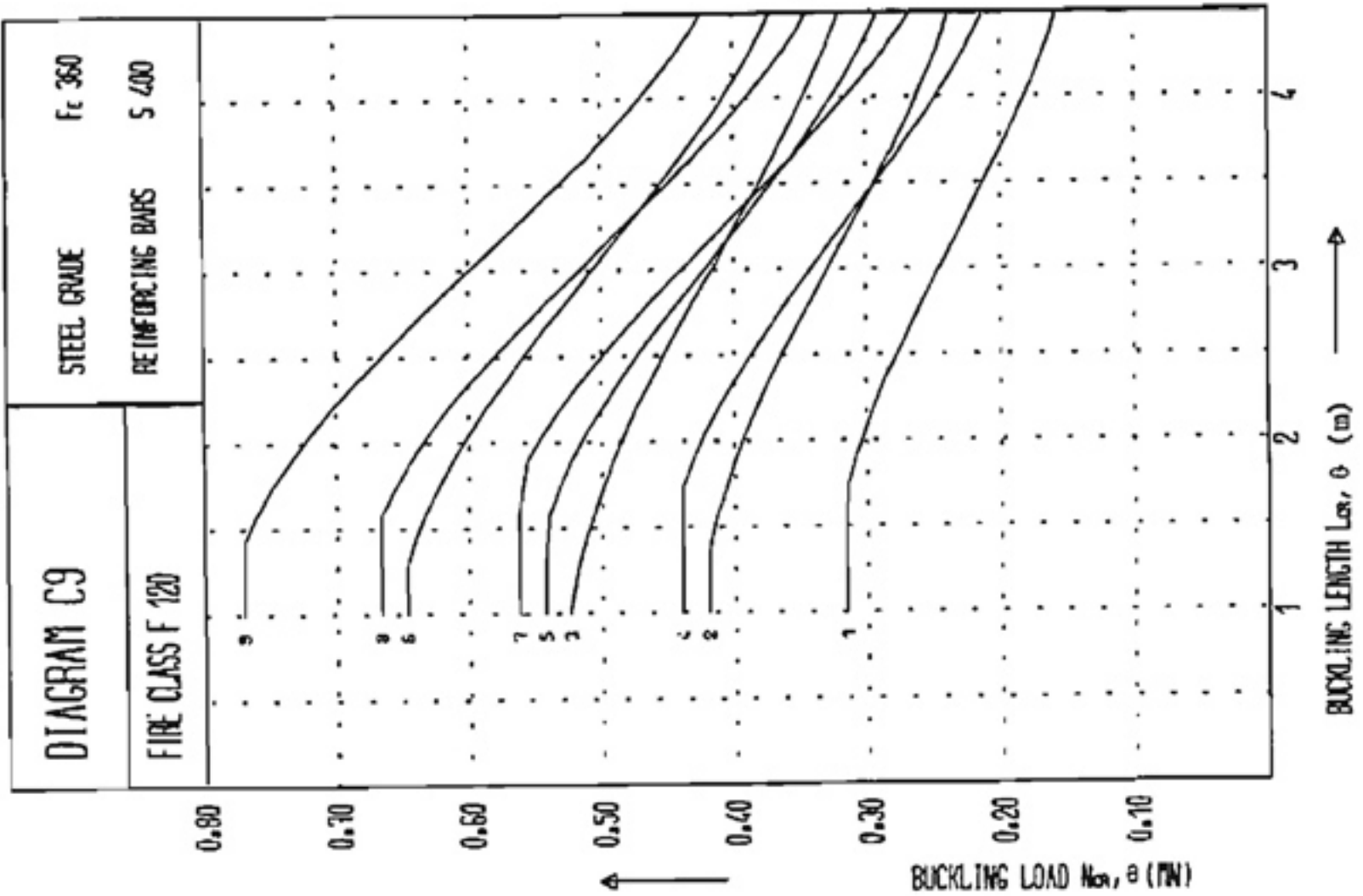
C20	1.0
C20	2.5
C20	4.0
C30	1.0
C30	2.5
C30	4.0
C40	1.0
C40	2.5
C40	4.0

1 8 9

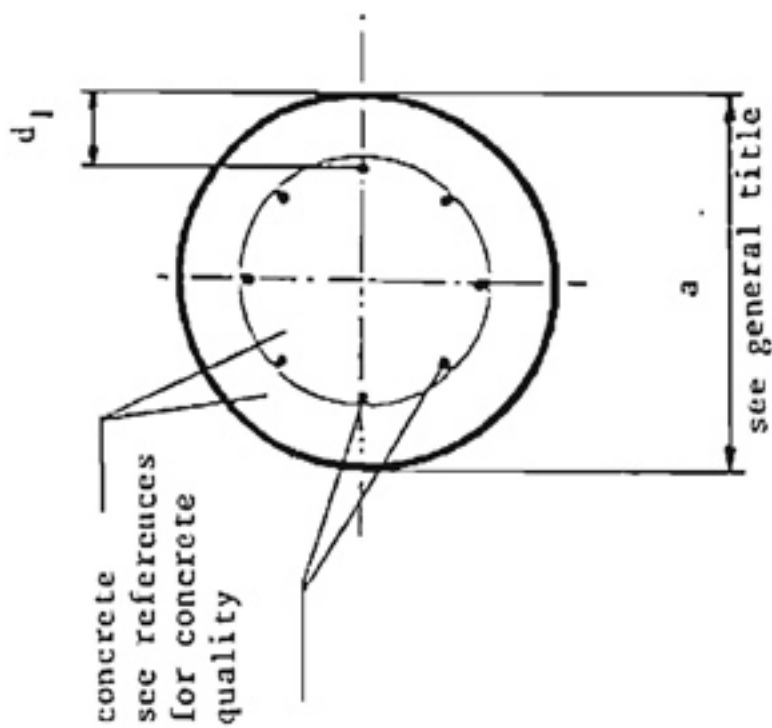
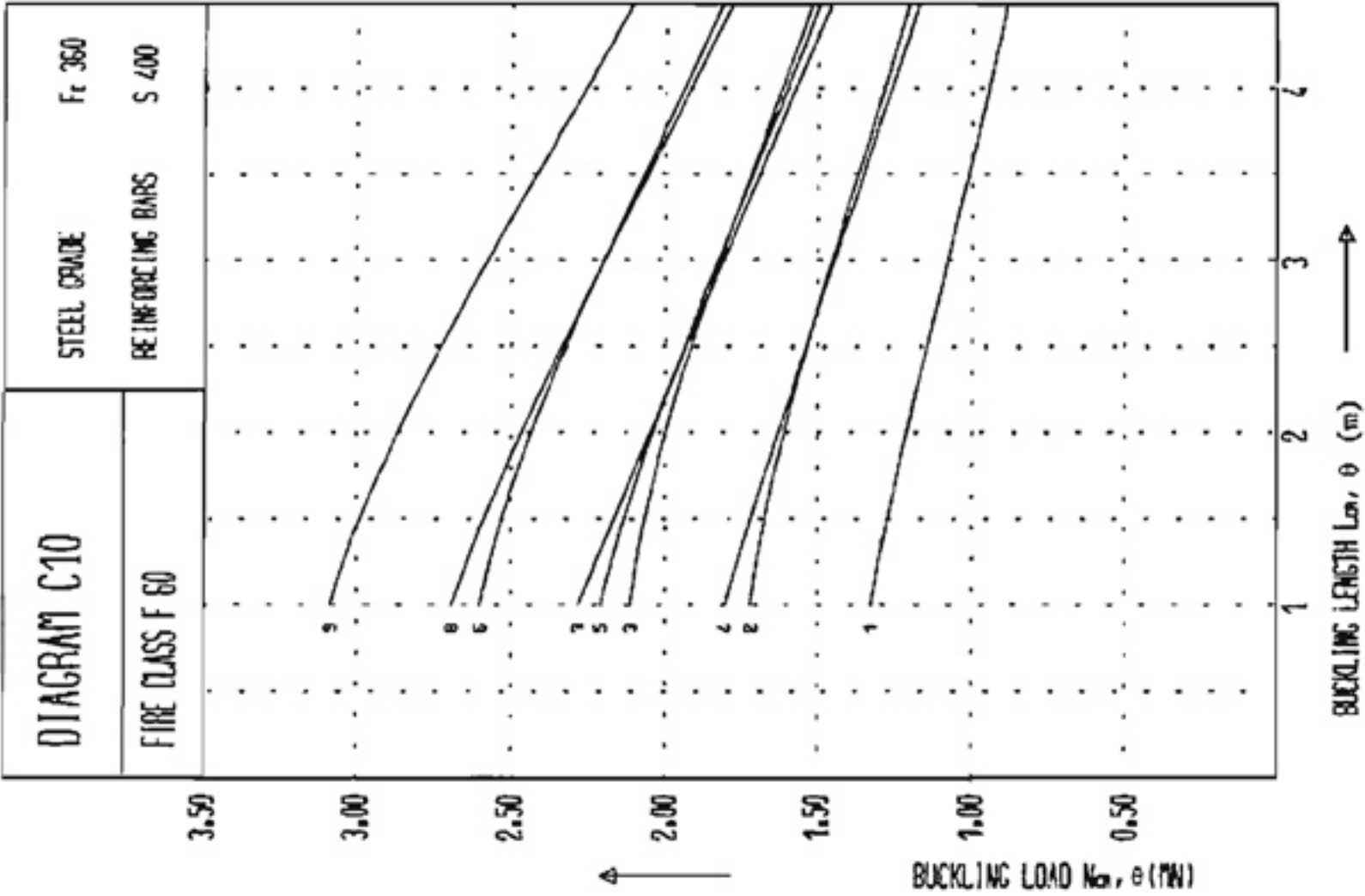
AXIAL LOAD DIAGRAMS FOR COLUMN WITH CIRCULAR HOLLOW SECTION 244.5x5.0



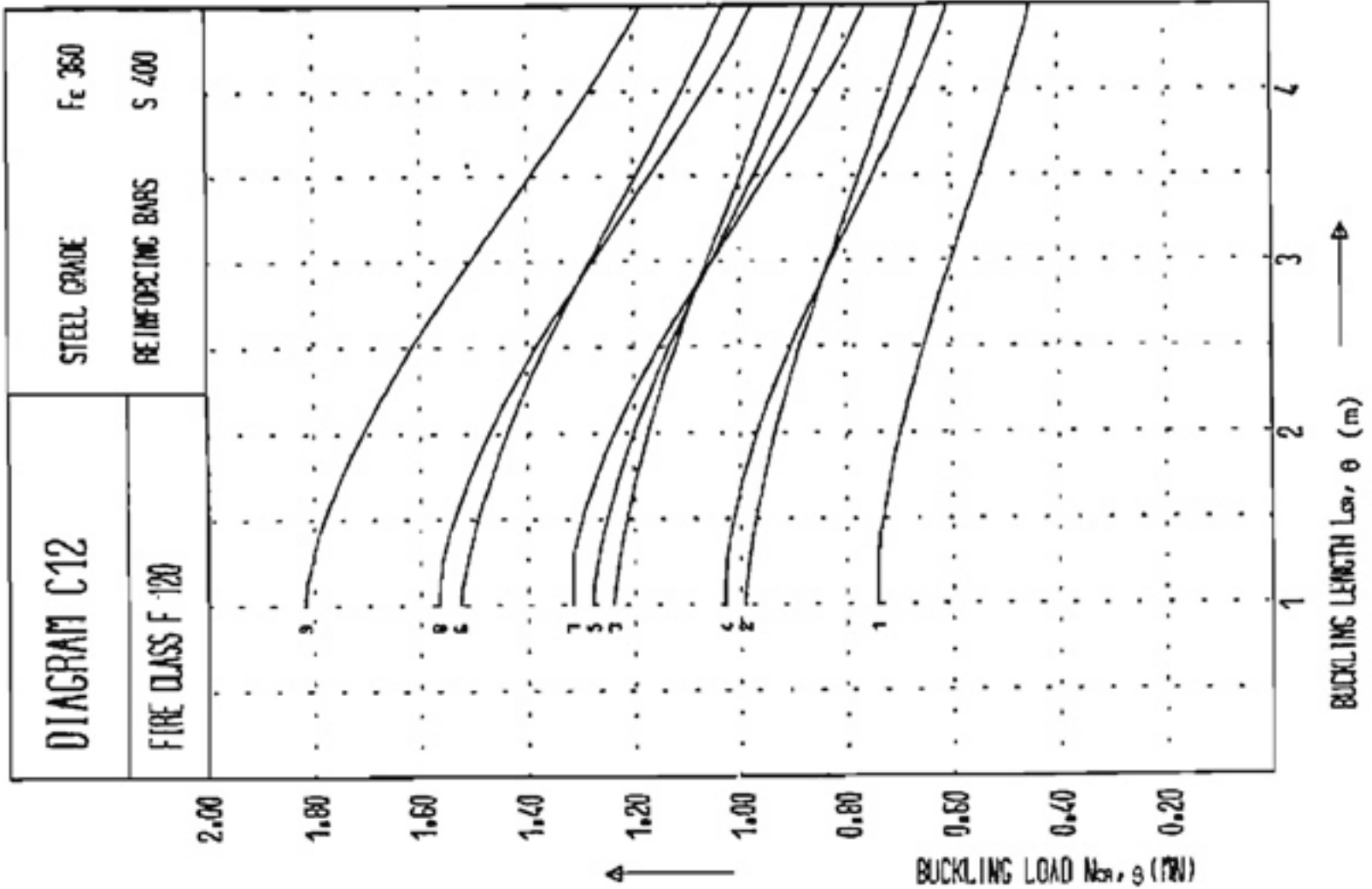
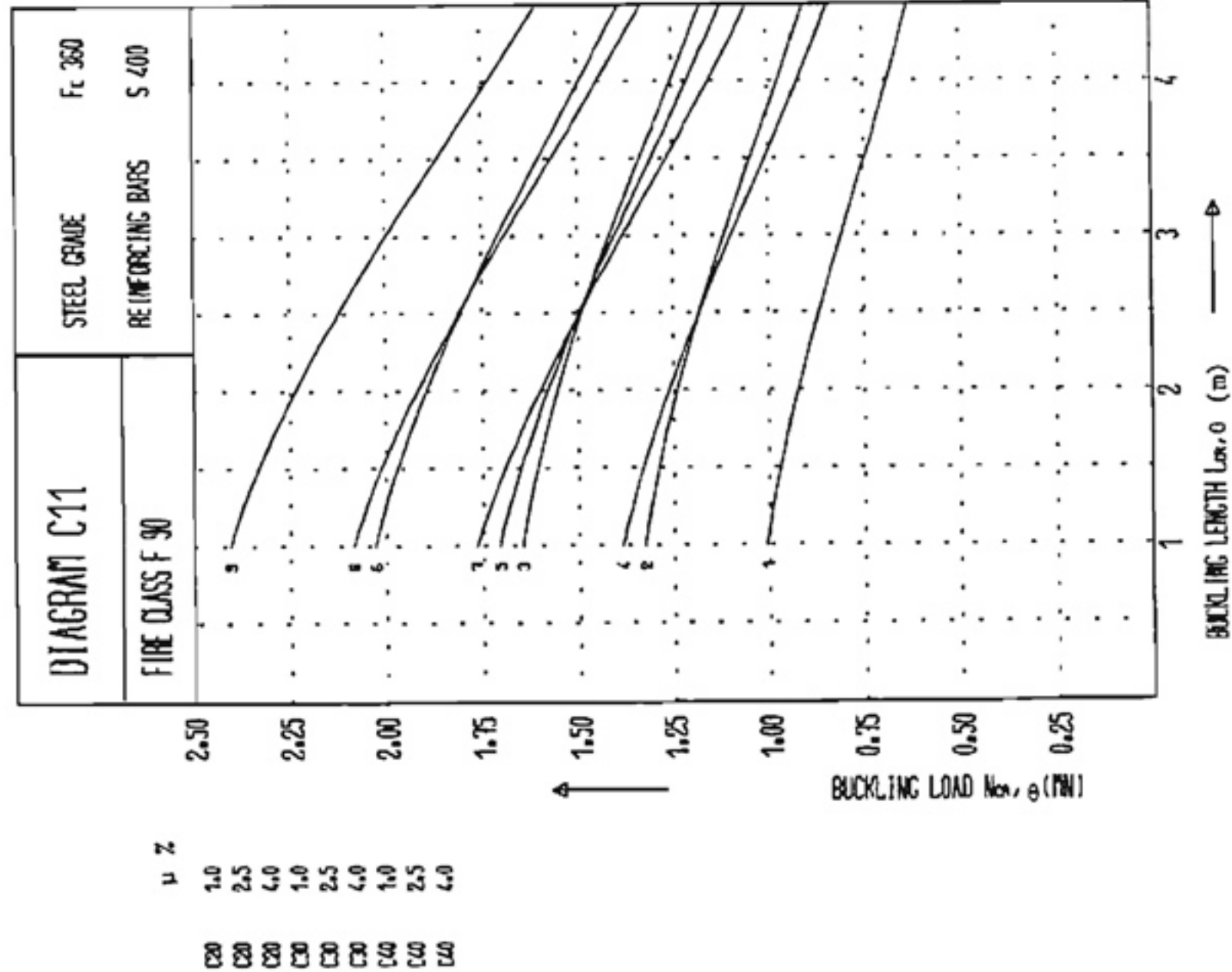
AXIAL LOAD DIAGRAMS FOR COLUMN WITH CIRCULAR HOLLOW SECTION 273.0x5.0



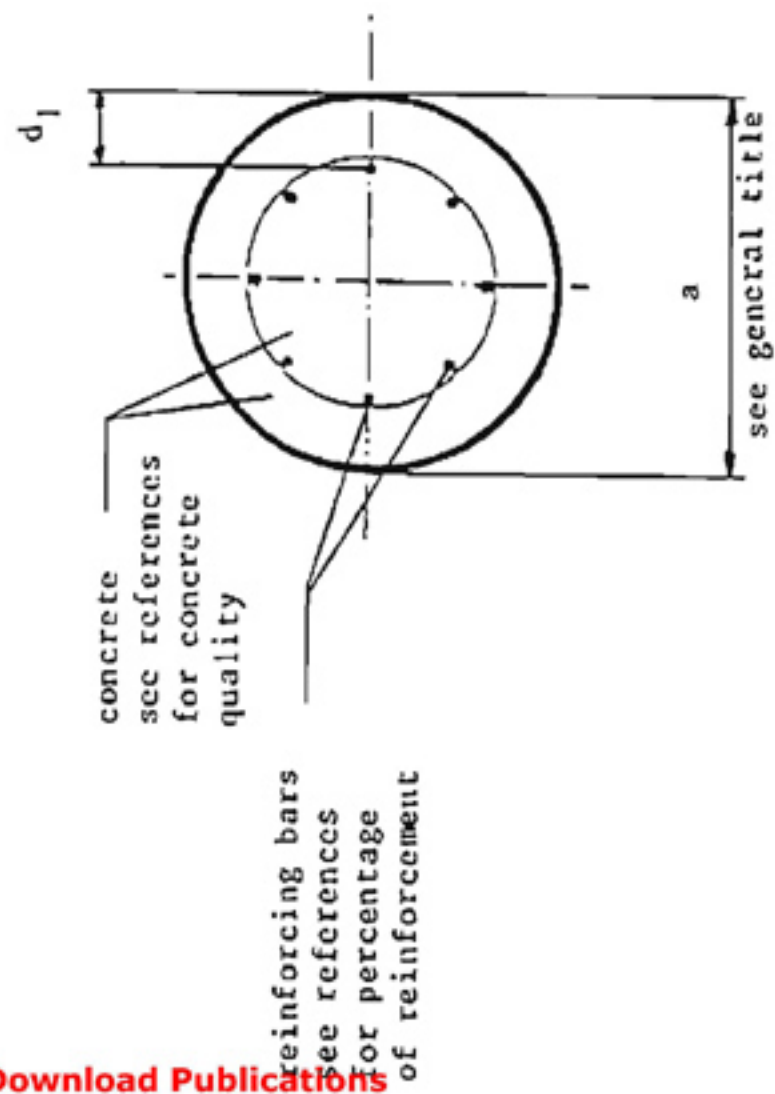
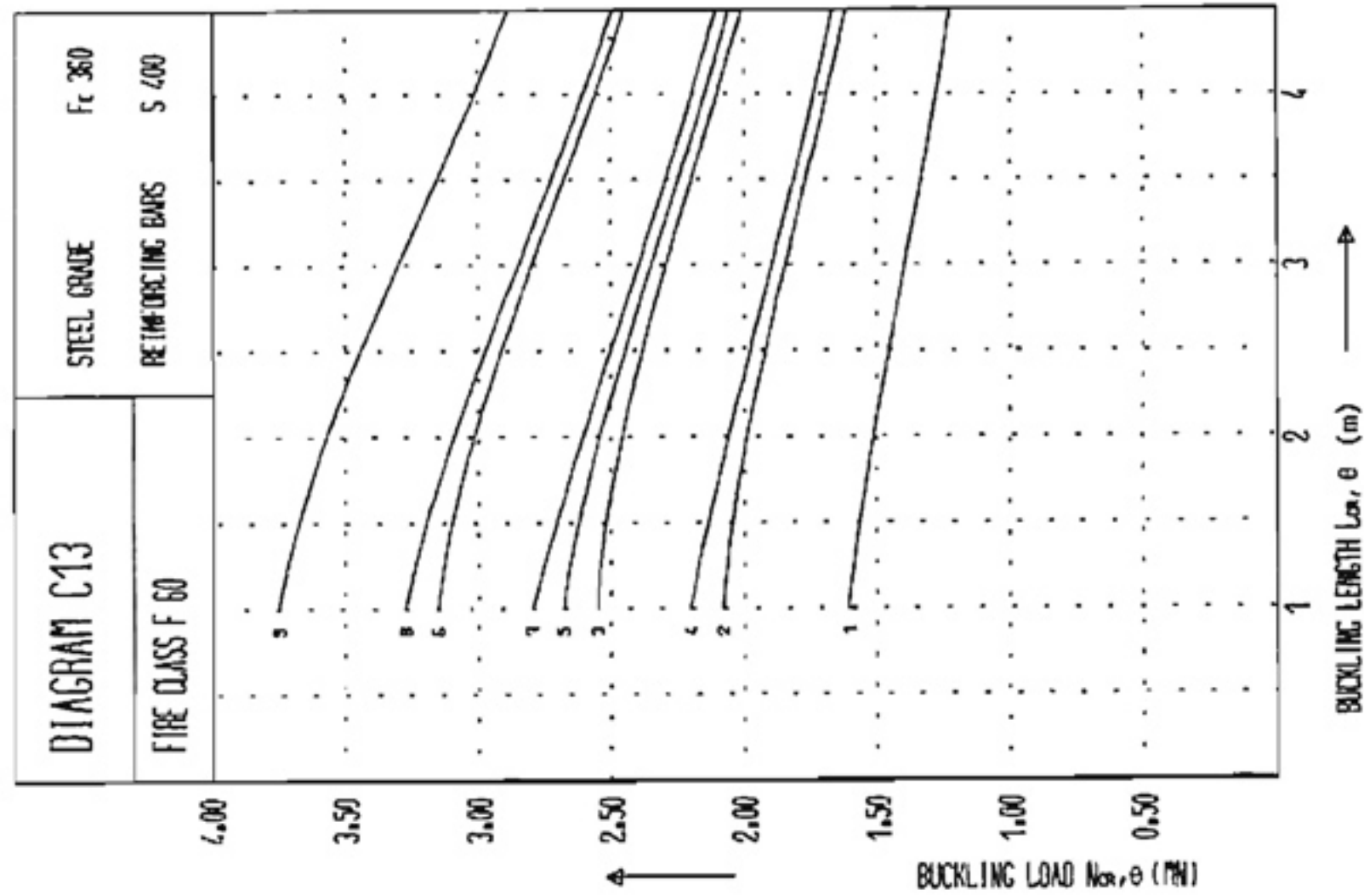
AXIAL LOAD DIAGRAMS FOR COLUMN WITH CIRCULAR HOLLOW SECTION 273.0x5.0



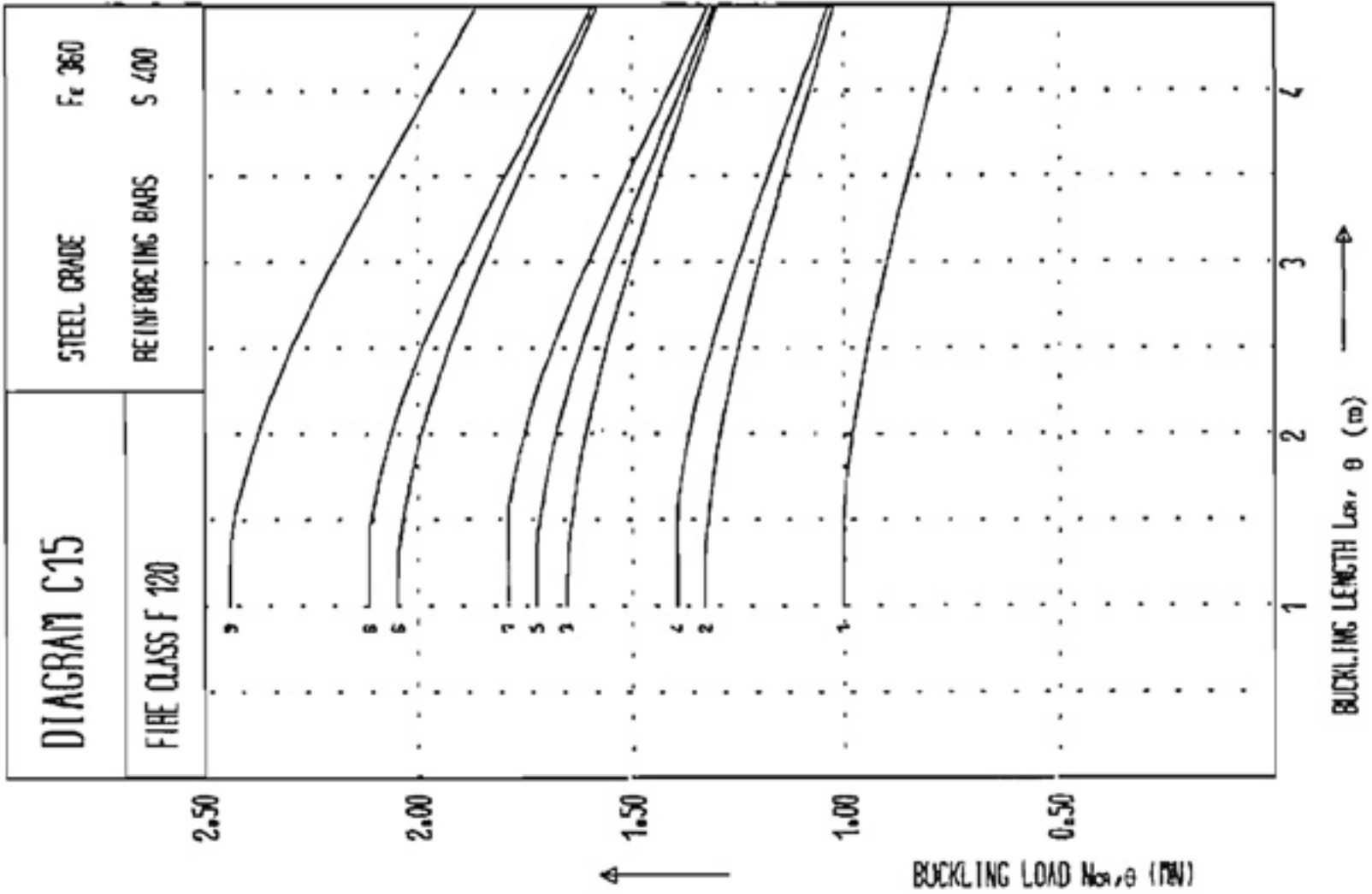
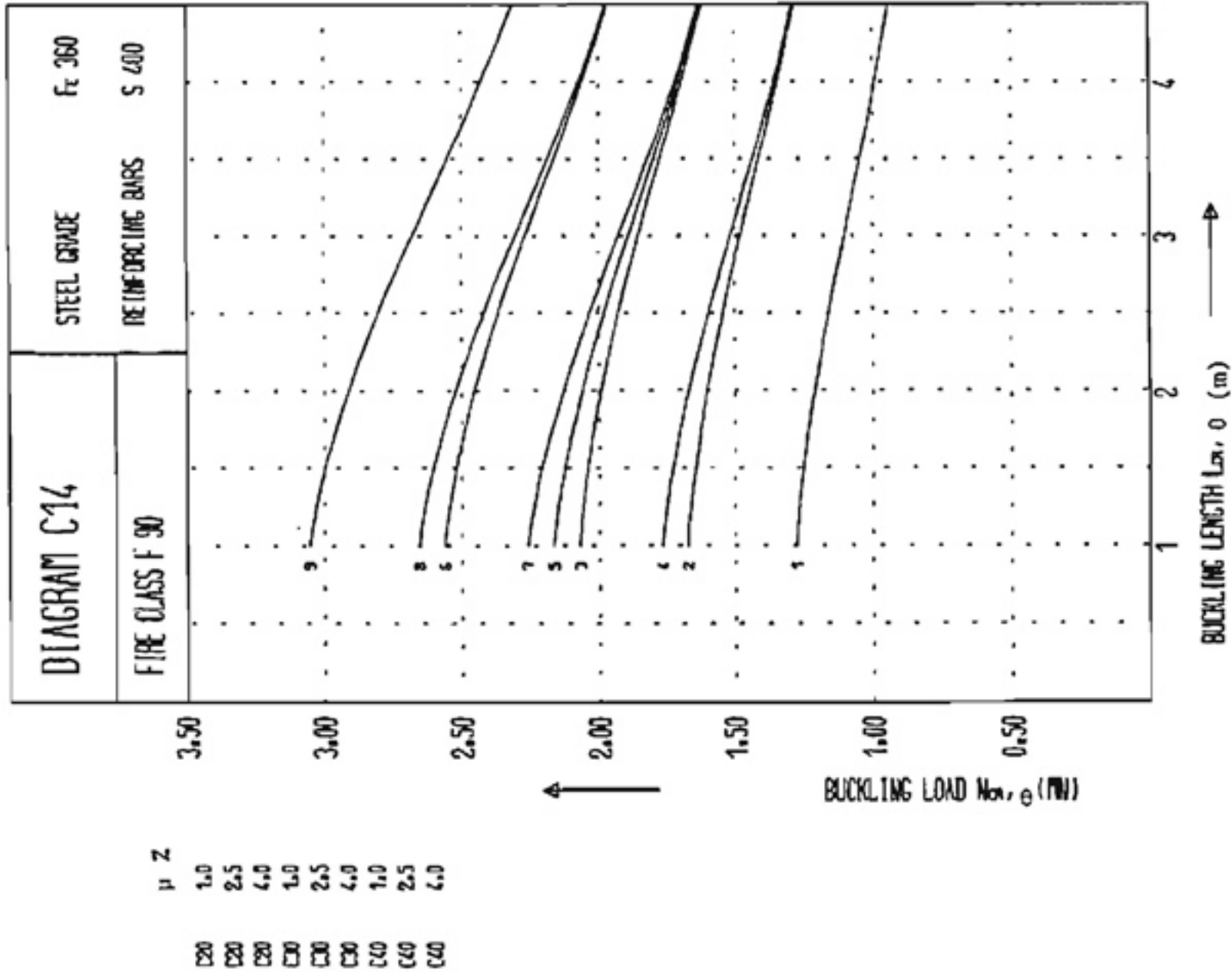
AXIAL LOAD DIAGRAMS FOR COLUMN WITH CIRCULAR HOLLOW SECTION 323.9x5.6



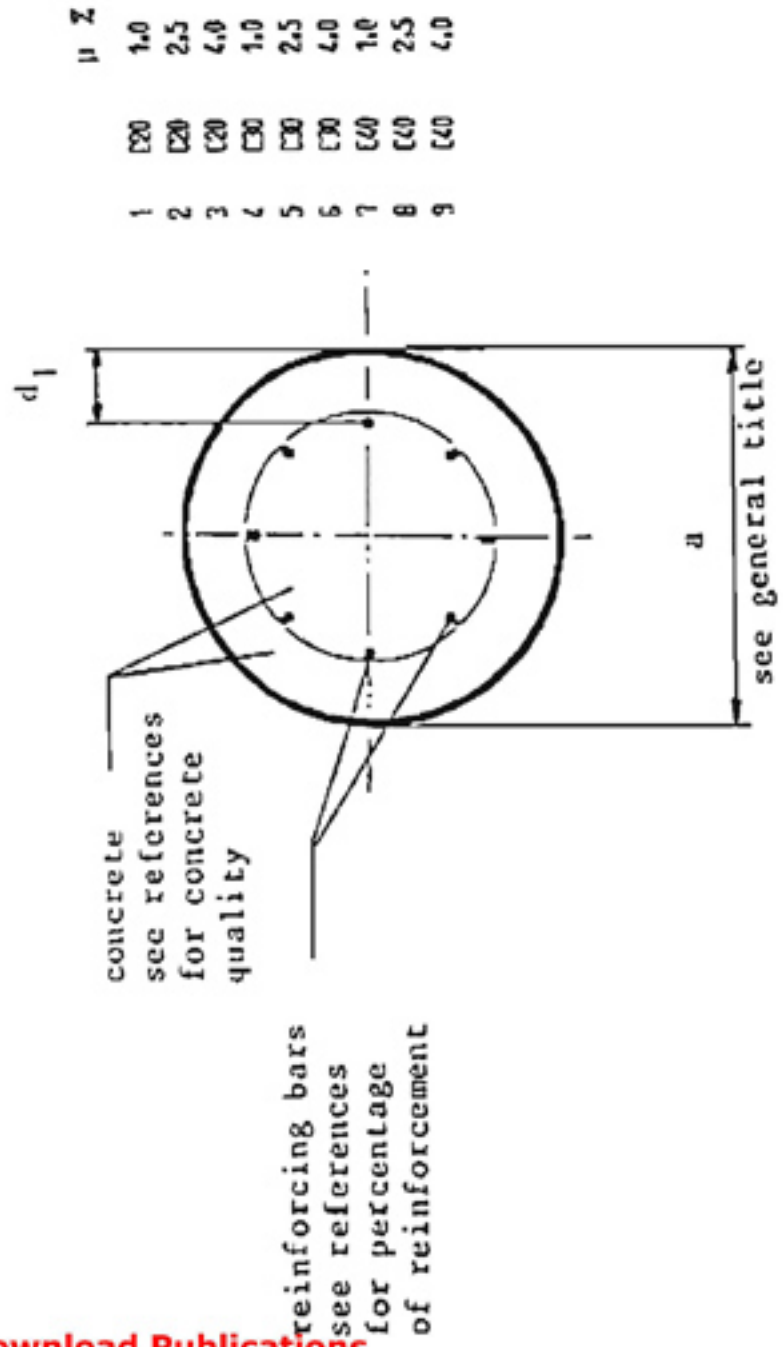
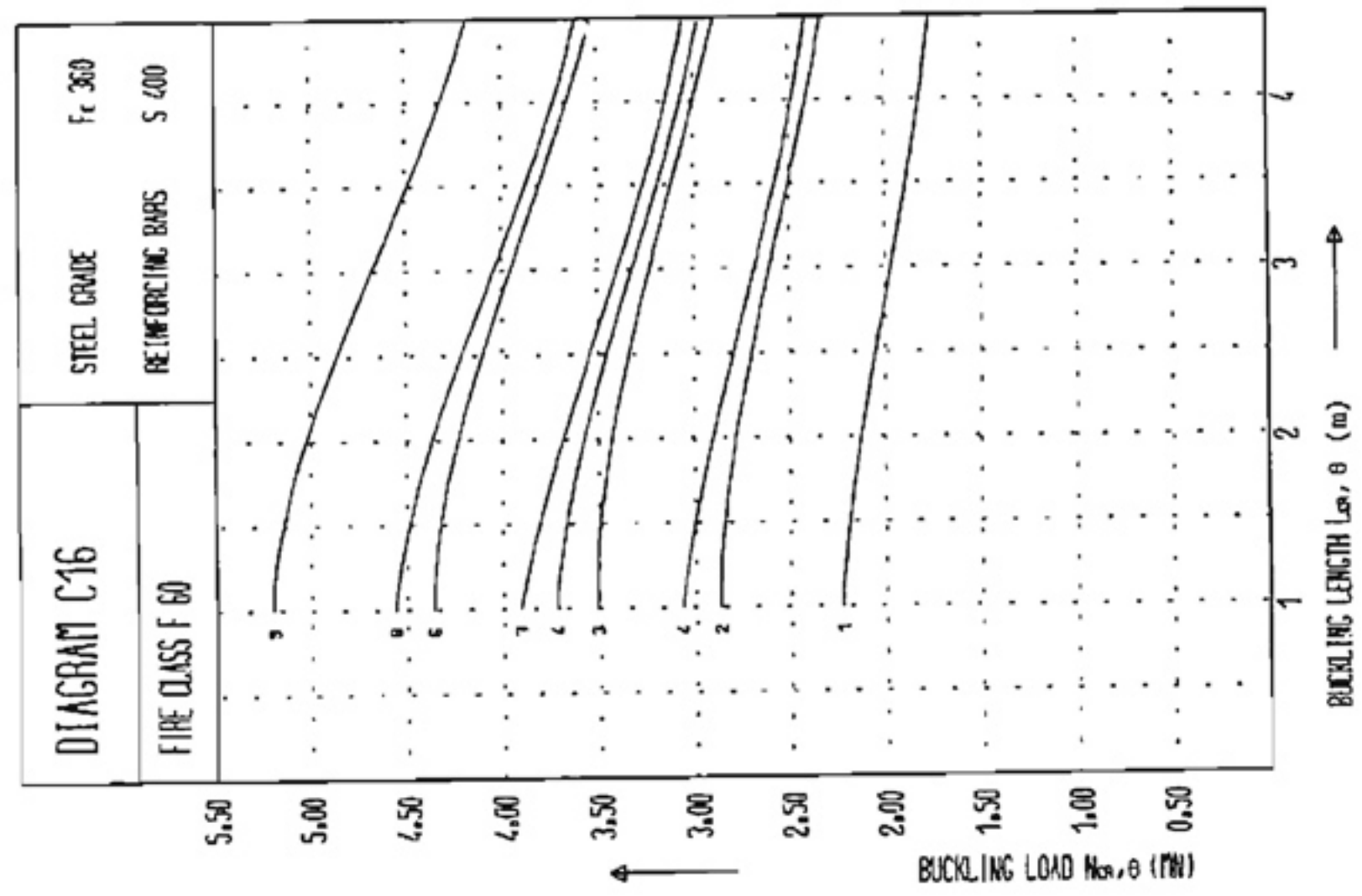
AXIAL LOAD DIAGRAMS FOR COLUMN WITH CIRCULAR HOLLOW SECTION 323.9x5.6



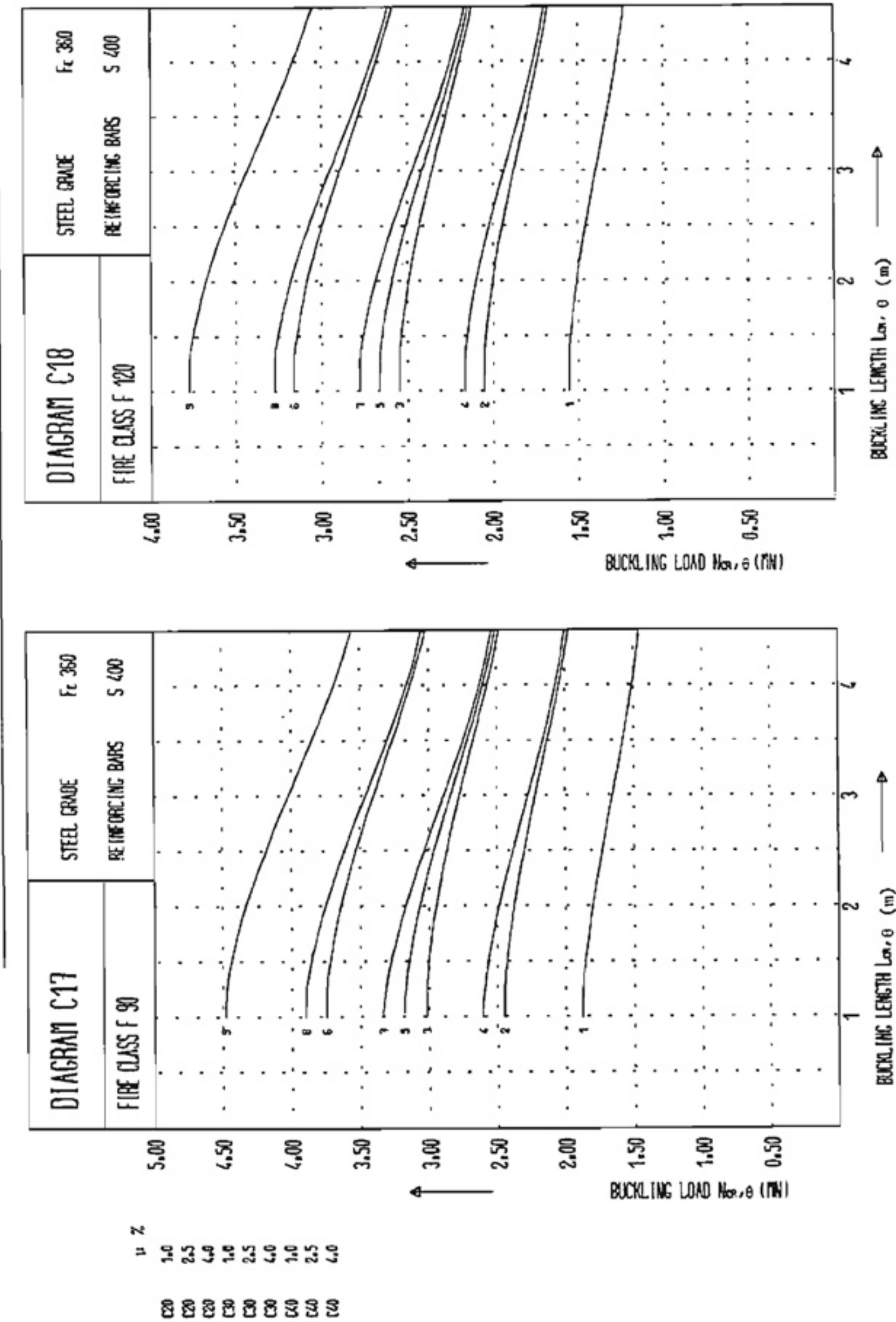
AXIAL LOAD DIAGRAMS FOR COLUMN WITH CIRCULAR HOLLOW SECTION 355₆×5₆



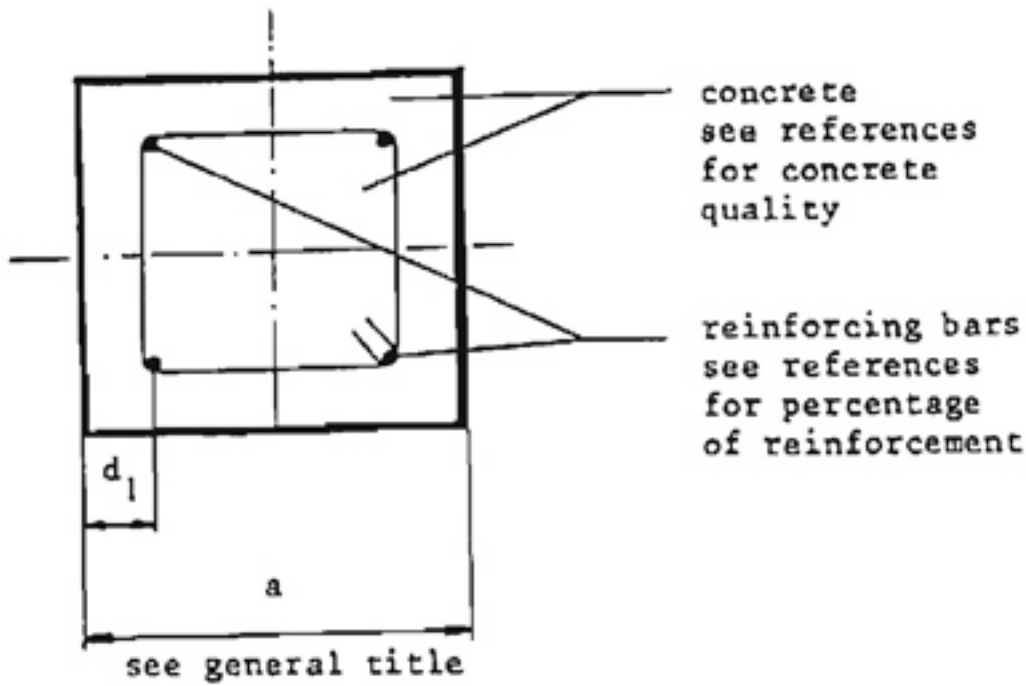
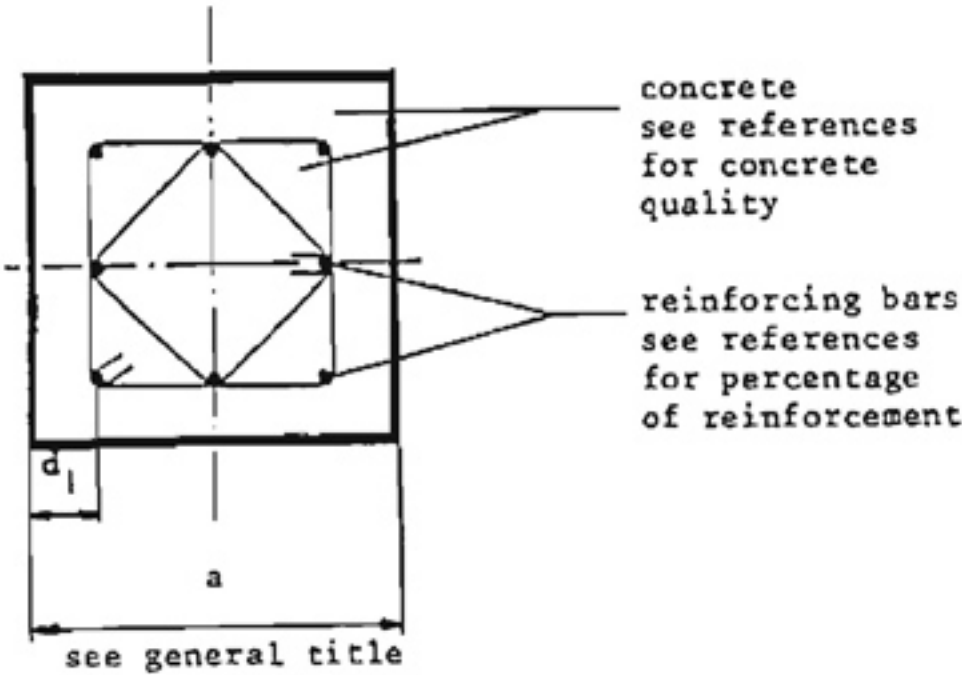
AXIAL LOAD DIAGRAMS FOR COLUMN WITH CIRCULAR HOLLOW SECTION 355.6x5.6

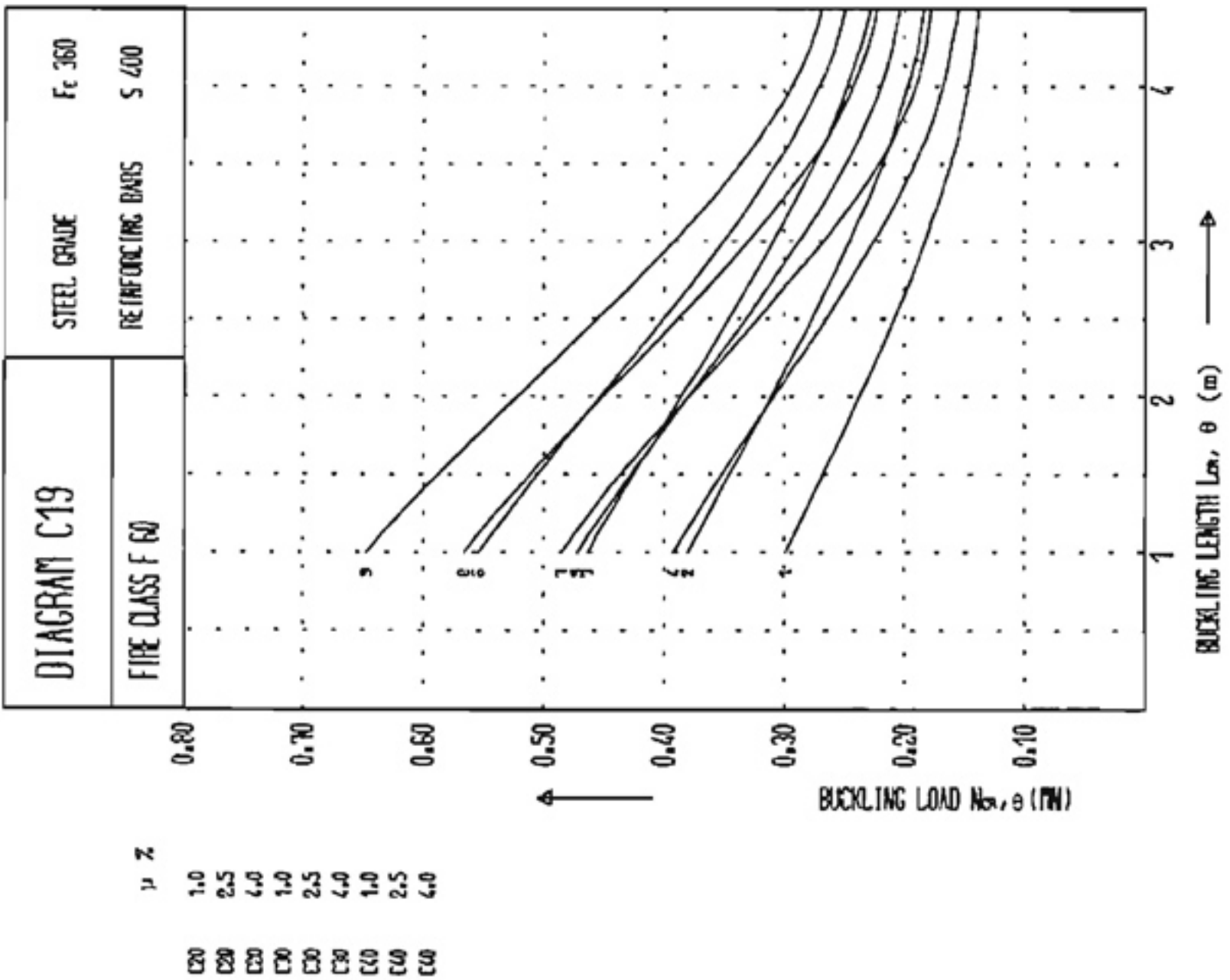
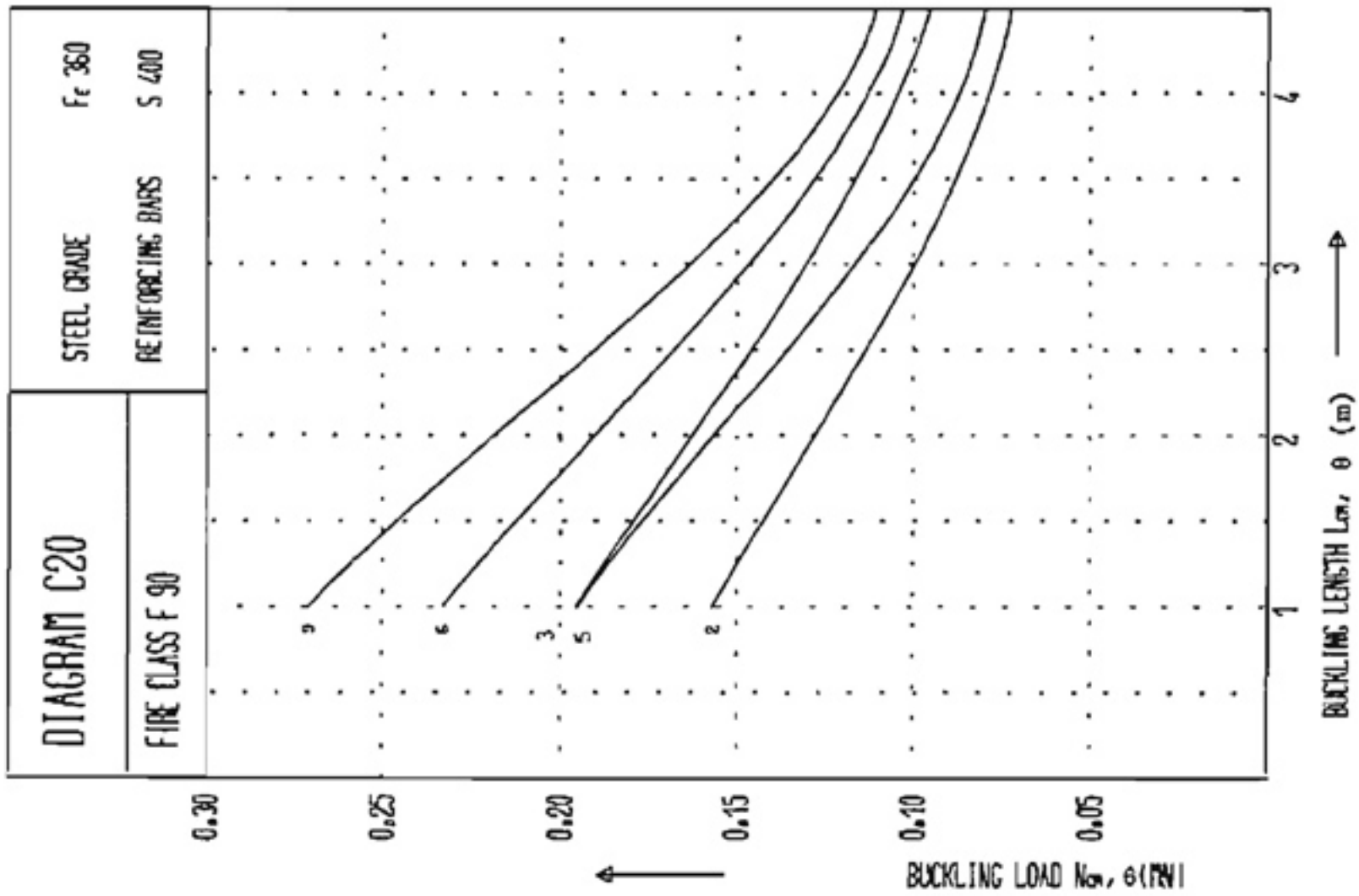


AXIAL LOAD DIAGRAMS FOR COLUMN WITH CIRCULAR HOLLOW SECTION 406_a4*6_a3

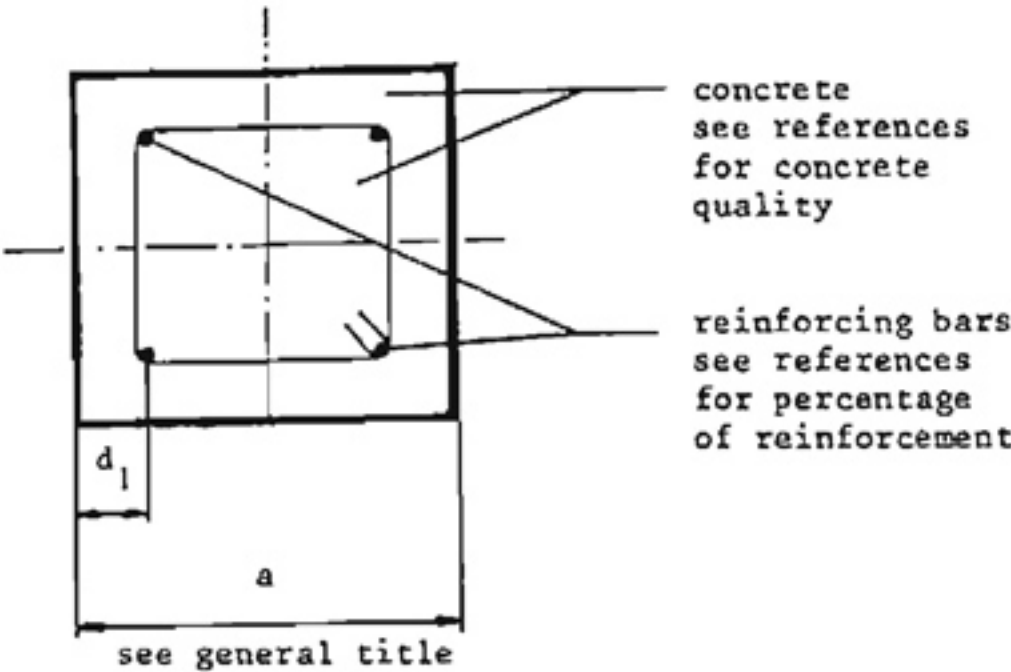
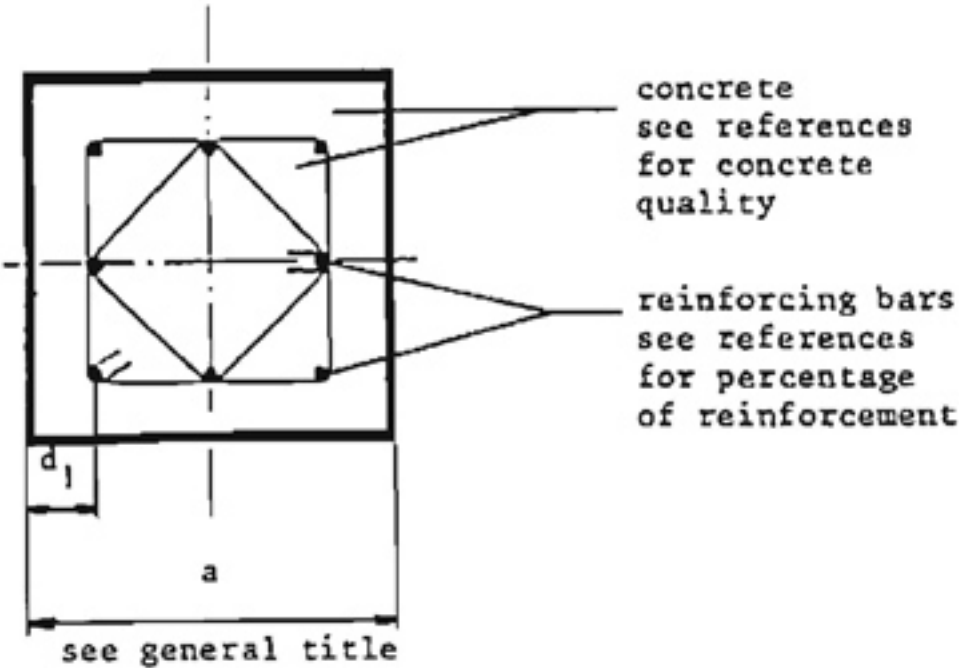


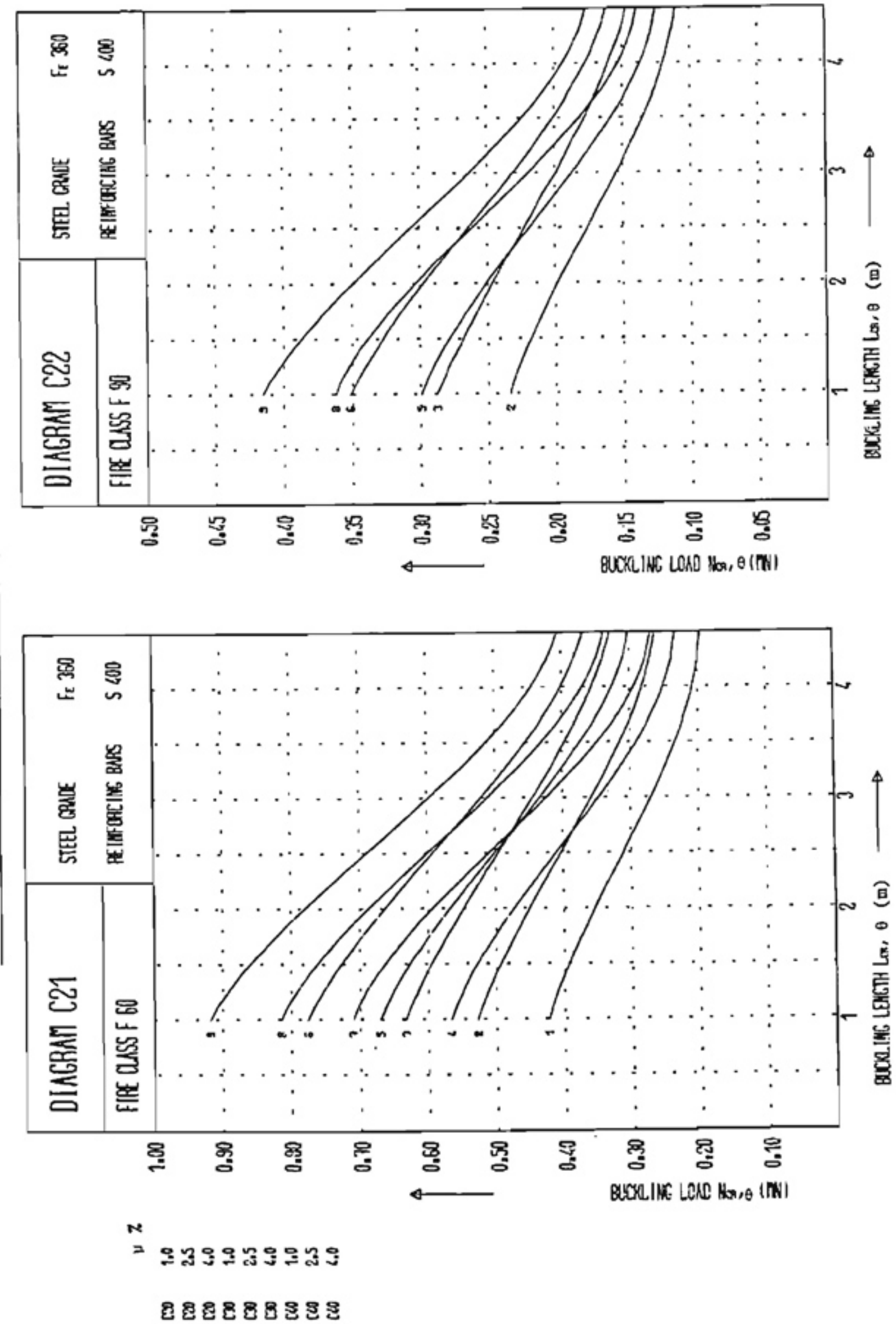
AXIAL LOAD DIAGRAMS FOR COLUMN WITH CIRCULAR HOLLOW SECTION 406.4x6.3

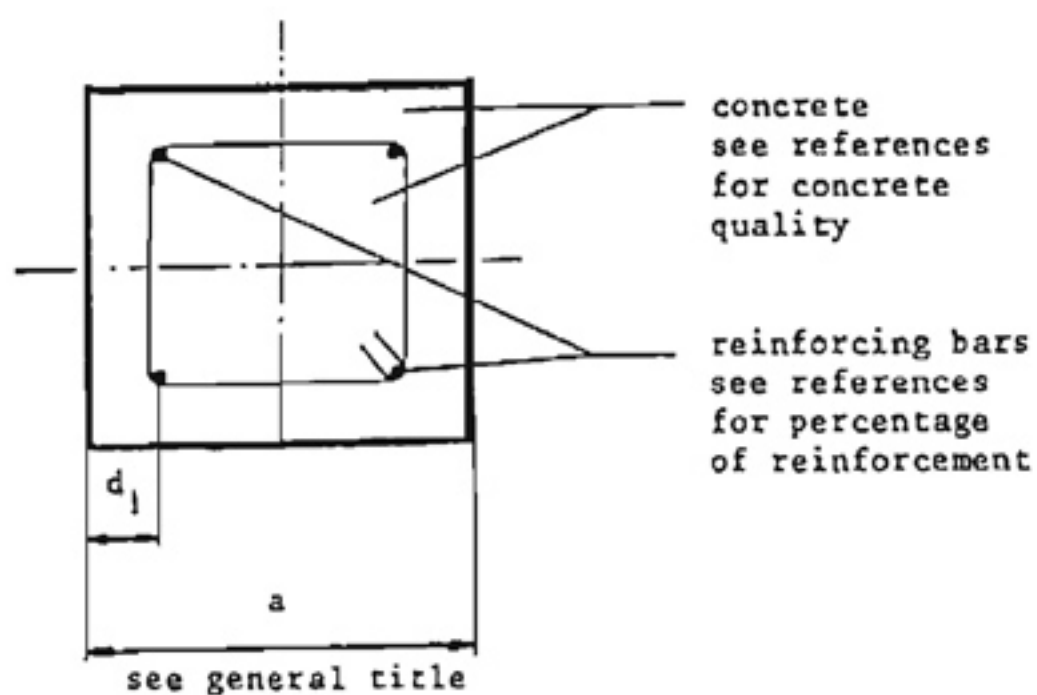
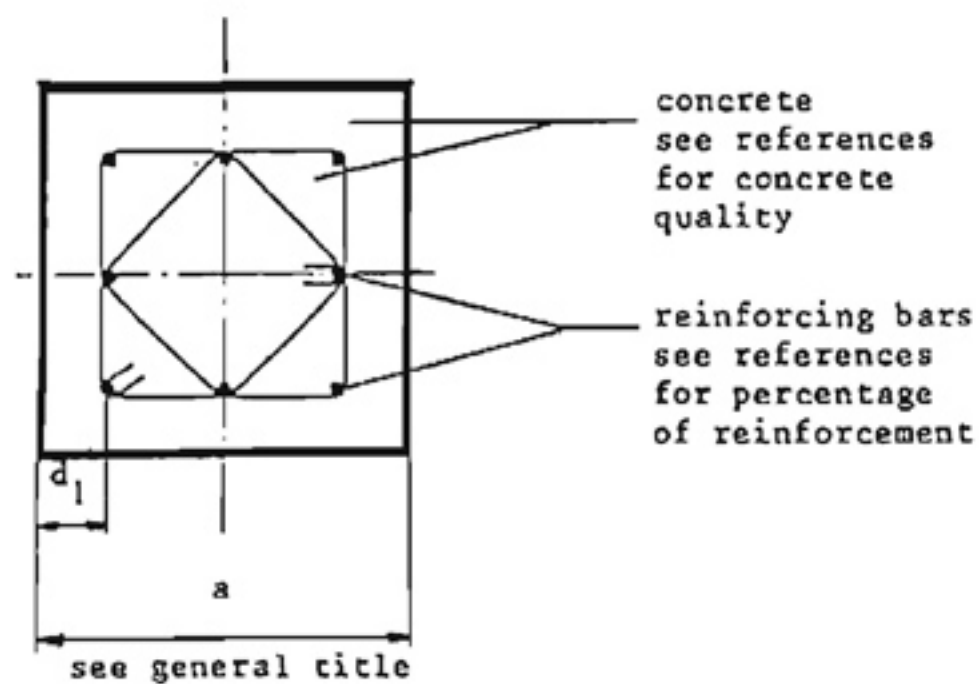


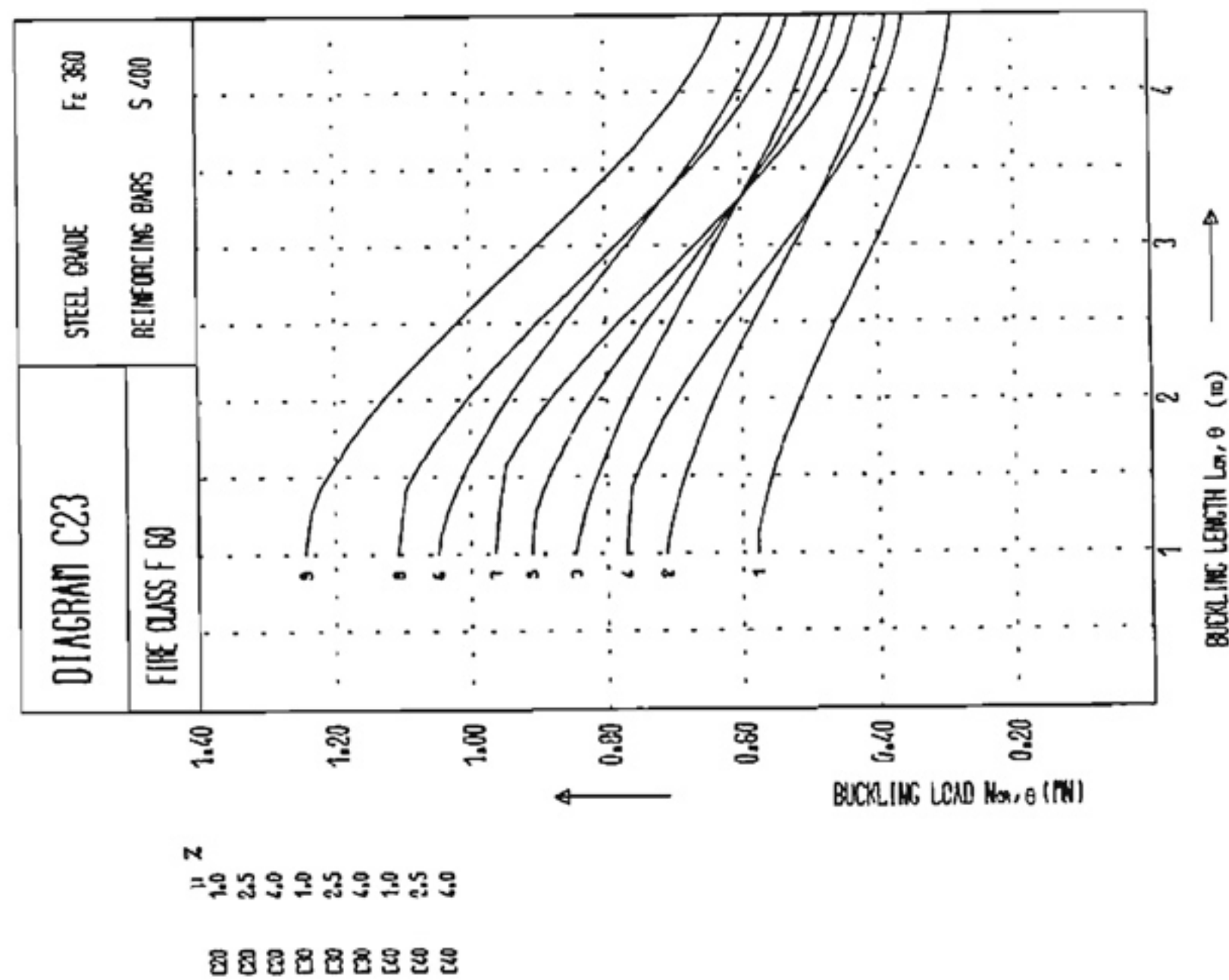
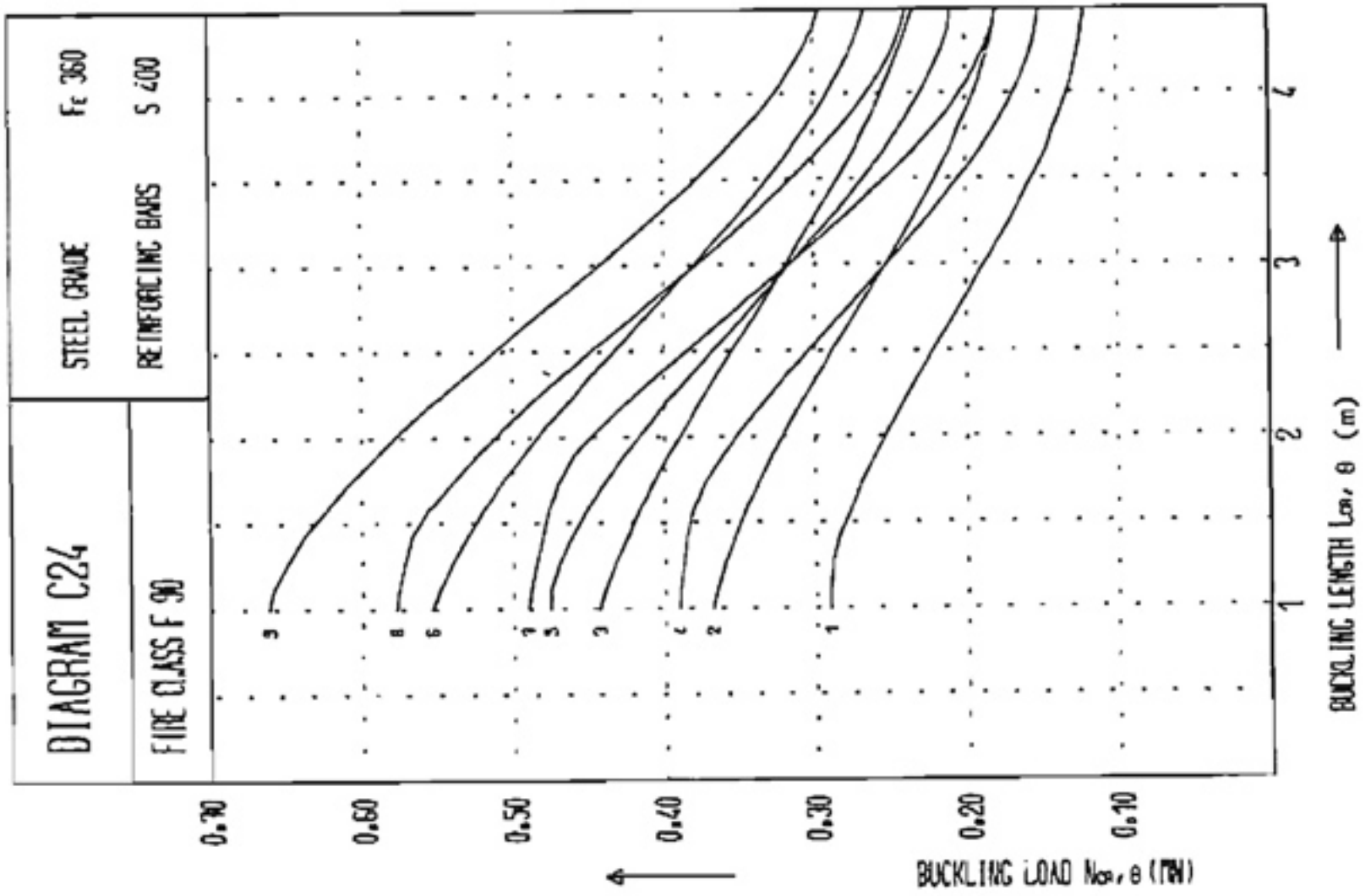


AXIAL LOAD DIAGRAMS FOR COLUMN WITH SQUARE HOLLOW SECTION 180x60.3

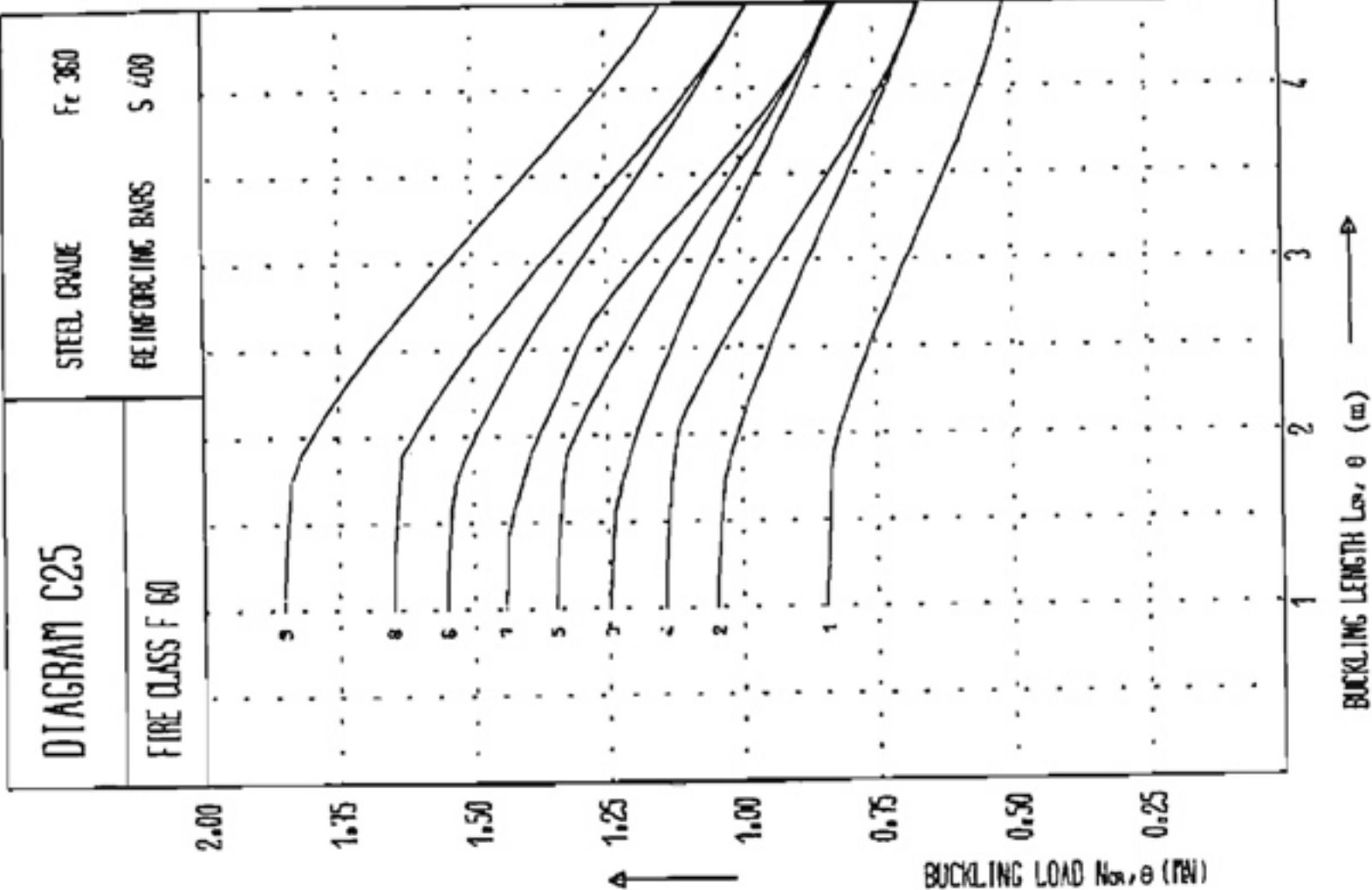
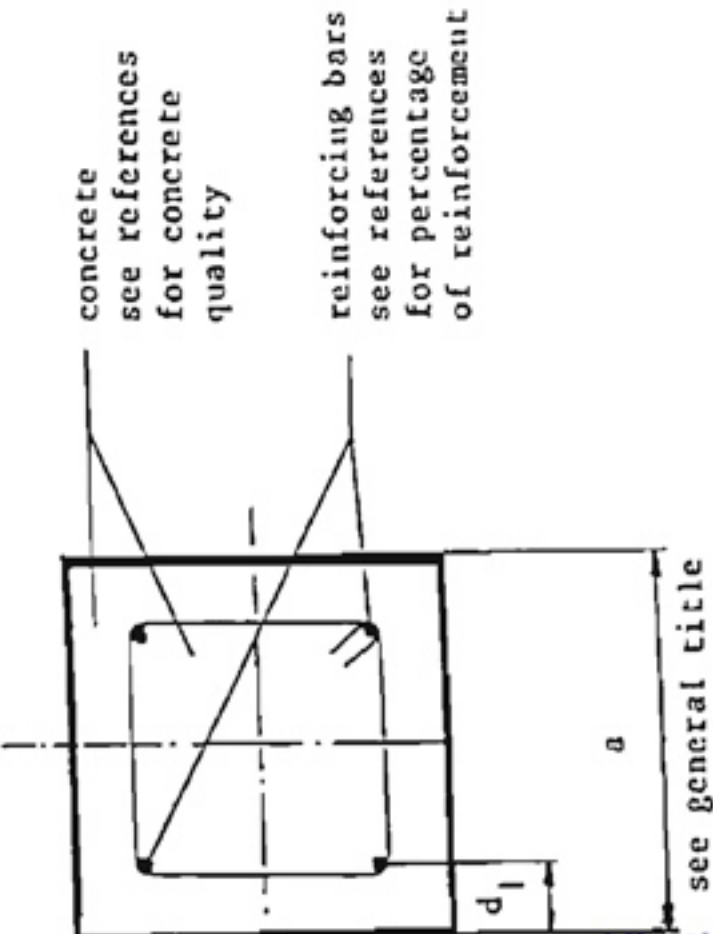
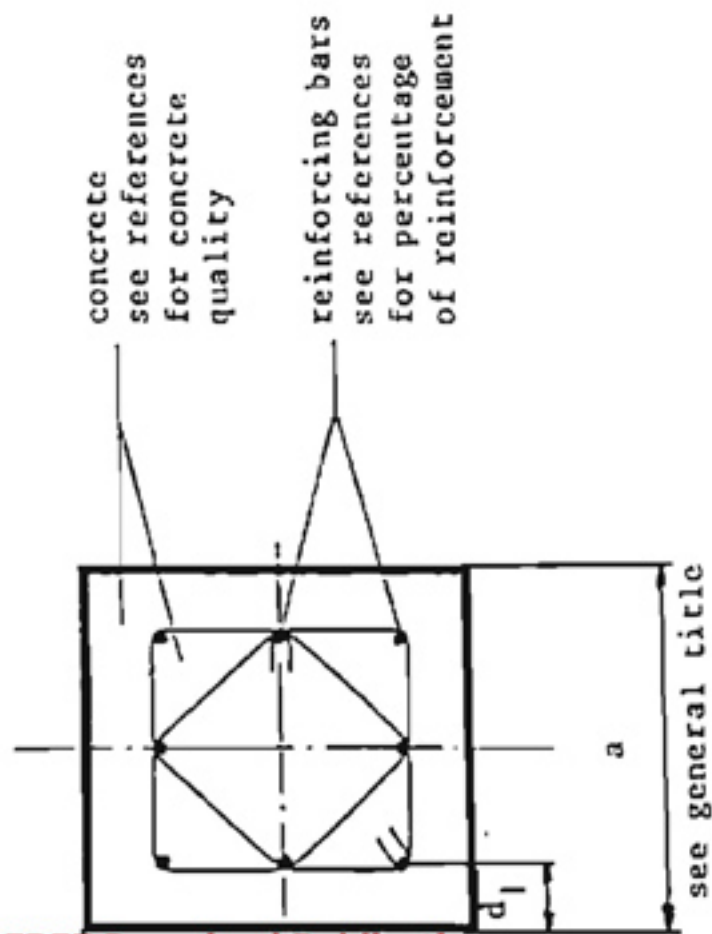




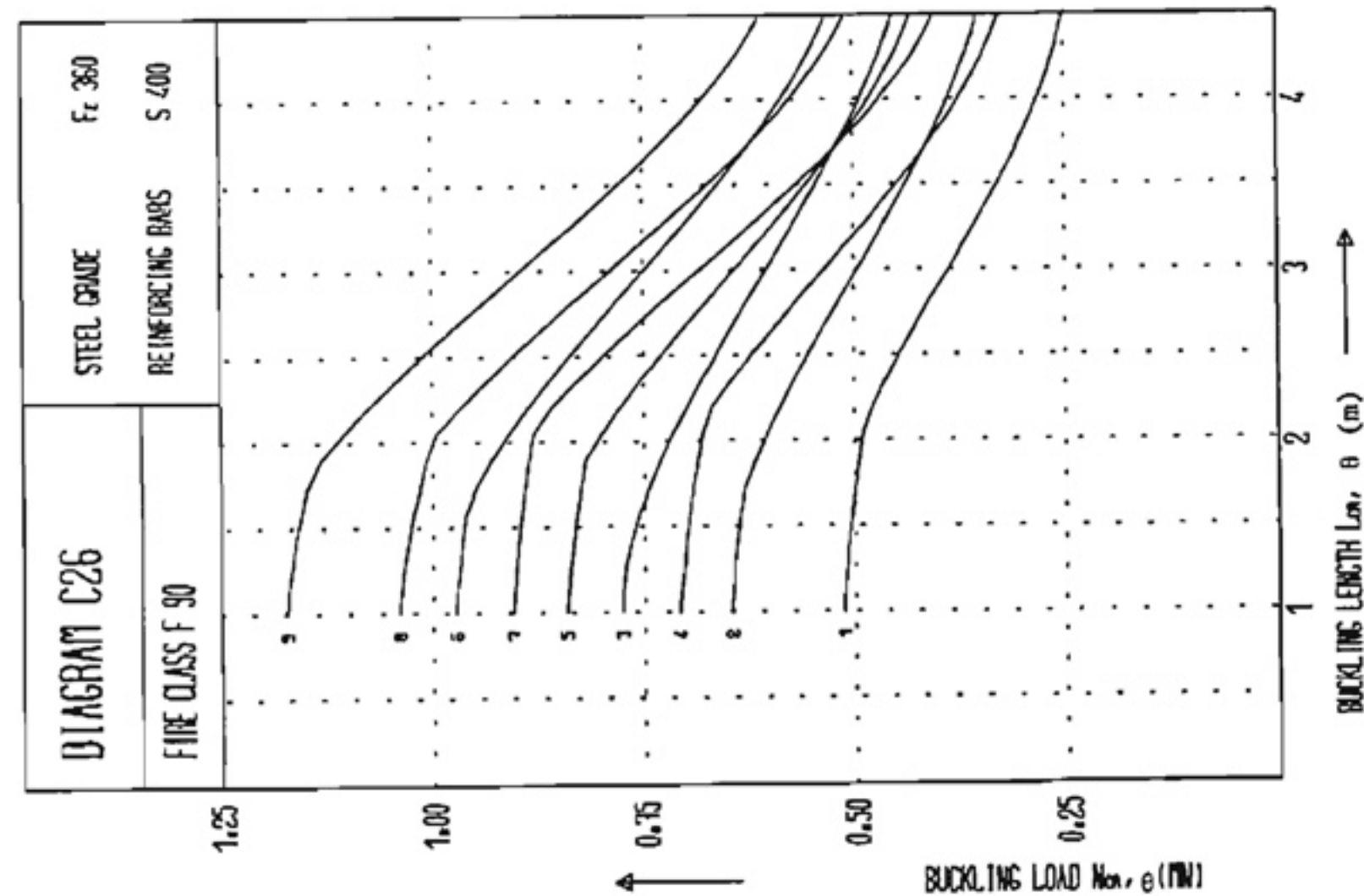
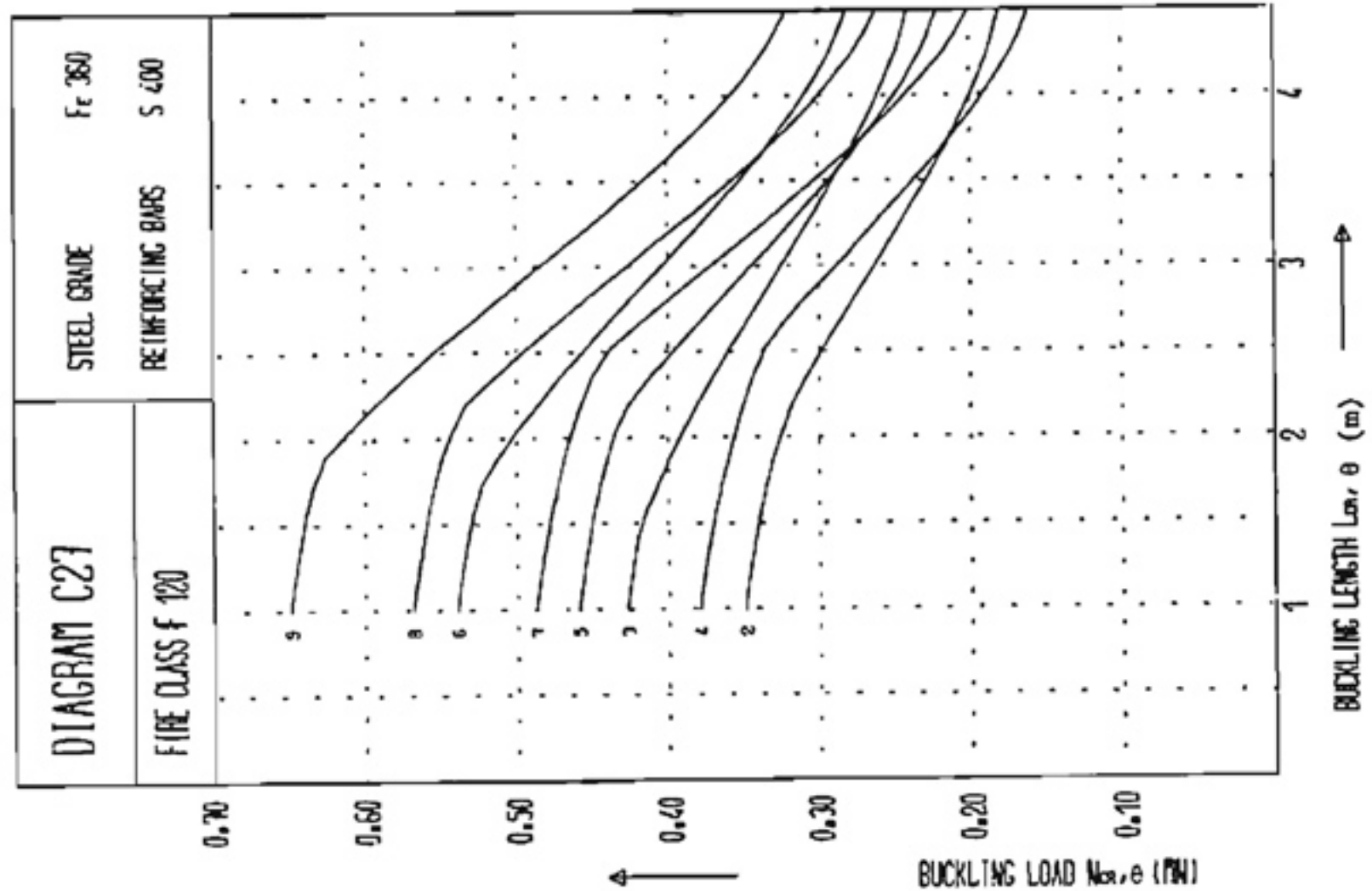




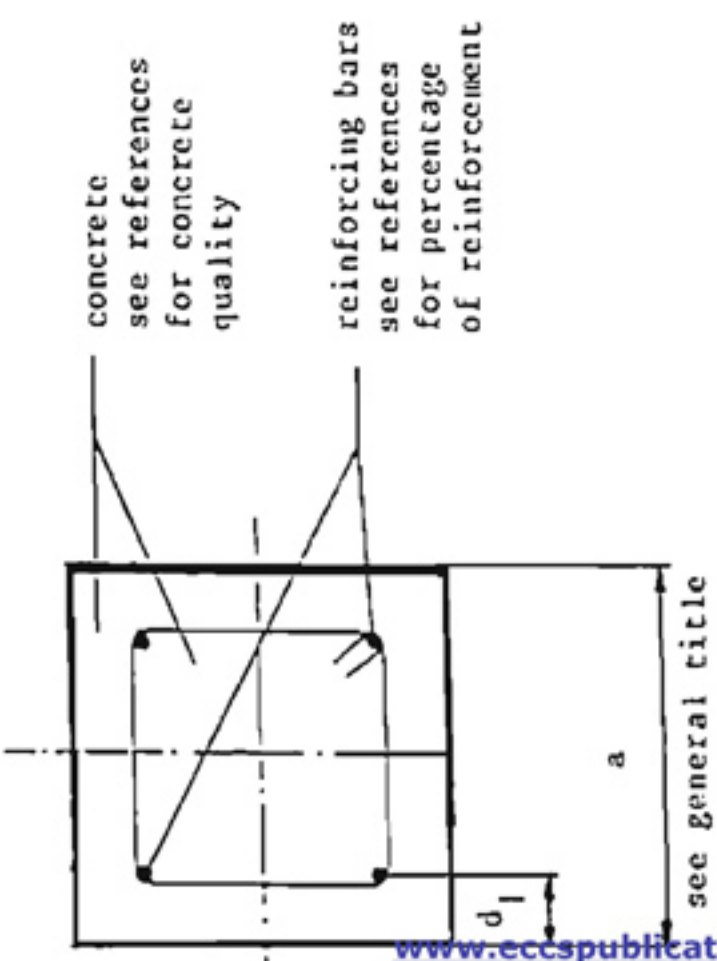
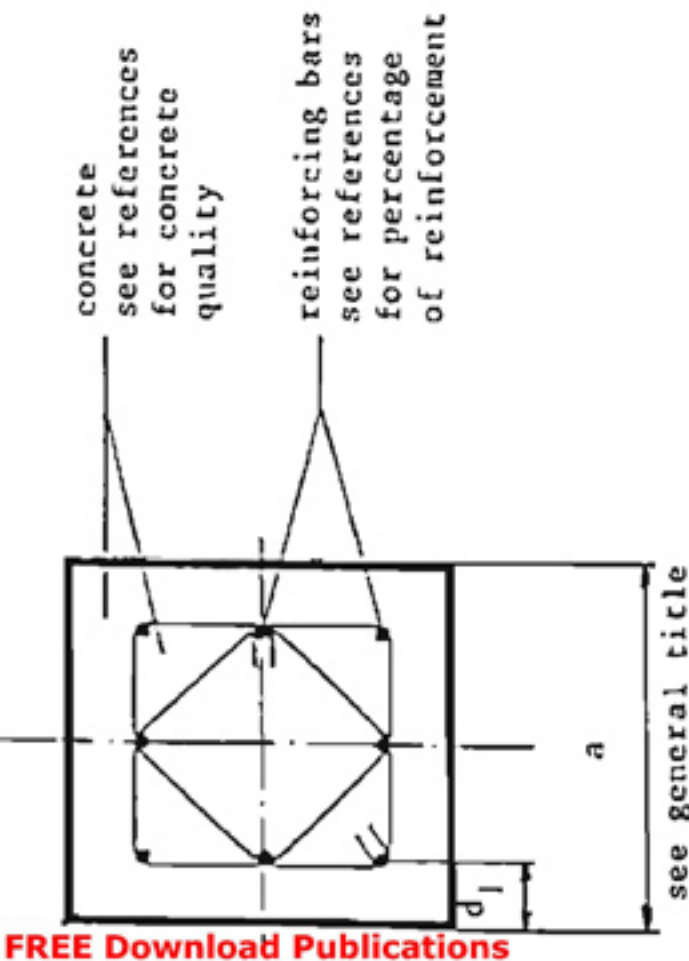
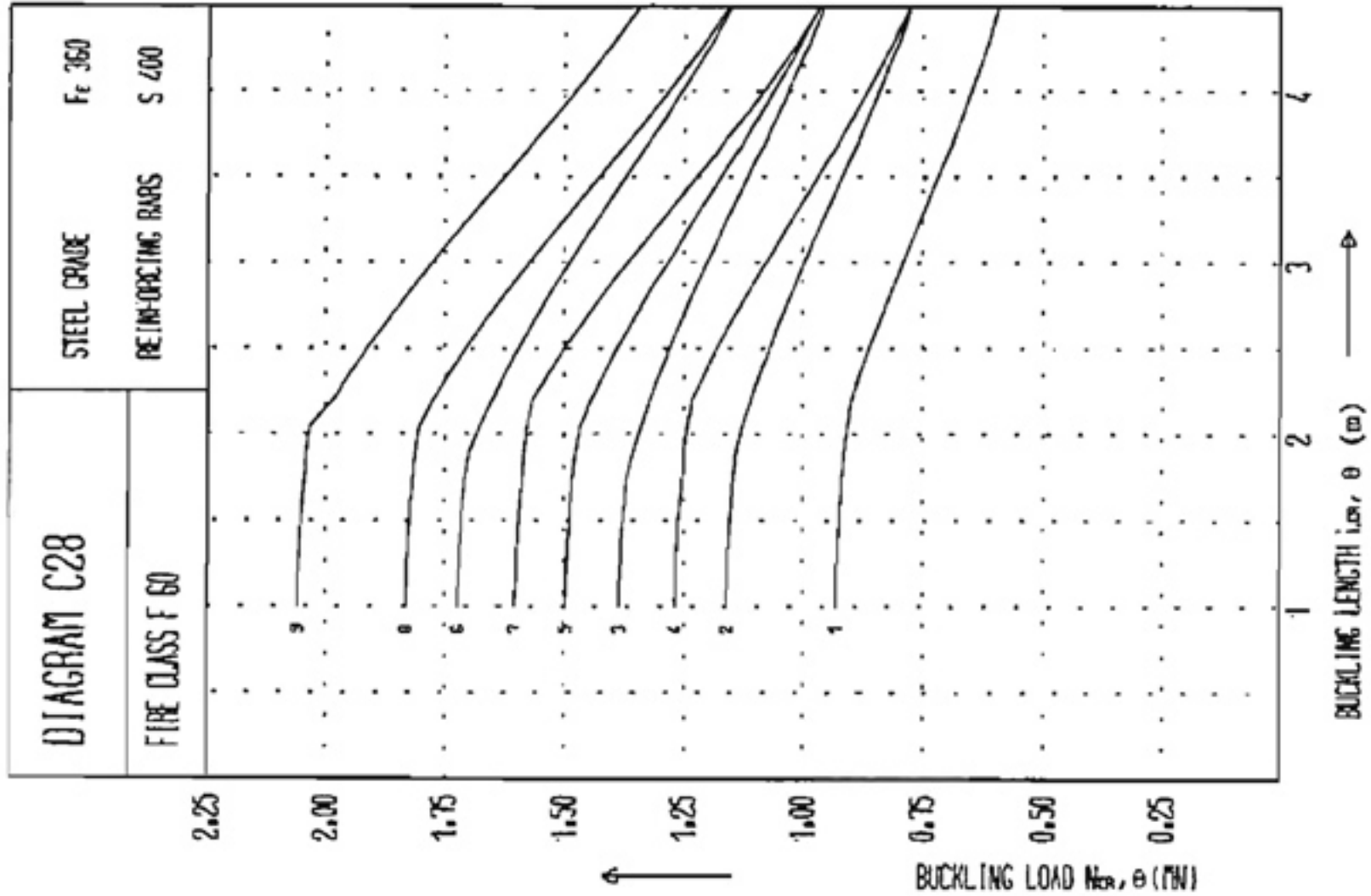
AXIAL LOAD DIAGRAMS FOR COLUMN WITH SQUARE HOLLOW SECTION 220x6x3



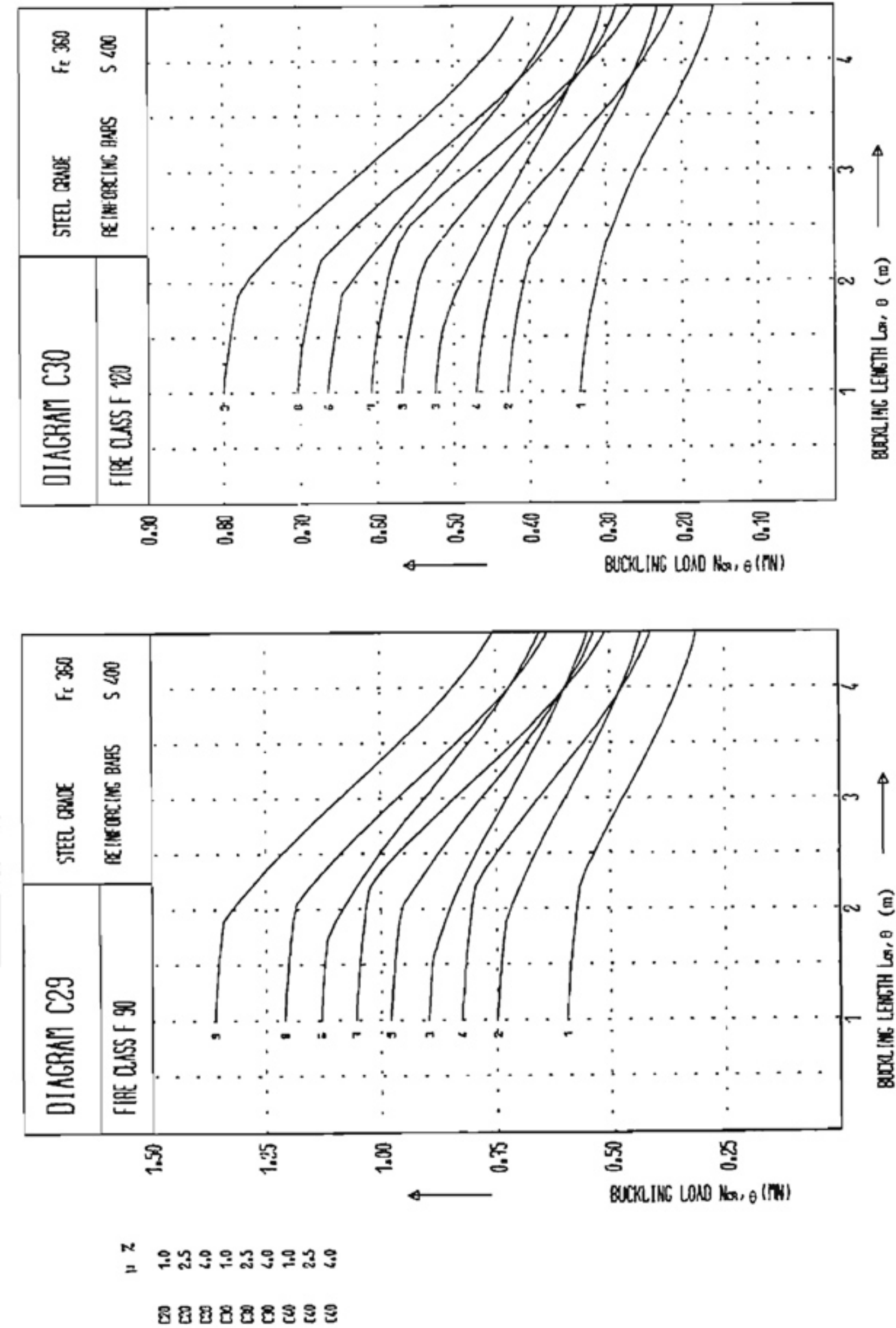
AXIAL LOAD DIAGRAMS FOR COLUMN WITH SQUARE HOLLOW SECTION 250*6*3



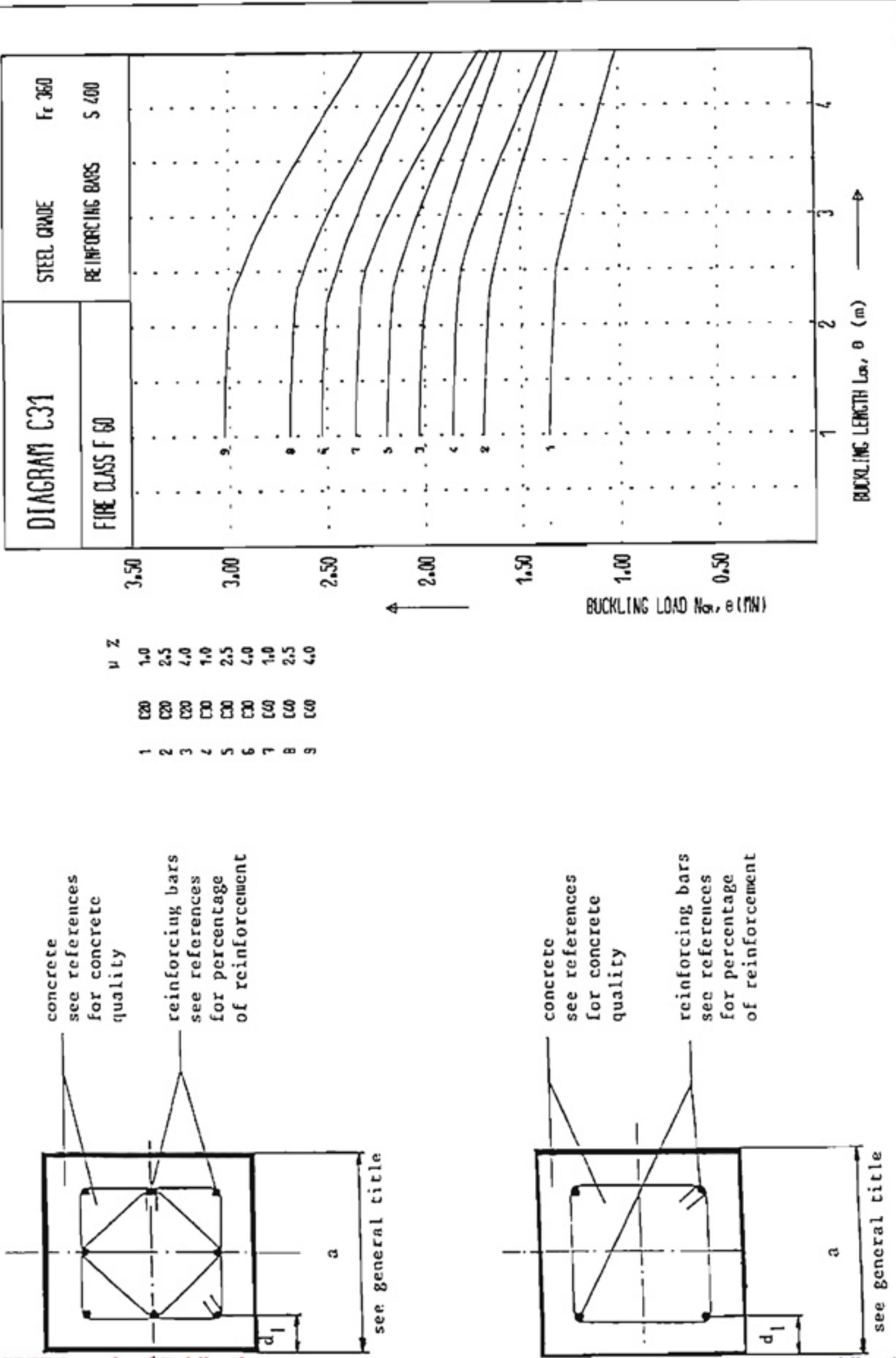
AXIAL LOAD DIAGRAMS FOR COLUMN WITH SQUARE HOLLOW SECTION 250*6.3



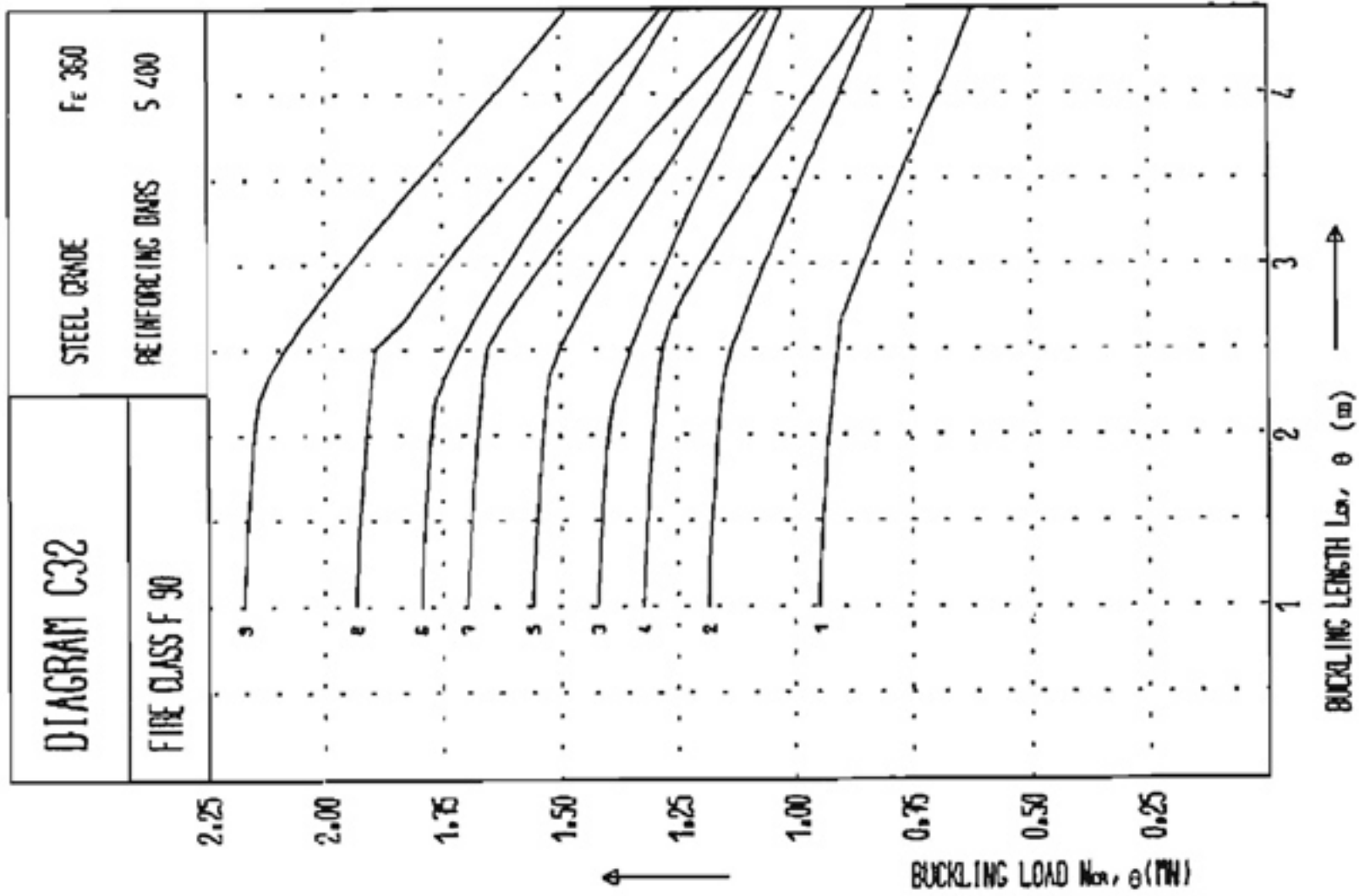
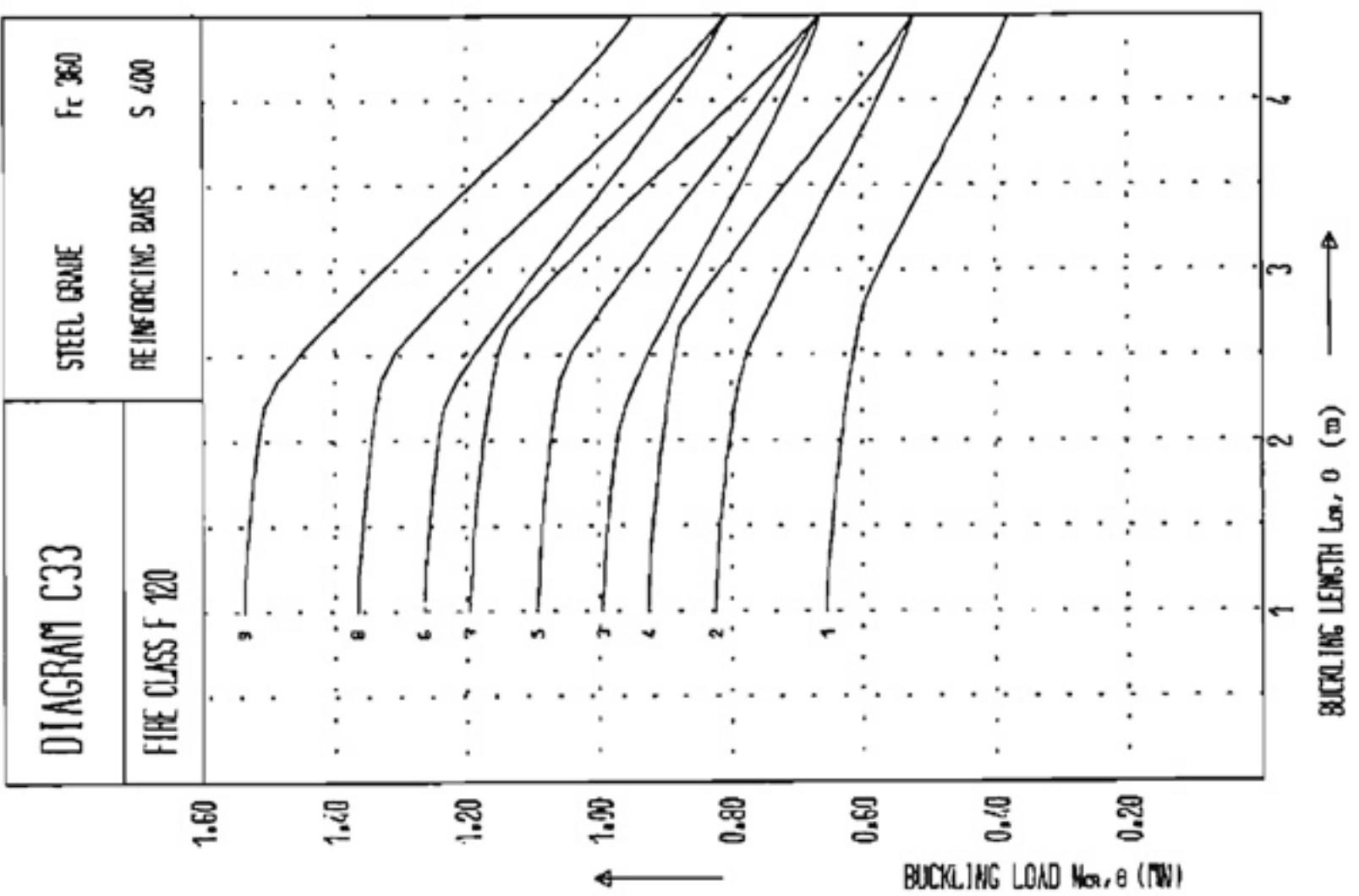
AXIAL LOAD DIAGRAMS FOR COLUMN WITH SQUARE HOLLOW SECTION 260x6



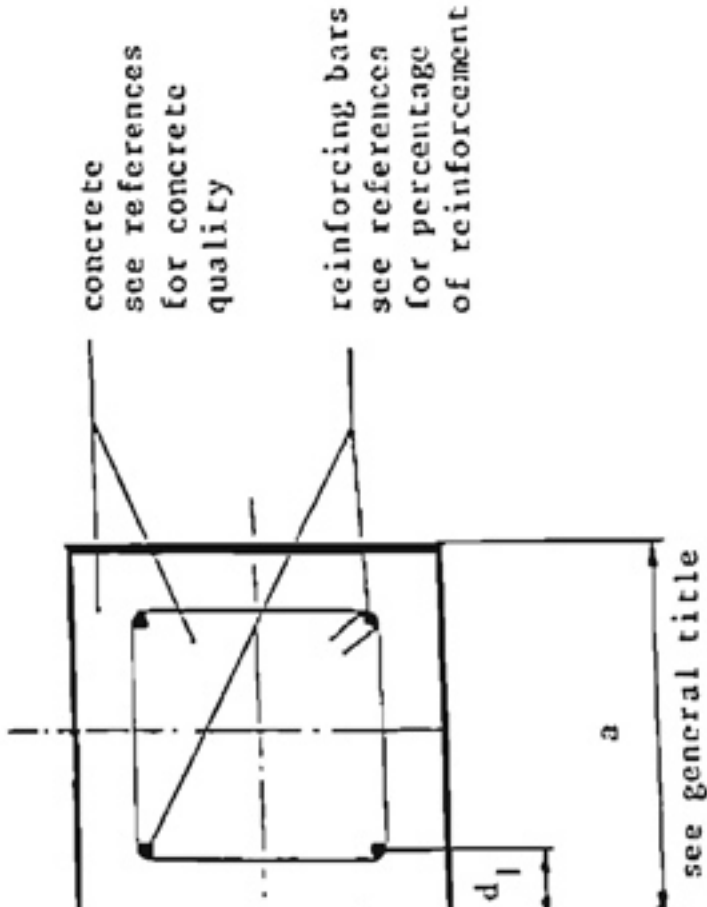
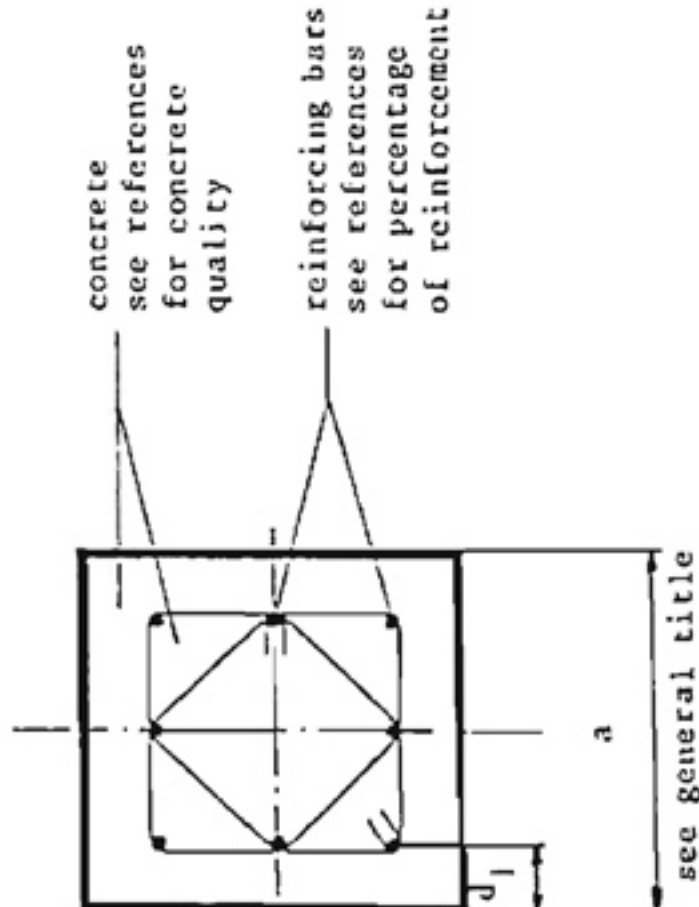
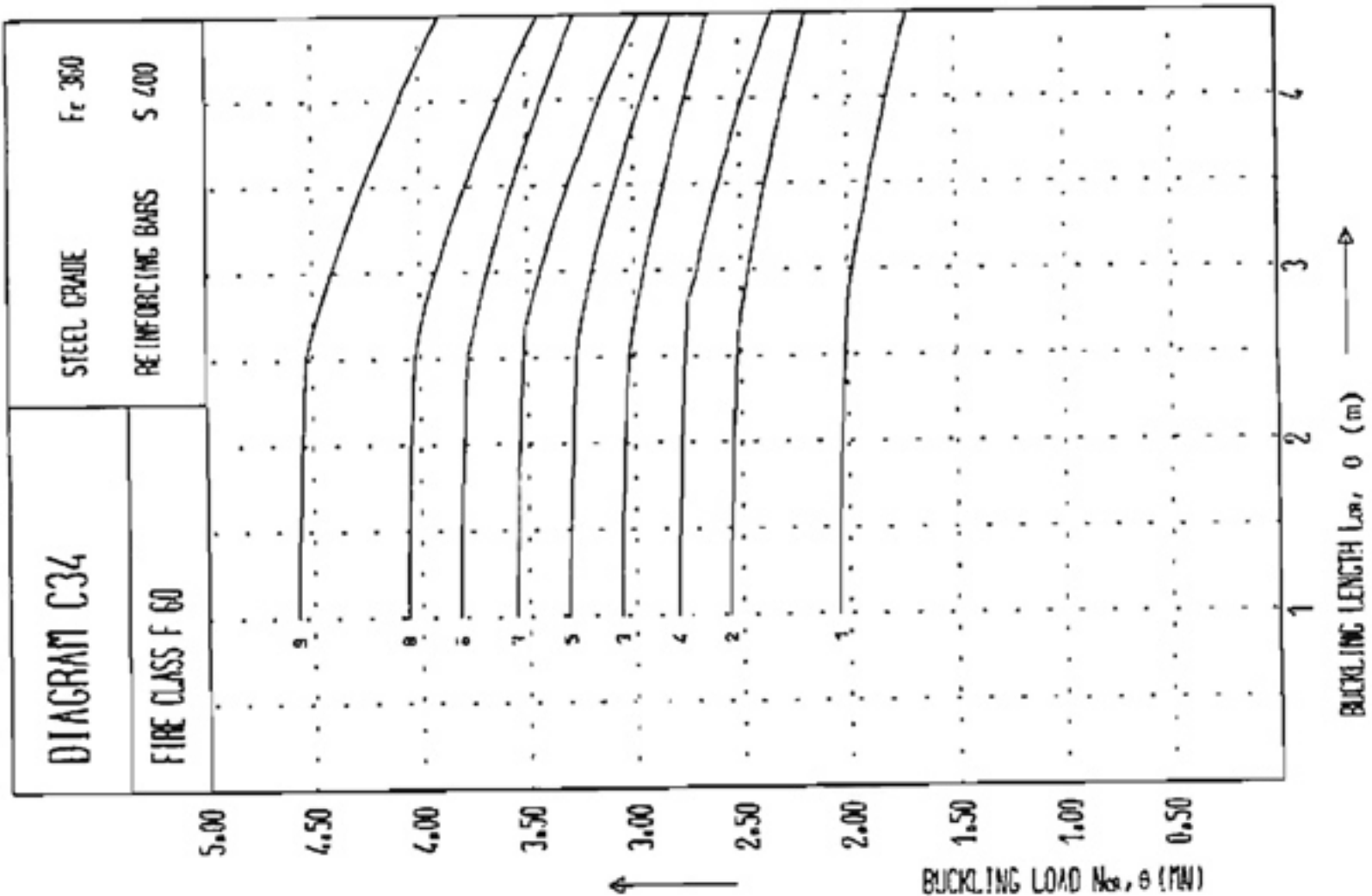
AXIAL LOAD DIAGRAMS FOR COLUMN WITH SQUARE HOLLOW SECTION 260*6*3



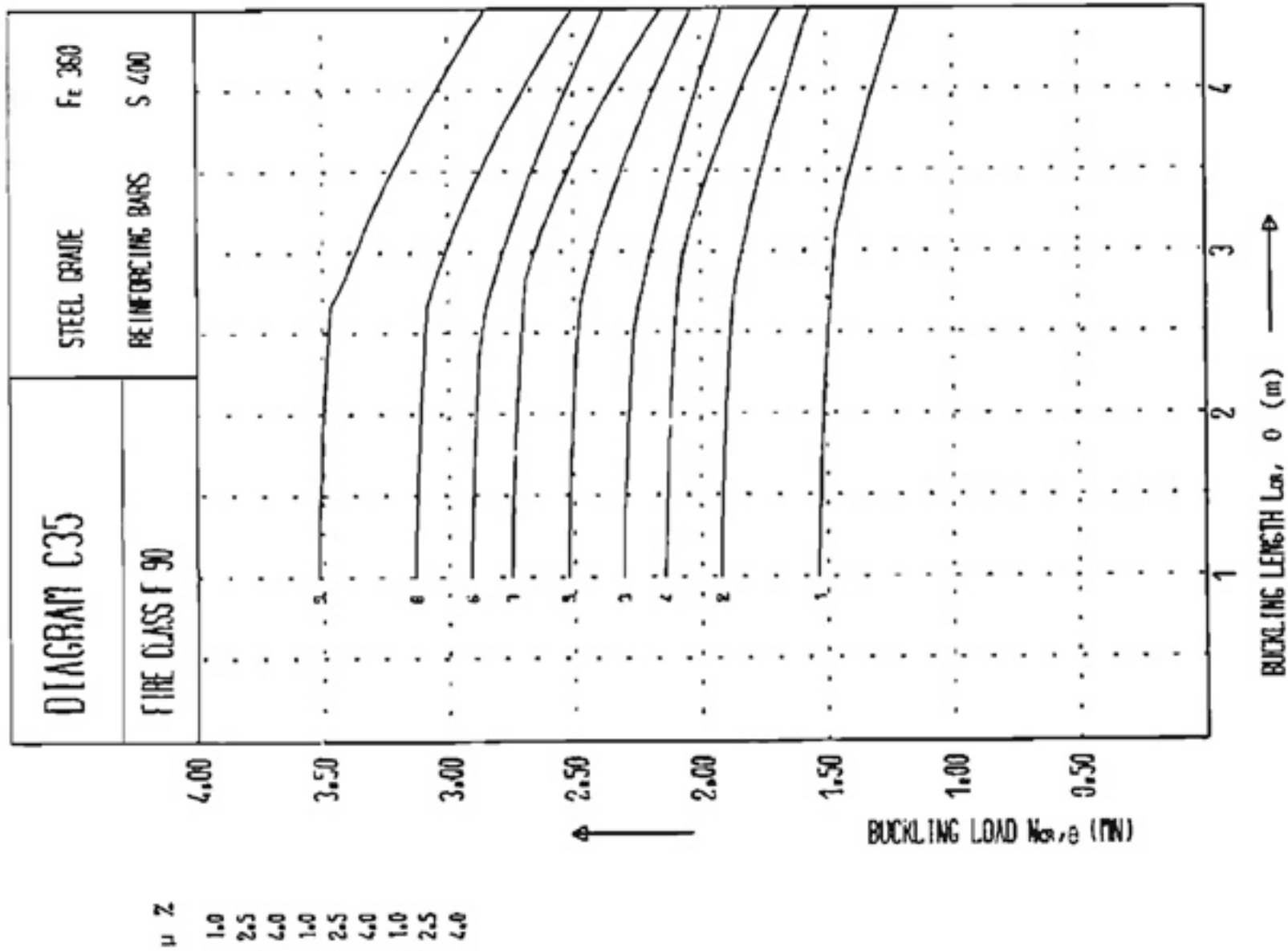
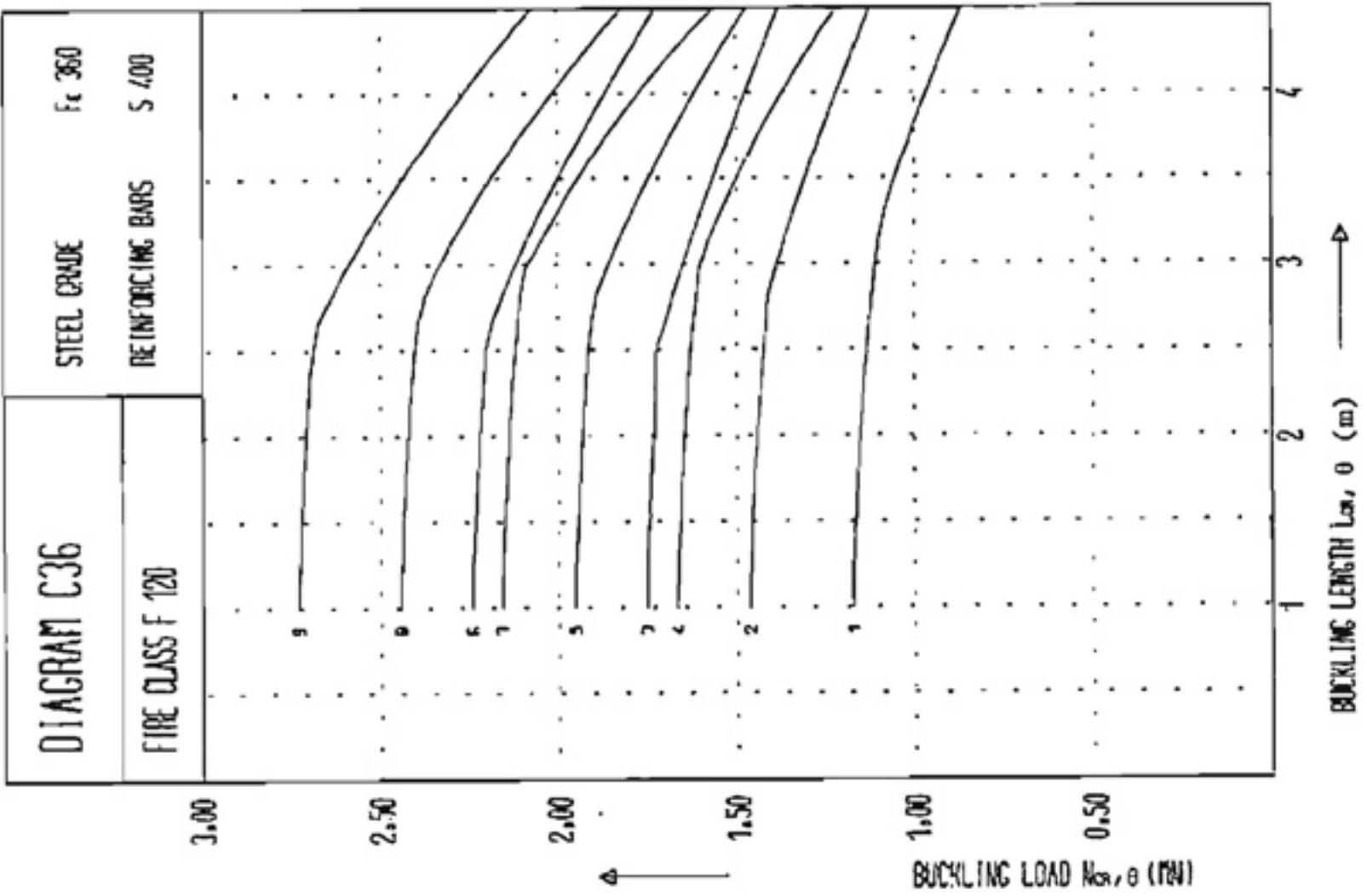
AXIAL LOAD DIAGRAMS FOR COLUMN WITH SQUARE HOLLOW SECTION 300*701



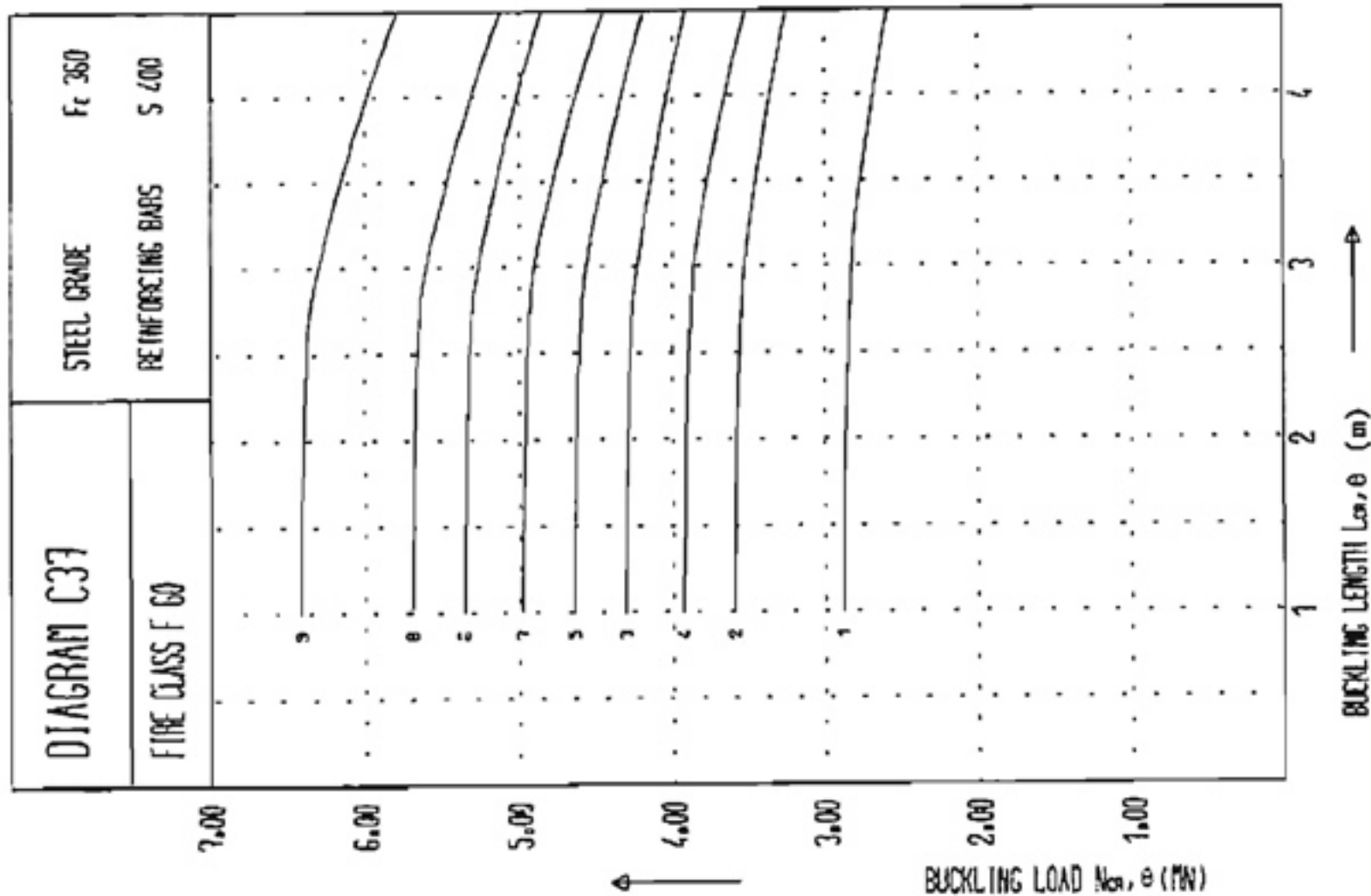
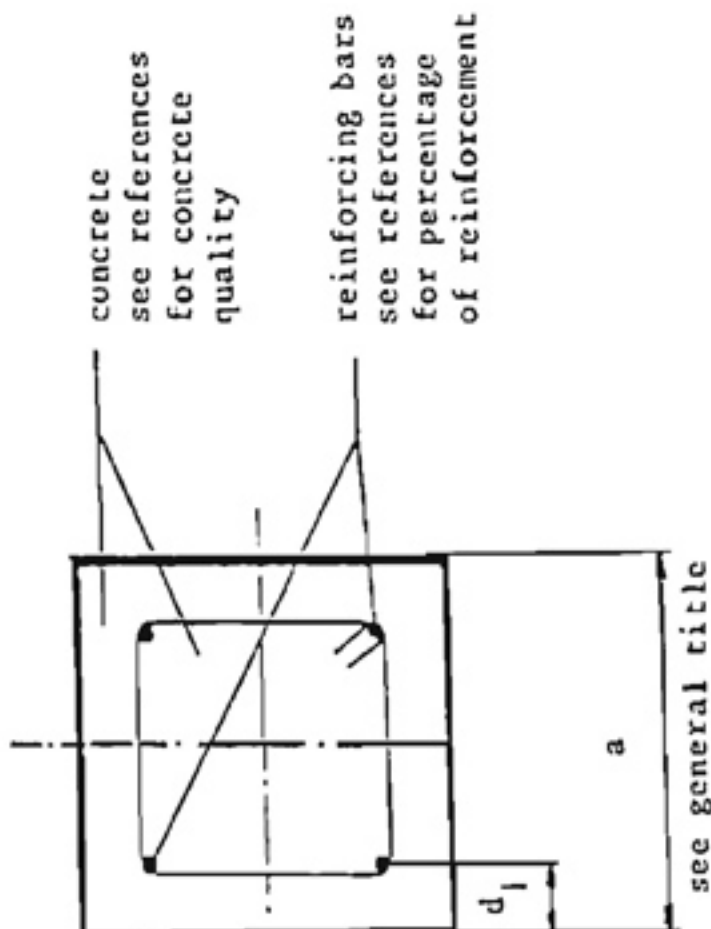
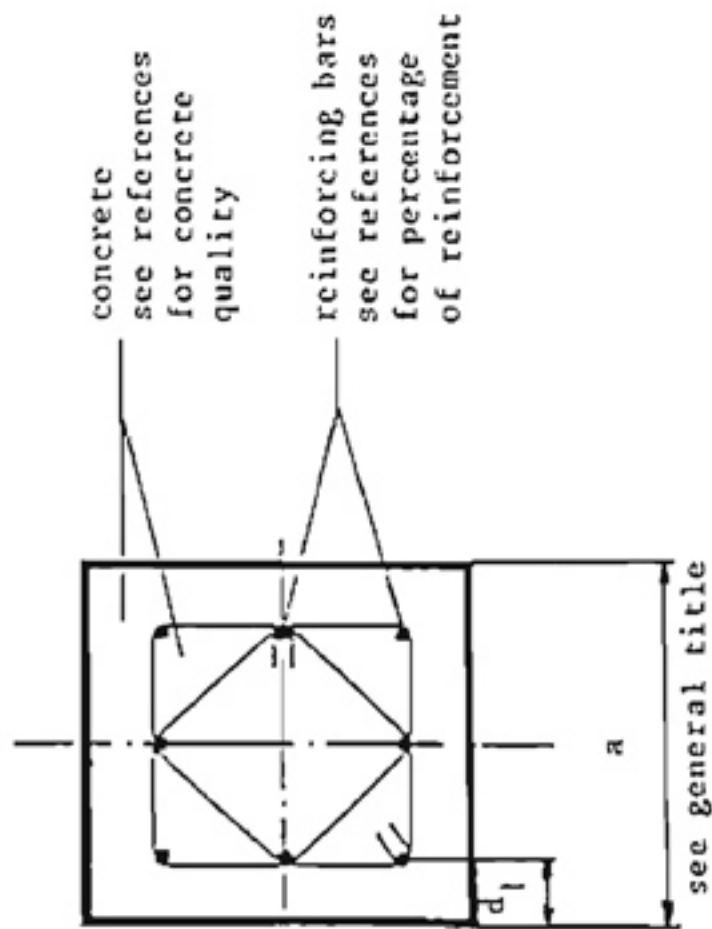
AXIAL LOAD DIAGRAMS FOR COLUMN WITH SQUARE HOLLOW SECTION 300*701



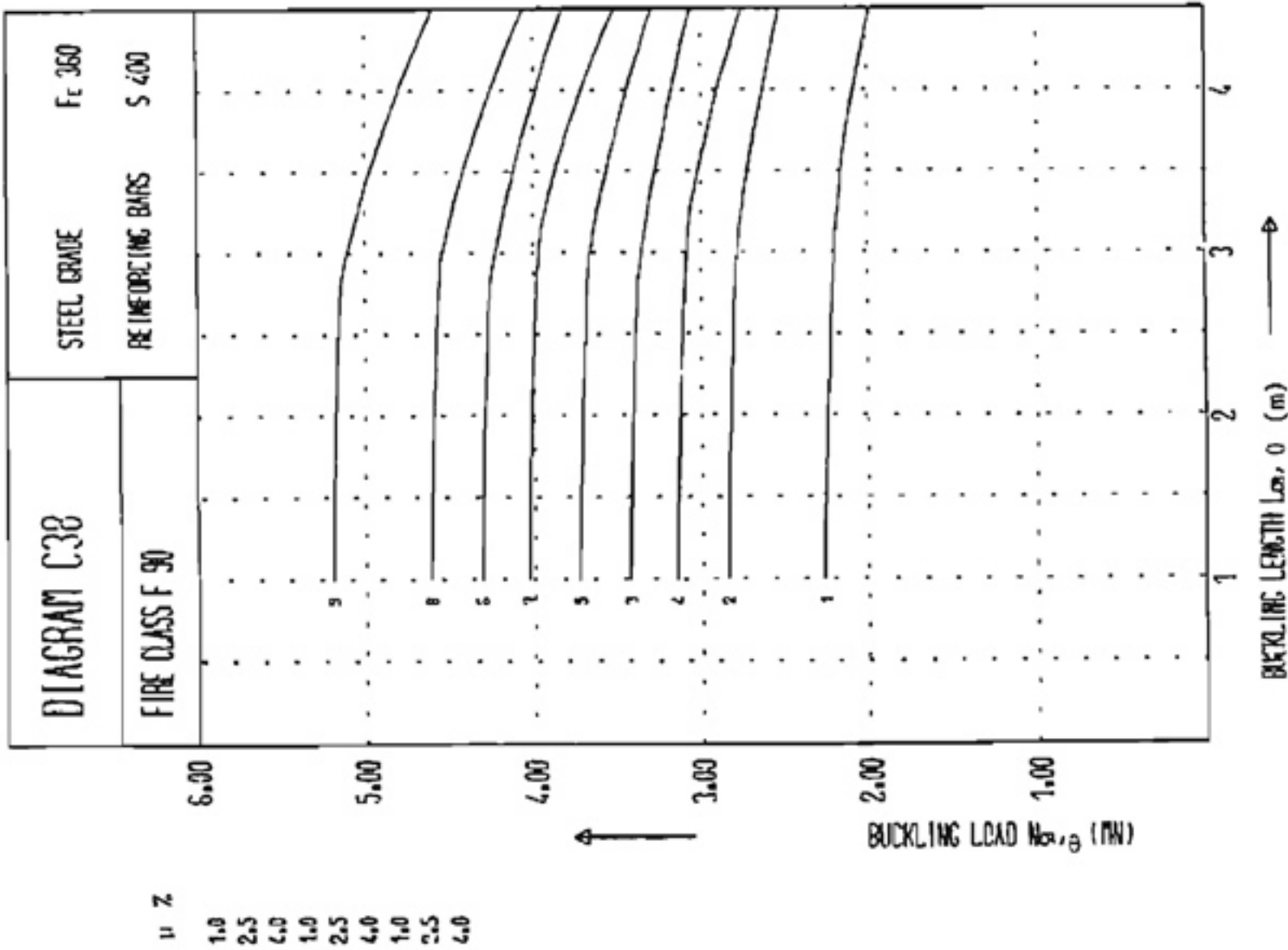
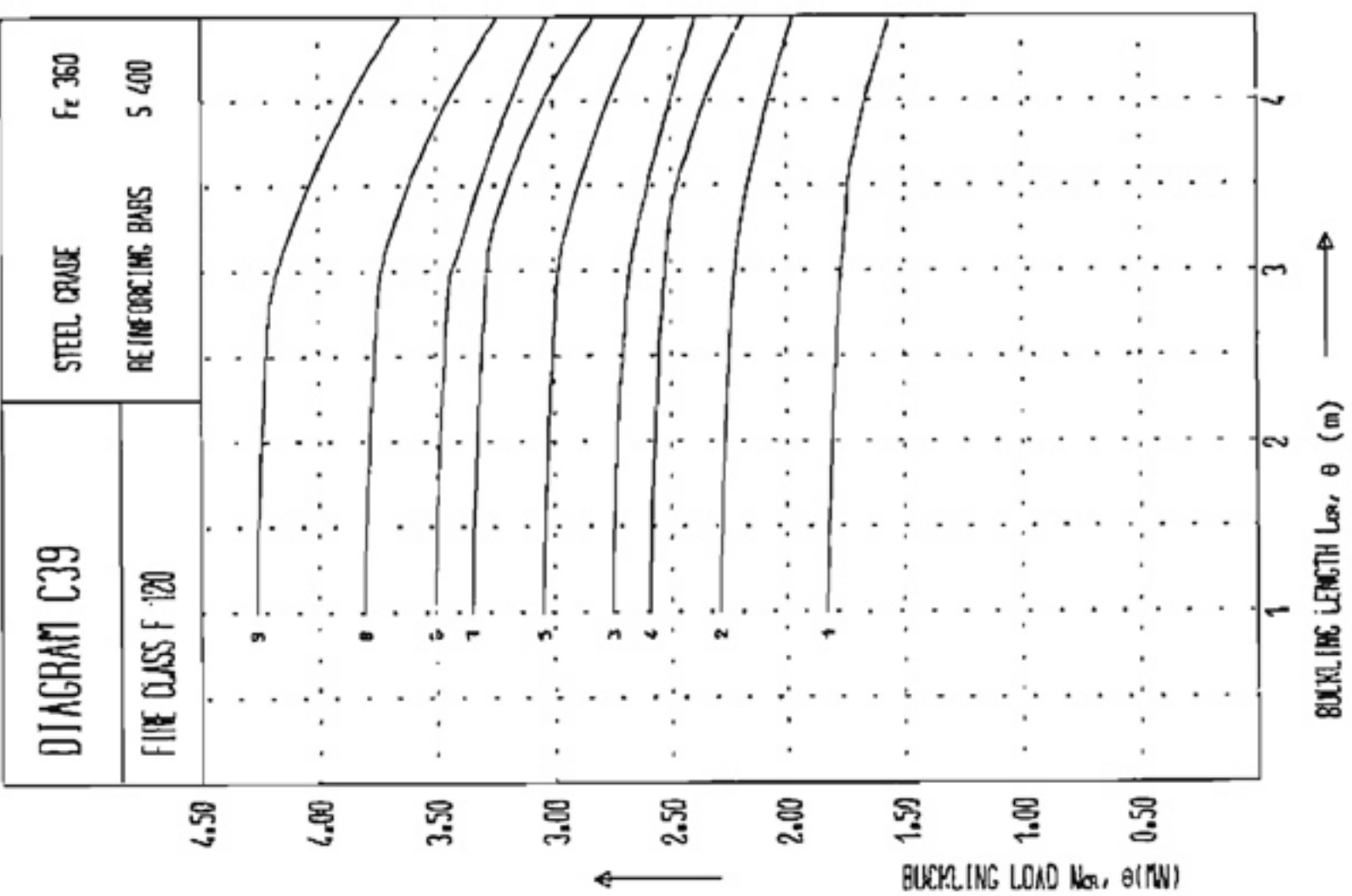
AXIAL LOAD DIAGRAMS FOR COLUMN WITH SQUARE HOLLOW SECTION 350x8n0



AXIAL LOAD DIAGRAMS FOR COLUMN WITH SQUARE HOLLOW SECTION 350*8*0



AXIAL LOAD DIAGRAMS FOR COLUMN WITH SQUARE HOLLOW SECTION 400x10.0



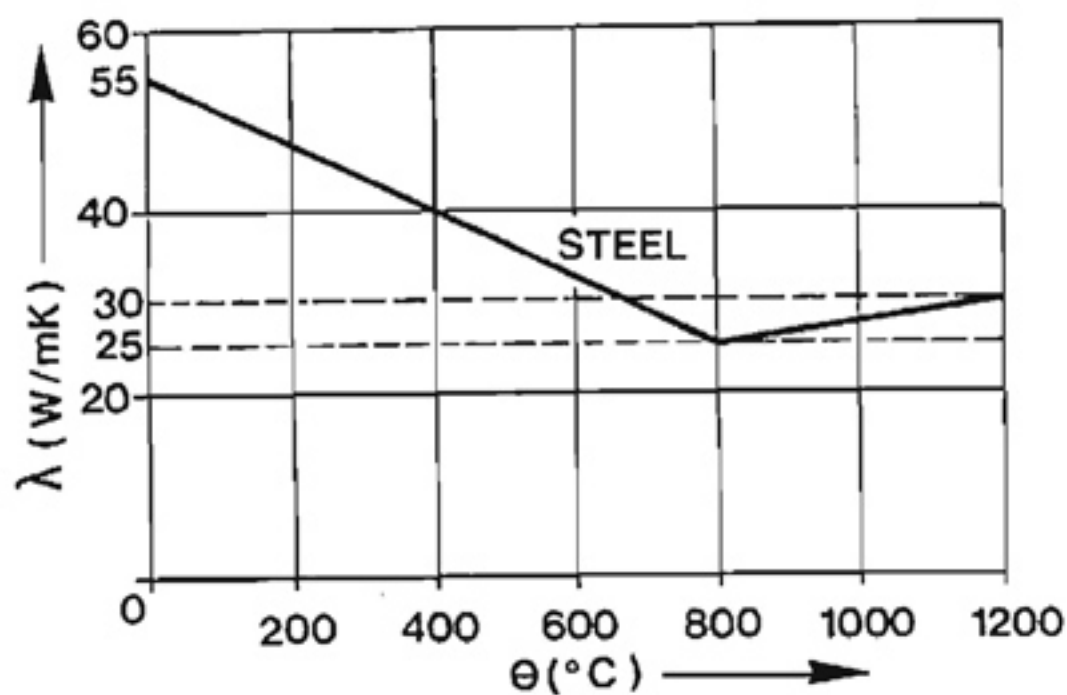
AXIAL LOAD DIAGRAMS FOR COLUMN WITH SQUARE HOLLOW SECTION 400x10x0

Appendix D: THERMO-MECHANICAL MATERIAL PROPERTIES AT
ELEVATED TEMPERATURES

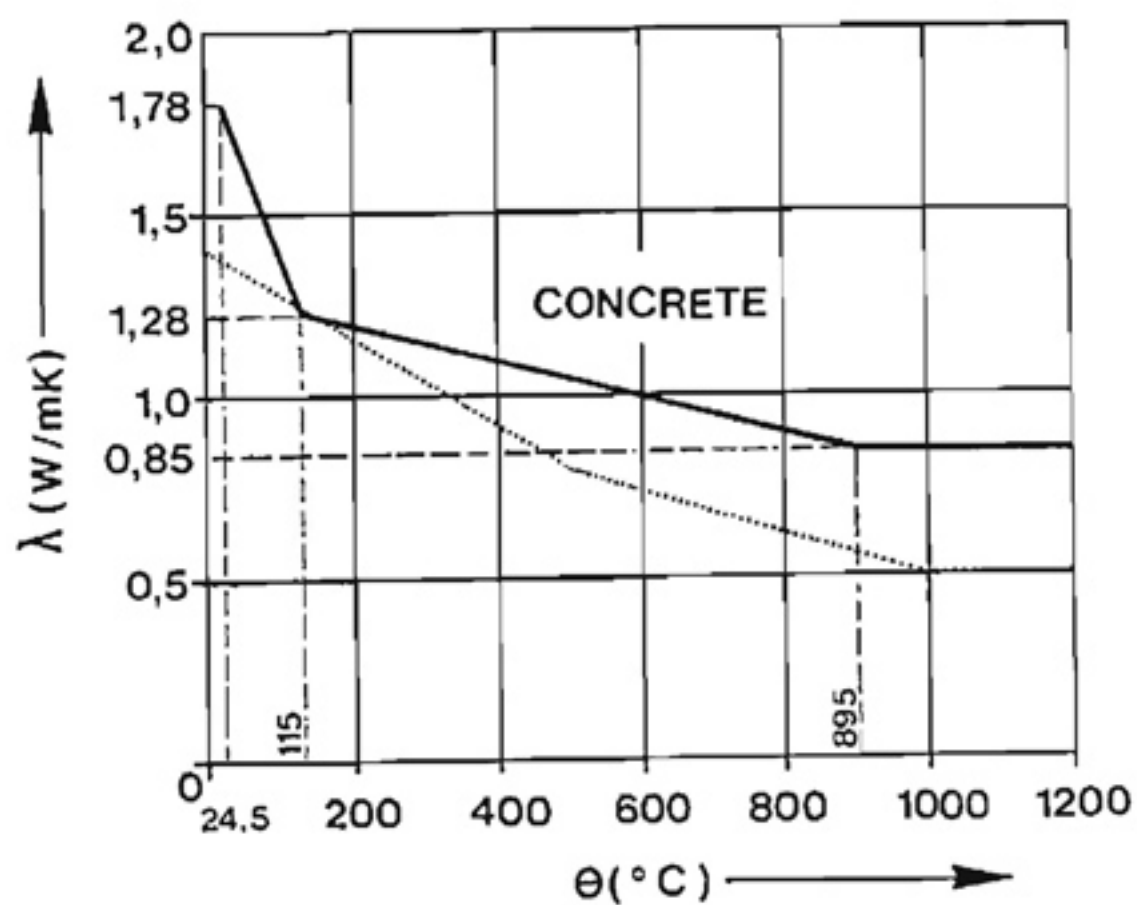
The material laws given in this appendix form the basis for the calculations shown in chapter 3 (full curves —), in chapter 4 (interrupted curves - -) and in chapter 5 (dotted curves).

The thermal properties of steel and concrete are given in diagrams D1 to D3.

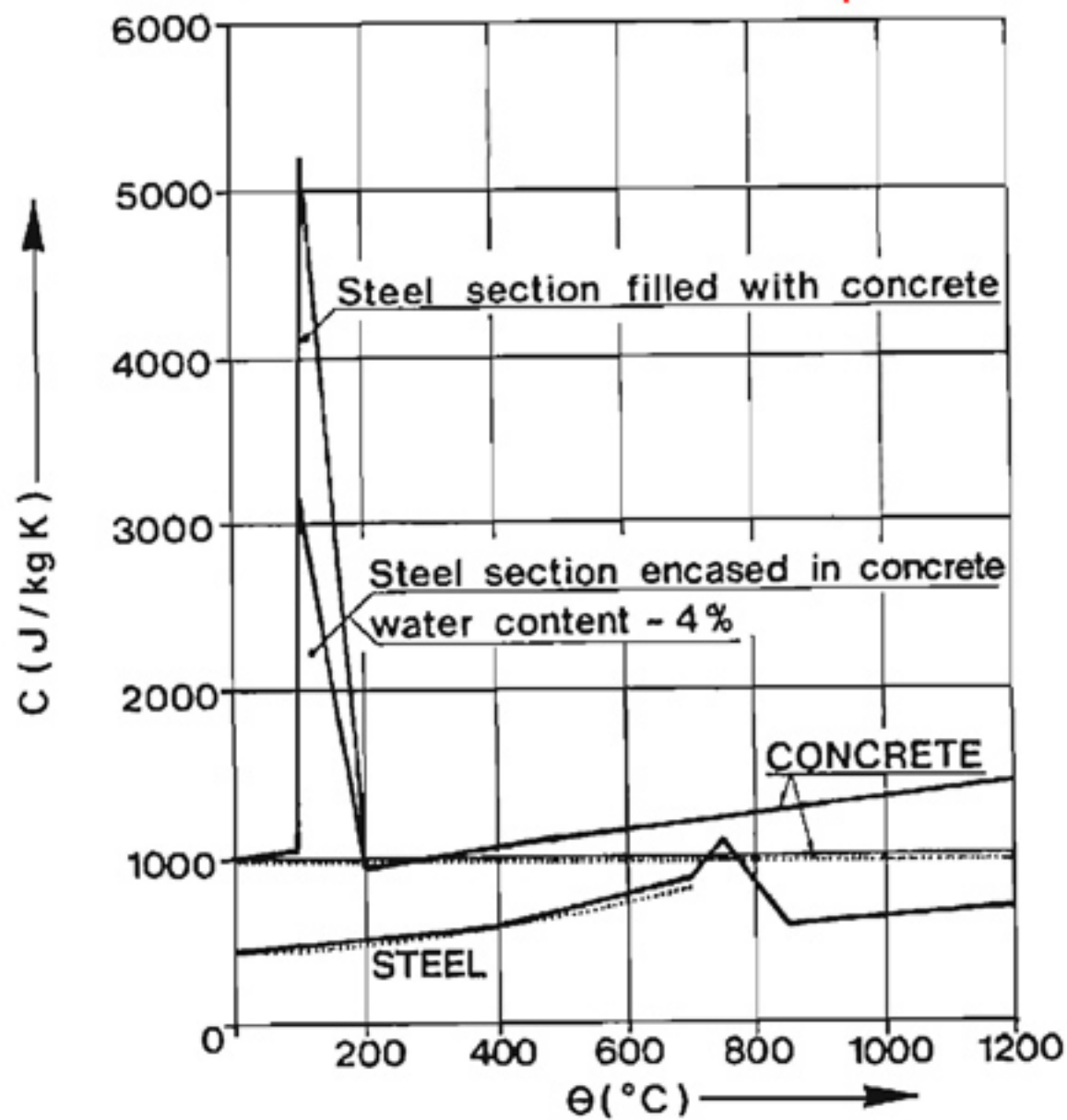
The mechanical properties of steel and concrete are given in diagrams D4 to D12.



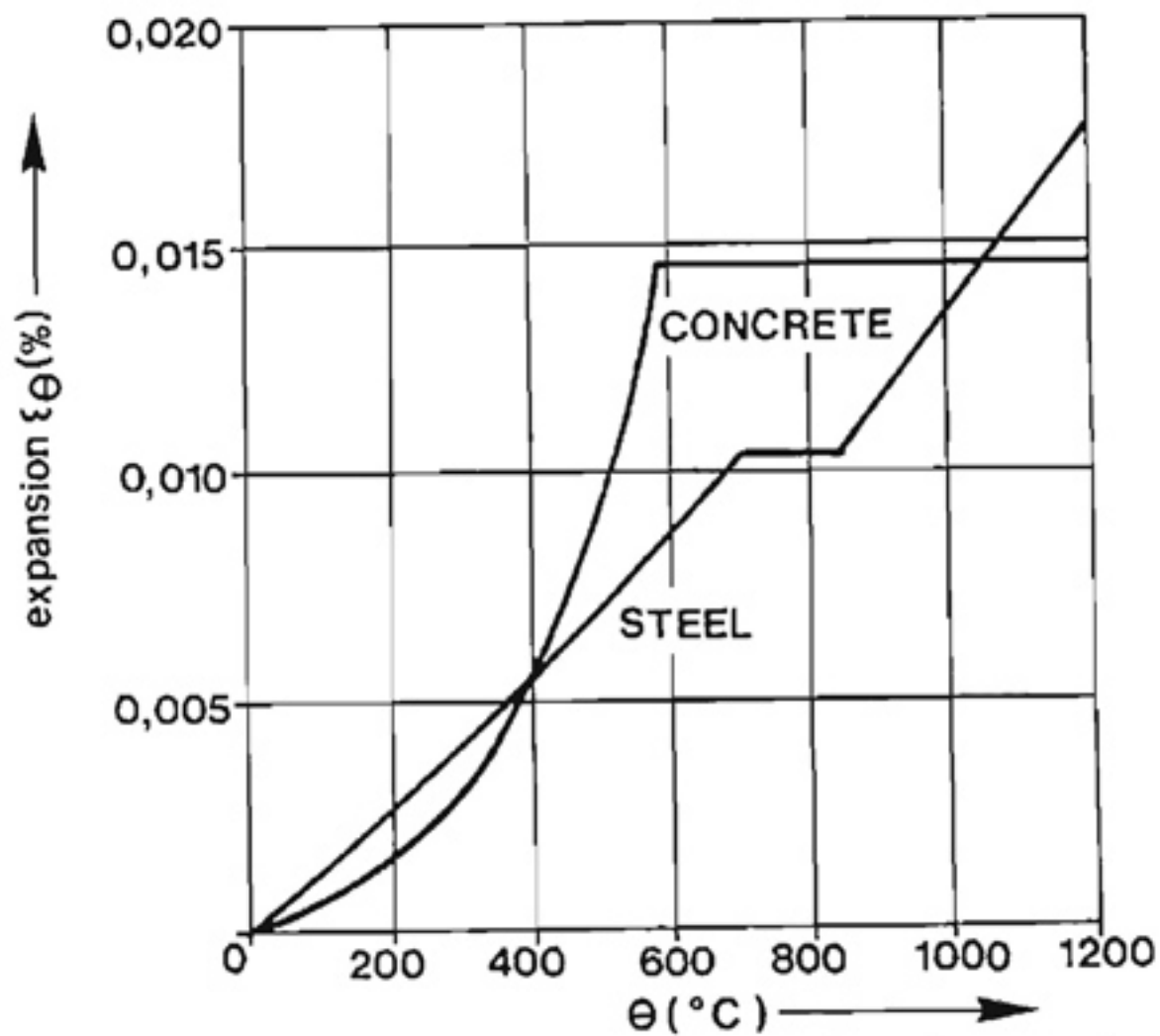
D.1/A Thermal conductivity of steel depending on temperature .



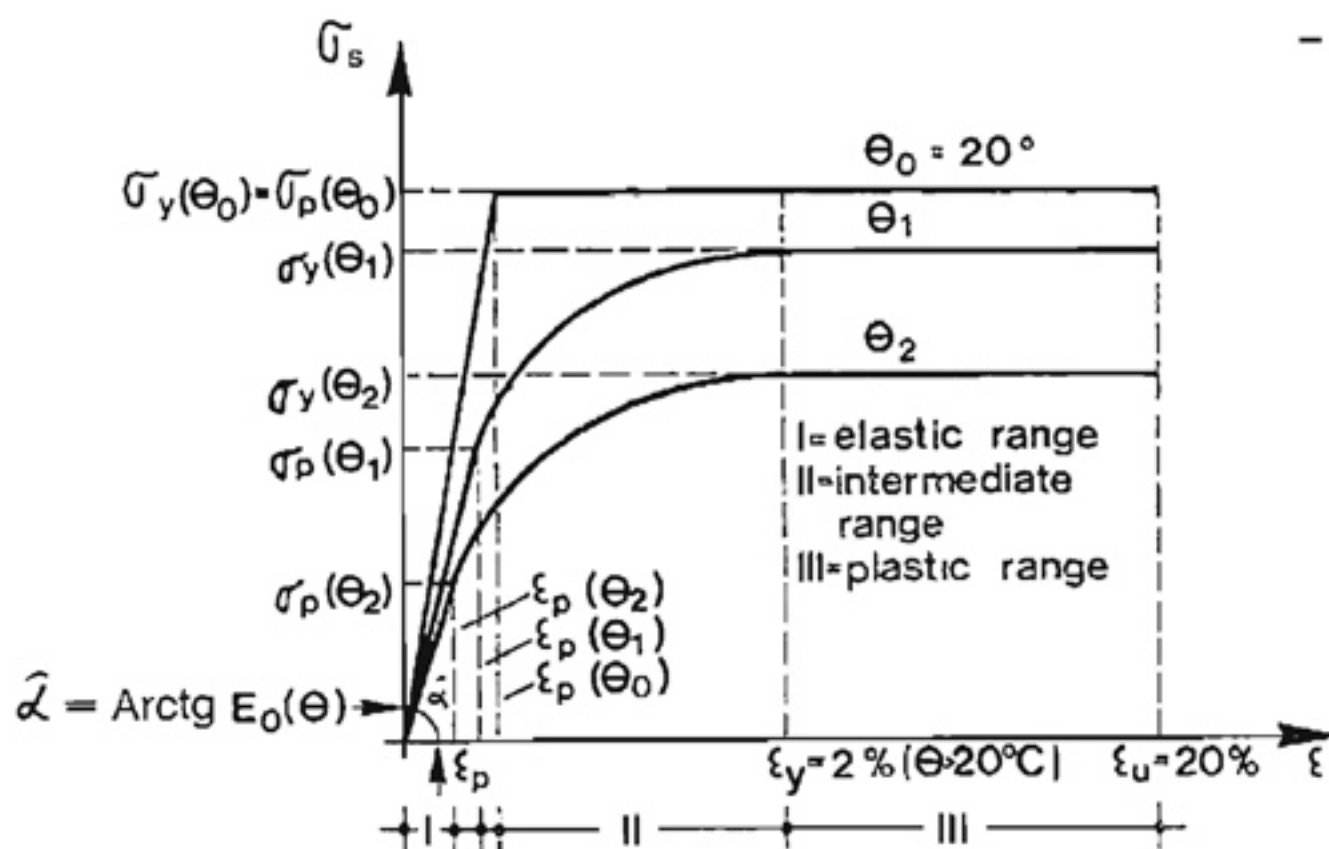
D.1/B Thermal conductivity of concrete depending on temperature .



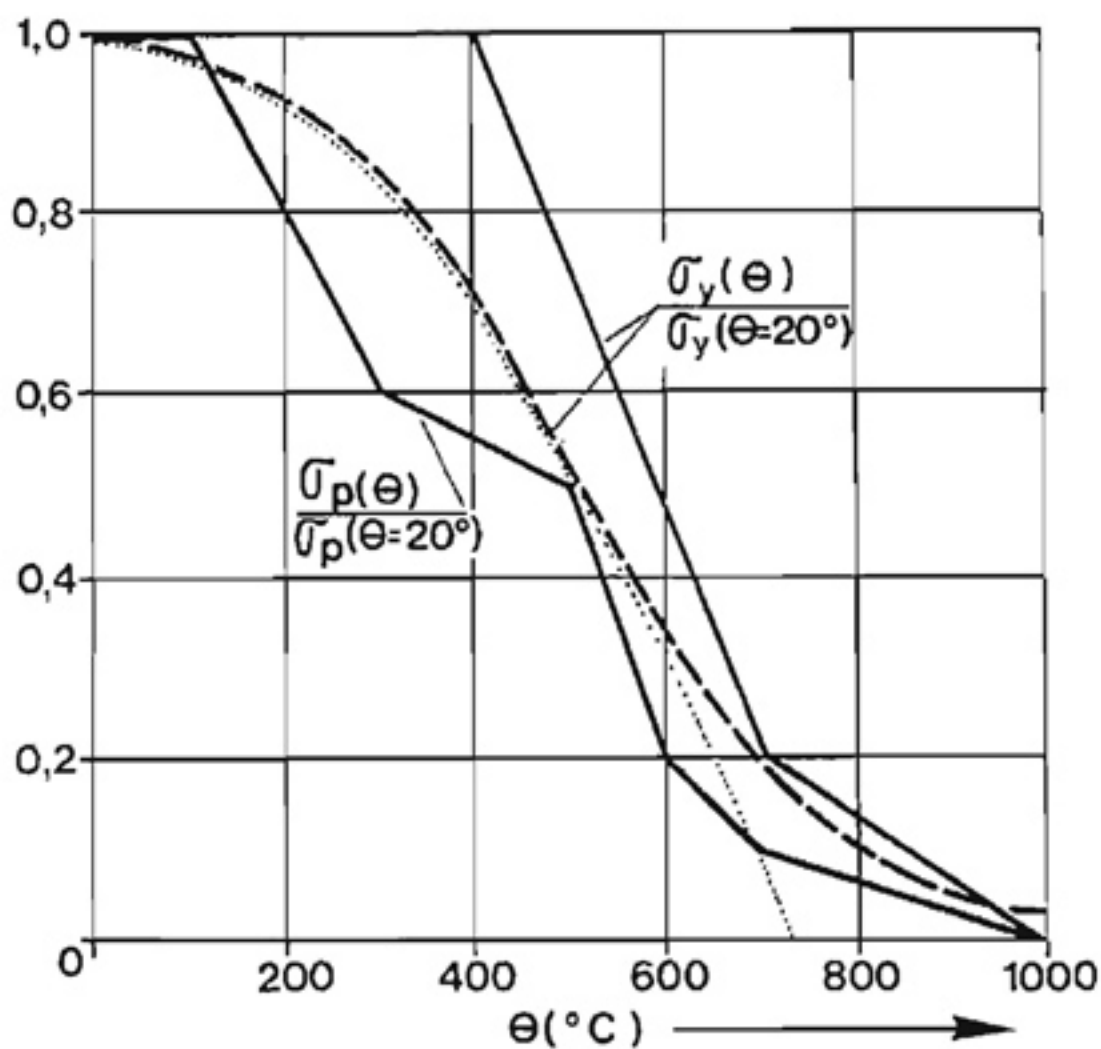
D.2 Specific heat of steel and concrete depending on temperature.



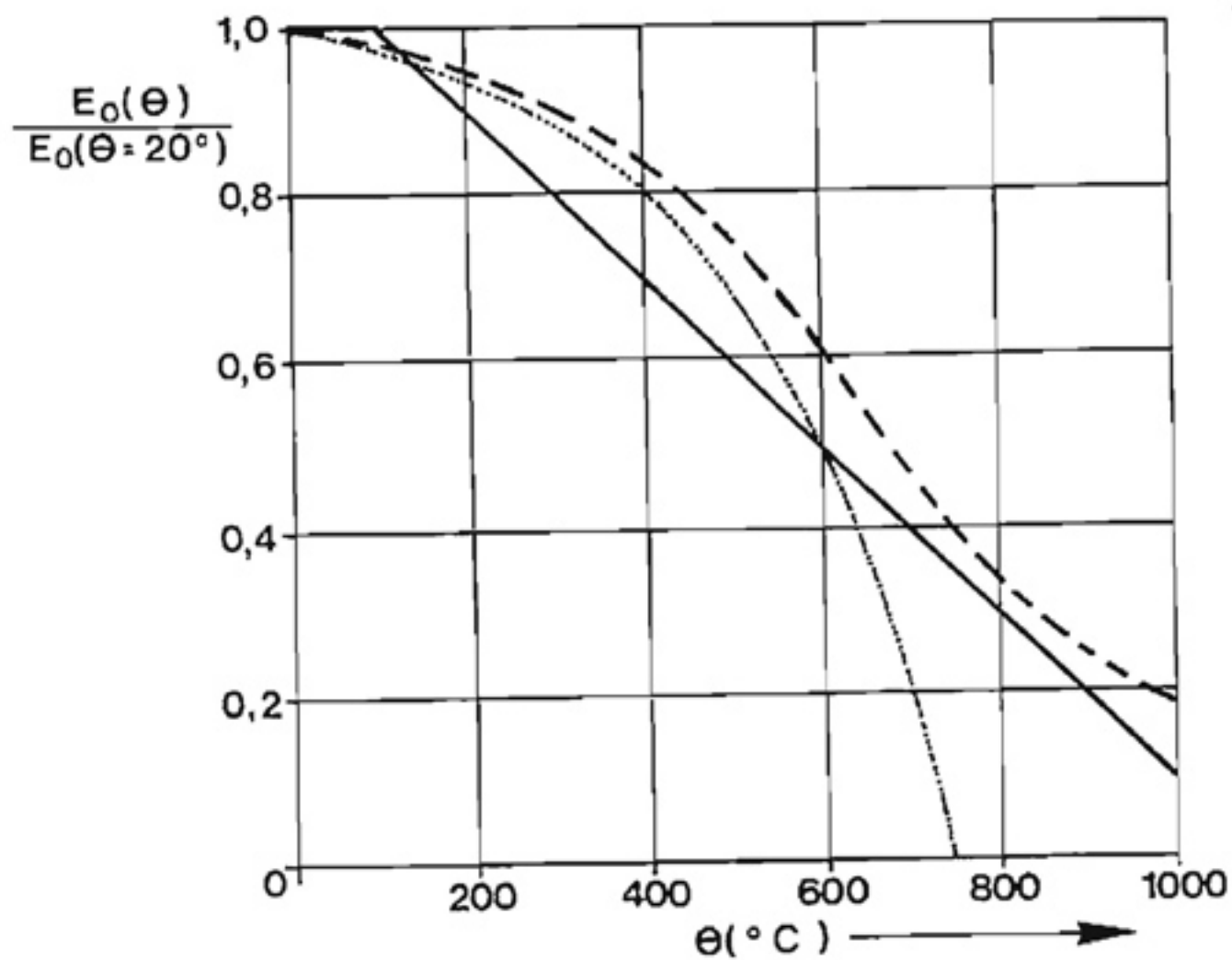
D.3 Thermal expansion of steel and concrete.



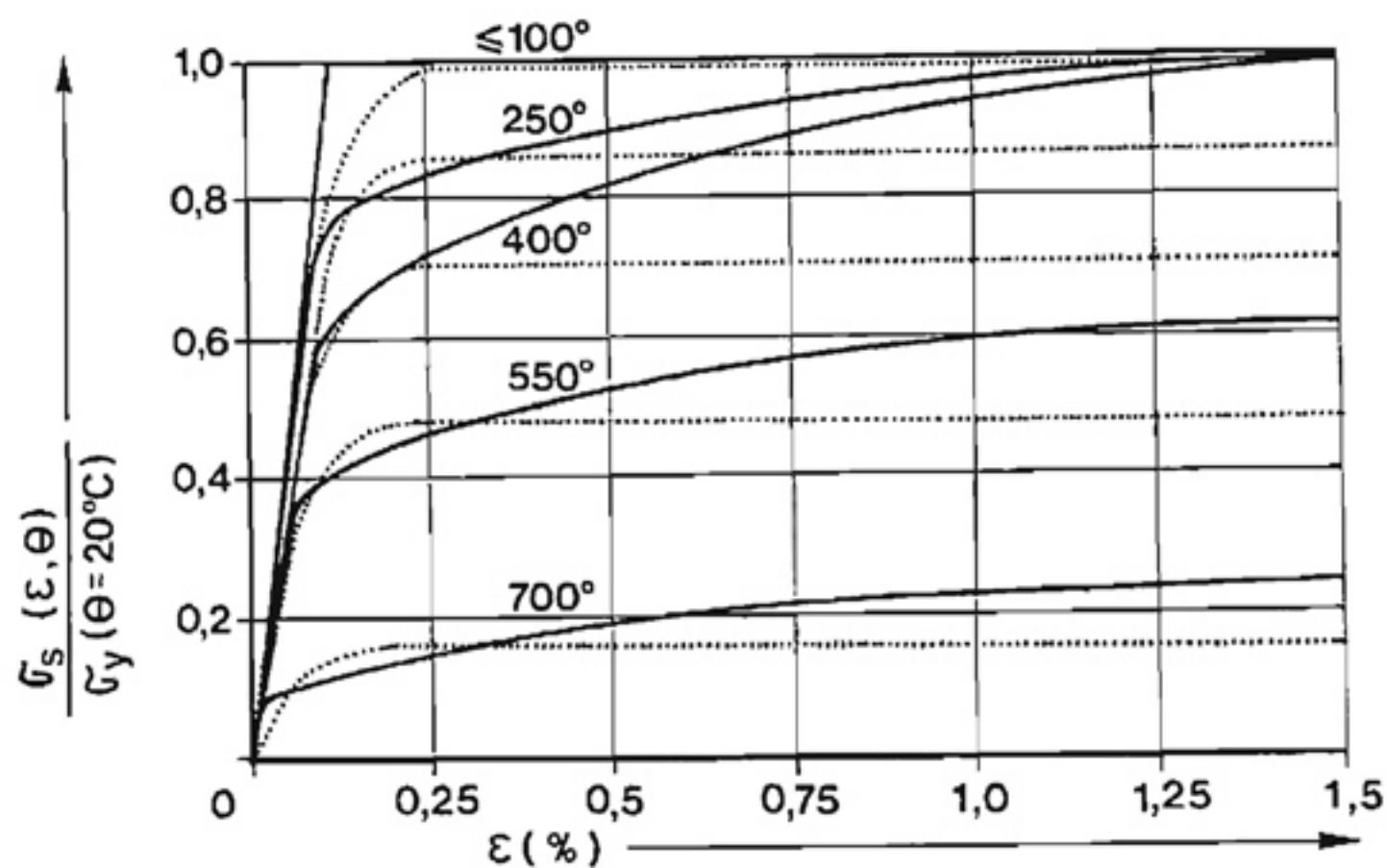
D.4 General σ - ϵ diagrams for steel as function of temperature.



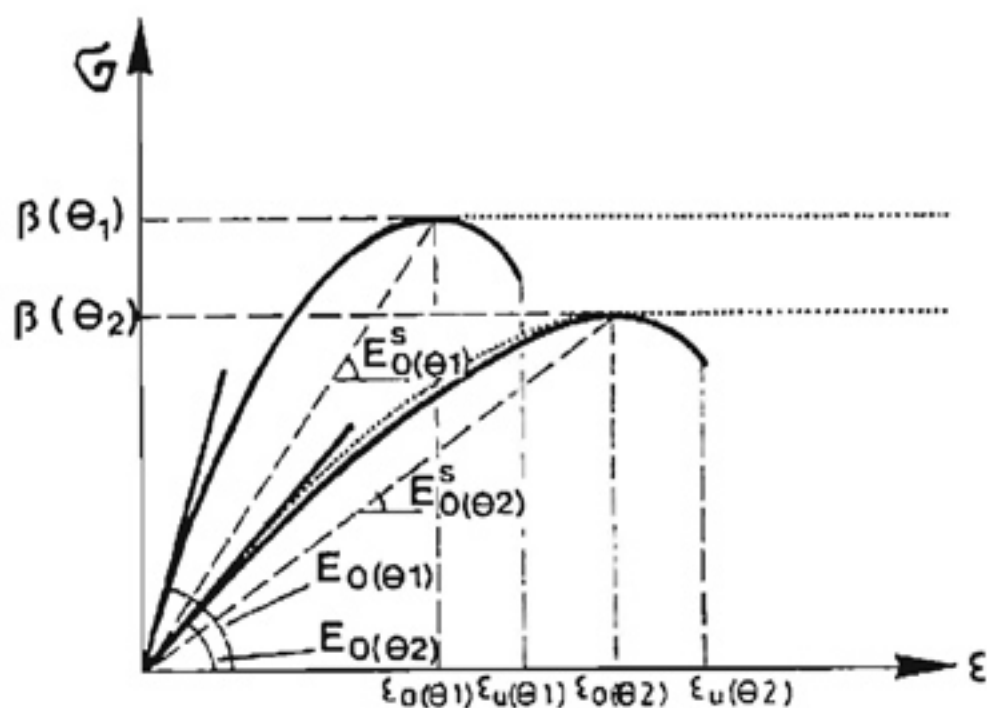
D.5 Relative yield point of steel as function of temperature.



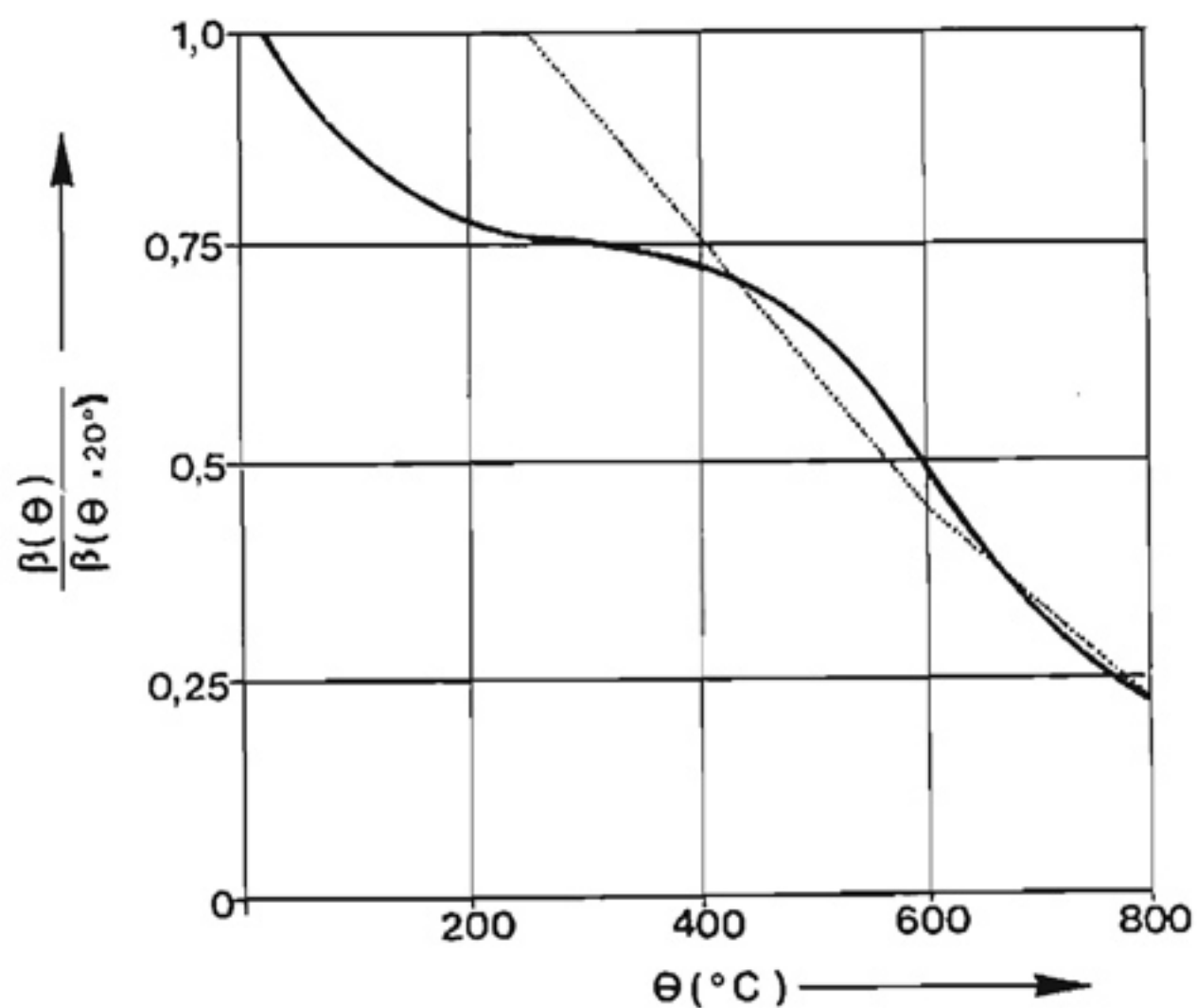
D.6 Elastic modulus of steel as function of temperature.



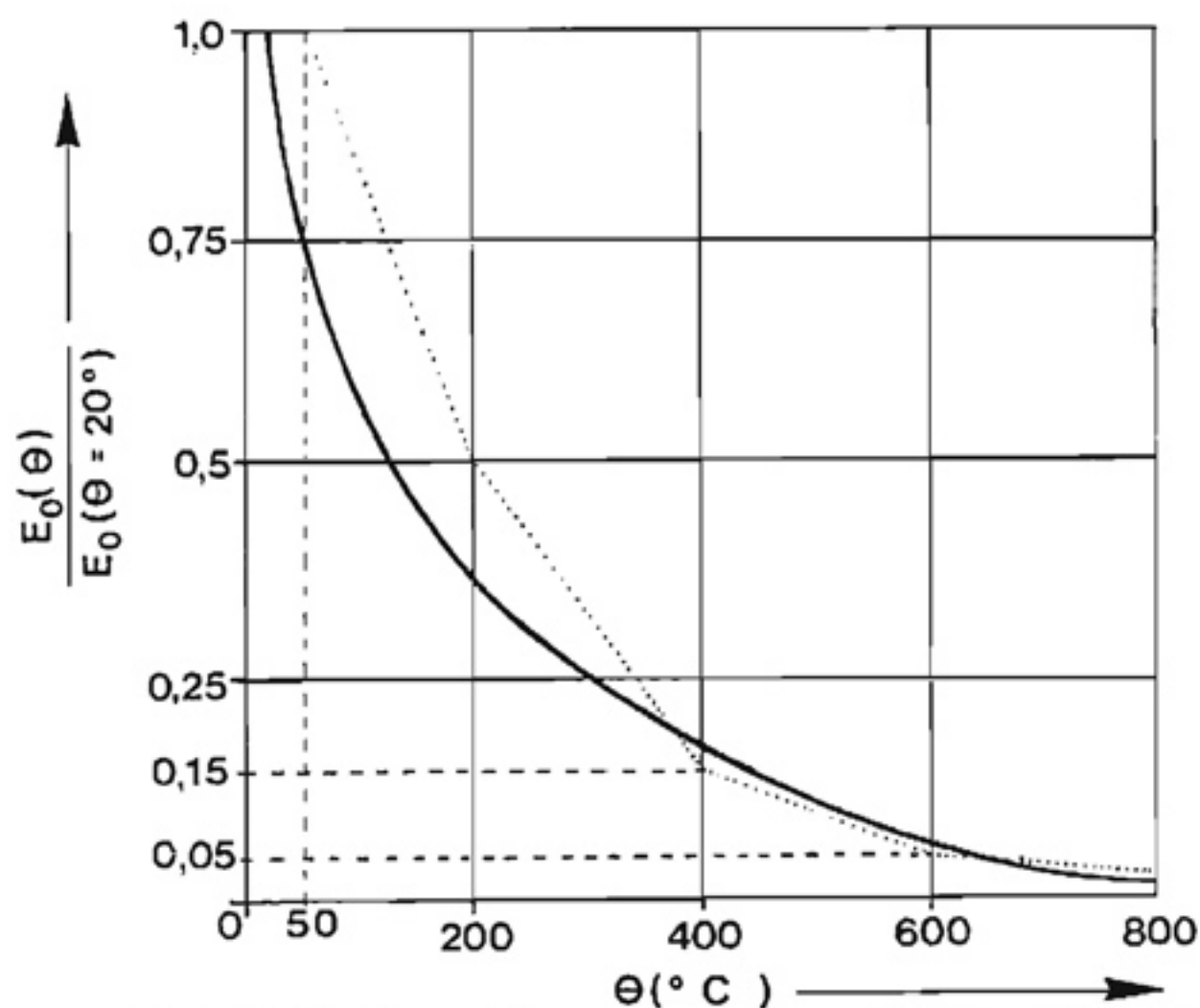
D.7 σ - ϵ diagram for Fe 360 at different temperatures.



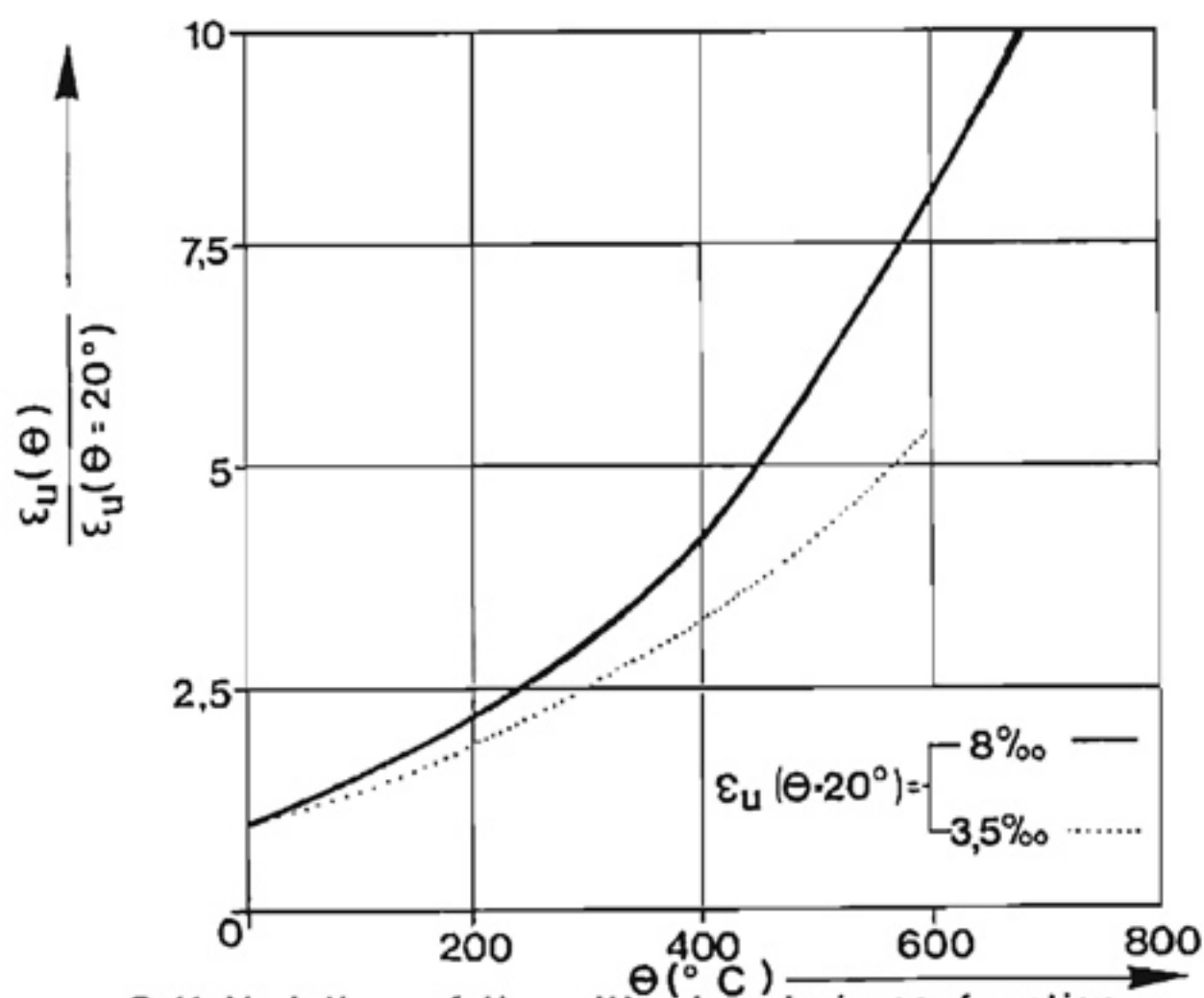
D.8 General σ - ϵ diagrams for concrete as function of temperature .



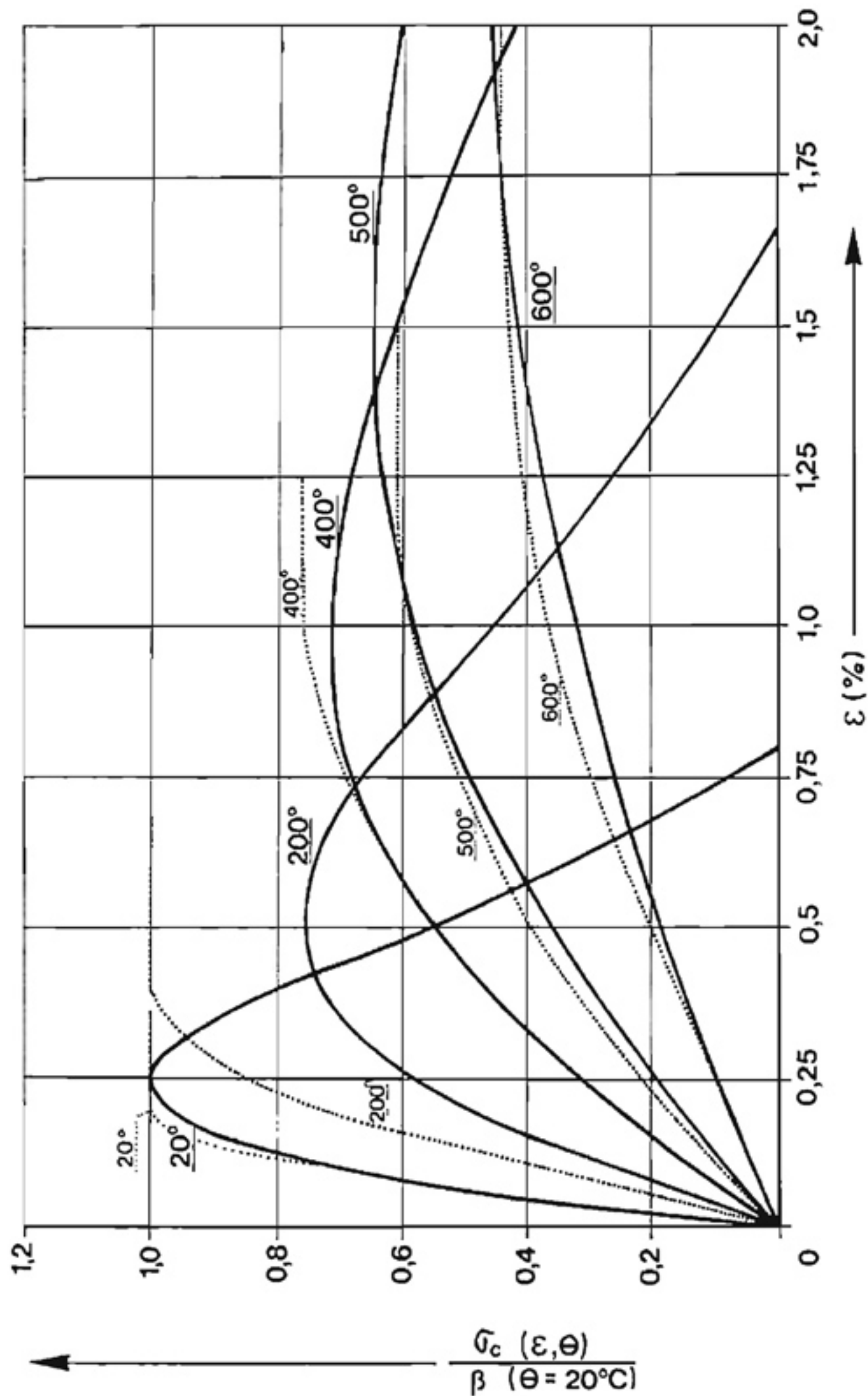
D.9 Reduction of the ultimate compressive strength of concrete as function of temperature .



D.10 Reduction of tangent young modulus of concrete as function of temperature.



D.11 Variation of the ultimate strain as function of temperature.



D. 12 σ - ϵ diagrams for concrete as function of different temperatures .

ECCS LIST OF PUBLICATIONS

April 1989

NB : the publications No 1 to 12 are also available from the ECCS General Secretariat in Brussels.

19. EUROPEAN RECOMMENDATIONS FOR THE STRESSED SKIN DESIGN OF STEEL STRUCTURES
English, 1977, 188pp
NLG 40,-
20. EUROPEAN RECOMMENDATIONS FOR THE TESTING OF PROFILED METAL SHEETS
English, 1976, 40pp
NLG 17,-
22. MANUAL ON STABILITY OF STEEL STRUCTURES
English, 1976, 333pp
NLG 125,-
23. EUROPEAN RECOMMENDATIONS FOR STEEL CONSTRUCTION
English, 1977, 366pp
NLG 90,-
24. DEFORMATIONS IN MULTI-STORY STRUCTURES, GRAVITATIONAL AND TEMPERATURE
Trilingual, 1978-1980-1981, 33pp
OUT OF PRINT
25. SURVEY OF RESEARCH AND DEVELOPMENT IN STEEL CONSTRUCTION 1971-1977
English, 1977, 173pp
OUT OF PRINT
26. EUROPEAN RECOMMENDATIONS FOR ALUMINIUM ALLOY STRUCTURES
English, 1978, 160pp
Sold and distributed by I.A.I., rue Montoyer, 47, B - 1040 BRUXELLES
BEF 200,-
27. RECOMMENDATIONS FOR THE CALCULATION OF WIND EFFECTS ON BUILDINGS AND STRUCTURES
English, 1978, 157pp
(see also revised edition 1987, publication No 52 of the present list)
OUT OF PRINT
28. COMPOSITE STRUCTURES (Model Code for)
English, 1981, 163pp
NLG 52,-
29. EUROPEAN RECOMMENDATIONS FOR STEEL CONSTRUCTION : BUCKLING OF SHELLS
English, 1984, 3rd edition, 97pp
NLG 30,-
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