

Design Guide of High Strength Concrete to Singapore Standard CP 65

DRAFT



TABLE OF CONTENT

| | |
|---|-----------|
| SECTION 1. INTRODUCTION | 5 |
| 1.1 Scope | 5 |
| 1.2 Basic information on HSC | 5 |
| SECTION 2. EXTENSION OF CP65 FOR HIGH STRENGTH CONCRETE | 7 |
| 2.1 Overview of Changes to CP65 | 7 |
| 2.2 Proposed Changes to CP65 : Part 1 : 1999 | 7 |
| Clause 2.5.3 Analysis of sections for the ultimate limit state. | 8 |
| Clause 3.4.4.2 Design charts. | 8 |
| Clause 3.4.4.4 design formulae for rectangular beams. | 8 |
| Clause 3.4.4.5 Design ultimate moments of resistance (flanged beams where the neutral axis falls below the flange). | 10 |
| Clause 3.4.5.2 Shear stress in beams. | 11 |
| Clause 3.4.5.3 Shear reinforcement: form, area and stress. | 11 |
| Clause 3.4.5.4 Concrete shear stresses. | 12 |
| Clause 3.4.5.8 Enhanced shear strength of sections close to supports. | 12 |
| Clause 3.4.5.12 Shear and axial compression. | 12 |
| Clause 3.5.5.2 Shear stresses. | 12 |
| Clause 3.5.5.3 Shear reinforcement. | 12 |
| Clause 3.6.4.6 Maximum design shear stress. | 13 |
| Clause 3.7.6.4 Maximum design shear stress at the column face. | 13 |
| Clause 3.7.7.2 Maximum design shear capacity. | 13 |
| Clause 3.11.4.5 Punching shear. | 13 |
| Clause 3.12.5.3 Minimum percentage of reinforcement. | 13 |
| Clause 3.12.7.4 Minimum percentage of reinforcement. | 13 |
| Clause 3.12.8.4 Values for design ultimate anchorage bond stress. | 14 |
| Clause 4.3.8.2 Maximum design shear stress. | 14 |
| Clause 4.3.8.7 | 14 |
| Shear reinforcement where V does not exceed $V_c + 0.4 b_v d$. | 14 |
| Clause 4.3.8.8 | 14 |
| Shear reinforcement where V exceeds $V_c + 0.4 b_v d$. | 14 |
| Section six. Concrete: materials, specification and construction. | 14 |
| 2.3 Proposed Changes to CP65 : Part 2 1996 (1999) | 15 |
| Section two. Clauses 2.4.5 and 2.4.6. | 15 |
| Section four. Fire resistance. | 15 |
| SECTION 3. GUIDE TO ENSURE PRODUCT CONFORMITY AND QUALITY FOR HIGH STRENGTH CONCRETE | 16 |
| 3.1 Introduction | 16 |
| 3.2 Preconstruction meeting | 16 |
| 3.3 Trial batches | 17 |
| 3.4 Prequalification of concrete suppliers and preconstruction testing | 17 |

| | | |
|------------|---|-----------|
| 3.5 | Quality Assurance and Quality Control | 18 |
| 3.6 | Testing | 19 |
| 3.7 | Prequalification of testing laboratories | 20 |
| 3.8 | Strength evaluation | 20 |
| 3.9 | Acknowledgement | 21 |

SECTION 4. GUIDELINE ON THE SUBMISSION TO BCA FOR THE USE OF HIGH STRENGTH CONCRETE 23

The following information are to be submitted in the form of a report: 23

| | | |
|-------|---|----|
| 4.1.1 | General Information | 23 |
| 4.1.2 | Information on the proposed HSC | 23 |
| 4.1.3 | Preconstruction Trial | 23 |
| 4.1.4 | Quality Assurance and Quality Control | 23 |
| 4.1.5 | Testing | 23 |
| 4.1.6 | Proposed solution in case of non-compliance | 24 |

SECTION 5. DESIGN GUIDELINE FOR THE USE OF HIGH STRENGTH CONCRETE 25

Summary of changes to CP65 for HSC 25

| | | |
|-------------|---|-----------|
| 5.1 | Cover for durability | 25 |
| 5.2 | Cover as fire protection | 25 |
| 5.3 | Flexural design | 25 |
| 5.4 | Shear | 26 |
| | Worked Example 1 | 27 |
| | Worked Example 2 | 27 |
| | Worked Example 3 | 28 |
| | Worked Example 4 | 29 |
| | Worked Example 5 | 29 |
| | Worked Example 7 | 30 |
| | Worked Example 8 | 32 |
| 5.5 | Deflection | 33 |
| 5.6 | Column design | 34 |
| 5.7 | Minimum reinforcement | 34 |
| 5.7.1 | Beams and slabs | 34 |
| 5.7.2 | Columns | 35 |
| 5.7.3 | Walls | 35 |
| 5.8 | Size and pitch of links in columns | 35 |
| 5.9 | Bond and anchorage | 35 |
| 5.10 | Shrinkage | 35 |

| | | |
|-------------------|--|-----------|
| 5.11 | Creep | 35 |
| SECTION 6. | SECTION 1. COLUMN DESIGN CHARTS | 36 |
| 6.1 | Introduction | 36 |
| 6.2 | Examples of column design | 36 |
| | Example 1: Rectangular column | 36 |
| | Example 2: Circular column | 39 |
| | Example 3: Rectangular column | 41 |
| | Example 4: Rectangular column | 43 |
| 6.3 | Design Charts for Rectangular Columns | 46 |
| 6.4 | Design Charts for Circular Columns | 72 |
| SECTION 7. | REFERENCE | 93 |

Section 1. Introduction

1.1 Scope

This guidance is intended for use by engineers familiar with the design of concrete structures using particularly CP65 but who have little or no experience of high strength concrete (HSC). The short time to publication necessitates that the design approach contained herein should be recognizable to these engineers. It is intended to provide safe design guidance, based on the best available information, especially in the areas not adequately covered by CP65. In this respect, the guidance and recommendations in the Concrete Society Technical Report 49 are very relevant and have been extensively adopted here where ever possible. However, it should not be noted that this guidance deals only with HSC made with normal weight aggregates and it should not be use for light weight aggregate concrete.

1.2 Basic information on HSC

The methods and technology for producing HSC are not basically different from those required for concrete of normal grade except that the emphasis on quality control is perhaps greater with HSC.

HSC can be produced with all of the cements and cement replacements normally available in the Singapore, including Portland cement, sulfate-resisting Portland cement, and combinations with pulverised fuel ash and ground granulated blast furnace silica fume slag. High early strength cements should preferably be avoided as a rapid rise in hydration temperature may cause problems of (internal) cracks or micro-cracks due to higher cementitious material content.

HSC can be produced with a wide range of aggregates, but smooth and or rounded aggregates may tend to exhibit aggregate bond failure at a relatively low strength. Crushed rock aggregates, of 10 to 20 mm size, which are not too angular and elongated, should preferably be used. However, it has been found that bond to a small particle of aggregate is greater than to a large particle and for that reason smaller size aggregates (say 10 to 17 mm) tend to give better results. Fine sands should be avoided, particularly those with high absorption.

Superplasticisers should be used to achieve maximum water reduction, although plasticisers may be adequate for lower strength HSC (C60 to C70). Silica fume (microsilica) can be used to enhance the strength at high levels. To facilitate handling, silica fume is often blended into a slurry with superplasticisers, or supplied as a densified powder.

The basic proportioning of an HSC mix follows the same method as for normal strength concrete, with the objective of producing a cohesive mix with minimum voids. This can be done by theoretical calculations or subjective laboratory trials.

The basic strength to water/cement ratio relationships used for producing normal strength concrete are equally valid when applied to HSC, except that the target water/cement ratio can be in the range 0.30-0.35 or even lower. It is essential to ensure full compaction at these levels. A higher ultimate strength can be obtained by designing a mix with a low initial strength gain and cementitious additions. This is partially due to avoidance of micro-cracking associated with high thermal gradients. This effect can be facilitated if strength compliance is measured at 56 instead of 28 days.

Increasing the cement content may not always produce higher strength. Above certain levels it may have little effect. An optimum amount of total cementitious material usually appears to be between 450 and 550 kg/m³.

HSC mixes tend to be very cohesive and a concrete with a measured slump of 50 mm may be difficult to place. As HSC is likely to be used in heavily reinforced sections, a higher workability, should be specified if honeycombing is to be avoided.

When superplasticisers are used, concrete tends to lose workability rapidly. HSC containing such materials must therefore be transported, placed and finished before they lose their effect. Many modern superplasticisers can retain reasonable workability for a period of about 100 minutes, but care is still needed, particularly on projects where ready-mixed concrete delivery trucks have long journey times. Often, in order to avoid drastic decreases in slump and resultant difficulty in placing, superplasticisers are only partly mixed on batching, the balance being added on site prior to pouring.

The same production and quality control techniques for normal strength concrete should also be applied to HSC. For HSC the importance of strict control over material quality as well as over the production and execution processes cannot be over-emphasized. In general, production control should include not only correct batching and mixing of ingredients, but also regular inspection and checking of the production equipment, e.g. the weighing and gauging equipment, mixers and control apparatus. With ready-mixed concrete supply, this control should extend to transport and delivery conditions as well.

The main activities for controlling quality on site are placing, compaction, curing and surface finishing. Site experience indicates that more compaction is normally needed for high strength concrete with high workability than for normal strength concrete of similar slump. As the loss in workability is more rapid, prompt finishing also becomes essential. Particular attention needs to be given to vibration at boundaries of individual loads to avoid 'pour lines'. To avoid plastic shrinkage, the finished concrete surface needs to be covered rapidly with water-retaining curing agents. As the quality of the structure with HSC is the main objective, it is essential that, in addition to the above, the accuracy of the formwork and the fixing details of the reinforcement and/or prestressing steel should also form part of the control activities.

It is also desirable to assess the in-situ strength of the concrete in the actual structure by some non-destructive methods (such as hammer tests or ultrasonic pulse velocity measurements) for comparison with compliance cube test results, to establish that no significant difference exists between the two sets of results. It should be borne in mind that factors which have only a second-order effect at lower strength levels may become of major importance at higher levels.

Section 2. Extension of CP65 for High Strength Concrete

2.1 Overview of Changes to CP65

The technology cost of producing high strength concrete with strengths in excess of grade 60 is commercially viable but current codes of practice CP65/BS8110 is only applicable to concretes up to grade 60. There is a need to address this need before Singapore adopts the Eurocode 2. In the interim, recommendations and provisions in CP65/BS8110 need to be reviewed so as to extend the codes use for high strength concrete. In this regard, High Strength Concrete (HSC) is defined as concrete with a specified characteristic cube strength in excess of 60 N/mm². The design recommendations herein are restricted to an upper limit to the characteristic cube strength of 105 N/mm². Unless stated otherwise, concrete strengths are quoted as cube strengths. Design methodology, including the design parameters and equations in the current standards are based mainly on previous experience, tests and surveys of structures or structural elements where the concrete strength rarely exceed 40 N/mm² and thus may not be fully relevant for applications using higher strength concrete.

The design methodology based on the idealised short-term (uniaxial) stress-strain diagram, accepted in the Codes for normal strength concrete, can generally be safely applied to higher strength concrete. However, the stress-strain diagrams, may need minor modifications to allow for the lower ultimate strain of HSC. With a reduction in ultimate strain comes a reduction in ductility. One way to enhance ductile behaviour is through the provision of longitudinal and transverse reinforcement. Even if only nominal amount is needed, some should be provided in the compression zone.

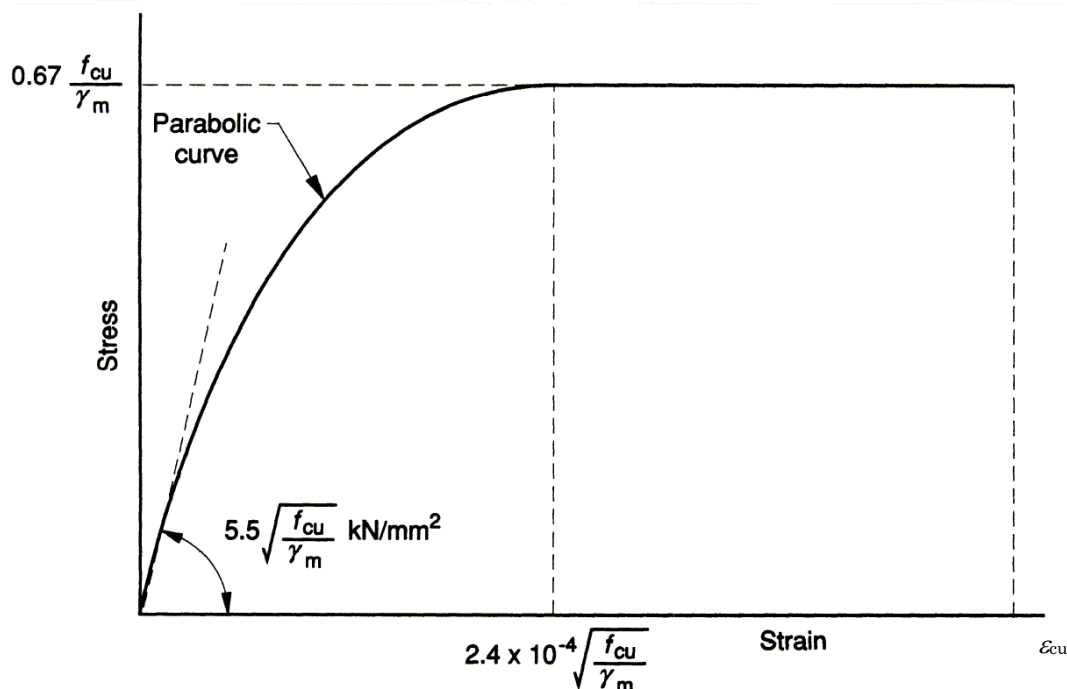
In beams, the requirements for minimum shear reinforcement should in most cases result in providing an adequate amount of confinement in the compression zone. For slabs, the amount of reinforcement in the compression zone should not be less than the minimum amount of reinforcement required in the tension zone. High strength precast floor units, such as hollow-core units, should not usually be used without a structural concrete topping having the required minimum amount of reinforcement, unless sufficient top steel is provided in the unit itself.

2.2 Proposed Changes to CP65 : Part 1 : 1999

The proposed changes to CP65 : Part 1 : 1999 taking into account High-Strength Concrete are as follows:

Clause 2.5.3 Analysis of sections for the ultimate limit state.

Figure 2.1 of CP65 is to be modified as shown below taking into account the behaviour of high-strength concrete.



For $f_{cu} < 60 \text{ N/mm}^2$, $\epsilon_{cu} = 0.0035$

For $f_{cu} > 60 \text{ N/mm}^2$, $\epsilon_{cu} = 0.0035 - (f_{cu} - 60)/50000$

NOTE 1 - 0.67 takes account of the relation between the cube strength and the bending strength in a flexural member. It is simply a coefficient and not a partial safety factor.

NOTE 2 - f_{cu} is in N/mm^2 .

Figure 2.1 (amended) - Short term design stress-strain curve for normal-weight concrete

Clause 3.4.4.2 Design charts.

The design charts in BS 8110-3 should not be used for the design of beams with a grade of concrete higher than C60.

Clause 3.4.4.4 design formulae for rectangular beams.

Figure 3.3 of CP65 is to be modified as shown below taking into account the behaviour of high-strength concrete. The following equations, which are based on the simplified stress block of Figure 3.3, are also applicable to flanged beams where the neutral axis lies within the flange:

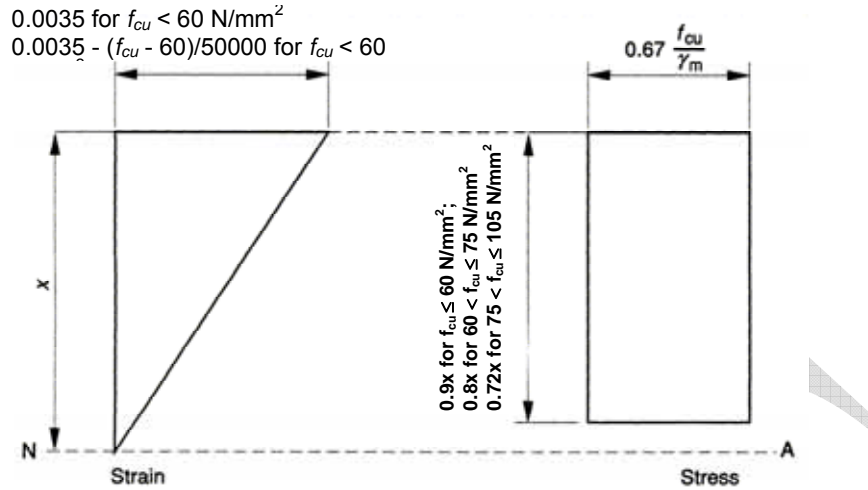


Figure 3.3 (amended) – Simplified stress block for concrete at ultimate limit state

Clause 3.4.4.4 to be replaced by the following:

In order to ensure large strains are developed in the tensile reinforcement, the depth of the neutral axis from the compression face should be limited.

Where redistribution of moments is not carried out or does not exceed 10%, the depth to the neutral axis, x , should be limited as follows:

$$x \leq 0.5d \text{ for } f_{cu} \leq 60 \text{ N/mm}^2;$$

$$x \leq 0.4d \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2; \text{ or}$$

$$x \leq 0.33d, \text{ for } 75 < f_{cu} \leq 105 \text{ N/mm}^2 \text{ and no moment redistribution.}$$

Where redistribution of moments exceeds 10%, the depth to the neutral axis, x , should be limited as follows:

$$x \leq (\beta_b - 0.4)d \text{ for } f_{cu} \leq 60 \text{ N/mm}^2; \text{ or}$$

$$x \leq (\beta_b - 0.5)d \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2;$$

where:

$$\beta_b = \frac{\text{moment at the section after redistribution}}{\text{moment at the section before redistribution}}$$

The following equations, which are based on the simplified stress block of amended Figure 2.1, are also applicable to flanged beams where the neutral axis lies within the flange.

$$K = M/bd^2f_{cu}$$

Where redistribution of moments is not carried out or does not exceed 10%:

$$K = 0.156 \text{ for } f_{cu} \leq 60 \text{ N/mm}^2; \text{ or}$$

$$0.120 \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2; \text{ or}$$

0.094 for $75 < f_{cu} \leq 105 \text{ N/mm}^2$ and no moment redistribution

Where redistribution exceeds 10%,

$$K' = 0.402(\beta_b - 0.4) - 0.18(\beta_b - 0.4)^2, \text{ for } f_{cu} \leq 60 \text{ N/mm}^2; \text{ or}$$

$$0.357(\beta_b - 0.5) - 0.143(\beta_b - 0.5)^2 \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2.$$

If $K \leq K'$, compression reinforcement is not required and:

$$z = d \left(0.5 + \sqrt{0.25 - \frac{K}{0.9}} \right)$$

but not greater than $0.95d$.

$$x = (d - z)/0.45, \quad \text{for } f_{cu} \leq 60 \text{ N/mm}^2; \text{ or}$$

$$(d - z)/0.40, \quad \text{for } 60 < f_{cu} \leq 75 \text{ N/mm}^2; \text{ or}$$

$$(d - z)/0.36, \quad \text{for } 75 < f_{cu} \leq 105 \text{ N/mm}^2.$$

$$A_s = M / 0.87 f_y z$$

If $K > K'$, compression reinforcement is required and:

$$z = d \left(0.5 + \sqrt{0.25 - \frac{K'}{0.9}} \right)$$

$$x = (d - z)/0.45, \quad \text{for } f_{cu} \leq 60 \text{ N/mm}^2; \text{ or}$$

$$(d - z)/0.40, \quad \text{for } 60 < f_{cu} \leq 75 \text{ N/mm}^2; \text{ or}$$

$$(d - z)/0.36, \quad \text{for } 75 < f_{cu} \leq 105 \text{ N/mm}^2$$

$$A_s' = \frac{(K - K') f_{cu} b_c d^2}{0.87 f_y (d - d')}$$

$$A_s = \frac{K' f_{cu} b_c d^2}{0.87 f_y z} + A_s'$$

If d'/x exceeds 0.43 (for $f_y = 460 \text{ N/mm}^2$), the compression stress will be less than $0.87 f_y$ and should be obtained from figure 2.2.

Clause 3.4.4.5 Design ultimate moments of resistance (flanged beams where the neutral axis falls below the flange).

Provided that the design ultimate moment is less than $\beta_f f_{cu} b d^2$ and that not more than 10% of redistribution has been carried out, the required area of tension steel may be calculated using the following equation:

$$A_s = \frac{M + k_1 f_{cu} b_w d (k_2 d - h_f)}{0.87 f_y (d - 0.5 h_f)} \quad \text{Amended equation 1}$$

where $k_1 = 0.1$ for $f_{cu} \leq 60 \text{ N/mm}^2$,

0.072 for $60 < f_{cu} \leq 75 \text{ N/mm}^2$ and

0.054 for $75 < f_{cu} \leq 105 \text{ N/mm}^2$; and

$k_2 = 0.45$ for $f_{cu} \leq 60 \text{ N/mm}^2$,

0.32 for $60 < f_{cu} \leq 75 \text{ N/mm}^2$ and

0.24 for $75 < f_{cu} \leq 105 \text{ N/mm}^2$

The amended equation is only applicable when:

$h_f < 0.45d$ for $f_{cu} \leq 60 \text{ N/mm}^2$; or

$h_f < 0.36d$ for $60 < f_{cu} \leq 75 \text{ N/mm}^2$; or

$h_f < 0.30d$ for $75 < f_{cu} \leq 105 \text{ N/mm}^2$ and no moment redistribution.

If the design ultimate moment exceeds $\beta_f f_{cu} b d^2$ or more than 10% redistribution has been carried out, the section may be designed by direct application of the assumptions given in clause 3.4.4.1. β_f in this expression is a value calculated from the following equation:

$$\beta_f = 0.45 \frac{h_f}{d} \left(1 - \frac{b_w}{b} \right) \left(1 - \frac{h_f}{2d} \right) + K' \frac{b_w}{b} \quad \text{Amended equation 2}$$

Clause 3.4.5.2 Shear stress in beams.

The last paragraph of Clause 3.4.5.2 to be replaced by the following.

“In no case should v exceed $0.8\sqrt{f_{cu}}$ or 7.0 N/mm^2 , whichever is the lesser, whatever shear reinforcement is provided (this limit includes an allowance for γ_m of 1.25).”

Clause 3.4.5.3 Shear reinforcement: form, area and stress.

Table 3.8 Form and area of shear reinforcement in beams to be modified:

- i) Maximum shear stress to be modified.
- ii) Note 2 to be modified taking into account high-strength concrete.

Table 3.8 (amended) Form and area of shear reinforcement in beams

| Value of v (N/mm ²) | Form of shear reinforcement to be provided | Area of shear reinforcement to be provided |
|--|---|--|
| $v < 0.5v_c$ throughout the beam | See note 1 | See note 1 |
| $0.5v_c < v < (v_c + v_r)$ (See note 2) | Minimum links for whole length of beam | $A_{sv} \geq v_r b_v s_v / 0.87f_{yv}$ (See note 2) |
| $(v_c + v_r) < v < 0.8\sqrt{f_{cu}}$, or 7.0 N/mm ² | Links or links combined with bent-up bars. Not more than 50% of the shear resistance provided by the steel may be in the form of bent-up bars (See note 3) | Where links only provided: $A_{sv} \geq b_v s_v (v - v_c) / 0.87f_{yv}$ Where links and bent-up bars provided: see clause 3.4.5.6 |
| Notes: 1. While minimum links should be provided in all beams of structural importance, it will be satisfactory to omit them in members of minor structural importance such as lintels or where the maximum design shear stress is less than half v_c 2. Minimum links provide a design shear resistance of v_r where $v_r = 0.4$ for $f_{cu} \leq 40$ N/mm ² or $0.4(f_{cu}/40)^{2/3}$ for $f_{cu} > 40$ N/mm ² but with the value of f_{cu} not to be taken as greater than 80 N/mm ² 3. See clause 3.4.5.5 for guidance on spacing of links and bent-up bars. | | |

Clause 3.4.5.4 Concrete shear stresses.

The last sentence in the table is to be modified to limit the value of f_{cu} to not greater than 80 N/mm².

Clause 3.4.5.8 Enhanced shear strength of sections close to supports.

Second paragraph, “the lesser of $0.8\sqrt{f_{cu}}$, or 5 N/mm²” be replaced by “the lesser of $0.8\sqrt{f_{cu}}$, or 7.0 N/mm²”.

Clause 3.4.5.12 Shear and axial compression.

Second last line is to be replaced by the following

“ v should not exceed $0.8\sqrt{f_{cu}}$ or 7.0 N/mm², whichever is the lesser”.

Clause 3.5.5.2 Shear stresses.

The last paragraph is to be replaced by the following.

“In no case should v exceed $0.8\sqrt{f_{cu}}$ or 7.0 N/mm², whichever is the lesser, whatever shear reinforcement is provided.”

Clause 3.5.5.3 Shear reinforcement.

Table 3.17 Form and area of shear reinforcement in solid slabs is to use v_r instead of a constant value of 0.4

Amended Table 3.17 - Form and area of shear reinforcement in solid slabs

| Value of v (N/mm ²) | Form of shear reinforcement to be provided | Area of shear reinforcement to be provided |
|--|--|---|
| $v < v_c$ | None required | None |
| $v_c < v < v_c + v_r$ | Minimum links in areas where $v > v_c$ | $A_{sv} \geq v_r b_s v / 0.87 f_{yv}$ |
| $(v_c + v_r) < v < 0.8\sqrt{f_{cu}}$ or 7.0 N/mm ² | Links and/or bent-up bars in any combination (but the spacing between links or bent- up bars need not be less than d) | Where links only provided: $A_{sv} \geq b_s v (v - v_c) / 0.87 f_{yv}$ Where bent-up bars only provided: $A_{sb} \geq b_s (v - v_c) /$ $\{0.87 f_{yv} (\cos \alpha + \sin \alpha \cot \beta)\}$ (see clause 3.4.5.7) |
| Notes: 1. $v_r = 0.4$ for $f_{cu} \leq 40$ N/mm ² or $0.4(f_{cu}/40)^{2/3}$ for $f_{cu} > 40$ N/mm ² with the value of f_{cu} not to be taken as greater than 80 N/mm ² 2. It is difficult to bend and fix shear reinforcement so that its effectiveness can be assured in slabs less than 200 mm deep. It is therefore not advisable to use shear reinforcement in such slabs. 3. The enhancement in design shear strength close to supports described in clauses 3.4.5.8 and 3.4.5.9 may also be applied to solid slabs. | | |

Clause 3.6.4.6 Maximum design shear stress.

“In no case should v exceed $0.8\sqrt{f_{cu}}$ or 7.0 N/mm², whichever is the lesser, whatever shear reinforcement is provided (this limit includes an allowance for γ_m of 1.25).”

Clause 3.7.6.4 Maximum design shear stress at the column face.

The maximum shear stress is to be modified.

The words “ $0.8\sqrt{f_{cu}}$, or 5 N/mm²” to be replaced by “ $0.8\sqrt{f_{cu}}$, or 7.0 N/mm²”.

Clause 3.7.7.2 Maximum design shear capacity.

The maximum shear stress is to be modified:

The words “ $0.8\sqrt{f_{cu}}$, or 5 N/mm²” is to be replaced by “ $0.8\sqrt{f_{cu}}$, or 7.0 N/mm²”.

Clause 3.11.4.5 Punching shear.

The maximum shear stress to be modified.

The words “ $0.8\sqrt{f_{cu}}$, or 5 N/mm²” be replaced by “ $0.8\sqrt{f_{cu}}$, or 7.0 N/mm²”.

Clause 3.12.5.3 Minimum percentage of reinforcement.

A note is to be added to Table 3.27: “For f_{cu} greater than 40 N/mm², the minimum percentage shown shall be multiplied by a factor of $(f_{cu}/40)^{2/3}$.”

Clause 3.12.7.4 Minimum percentage of reinforcement.

The minimum horizontal reinforcement is to be increased for $f_{cu} > 60$ N/mm², such that:

$$0.25 \leq 100 A_s / A_c \leq [0.25 \times (f_{cu} / 40)^{2/3}]$$

Clause 3.12.8.4 Values for design ultimate anchorage bond stress.

The definition “ f_{bu} is the design ultimate anchorage bond stress” is to be replaced by “ f_{bu} is the design ultimate anchorage bond stress with f_{cu} limited to 60 N/mm²”

Clause 4.3.8.2 Maximum design shear stress.

The maximum shear stress not exceeding 5 N/mm² is to be modified:

The words, “ $0.8 \sqrt{f_{cu}}$, or 5 N/mm²” be replaced by “ $0.8 \sqrt{f_{cu}}$, or 7.0 N/mm²”.

Clause 4.3.8.7 Shear reinforcement where V does not exceed $V_c + 0.4 b_v d$.

Reference Clause 3.5.5.3, the 0.4 value is to be replaced by v_r .

In equation 56, use v_r instead of a constant value of 0.4

v_r is defined in Note 1 in Amended Table 3.17.

Clause 4.3.8.8 Shear reinforcement where V exceeds $V_c + 0.4 b_v d$.

“ $v_c + 0.4 b_v d$.” to be replaced by “ $v_c + v_r b_v d$.”

Section six. Concrete: materials, specification and construction.

To refer to SS289 or BS EN 206-1 and BS 8500 instead of BS 5328.

2.3 Proposed Changes to CP65 : Part 2 1996 (1999)

Section two. Clauses 2.4.5 and 2.4.6.

Recommendations for the combinations of shear and torsion to take into account of high-strength concrete, Table 2.3 to be modified.

Amended Table 2.3:

| Concrete grade | v_t min (N/mm ²) | v_{tu} (N/mm ²) |
|---|--------------------------------|-------------------------------|
| 25 | 0.33 | 4.00 |
| 30 | 0.37 | 4.38 |
| 40 | 0.40 | 5.00 |
| 50 | 0.47 | 5.65 |
| 60 | 0.52 | 6.20 |
| 80 or above | 0.60 | 7.0 |
| Notes: 1. Allowance is made for γ_m . 2. $v_{t \text{ min}}$ is the minimum torsional shear stress, above which reinforcement is required. 3. Values of $v_{t \text{ min}}$ and v_{tu} (in N/mm ²) are derived from the equations: $v_{t \text{ min}} = 0.067 \sqrt{f_{cu}} \text{ but not more than } 0.6 \text{ N/mm}^2;$ $v_{tu} = 0.8 \sqrt{f_{cu}} \text{ but not more than } 7.0 \text{ N/mm}^2$ | | |

Section four. Fire resistance.

In a completed structure where the concrete elements are sufficiently dry such that moisture content is less than 3% by volume, spalling is less likely to occur and the fire resistance of members with high strength concrete should then be as good as, if not better than that of low to medium made concrete, other conditions remaining the same. In normal strength range concrete, it usually takes 9 to 12 months to attain the level of moisture content mentioned above. When the moisture content is higher than 3% by volume, HSC members are more susceptible to spalling, possibly explosive spalling, in fire. HSC is less permeable than normal concrete, so vapour and pore pressure due to heat near the surface builds up quickly, and spalling (local breakdown and removal of surface material) occurs as soon as the pressure exceeds the tensile capacity of the concrete.

It has been found that the risk of spalling can be reduced by the addition of polypropylene or steel fibres to the concrete. Polypropylene fibres have been found to be particularly effective, although the mechanism by which they prevent spalling is not well understood. Under fire conditions the fibres will melt, but it has been suggested that they form voids or micro-cracks in the concrete which effectively relieve the pressure from the expanding moisture. Steel fibres provide some reinforcement to the outer skin of the concrete, tying it into the main body of the member and again limiting spalling. In addition, the behaviour of high strength concrete in fire is particularly influenced by the choice of aggregate.

“For concrete compressive strength greater than 60 MPa, the possible reduction in strength at elevated temperatures and the associated risk of spalling should be investigated, taking into account the relevant factors including moisture content, type of aggregate, permeability of concrete, possible heating rate and the silica fume content.

Specialist literature and testing should be referenced for the fire resisting design of high strength concrete structures.”

Section 3. Guide to ensure product conformity and quality for high strength concrete

3.1 Introduction

The methods and technology for producing high strength concrete (HSC) are not basically different from those required for concrete of low to medium grade except that the emphasis on quality control is perhaps greater with HSC. HSC can be produced with all of the cements and cement replacements normally available in Singapore, including ordinary Portland cement, sulfate-resisting Portland cement, and combinations with pulverised fuel ash and ground granulated blast furnace slag. When the specified strength substantially exceeds that produced previously in a particular market area, special measures are necessary to make a successful progression to the use of the higher-strength concrete. The measures recommended in this guide should be applied for concrete with compressive strength greater than about 60 MPa.

The quality of high-strength concrete is controlled by the quality and uniformity of the ingredients, and by the mixing, placing, and curing conditions. A high level of quality control is essential for those involved in the production, testing, transportation, placing, and curing of the concrete. Careful consideration of placing restrictions, workability, difficulties during transportation, field curing requirements, and the inspection and testing process is required. Thorough planning and teamwork by the inspector, contractor, architect/engineer, producer, and owner are essential for the successful use of high-strength concrete. A preconstruction meeting is essential to clarify the roles of the members of the construction team and review the planned quality control and testing program. Special attention is required during the trial-batch phase to assure that selected mixtures will perform as required under field conditions. Planning for inspection and testing of high strength concrete involves giving attention to personnel requirements, equipment needs, test methods, and the preparation and handling of test specimens.

3.2 Preconstruction meeting

Project participants should meet before construction to clarify contract requirements, discuss planned placing conditions and procedures, and review the planned inspection and testing programs of the various parties. The effects on the concrete of time, temperature, placing, consolidation, and curing should be reviewed. Acceptance criteria for standard cured test specimens, in-place tests, and core test results should be established. The capabilities and qualifications of the contractor's work force, the inspection staff, and the testing and batching facilities should also be reviewed. The preconstruction meeting should establish lines of communication and identify responsibilities. It is especially important to review the procedures the inspector will follow when non-compliance with contract requirements is found or suspected. Such advance understanding minimizes future disputes, and allows members of the construction team to participate in the quality process. Timely and accurate reporting are important. Arrangements should be made to distribute inspection reports and test data as soon as possible. Trial production batches should have established a workable mixture, but it may be necessary to make adjustments due to site conditions, such as changing weather. Since high strength concrete relies on a low water-cementitious materials ratio for strength potential, responsibility for field addition of water and admixtures should be discussed and defined clearly. Participation of the ready-mixed concrete producer is essential in the discussion since the producer is familiar with and responsible for the product. Individuals should be identified, such as the concrete supplier's quality control personnel, who will have the authority to add admixtures or water at the site. To permit verification that the concrete provided conforms to established requirements, procedures should be established for documenting what, when, and how much was added to the concrete at the site.

3.3 Trial batches

Where historical data are not available, the development of an optimum high-strength concrete mixture requires trial batches. Materials and proportions initially should be evaluated in the laboratory to determine the appropriate material proportions and their relative characteristics. Sufficient lead time should be allowed, since high-strength mixtures containing fly ash, silica fume, or ground granulated blast furnace slag are often evaluated at 56 and 90 days. After the work has been completed in the laboratory, production-sized batches are recommended because laboratory trial batches sometimes exhibit strengths and other properties different from those achieved in actual production. For instance, the efficiency of small laboratory mixers is much less than that of production mixers, which can affect the dispersion and performance of chemical and mineral admixtures. Since high-strength concretes usually contain both chemical and mineral admixtures, including silica fume, and a high volume of cementitious materials, they tend to be more sticky than conventional concrete mixtures. Production trials can be used to establish optimum batching and mixing sequences that can reduce problems prior to the start of the project. Where truck mixing is used, the maximum load that can be mixed adequately should be determined, but practice has shown that this usually is less than 90 percent of the truck's rated mixing capacity. Based on experience, batches of high-strength concrete smaller than 3 m³ (4 yd³) should not be mixed in truck mixers.

3.4 Prequalification of concrete suppliers and preconstruction testing

Bidders should be prequalified prior to the award of a supply contract for concrete. Where the specified strength has been widely produced for previous projects, a review of available test data may adequately measure performance. When a strength higher than previously supplied is specified, or where there is limited experience in the supply of that strength concrete, a more detailed prequalification procedure should be carried out. This should generally include the production of a trial batch of the proposed mixture proportions. The trial concrete should be cast into monoliths representative of typical structural sizes on the project. Fresh concrete should be tested for slump, air content, and temperature. Hardened concrete should be tested to determine compressive strength and modulus of elasticity based on standard-cured cylinders and on cores drilled from the monolith. Strengths of cores and standard-cured cylinders tested at the same age should be correlated. In massive elements, core strength may vary with distance from the surface due to different temperature histories. Therefore, relationships should be established for a specific core depth. If cores need to be removed during construction, the correlation allows interpretation of core strength results. The monolith also should be instrumented to determine the maximum internal temperature and the temperature gradients developed throughout the cross section.

Qualified suppliers can be selected based on their successful preconstruction trials. After the start of construction, further trials are desirable to confirm the field performance of the submitted and accepted mixtures. Further testing may also be required on full-scale mock-ups of structural subassemblies to determine the potential for cracking problems, such as at the interface between structural elements of different thickness.

Laboratory and field tests should be performed to evaluate the effects of environmental conditions on the properties of freshly-mixed and hardened concrete. In particular, slump loss between the batch plant and the project site should be evaluated to assure adequate slump at the time of placing. During periods of high temperature or low humidity, it may be necessary to adjust the concrete mixture using retarding or high-range water-reducing admixtures in varied dosage rates and addition sequences.

In-place strength - If in-place testing is to be used, it is recommended that a correlation with standard-cured cylinders be made at the prequalification trials.

Temperature considerations - Each high-strength concrete mixture has unique heat evolution and heat dissipation characteristics for a particular curing environment. Maximum temperatures and

thermal gradients, and their effects on constructability and long-term design properties, should be determined during preconstruction trials. Computer simulation of the likely thermal history can be used to establish appropriate curing and protection. In addition, temperature-matched curing systems may be used to evaluate the effects of temperature history on strength development.

The Qualified Person should understand the effects of heat generation in the various structural elements and address these in the project specifications. Specifications for mass concrete often place limit the temperature difference between the concrete interior and surface.

3.5 Quality Assurance and Quality Control

Quality assurance (QA) - actions taken by the Qualified Person to provide assurance that what is being done and what is being provided are in accordance with the applicable standards of good practice for the work.

Quality control (QC) - actions taken by a producer or contractor to provide control over what is being done and what is being provided so that the applicable standards of good practice for the work are followed.

Comprehensive and timely QA/QC permit confidence in the use of advanced design procedures, frequently expedite construction, and improve quality in the finished product. Conversely, the results of poor QA/QC can be costly for all parties involved. QA/QC personnel must be experienced with their respective duties, including the batching, placing, curing, and testing of high-strength concrete. QA/QC personnel should be able to provide evidence of such training or experience, or both.

QA/QC personnel should concentrate their efforts at the concrete plant to ensure consistently acceptable batching is achieved. For example, QA/QC personnel should ensure that the facilities, moisture meters, scales, and mixers (central or truck, or both) meet the project specification requirements and that materials and procedures are as established in the planning stages. QA/QC personnel should be aware of the importance of batching high-strength concrete, such as using proper sequencing of ingredients, especially when pozzolans or ground slag are used. Scales, flow meters, and dispensers should be checked monthly for accuracy, and should be calibrated every six months. Moisture meters should be checked daily. These checks and calibrations should be documented. Plants that produce high-strength concrete should have printed records for all materials batched. Entries showing deviations from accepted mixture proportions are provided with some plant systems.

The QC or QA inspector should be present at the batching console during batching and should verify that the accepted types and amounts of materials are batched. Batch weights should fall within the allowable tolerances set forth by project specifications. Moisture content tests should be repeated after rain and the other tests should be repeated after deliveries of new batches of materials. High-strength concrete may rely on a combination of chemical and mineral admixtures to enhance strength development. Certain combinations of admixtures and portland cements exhibit different strength development curves. Therefore, it is important for QA/QC personnel to watch for deviations in the type or brands of mixture ingredients from those submitted and accepted. Substitutions should not be allowed without the prior understanding of all parties. Reference samples of cementitious materials should be taken at least once per day or per shipment in case tests are needed later to investigate low strengths or other deficiencies. Sources of additional mixture water such as “wash water” or any “left-over” concrete remaining in the truck drum prior to batching should be identified. These should be emptied from the truck prior to batching.

The QA/QC personnel should recognize that prolonged mixing will cause slump loss and result in lower workability. Adequate job control must be established to prevent delays. When practical, withholding some of the high-range water-reducing admixture until the truck arrives at the job

site or site-addition of high-range water-reducing admixtures may be desirable. Newer high-range water-reducing admixtures with extended slump retention characteristics may preclude the need for job-site additions of admixture to recover slump. Truck mixers should rotate at agitation speed while waiting for discharge at the site. Failure to do so may lead to severe slump loss.

When materials are added at the site, proper mixing is required to avoid non-uniformity and segregation. QA/QC personnel should pay close attention to site mixing and should verify that the mixture is uniform.

The concrete truck driver should provide a delivery ticket and every ticket should be reviewed by the inspector prior to discharge of concrete.

Chemical admixtures can be used to increase workability time. High-range water-reducing admixtures often are used to increase the fluidity of concrete for a definite time period. QA/ QC personnel should be aware of that time frame and should know whether redosing with additional admixture is permitted. If the batch is redosed, the amount of admixture added to the truck should be recorded. Addition of water at the job site should be permitted only if this was agreed to at the preconstruction meeting and provided that the maximum specified water-cementitious materials ratio is not exceeded.

QA/QC personnel should verify that forms, reinforcing steel, and embedded items are ready and that the placing equipment and vibration equipment (including standby equipment) are in working order prior to the contractor placing concrete. All concrete should be consolidated quickly and thoroughly.

The potential strength and durability of high-strength concrete will be fully developed only if it is properly cured for an adequate period prior to being placed in service or being subjected to construction loading. Therefore, appropriate curing methods for various structural elements should be selected in advance. QA/QC personnel should verify that the accepted methods are properly employed in the work.

High-strength concretes usually do not exhibit much bleeding, and without protection from loss of surface moisture, plastic shrinkage cracks have a tendency to form on exposed surfaces. Curing should begin immediately after finishing, and in some cases other protective measures should be used during the finishing process. Curing methods include fog misting, applying an evaporation retarder, covering with polyethylene sheeting, or applying a curing compound.

The inspector should monitor and record ambient temperatures and temperatures at the surface and center of large concrete components so that the design/construction team can effectively make any adjustments, such as changes in mixture proportions or the use of insulating forms, during the course of the project. Concrete delivered at temperatures exceeding specification limits should be rejected, unless alternative procedures have been agreed to at the preconstruction meeting. The inspector should monitor that curing procedures are according to project specifications, particularly those at early ages to control the formation of plastic shrinkage cracks.

3.6 Testing

Measurement of mechanical properties during construction provides the basic information needed to evaluate whether design considerations are met and the concrete is acceptable. Experience indicates that the measured strength of high-strength concrete is more sensitive to testing variables than is normal-strength concrete. Therefore, the quality of these measurements is very important.

Adequate planning, with review of personnel and laboratory qualifications, and strict adherence to standard procedures should help prevent questions about the quality of testing during

construction. The laboratory should be accredited. Field and laboratory testing personnel should be experienced and trained properly.

The Qualified Person can generally take advantage of the fact that high-strength concrete containing fly ash or ground granulated blast-furnace slag develops considerable strength at later ages, such as 56 and 90 days. It is common to specify more test specimens than would normally be required.

Since much of the interest in high-strength structural concrete is limited to compressive strength, these measurements are of primary concern. The primary function of standard laboratory-cured specimens is to provide assurance that the concrete mixture as delivered to the job site has the potential to meet design specification requirements. The potential strength and variability of the concrete can be established only by specimens made, cured, and tested under standard conditions.

Regardless of specimen size, the size used to evaluate trial mixture proportions should be consistent with the size specified for acceptance testing, and should be acceptable to the Qualified Person. If necessary, the relationship between the compressive strengths of the two specimen sizes can be determined at the laboratory or field trial stage using the testing machine that will be used for the project.

The curing of high-strength concrete in a structural element normally varies from the curing of representative standard specimens from samples. If an accurate determination of the strength in a high-strength concrete component is required, temperature-matched cured test specimens should be used instead of standard-cured test specimens.

Testing apparatus —Due to the higher loads carried by high-strength concrete test specimens, compression machine characteristics influence results. Machine characteristics that may affect the measured strength include calibration accuracy, longitudinal and lateral stiffness, alignment of the machine components, type of platens, and the behavior of the platen spherical seating.

3.7 Prequalification of testing laboratories

A laboratory should be examined from two perspectives: how it has performed in the past and how well it is equipped to perform properly in the future. The qualifications and experience of technicians and inspectors should be reviewed. The laboratory should be accredited.

3.8 Strength evaluation

High-strength concretes may continue to gain significant strengths after the acceptance test age, especially if fly ash or ground granulated blast-furnace slag are used. During the evaluations to establish mixture proportions, a strength development curve should be established indicating potential strength over time. However, if questions arise concerning the load-carrying capacity of a structure may be investigated by analysis using core test results or by load testing. In cases where load testing a structure is not practical, analytical investigations using the strength results from extracted cores, or in-place tests are more appropriate. Tests to evaluate the durability of the concrete should be performed separately on cores other than those used for strength tests.

As mentioned earlier, a correlation curve should be established for each high-strength mixture to relate the strength of extracted cores to the strength of specimens used for acceptance testing. Then, if coring becomes necessary, the relationship has been established, agreed upon, and is ready for conclusive interpretation.

3.9 Acknowledgement

This guide has been adapted mostly from ACI 363.2R-98 Guide to Quality Control and Testing of High-Strength Concrete.

Relevant References

American Concrete Institute

- 116R Cement and Concrete Terminology
- 201.2R Guide to Durable Concrete
- 207.2R Effect of Restraint, Volume Change, and Reinforcement on Cracking of Massive Concrete
- 211.1 Standard Practice for Selecting Proportions for Normal, Heavyweight and Mass Concrete
- 211.4R Guide for Selecting Proportions for High-Strength Concrete with Portland Cement and Fly Ash
- 212.3R Chemical Admixtures for Concrete
- 214 Recommended Practice for Evaluation of Strength Test Results of Concrete
- 228.1R In-Place Methods to Estimate Concrete Strength
- 304R Guide for Measuring, Mixing, Transporting and Placing Concrete
- 308 Standard Practice for Curing Concrete
- 309R Guide for Consolidation of Concrete
- 311.4R Guide for Concrete Inspection
- 318 Building Code Requirements for Structural Concrete
- 363R State-of-the-Art Report on High-Strength Concrete

American Society for Testing and Materials

- C 31 Practice for Making and Curing Concrete Test Specimens in the Field
- C 33 Specification for Concrete Aggregates
- C 39 Test Method for Compressive Strength of Cylindrical Concrete Specimens
- C 94 Specification for Ready-Mixed Concrete
- C 117 Test Method for Materials Finer than 75- μ m (No. 200) Sieve in Mineral Aggregates by Washing
- C 136 Test Method for Sieve Analysis of Fine and Coarse Aggregates
- C 172 Practice for Sampling Freshly Mixed Concrete

| | |
|--------|---|
| C 192 | Practice for Making and Curing Concrete Test Specimens in the Laboratory |
| C 470 | Specification for Molds for Forming Concrete Test Cylinders Vertically |
| C 566 | Test Method for Total Moisture Content Aggregates by Drying |
| C 617 | Practice for Capping Cylindrical Concrete Specimens |
| C 684 | Test Method for Making, Accelerated Curing, and Testing Concrete Compression Test Specimens |
| C 1077 | Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation |
| C 1231 | Practice for Use of Unbonded Caps in Determination of Compressive Strength of Hardened Concrete Cylinders |

British Standards Institution

| | |
|-------------|---|
| BS 1881 | Testing concrete |
| BS 8500 | Concrete. Complementary British Standard to BS EN 206-1 |
| BS EN 197 | Cement |
| BS EN 206-1 | Concrete. Specification, performance, production and conformity |
| BS EN 450 | Fly ash for concrete. |
| BS EN 934 | Admixtures for concrete, mortar and grout. |
| BS EN 1008 | Mixing water for concrete |
| BS EN 12350 | Testing fresh concrete |
| BS EN 12390 | Testing hardened concrete |
| BS EN 12504 | Testing concrete in structures |
| BS EN 12620 | Aggregates for concrete |
| BS EN 13263 | Silica fume for concrete. |
| BS EN 13791 | Assessment of in-situ compressive strength in structures and pre-cast concrete components |
| BS EN 15167 | Ground granulated blast furnace slag for use in concrete, mortar and grout |

Section 4. Guideline on the submission to BCA for the use of High Strength Concrete

The following information are to be submitted in the form of a report:

4.1.1 General Information

Project name, Developer, Consultant, Contractor, Concrete Supplier and Specialist

Information on past experience of consultant, contractor and concrete supplier including project data relating to the use of HSC

4.1.2 Information on the proposed HSC

Locations to be used and quantity etc

Details on mix design, characteristic strength and its test age (if it is not 28 days), workability, method of curing

Standards and/or Code of Practice to be used

4.1.3 Preconstruction Trial

Number and size of trial mix

Any production-sized trials? If yes, provide details

Any full-scale trial casting of typical structural sub-assemblages? If yes, provide details

Details on plan to measure development of strength and temperature

4.1.4 Quality Assurance and Quality Control

Organisational chart, qualification and experience of personnel

Quality Assurance Plan e.g. inspection, testing etc.

Quality Control Plan in mixing, delivery, placing, curing

4.1.5 Testing

Types of test other than for strength e.g. modulus of elasticity, modulus of rupture, shrinkage, creep,

Details of test programme e.g. frequency, number, specimen size etc.

Acceptance criteria for standard cured test specimens, in-place tests, and core test results. (correlation for purpose of interpretation may be useful)

Test laboratory : Name, Accredited Lab(?), Past experience, Information on equipment and facilities for handling, testing etc of HSC

4.1.6 Proposed solution in case of non-compliance

DRAFT

Section 5. Design Guideline for the use of High Strength Concrete

Summary of changes to CP65 for HSC

5.1 Cover for durability

From the point of view of durability, though it should be possible to reduce the covers to the steel in HSC from those which are specified in Table 3.4 of CP65 for relatively low strength concretes, it is recommended that the covers appropriate to C50 concrete should be used for higher grades.

5.2 Cover as fire protection

No change to the current cover requirements is recommended. However, for concrete compressive strength greater than 60 MPa, the possible reduction of strength at elevated temperatures and the associated risk of spalling should be investigated, taking into account the relevant factors including moisture content, type of aggregate, permeability of concrete, possible heating rate and the silica fume content. Specialist literature and testing should be referenced for the fire resistance design of high strength concrete structures.

5.3 Flexural design

The principles of analysis in CP65 can be applied in design using high strength concrete. Design methodology based on the idealised short-term stress-strain (uniaxial) diagram in CP65 also applies. The principal significant difference between normal and high strength concrete is that ductility decreases as concrete strength increase. Hence, the ultimate strain in compression has to be reduced as strength increases and the compressive stress modified. In CP 65 Part 1, the ultimate concrete strain for flexural design is taken as a constant value of 0.0035. For HSC, the maximum ultimate strain limit of the diagram is modified by the following equations:

For $f_{cu} < 60 \text{ N/mm}^2$ CP65 stress block is applicable

For $f_{cu} > 60 \text{ N/mm}^2$, $\epsilon_{cu} = 0.0035 - (f_{cu} - 60)/50000$

The ultimate strain, ϵ_{cu} , decreases with increase in the grade of concrete and if it is less than the strain at the tangent point then the stress is reduced accordingly. Hence, the design charts in BS 8110-3 should not be used for the design of beams higher than C60.

The simplified design procedure based on a rectangular stress-block may also be acceptable with the same limitations on strain. Therefore, the simplified stress block in Figure 3.3 is also modified. Some longitudinal and transverse reinforcement should be provided in the compression zone.

HSC concrete beams exhibit a lower flexural ductility than those of normal strength concrete for a similar tension steel ratio. In order to maintain a similar level of flexural ductility, the tension steel ratio needs to be lowered as concrete strength increases. To ensure a degree flexural ductility, CP 65 restricts the neutral axis depth to 0.50 of the effective depth 'd' where redistribution is less than 10%. This is to be modified as:

$f_{cu} \leq 60 \text{ N/mm}^2$, $x \leq 0.5d$

$60 < f_{cu} \leq 75 \text{ N/mm}^2$, $x \leq 0.4d$

$$75 < f_{cu} < 105 \text{ N/mm}^2, x \leq 0.33d$$

5.4 Shear

The design approach for shear in CP 65 is unchanged. The maximum shear stress for HSC is increased to 7 N/mm^2 but no increase in design shear strength beyond that for concrete of grade 80 N/mm^2 should be allowed. However, allowing for a higher shear stress for higher strength concrete necessitates the requirement for a greater area of minimum links for concrete strengths greater than 40 N/mm^2 . Minimum links are provided to ensure that when shear cracking occurs, the tension originally carried by the concrete before cracking can be transferred to the shear reinforcement without causing it to yield. In CP 65, minimum links are designed for a shear resistance of 0.40 N/mm^2 but this is based on a maximum concrete strength of 40 N/mm^2 . As the concrete strength increases, the shear force at which cracking occurs also increases and the area of shear reinforcement has to be increased to prevent it from yielding at the time of shear cracking. The required minimum area of reinforcement is proportional to tensile strength of concrete, which is proportional to its compressive strength to the power $2/3$. Therefore, for concrete strength higher than 40 N/mm^2 the minimum shear reinforcement has to be increased by a factor of $(f_{cu}/40)^{2/3}$ up to a limit of 80 MPa .

The affected clauses are

3.4.5.2,
3.4.5.3,
3.4.5.4,
3.4.5.8,
3.4.5.12
3.5.5.2,
3.5.5.3,
3.6.4.6,
3.7.6.4
3.7.7.2,
4.3.8.2,
4.3.8.7,
4.3.8.8,

5.4.1.1 Worked Examples on Shear in Beams

5.4.1.1.1 Normal grade concrete $f_{cu} = 35 \text{ MPa}$;

$$v_r = 0.4 \quad \text{MPa} \quad \text{as}$$

$$f_{cu} \leq 40 \text{ MPa}$$

Worked Example 1

$$V = 80 \text{ kN};$$

$$\text{Section} \quad : \quad b = 400 \quad \text{mm};$$

$$d = 700 - 40 - 16 = 644 \text{ mm};$$

$$\text{Longitudinal steel : 4T32,}$$

$$A_{st} = 3217 \quad \text{mm}^2;$$

$$\frac{100A_{st}}{bd} = 1.25;$$

$$v_c = 0.79 \left(\frac{f_{cu}}{25} \right)^{\frac{1}{3}} \left(\frac{100A_s}{b_v d} \right)^{\frac{1}{3}} \left(\frac{400}{d} \right)^{\frac{1}{4}} \frac{1}{\gamma_m} = 0.79 \left(\frac{35}{25} \right)^{\frac{1}{3}} (1.25)^{\frac{1}{3}} (1)^{\frac{1}{4}} \frac{1}{1.25} = 0.76 \text{ MPa}$$

where $\left(\frac{400}{d} \right)^{\frac{1}{4}}$ is taken as 1 as the beam is of structural importance and at least minimum links is to be provided.

$$v = \frac{80 \times 10^3}{400 \times 644} = 0.31 \text{ MPa} < \frac{1}{2} v_c = \frac{0.76}{2} = 0.38 \text{ MPa}.$$

$$\text{So minimum links of } \frac{A_{sv}}{s_v} \geq \frac{v_r b_v}{0.87 f_{sv}} = \frac{0.4 \times 400}{0.87 \times 460} = 0.40 \text{ mm is required.}$$

$$\text{Use T10 - 300 c/c s.s. } \frac{A_{sv}}{s_v} \text{ provided is } \frac{78.5 \times 2}{300} = 0.52 \text{ mm} > 0.40 \text{ mm, O.K.}$$

Worked Example 2

Same section and longitudinal bars as in Worked Example 1, but $V = 180 \text{ kN}$;

$$v = \frac{180 \times 10^3}{400 \times 644} = 0.70 \text{ MPa}$$

$$\text{As } 0.5v_c = 0.38 < v = 0.7 < (v_c + v_r) = 0.76 + 0.4 = 1.16,$$

provide minimum links of T10 – 300 c/c s.s. as in Worked Example 1.

Worked Example 3

Same section and longitudinal bars as in Worked Example 1, but $V = 750$ kN;

$$v = \frac{750 \times 10^3}{400 \times 644} = 2.91 \text{ MPa.}$$

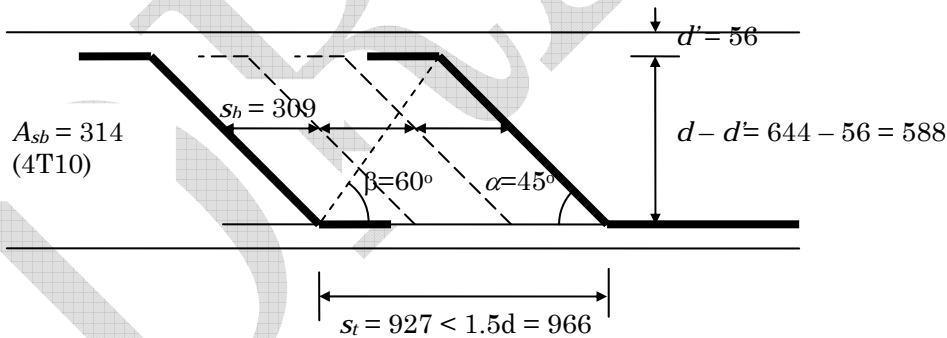
As $(v_c + v_r) = 0.76 + 0.4 = 1.16 < v = 2.91 < 0.8\sqrt{f_{cu}} = 4.73$, shear reinforcement in excess of minimum links is required.

$$\frac{A_{sv}}{s_v} = \frac{b(v - v_c)}{0.87 f_{yv}} = \frac{400(2.91 - 0.76)}{0.87 \times 460} = 2.15 \text{ mm;}$$

Use T12 – 200 c/c d.s. $\frac{A_{sv}}{s_v}$ provided is 2.26 mm, O.K.

The shear taken up by steel is $2.15 \times 400 \times 644 \times 10^{-3} = 553.8$ kN.

Alternatively, if some of the shear resistance taken up by steel is provided by bent-up bars, the arrangement should be in accordance with the following diagram:



The 4T10 inclined bars will provide a shear resistance of

$$A_{sb} (0.87 f_{yv}) (\cos \alpha + \sin \alpha \cot \beta) \frac{d - d'}{s_b}$$

$$= 314 \times 0.87 \times 460 \times (\cos 45^\circ + \sin 45^\circ \cot 60^\circ) \times \frac{588}{309} \times 10^{-3} = 266.7 \text{ kN}$$

< half of shear taken up by steel which is $\frac{553.8}{2} = 276.9$ kN, O.K. as the Code requires at least half of the shear resistance to be taken up by links.

The remaining shear of $553.8 - 266.7 = 287.1$ kN is to be taken up by links where
$$v_{link} = \frac{287.1 \times 10^3}{400 \times 644} = 1.11 \text{ MPa}$$

The links can then be reduced to

$$\frac{A_{sv}}{s_v} = \frac{bv_{link}}{0.87f_{yv}} = \frac{400 \times 1.11}{0.87 \times 460} = 1.11 \text{ mm}$$

Use T12 – 175 c/c s.s. $\frac{A_{sv}}{s_v}$ provided is 1.29 mm, O.K.

Worked Example 4

Same section and longitudinal bars as in Worked Example 1, but $V = 1300$ kN;

$$v = \frac{1300 \times 10^3}{400 \times 644} = 5.05 \text{ MPa.}$$

As $v = 5.05 > 0.8\sqrt{f_{cu}} = 4.73$, section has to be enlarged.

5.4.1.1.2 High Strength concrete $f_{cu} = 75 \text{ MPa}$;

$$v_r = 0.4(f_{cu}/40)^{2/3} = 0.4(75/40)^{2/3} = 0.61 \text{ MPa}$$

Worked Example 5

$V = 80$ kN;

Section : $b = 400$ mm; $d = 700 - 40 - 16 = 644$ mm;

Longitudinal steel : 4T32, $A_{st} = 3217 \text{ mm}^2$; $\frac{100A_{st}}{bd} = 1.25$;

$$v_c = 0.79 \left(\frac{f_{cu}}{25} \right)^{\frac{1}{3}} \left(\frac{100A_s}{b_v d} \right)^{\frac{1}{3}} \left(\frac{400}{d} \right)^{\frac{1}{4}} \frac{1}{\gamma_m} = 0.79 \left(\frac{75}{25} \right)^{1/3} (1.25)^{1/3} (1)^{1/4} \frac{1}{1.25} = 0.98 \text{ MPa}$$

where $\left(\frac{400}{d}\right)^{\frac{1}{4}}$ is taken as 1 as the beam is of structural importance and at least minimum links is to be provided.

$$v = \frac{80 \times 10^3}{400 \times 644} = 0.31 \text{ MPa} < \frac{1}{2} v_c = \frac{0.98}{2} = 0.49 \text{ MPa.}$$

So minimum links of $\frac{A_{sv}}{s_v} \geq \frac{v_r b_v}{0.87 f_{sv}} = \frac{0.61 \times 400}{0.87 \times 460} = 0.61 \text{ mm}$ is required.

Use T10 – 250 c/c s.s. $\frac{A_{sv}}{s_v}$ provided is $\frac{78.5 \times 2}{250} = 0.63 \text{ mm} > 0.61 \text{ mm}$, O.K.

Worked Example 6

Same section and longitudinal bars as in Worked Example 5, but $V = 180 \text{ kN}$;

$$v = \frac{180 \times 10^3}{400 \times 644} = 0.70 \text{ MPa}$$

As $0.5v_c = 0.49 < v = 0.7 < (v_c + v_r) = 0.98 + 0.61 = 1.59$,

provide minimum links of T10 – 250 c/c s.s. as in Worked Example 5.

Worked Example 7

Same section and longitudinal bars as in Worked Example 5, but $V = 750 \text{ kN}$;

$$v = \frac{750 \times 10^3}{400 \times 644} = 2.91 \text{ MPa.}$$

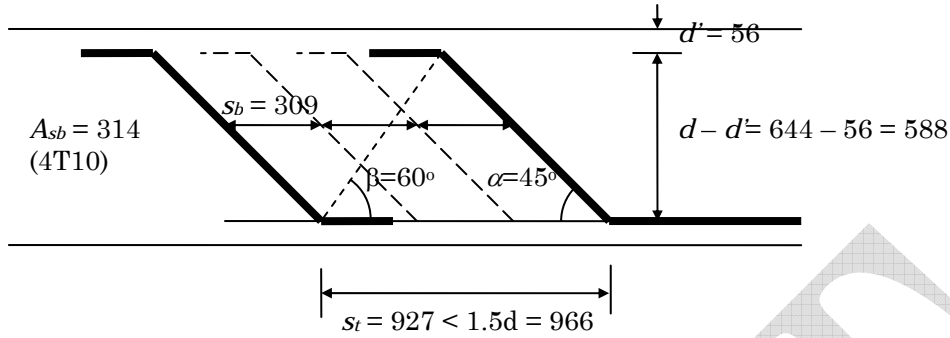
As $(v_c + v_r) = 0.98 + 0.61 = 1.59 < v = 2.91 < 0.8\sqrt{f_{cu}} = 6.45$, shear reinforcement in excess of minimum links is required.

$$\frac{A_{sv}}{s_v} = \frac{b(v - v_c)}{0.87 f_{yv}} = \frac{400(2.91 - 0.98)}{0.87 \times 460} = 1.93 \text{ mm;}$$

Use T12 – 200 c/c d.s. $\frac{A_{sv}}{s_v}$ provided is 2.26 mm, O.K.

The shear taken up by steel is $1.93 \times 400 \times 644 \times 10^{-3} = 497.2 \text{ kN}$.

Alternatively, if some of the shear resistance taken up by steel is provided by bent-up bars, the arrangement should be in accordance with the following diagram :



The 4T10 inclined bars will provide a shear resistance of

$$A_{sb} (0.87 f_{yv}) (\cos \alpha + \sin \alpha \cot \beta) \frac{d - d'}{s_b}$$

$$= 314 \times 0.87 \times 460 (\cos 45^\circ + \sin 45^\circ \cot 60^\circ) \frac{588}{309} \times 10^{-3} = 266.7 \text{ kN}$$

slightly greater than half of shear taken up by steel which is $\frac{497.2}{2} = 248.6 \text{ kN}$. As the Code requires at least half of the shear resistance to be taken up by links, the shear to be taken by links should be at least 248.6 kN.

$$\text{So } v_{link} = \frac{248.6 \times 10^3}{400 \times 644} = 0.97 \text{ MPa}$$

$$\text{The links can then be reduced to } \frac{A_{sv}}{s_v} = \frac{bv_{link}}{0.87 f_{yv}} = \frac{400 \times 0.97}{0.87 \times 460} = 0.97 \text{ mm}$$

Use T12 – 200 c/c s.s. $\frac{A_{sv}}{s_v}$ provided is 1.13 mm, O.K.

Worked Example 8

Same section and longitudinal bars as in Worked Example 5, but $V = 1800$ kN;

$$v = \frac{1800 \times 10^3}{400 \times 644} = 6.98 \text{ MPa.}$$

As $v = 6.98 > 0.8\sqrt{f_{cu}} = 6.92$, section has to be enlarged.

DRAFT

5.5 Deflection

Eurocode 2 makes allowance for reduced creep effects with increase in concrete strength but the basic span/effective depth ratios given in Tables 3.10, 3.10 and 3.11 of CP 65: Part 1 do not take account of the effect of increased concrete strength. The calculation method given in CP 65: Part 2 takes account of the elastic modulus, E_c , and shrinkage and creep coefficients but fixes the value of tension stiffening to 1 N/mm² in the short term and 0.55 N/mm² in the long term.

In order to modify Tables 3.10, 3.11 and 3.12 of CP 65 it is proposed that the E value is calculated in accordance with equation 17 of CP 65: Part 2. The creep coefficient for high strength concrete, ϕ_{hsc} should be adjusted such that:

$$\phi_{hsc} = \sqrt{[40 / (f_{cu} + 10)]} \times \phi$$

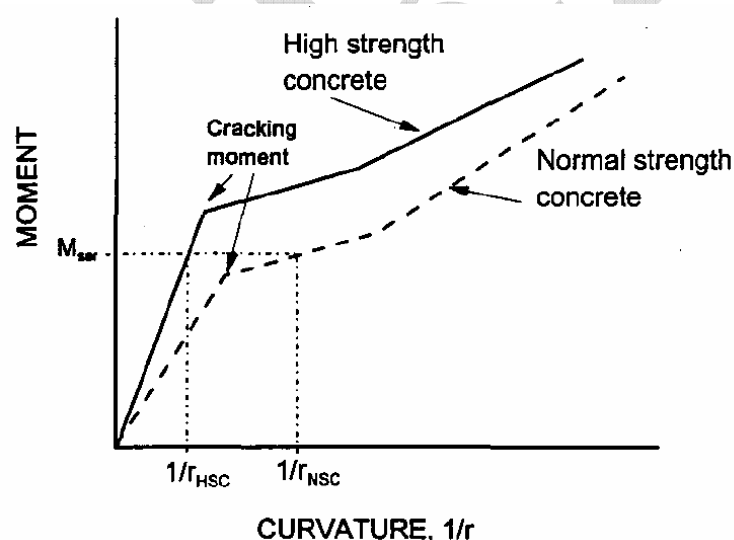
It is assumed that Table 3.10 of CP 65 is based on $f_{cu} = 35$ N/mm² and a creep value of 2. This agrees reasonably well with that of EC2 using the following values; RH 80%, section depth 300 mm, time of applying the load 14 days. These values correspond to the floors of a building exposed to the weather for several months of its initial life.

The expression used to determine the shrinkage coefficient has been adapted from Equation (2.1-76) in Model Code 1990

$$\epsilon_{sh} = [200 + 45 (6 - f_{cu}/13)] \times 10^{-6}$$

It is also assumed that the permanent load/total load ratio for Table 3.10 is 0.75.

A more important advantage that can be gained from the use of high strength concrete in controlling deflections arises from its higher tensile strength, and hence the higher cracking moment. Figure 5-1 shows how this affects the moment/curvature. In order to take advantage of the change in properties, tension stiffening should be considered.



**Figure 5-1: Moment vs curvature for high strength and normal strength concrete.
(from CSTR49)**

The method used to adapt Table 3.11 of CP 65: Part 1 follows that given for calculating mid-span deflection given in Clause 3.7 of CP 65: Part 2. Values of curvature were calculated assuming

tension stiffening in accordance with Scott (1983). The tension strength of concrete was assumed to be $0.45 \sqrt{f_{cu}}$ (see CP 65, CI 4.3.5.2 (b)).

The K values given in Table 3.1 of CP 65: Part 2, and hence the modification factors calculated, are conservative since they are based on the stiffness values at the most cracked position of the beam or cantilever.

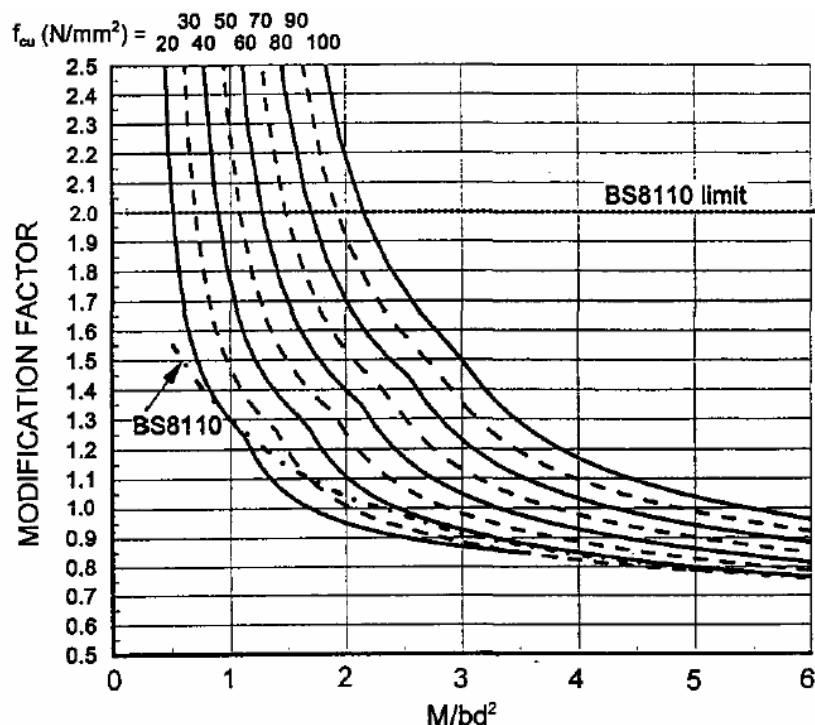


Figure 5-2: Modification factor vs M/bd^2 for a service stress of 307 N/mm² ($f_y = 460$ N/mm²). (from CSTR49)

Figure 5-2 provides an extension to Table 3.11 for a service stress of 307 N/mm² ($f_y = 460$ N/mm²), showing values of the modification factor plotted against M/bd^2 for concrete strengths from $f_{cu} = 20$ to 100 N/mm². It should be noted that if the beam or slab is restrained in such a way that it cracks significantly under strain loading (e.g. shrinkage or temperature) this modification factor may not be conservative. For this reason the value of span/effective depth ratio should not be taken as greater than 45.

5.6 Column design

The analysis and design of columns may follow the current provisions of CP 65. The design charts given in BS8110: Part 3 do not apply to HSC, and the charts in this guidance report can be used. The effects of confinement steel on the compressive strength are not included here but can be obtained elsewhere.

5.7 Minimum reinforcement

5.7.1 Beams and slabs

CP 65, CI 3.12.5.3. The minimum reinforcement, for $f_y = 460$ N/mm², should be increased for $f_{cu} > 40$ N/mm², such that:

$$0.13 \leq 100 A_s / A_c \leq [0.13 \times (f_{cu} / 40)^{2/3}]$$

CP 65, CI 3.4.5.3. The minimum reinforcement should be increased for $f_{cu} > 40$ N/mm², such that:

$$0.4 b_v \leq (A_{sv} / s_v \times 0.87 f_y) \geq [0.4 b_v \times (f_{cu} / 40)^{2/3}]$$

but with the value of f_{cu} not to be taken as greater than 80 N/mm²

5.7.2 Columns

CP 65, Cl 3.12.5.3. The minimum compression reinforcement should be increased for $f_{cu} > 60$ N/mm², such that:

$$0.4 \leq 100 A_{c,min} / A_{cc} \geq [0.4 + 0.01(f_{cu} - 60)]$$

5.7.3 Walls

CP 65, Cl 3.12.7.4. The minimum horizontal reinforcement, $f_y = 460$ N/mm², should be increased for $f_{cu} > 60$ N/mm², such that:

$$0.25 \leq 100 A_s / A_c \geq [0.25 \times (f_{cu} / 40)^{2/3}]$$

5.8 Size and pitch of links in columns

Link spacing should not be more than 10 times the diameter of the longitudinal reinforcement and should be ribbed bars of minimum diameter 10 mm.

5.9 Bond and anchorage

The values of the bond coefficient β , in Table 3.28 of CP 65 remain unchanged but f_{cu} is to be limited to 60 N/mm²

5.10 Shrinkage

The ultimate shrinkage strain given in CP 65 (100×10^{-6} for outdoor exposure and 280×10^{-6} for indoor exposure) for medium grade concrete, should be used.

5.11 Creep

The creep coefficients given in CP 65: Part 2 Figure 7.1) may be reduced by 20%, unless the age at loading is less than 24 hours.

Section 6. Section 1. Column design charts

6.1 Introduction

Ninety column design charts have been prepared for concrete strengths from C60 to C105 in steps of 5 N/mm² and reinforcement ratios between 1.0 to 10% in steps of 1%. For rectangular sections, values of d/h have been chosen from 0.75 to 0.95 in steps of 0.05, and for circular columns, values of h_s/h between 0.6 and 0.9 in steps of 0.1. For all charts f_y is 460 N/mm² and the partial safety factor for reinforcement is $\gamma_m = 1.15$. Modulus of elasticity of reinforcement steel has been taken as 200,000 N/mm². The parabolic-rectangle stress block was used for concrete stress distribution.

The centre line of the reinforcement is shown on a diagram for each graph. For the rectangular sections it is assumed that all the reinforcement is distributed symmetrically over the length of the two centre lines. For the circular sections a specific practical arrangement of bars has been chosen to fit the conditions of the chart.

The charts have been prepared taking account of the concrete displaced by the reinforcement. For situations where significant reinforcement is placed in the sides of a rectangular column a conservative approach is to calculate an effective value of d , d_{eff} , for the reinforcement in the least compressed half of the column as shown in Figure 6-1. This value should be used with the charts assuming the full value of A_{sc} .

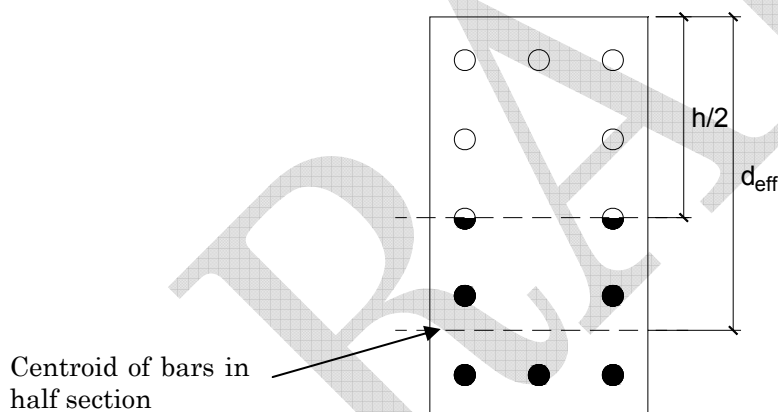


Figure 6-1 Position of d_{eff} in a rectangular column

6.2 Examples of column design

Example 1: Rectangular column

Rectangular column: 800 x 350 mm, $f_{cu} = 85$ N/mm²

Reinforcement: 12T25 (3 on the short side, 5 on the long side, see Figure 6-2)

Nominal cover: 30 mm, link diameter: 10 mm

Loading: Axial load, $N = 10\,000$ kN

Moment, $M_x = 500$ kNm

Moment, $M_y = 200$ kNm

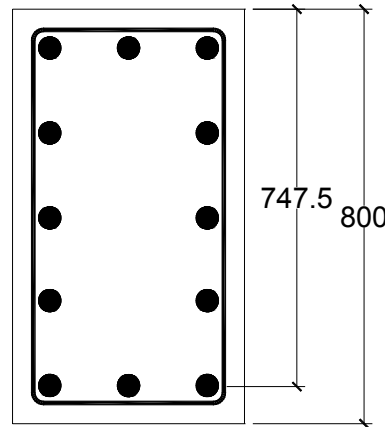


Figure 6-2 Rectangular column reinforcement arrangement

$$100 A_{sc}/bh = 100 \times 12 \times \pi \times 25^2 / (4 \times 350 \times 800) = 2.10$$

$$N/bh = 10000 \times 1000 / (350 \times 800) = 35.71 \text{ N/mm}^2$$

In the x-direction:

$$d = 800 - 30 - 10 - 25/2 = 747.5 \text{ mm}$$

$$d_{\text{eff}} = [3 \times 747.5 + 2 \times (747.5 + 800/2)/2 + 1 \times 400] / 6 = 631.7 \text{ mm}$$

$$d_{\text{eff}}/h = 631.7/800 = 0.79$$

$$M_x/bh^2 = 500 \times 10^6 / (350 \times 800^2) = 2.23 \text{ N/mm}^2$$

From the chart for $f_{cu} = 85 \text{ N/mm}^2$, $d/h = 0.8$ (reproduced in Figure 6-3)

$$M_{x,\text{max}}/bh^2 = 3.45 \text{ N/mm}^2$$

In the y-direction:

$$d = 350 - 30 - 10 - 25/2 = 297.5 \text{ mm}$$

$$d_{\text{eff}} = (5 \times 297.5 + 1 \times 175) / 6 = 277.1 \text{ mm}$$

$$d_{\text{eff}}/h = 277.1/350 = 0.79$$

$$M_y/bh^2 = 200 \times 10^6 / (800 \times 350^2) = 2.04 \text{ N/mm}^2$$

From the chart for $f_{cu} = 85 \text{ N/mm}^2$, $d/h = 0.8$ (reproduced in Figure 6-4):

$$M_{y,\text{max}}/bh^2 = 3.4 \text{ N/mm}^2$$

Biaxial bending (Check to BS 8110 Clause 3.8.4.5):

$$N/bhf_{cu} = 10000 \times 1000 / (350 \times 800 \times 85) = 0.42$$

From Table 3.22 of BS 8110:

$$\beta = 0.53 - (0.53 - 0.42) \times 0.02/0.1 = 0.51$$

$$M_x/h' = 500 \times 1000 / 631.7 = 792 \text{ kN}$$

$$M_y/b' = 200 \times 1000/277.1 = 722 \text{ kN}$$

$M_x/h' > M_y/b'$, hence use equation 40 :

$$M_x' = 500 + 0.51 \times 200 \times 631.7/277.1 = 732 \text{ kNm}$$

$$M_{x,\max} = 3.4 \times 350 \times 800^2/10^6 = 762 \text{ kNm} (> 732 \text{ kNm}) \quad \text{OK}$$

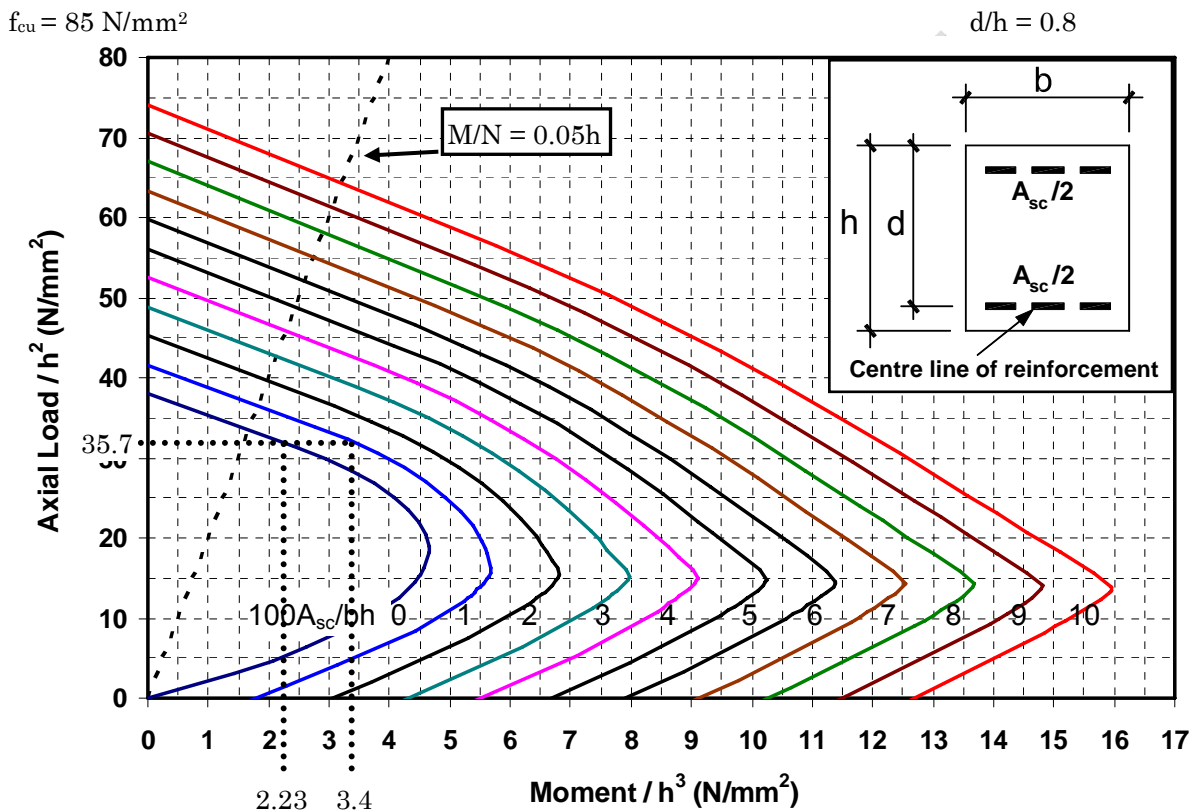


Figure 6-3 Column moment/axial load chart for $f_{cu} = 85 \text{ N/mm}^2$ and $d/h = 0.8$ used to establish $M_{x,\max}/bh^2$

$$f_{cu} = 85 \text{ N/mm}^2$$

$$d/h = 0.8$$

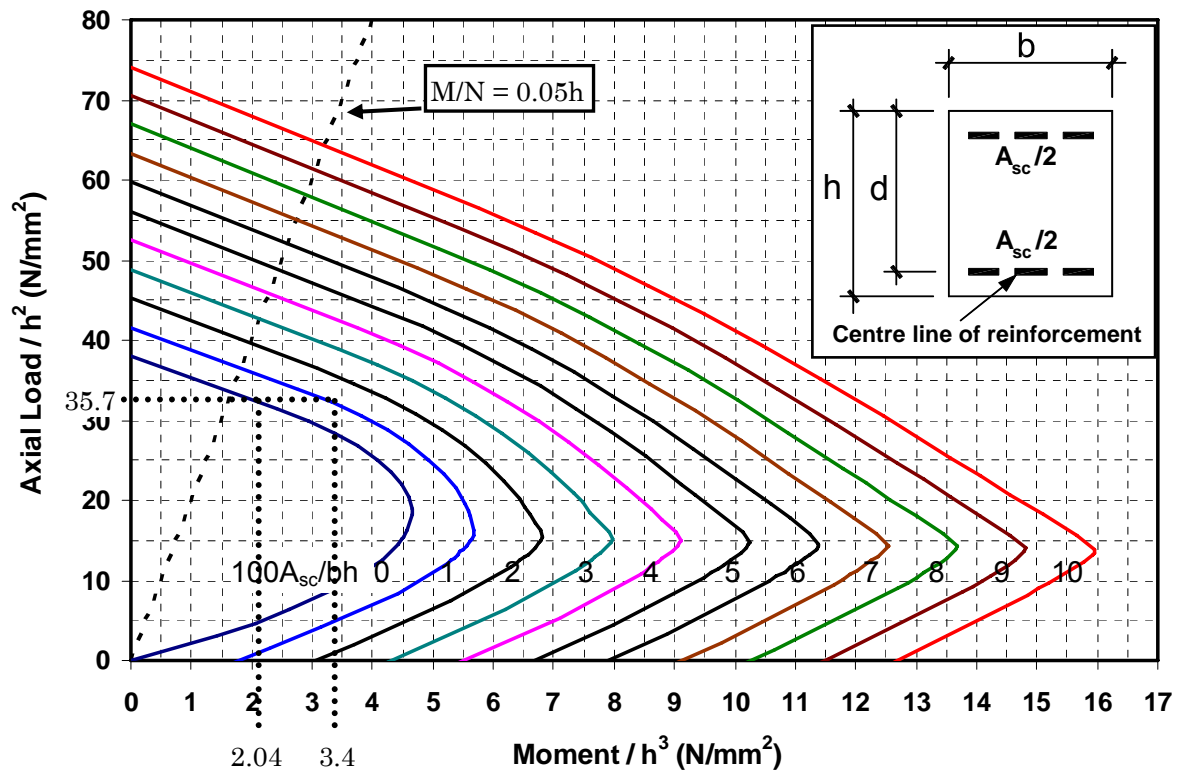


Figure 6-4 Column moment/axial load chart for $f_{cu} = 85 \text{ N/mm}^2$ and $d/h = 0.8$ used to establish $M_{y,max}/bh^2$

Example 2: Circular column

Circular column: 750mm diameter, $f_{cu} = 100 \text{ N/mm}^2$

Reinforcement: 12T32 (see Figure 6-5) , nominal cover: 25 mm, link diameter: 12 mm

Loading: Axial load, $N = 15\,000 \text{ kN}$; Moment, $M = 1500 \text{ kNm}$

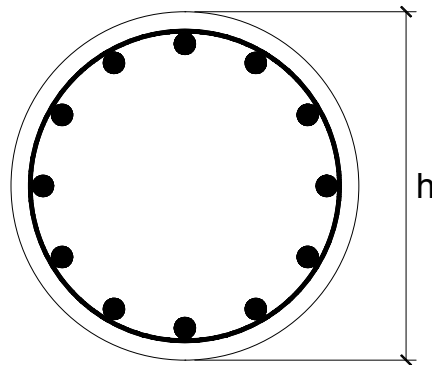


Figure 6-5 Circular column reinforcement arrangement

$$h_s = 750 - 2 \times (32/2 + 25 + 12) = 644 \text{ mm}$$

$$h_s/h = 644/750 = 0.86$$

$$100A_{sc}/A_c = 100 \times 12 \times 32^2 / 750^2 = 2.2$$

$$N/h^2 = 15000 \times 1000 / 750^2 = 26.7 \text{ N/mm}^2$$

$$M/h^3 = 1500 \times 10^6 / 750^3 = 3.6 \text{ N/mm}^2$$

From the chart for $f_{cu} = 100 \text{ N/mm}^2$ and $h_s/h = 0.8$ (reproduced as Figure 6-6. Note that the chart for $h_s/h = 0.9$ is less conservative)

For $N/h^2 = 26.7 \text{ N/mm}^2$

$$M_{max}/h^3 = 3.6 \text{ N/mm}^2$$

$$M_{max} = 3.6 \times 750^3 / 10^6 = 1519 \text{ kNm, greater than } M = 1500 \text{ kNm applied,}$$

Hence **OK**

$$f_{cu} = 100 \text{ N/mm}^2$$

$$h_s/h = 0.8$$

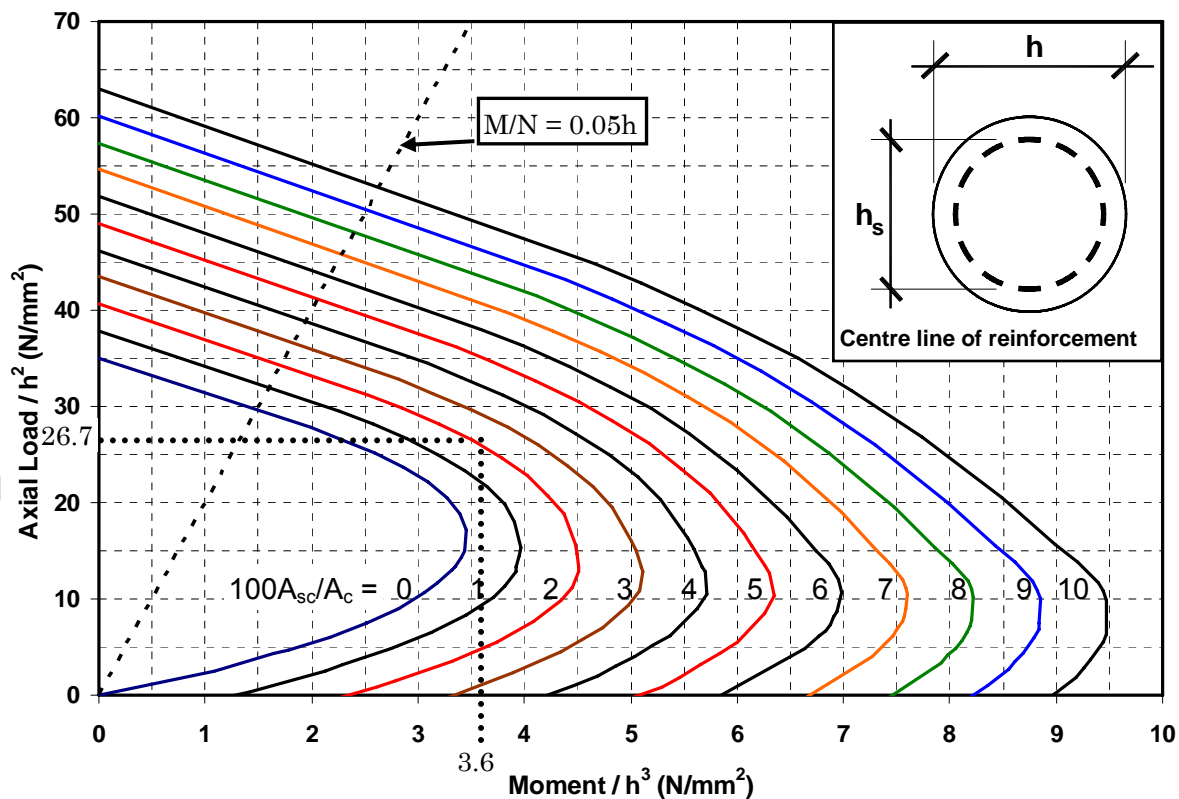


Figure 6-6: Column moment/axial load chart for $f_{cu} = 100 \text{ N/mm}^2$ and $h_s/h = 0.8$ used to establish M_{max}/h^3

Example 3: Rectangular column

Rectangular column: 800 x 400 mm, $f_{cu} = 95$ N/mm²
 Reinforcement: 12T32 (3 on the short side, 5 on the long side, see Figure 6-7)

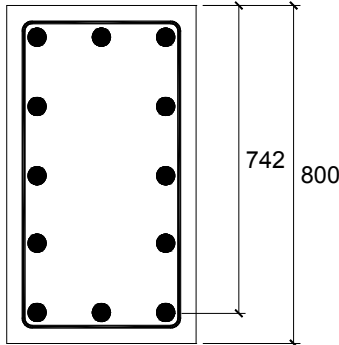


Figure 6-7 Rectangular column reinforcement arrangement

Nominal cover: 30 mm, link diameter: 12 mm

Loading: Axial load, $N = 12\,000$ kN

Moment, $M_x = 850$ kNm

Moment, $M_y = 400$ kNm

$$100 A_{sc}/bh = 100 \times 12 \times \pi \times 32^2 / (4 \times 400 \times 800) = 3.02$$

$$N/bh = 12\,000 \times 1000 / (400 \times 800) = 37.50 \text{ N/mm}^2$$

In the x-direction:

$$d = 800 - 30 - 12 - 32/2 = 742 \text{ mm}$$

$$d_{eff} = [3 \times 742 + 2 \times (742 + 800/2)/2 + 1 \times 400] / 6 = 628 \text{ mm}$$

$$d_{eff}/h = 628/800 = 0.79$$

In the y-direction:

$$d = 400 - 30 - 12 - 32/2 = 342 \text{ mm}$$

$$d_{eff} = (5 \times 342 + 1 \times 200) / 6 = 318.3 \text{ mm}$$

$$d_{eff}/h = 318.3/400 = 0.80$$

In both cases the chart for $f_{cu} = 95$ N/mm², $d/h = 0.8$ (reproduced in Figure 6-8) can be used to get M_{max}/bh^2 corresponding with $N/bh = 37.50$ N/mm² and $100 A_{sc}/bh = 3.02$

$$M_{max}/bh^2 = 5.1 \text{ N/mm}^2$$

$$M_{x,max} = 5.1 \times 400 \times 800^2 / 10^6 = 1306 \text{ kNm} (>850 \text{ kNm}) \text{ OK}$$

$$M_{y,max} = 5.1 \times 800 \times 400^2 / 10^6 = 653 \text{ kNm} (>400 \text{ kNm}) \text{ OK}$$

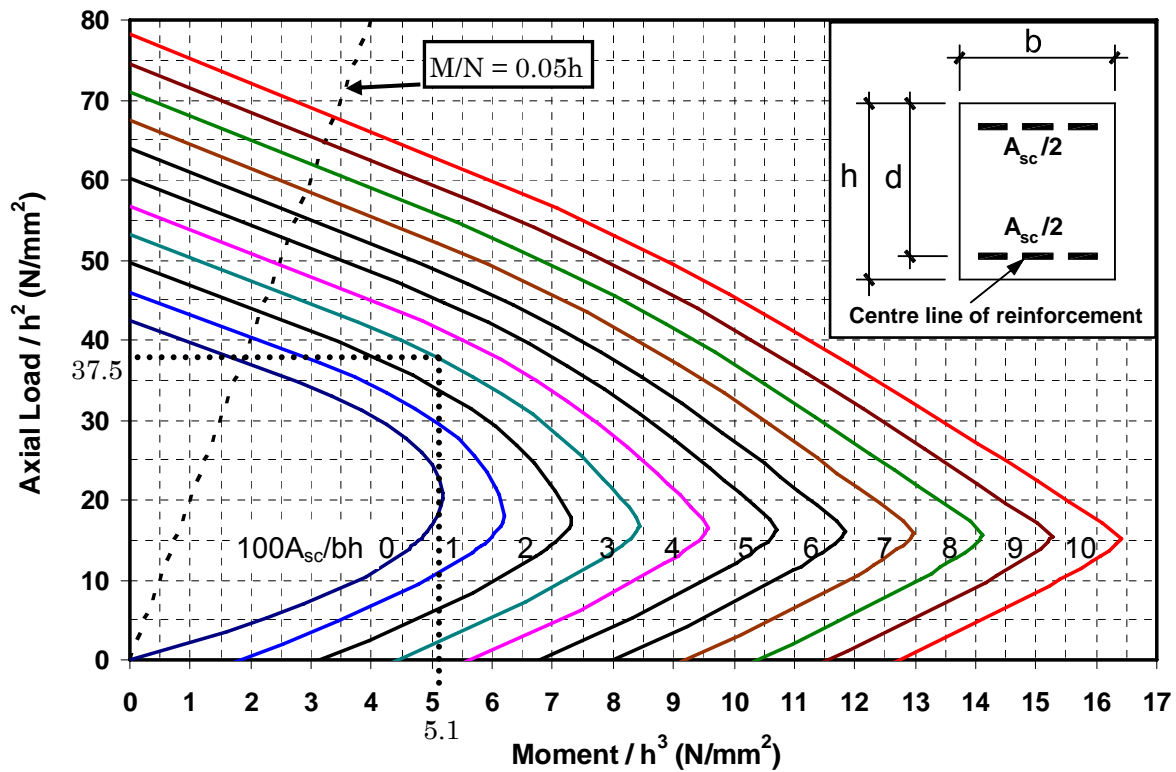
$f_{cu} = 95 \text{ N/mm}^2$ $d/h = 0.80$ 

Figure 6-8: Column moment/axial load chart for $f_{cu} = 95 \text{ N/mm}^2$ and $d/h = 0.8$ used to establish M_{\max}/bh^2

Biaxial bending (Check to BS 8110 Clause 3.8.4.5):

$$N/bhf_{cu} = 12\,000 \times 1000 / (400 \times 800 \times 95) = 0.4$$

From Table 3.22 of BS 8110:

$$\beta = 0.53$$

$$M_x/h' = 800 \times 1000 / 628 = 1268 \text{ kNm}$$

$$M_y/b' = 450 \times 1000 / 342 = 1257 \text{ kNm}$$

$M_x/h' > M_y/b'$ hence use equation 40 :

$$M_x' = 800 + 0.53 \times 450 \times 628 / 342 = 1268 \text{ kNm}$$

$M_{x,\max} > M_x'$ **OK**

Example 4: Rectangular column

Rectangular column: 700 x 400 mm, $f_{cu} = 95 \text{ N/mm}^2$

Reinforcement: 10T32 (4 on the short side, 3 on the long side, see Figure 6-9)

Nominal cover: 30 mm, link diameter: 13 mm

Loading: Axial load, $N = 10\,000 \text{ kN}$

Moment, $M_x = 700 \text{ kNm}$

Moment, $M_y = 300 \text{ kNm}$

$$100 A_{sc}/bh = 100 \times 10 \times \pi \times 32^2 / (4 \times 400 \times 700) = 2.87$$

$$N/bh = 10\,000 \times 1000 / (400 \times 700) = 35.71 \text{ N/mm}^2$$

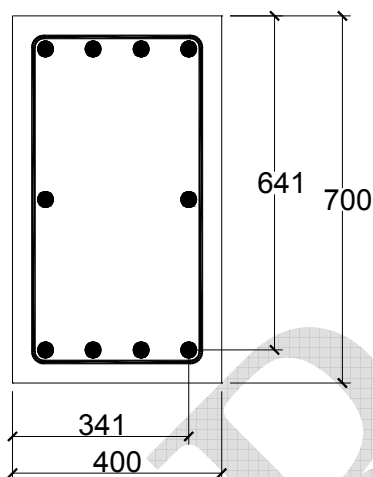


Figure 6-9 Rectangular column reinforcement arrangement

In the x-direction:

$$d = 700 - 30 - 13 - 32/2 = 641 \text{ mm}$$

$$d_{eff} = [4 \times 641 + 1 \times 350] / 5 = 583 \text{ mm}$$

$$d_{eff}/h = 583/700 = 0.83$$

From the chart for $f_{cu} = 95 \text{ N/mm}^2$, $d/h = 0.8$ (reproduced in Figure 6-10), $N/bh = 35.71 \text{ N/mm}^2$, $100 A_{sc}/bh = 2.87$ we get $M_{x,max}/bh^2 = 5.4 \text{ N/mm}^2$. Hence

$$M_{x,max} = 5.4 \times 400 \times 700^2 / 10^6 = 1058 \text{ kNm} (>700 \text{ kNm}) \text{ OK}$$

In the y-direction:

$$d = 400 - 30 - 13 - 32/2 = 341 \text{ mm}$$

$$d_{eff} = [3 \times 341 + 2 \times [200 + (341 - 200)/3]]/5 = 303 \text{ mm}$$

$$d_{eff}/h = 303/400 = 0.76$$

From the chart for $f_{cu} = 95 \text{ N/mm}^2$, $d/h = 0.75$ (reproduced in Figure 6-11), $N/bh = 35.71 \text{ N/mm}^2$, $100 A_{sc}/bh = 2.87$ we get $M_{y,max}/bh^2 = 5.1 \text{ N/mm}^2$

$$M_{y,max} = 5.1 \times 700 \times 400^2 / 10^6 = 571 \text{ kNm} (> 300 \text{ kNm}) \text{ OK}$$

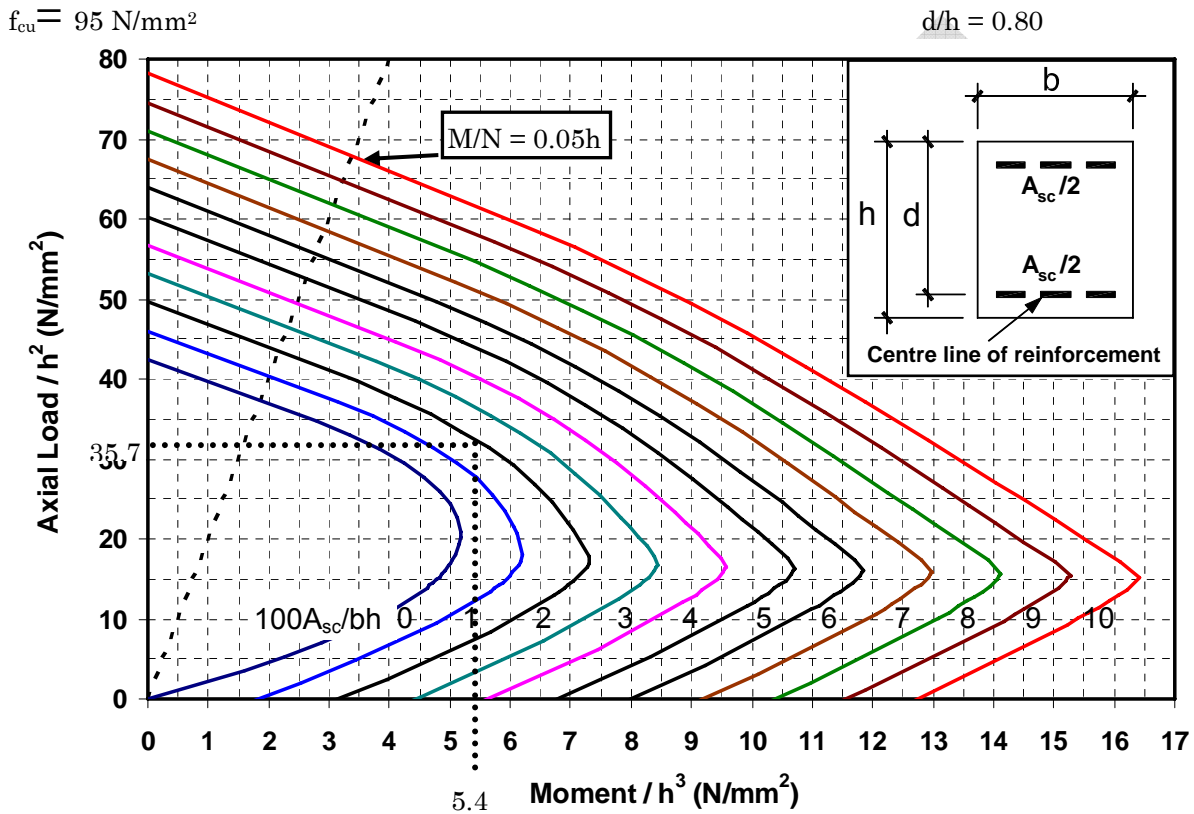


Figure 6-10: Column moment/axial load chart for $f_{cu} = 95 \text{ N/mm}^2$ and $d/h = 0.8$ used to establish M_{max}/bh^2

$$f_{cu} = 95 \text{ N/mm}^2$$

$$d/h = 0.75$$

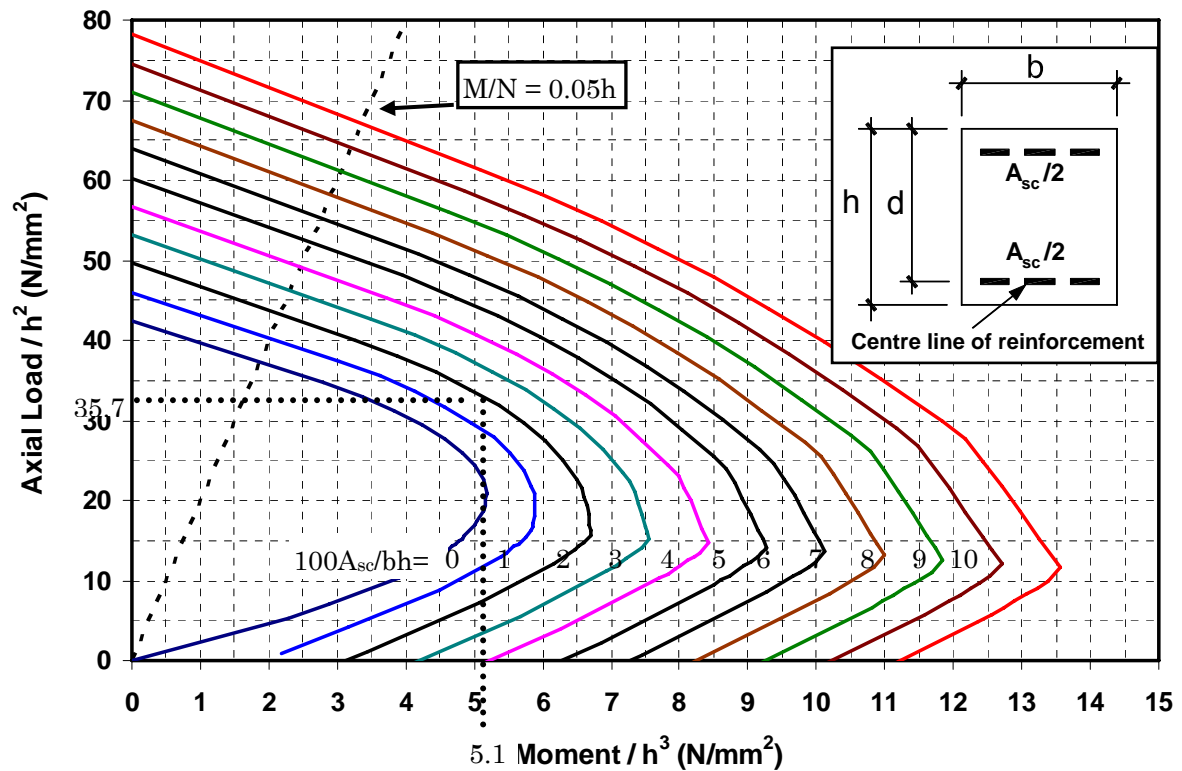


Figure 6-11 Column moment/axial load chart for $f_{cu} = 95 \text{ N/mm}^2$ and $d/h = 0.75$ used to establish M_{\max}/bh^2

Biaxial bending (Check to BS 8110 Clause 3.8.4.5):

$$N/bhf_{cu} = 10\,000 \times 1000 / (400 \times 700 \times 95) = 0.38$$

From Table 3.22 of BS 8110:

$$\beta = 0.65 - (0.65 - 0.53) \times 0.08 / 0.1 = 0.55$$

$$M_x/h' = 700 \times 1000 / 583 = 1201 \text{ kN}$$

$$M_y/b' = 300 \times 1000 / 303 = 989 \text{ kN}$$

$M_x/h' > M_y/b'$ hence use equation 40:

$$M_x' = 700 + 0.55 \times 300 \times 583 / 303 = 1019 \text{ kNm}$$

$M_{x,\max} > M_x'$ **OK**

6.3 Design Charts for Rectangular Columns

A total of fifty design charts for rectangular columns have been prepared for concrete strengths from 60 N/mm² to 105 N/mm² in steps of 5 N/mm². The value of h_s/h have been chosen between 0.6 and 0.9 in steps of 0.1. Reinforcement ratios have been taken from 1.0 to 10% in steps of 1%. The centre line of the reinforcement is shown on a diagram of each graph. It is assumed that all the reinforcement is distributed symmetrically over the length of the two centre lines.

For all charts f_y is taken as 460 N/mm² and the partial safety factor for reinforcement is $\gamma_m = 1.15$. Modulus of elasticity of reinforcement steel has been taken as 200,000 N/mm².

The parabolic-rectangle stress block was used for concrete stress distribution.

Chart 6.1

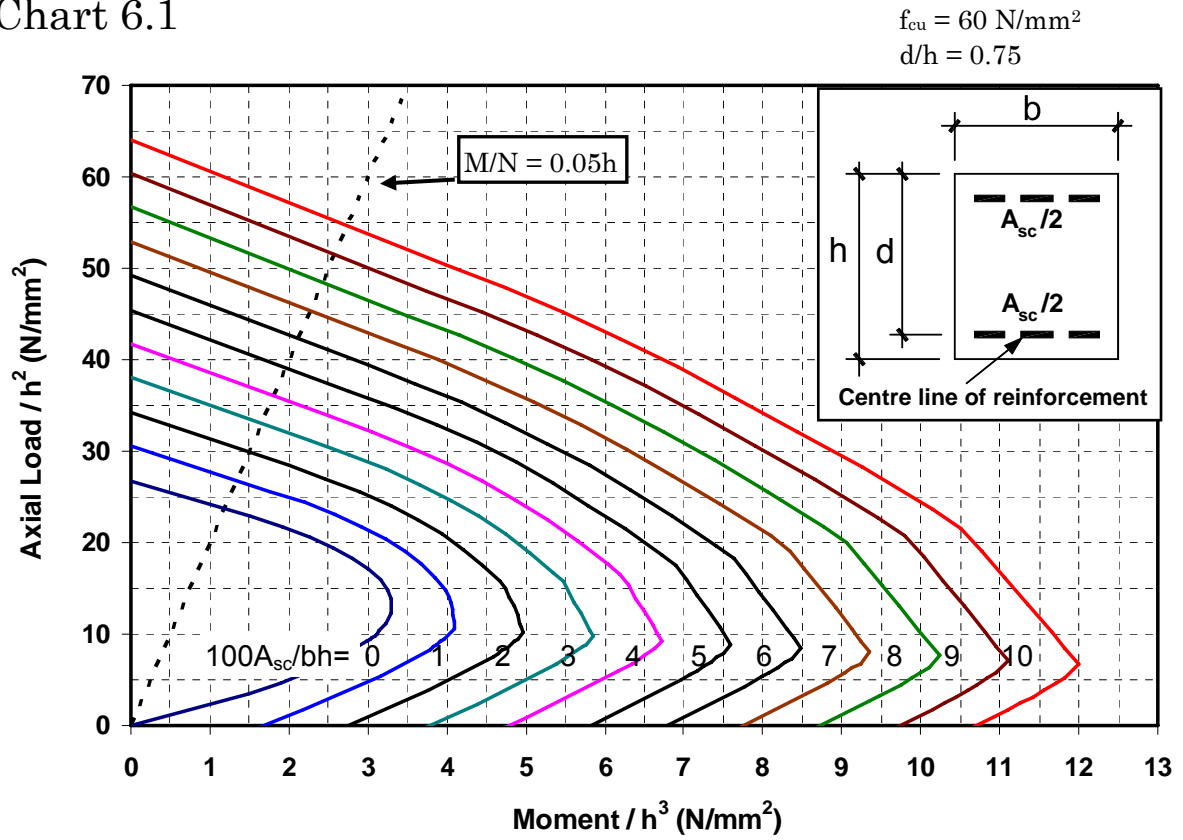


Chart 6.2

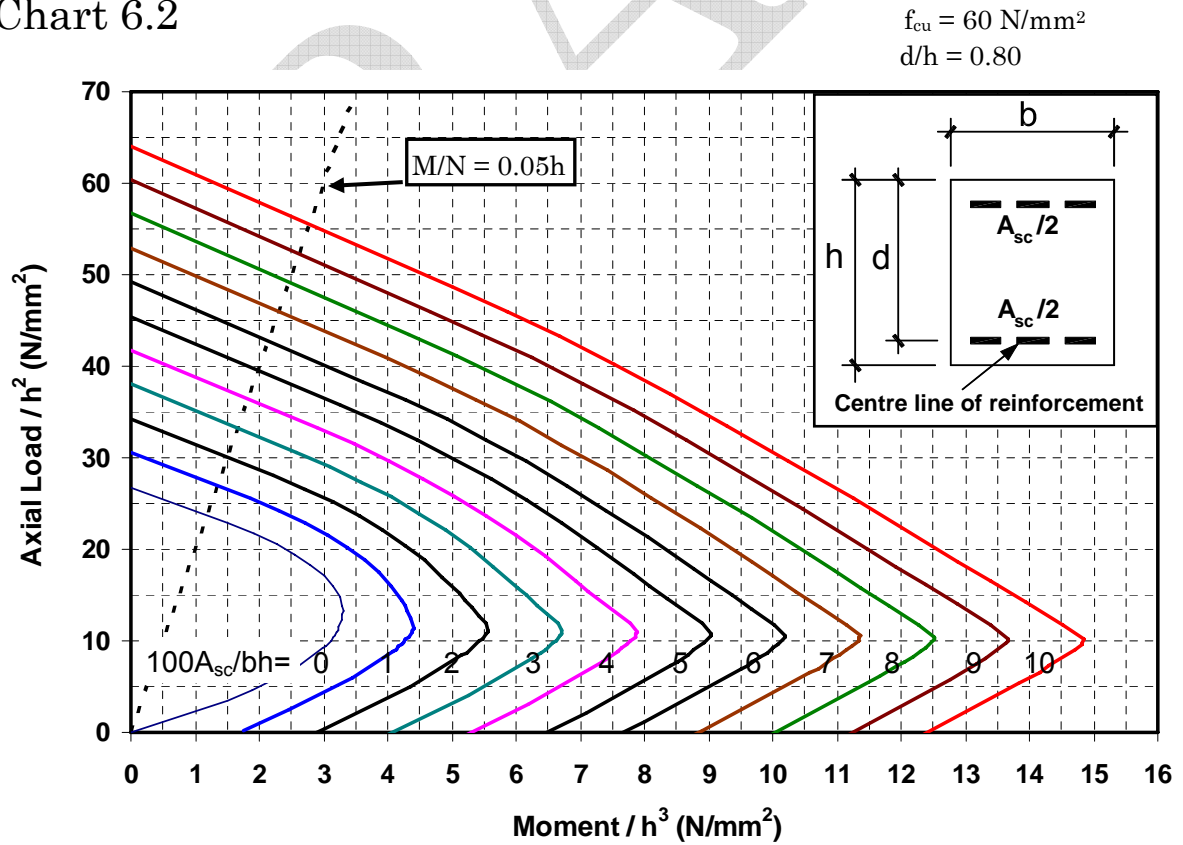


Chart 6.3

$$f_{cu} = 60 \text{ N/mm}^2$$

$$d/h = 0.85$$

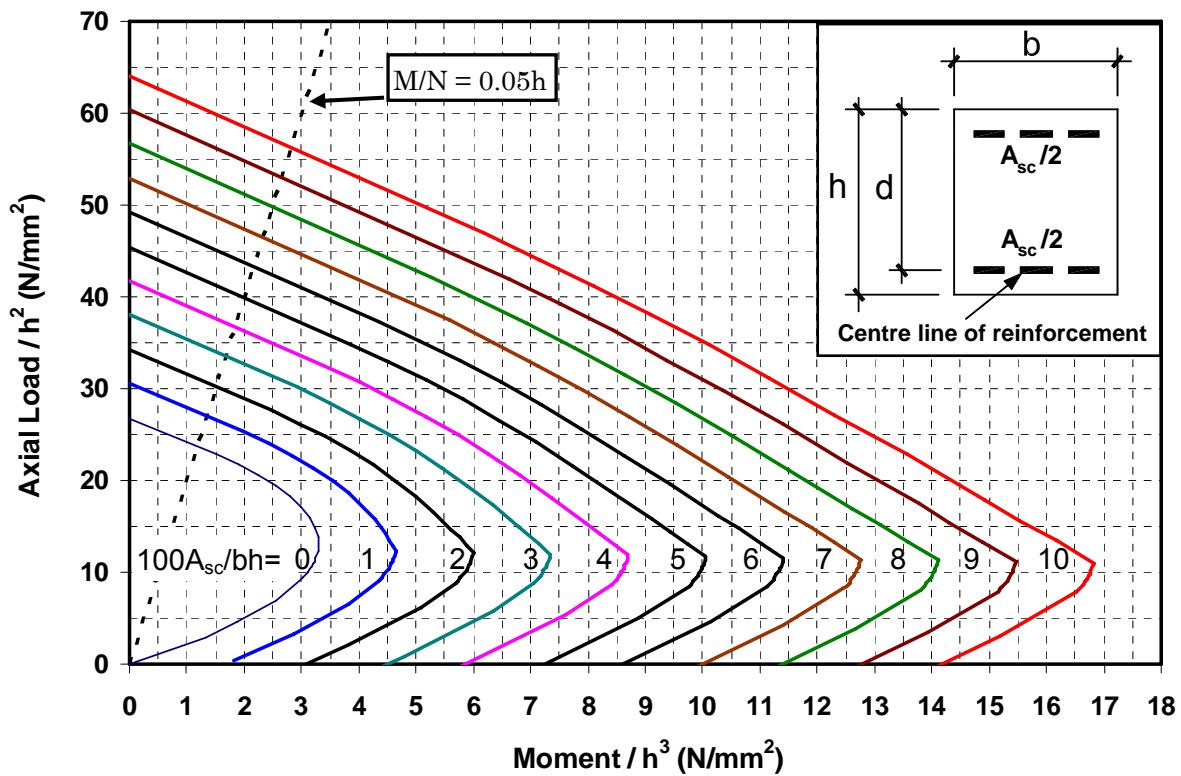


Chart 6.4

$$f_{cu} = 60 \text{ N/mm}^2$$

$$d/h = 0.90$$

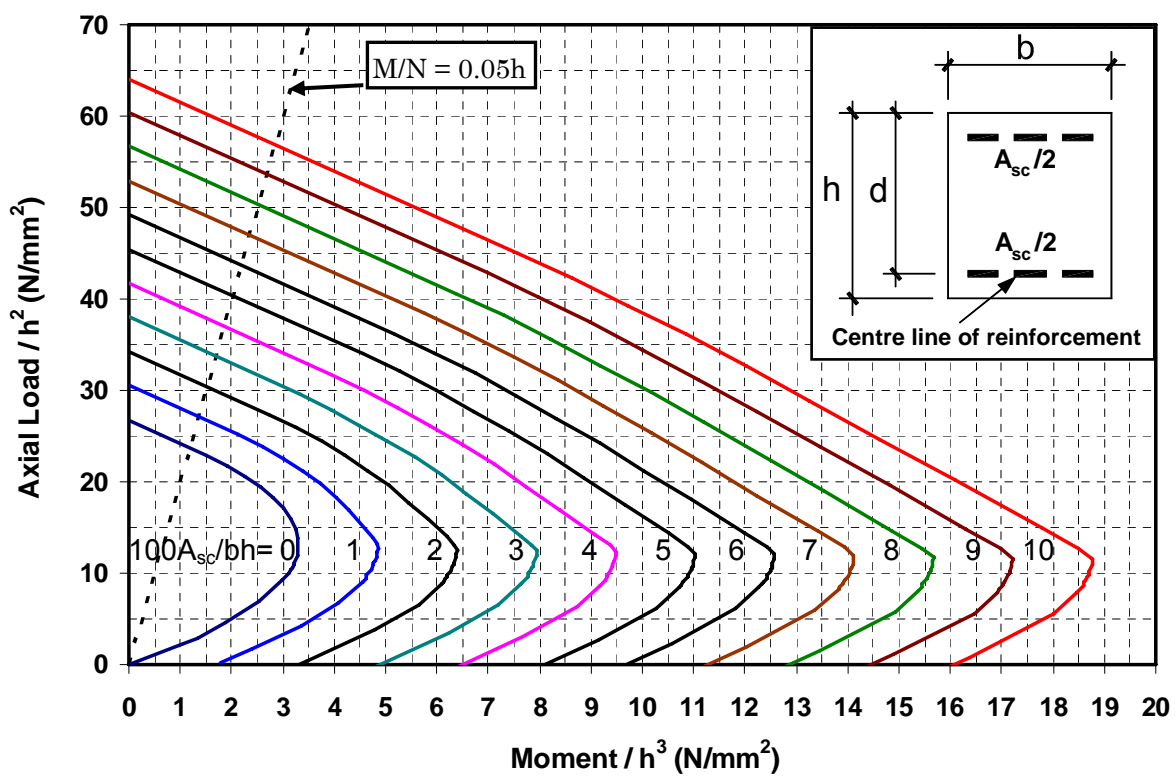


Chart 6.5

$f_{cu} = 60 \text{ N/mm}^2$
 $d/h = 0.95$

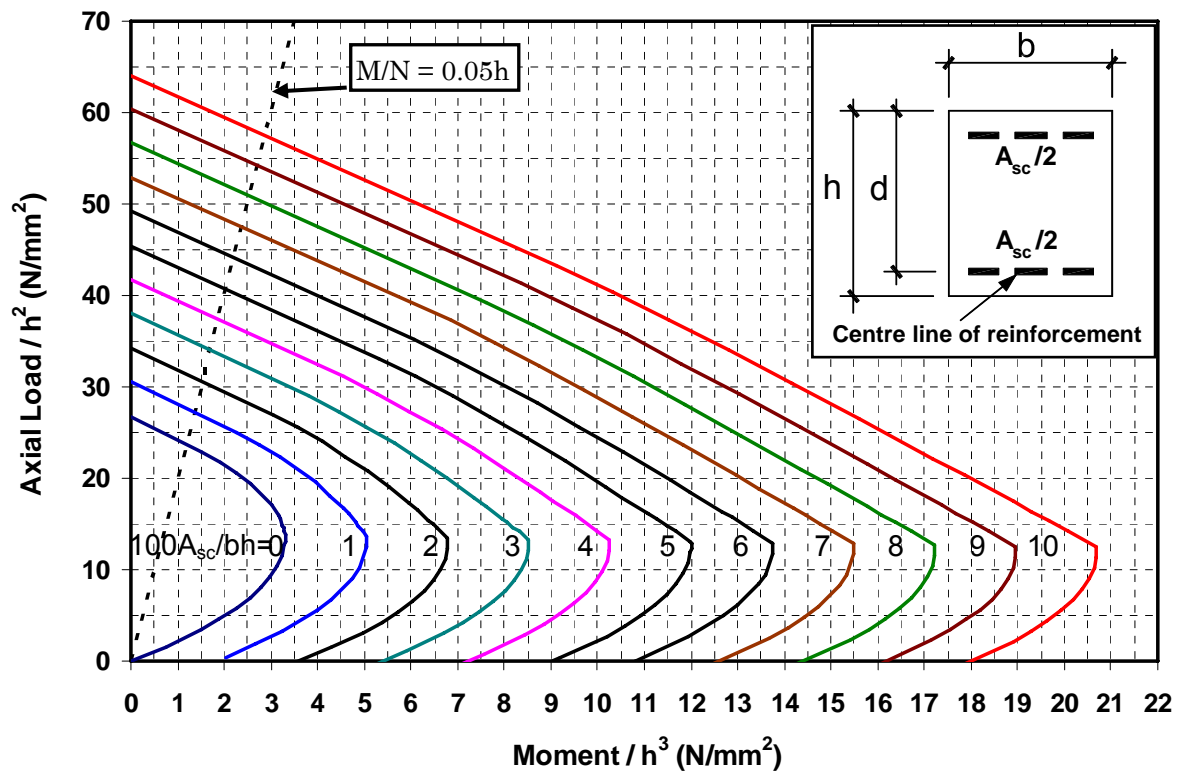


Chart 6.6

$f_{cu} = 65 \text{ N/mm}^2$
 $d/h = 0.75$

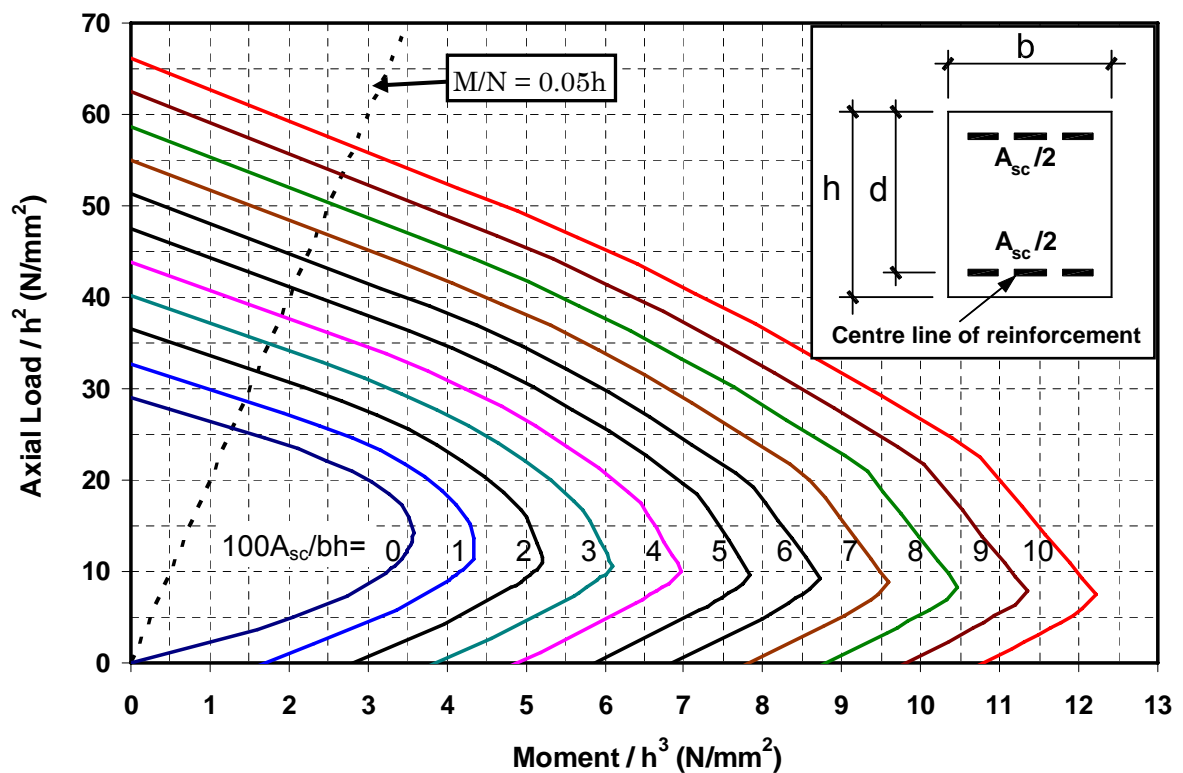


Chart 6.7

$f_{cu} = 65 \text{ N/mm}^2$
 $d/h = 0.80$

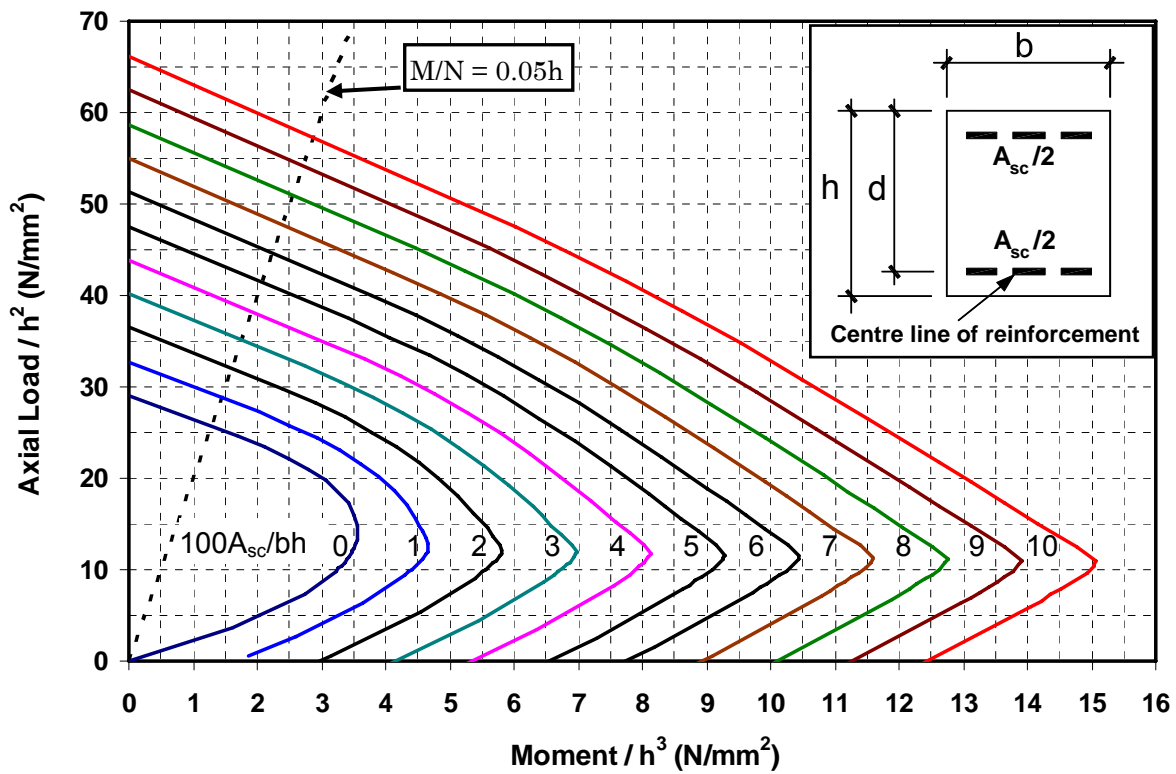


Chart 6.8

$f_{cu} = 65 \text{ N/mm}^2$
 $d/h = 0.85$

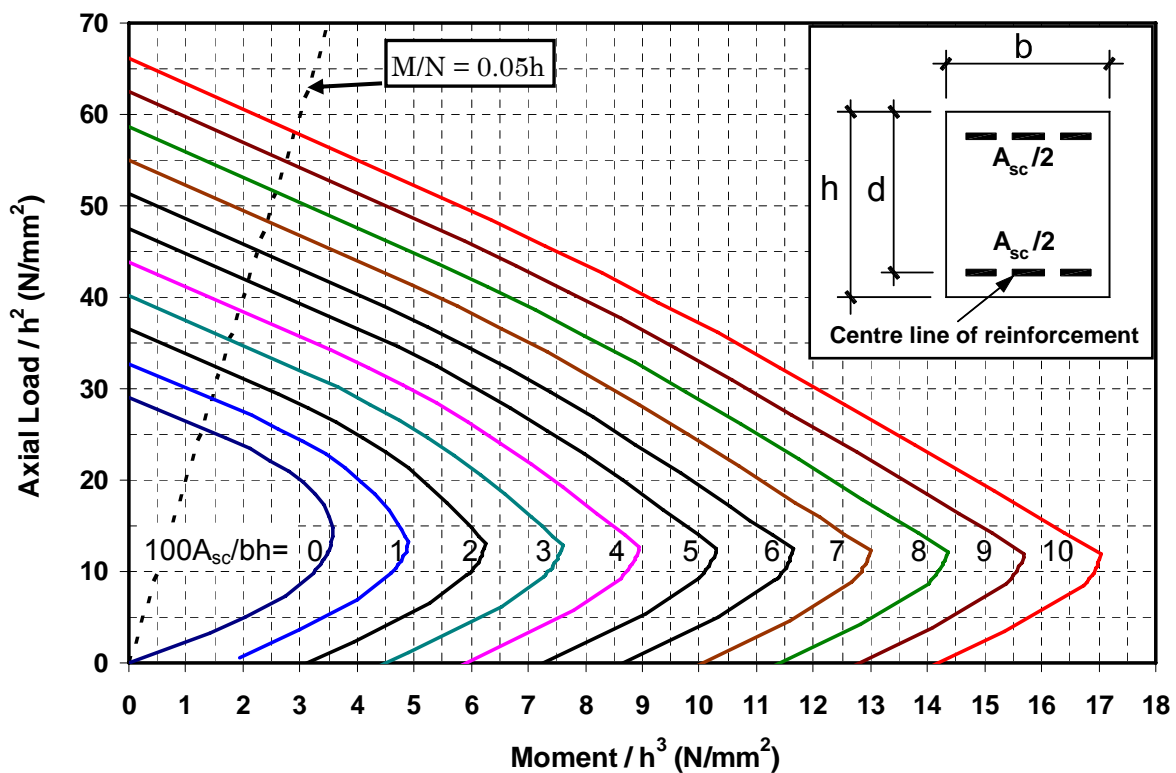


Chart 6.9

$f_{cu} = 65 \text{ N/mm}^2$
 $d/h = 0.90$

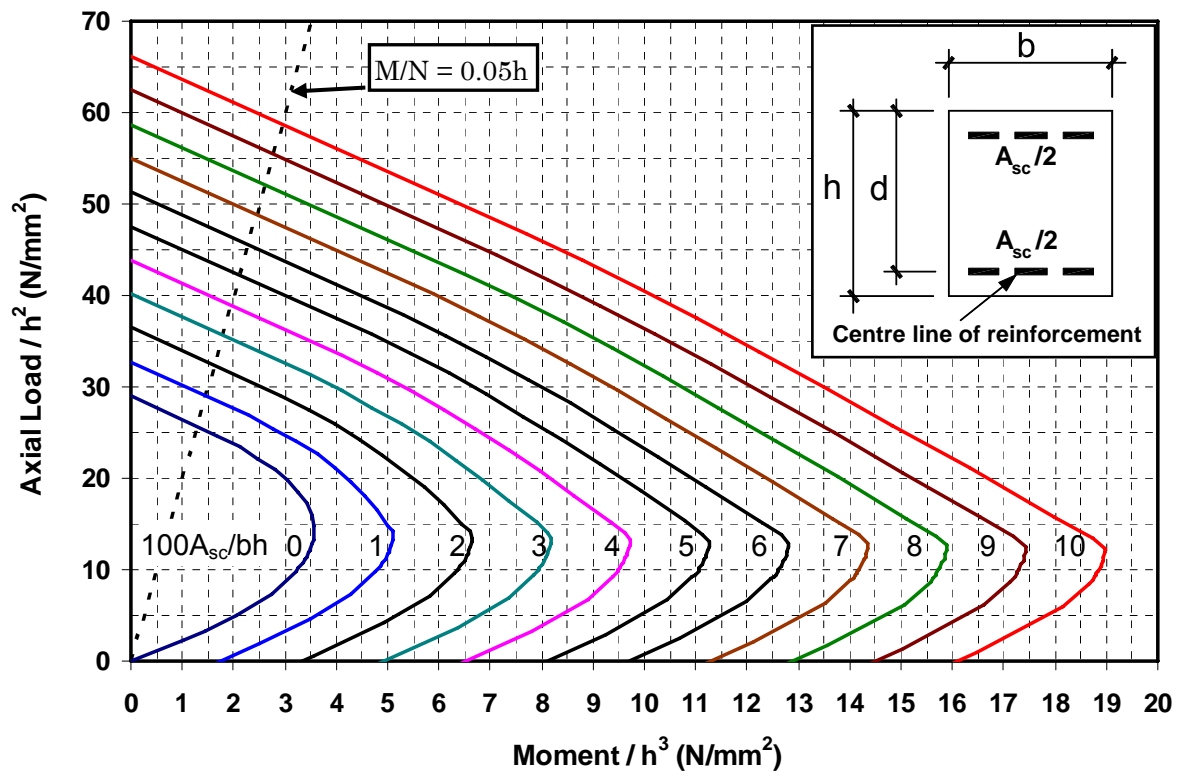


Chart 6.10

$f_{cu} = 65 \text{ N/mm}^2$
 $d/h = 0.95$

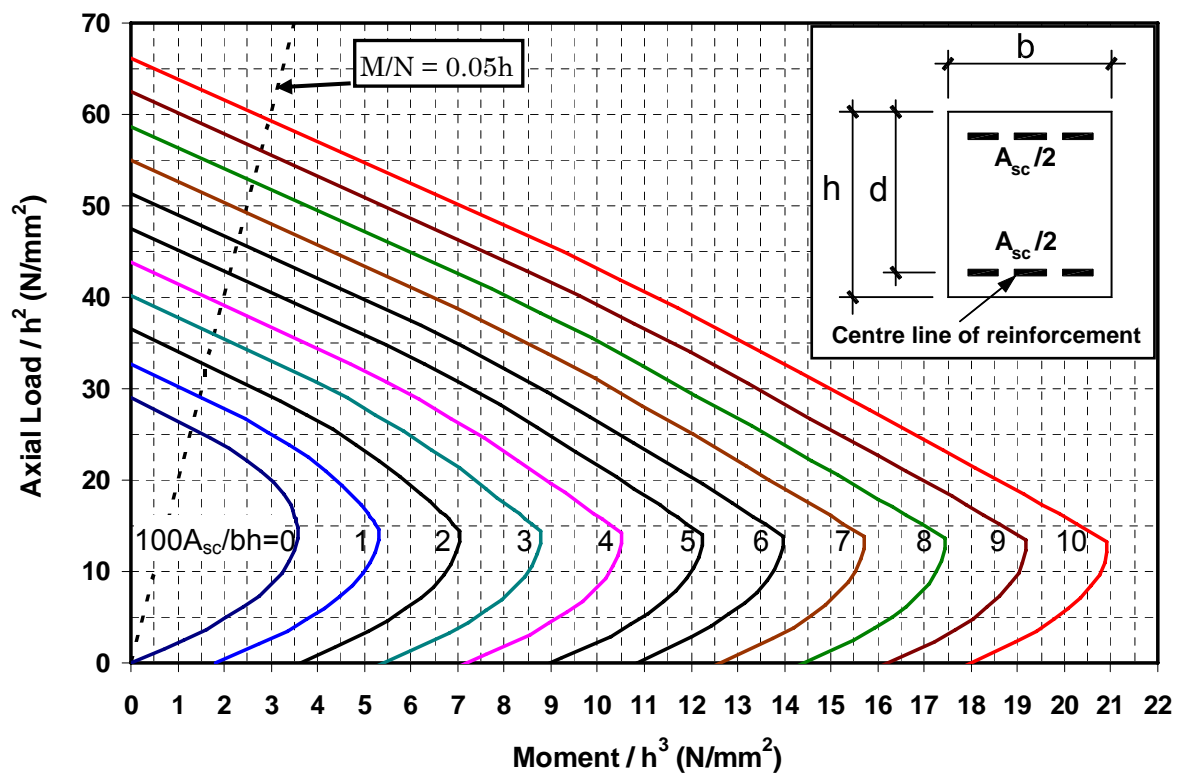


Chart 6.11

$f_{cu} = 70 \text{ N/mm}^2$
 $d/h = 0.75$

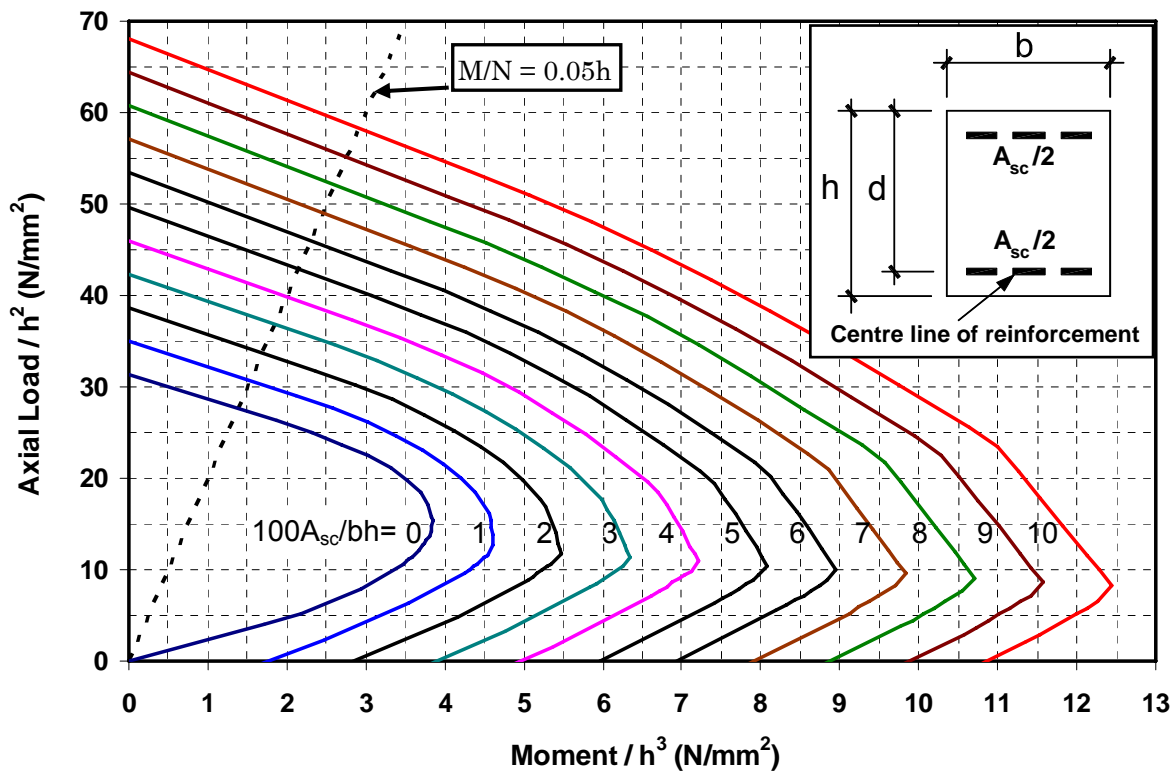


Chart 6.12

$f_{cu} = 70 \text{ N/mm}^2$
 $d/h = 0.80$

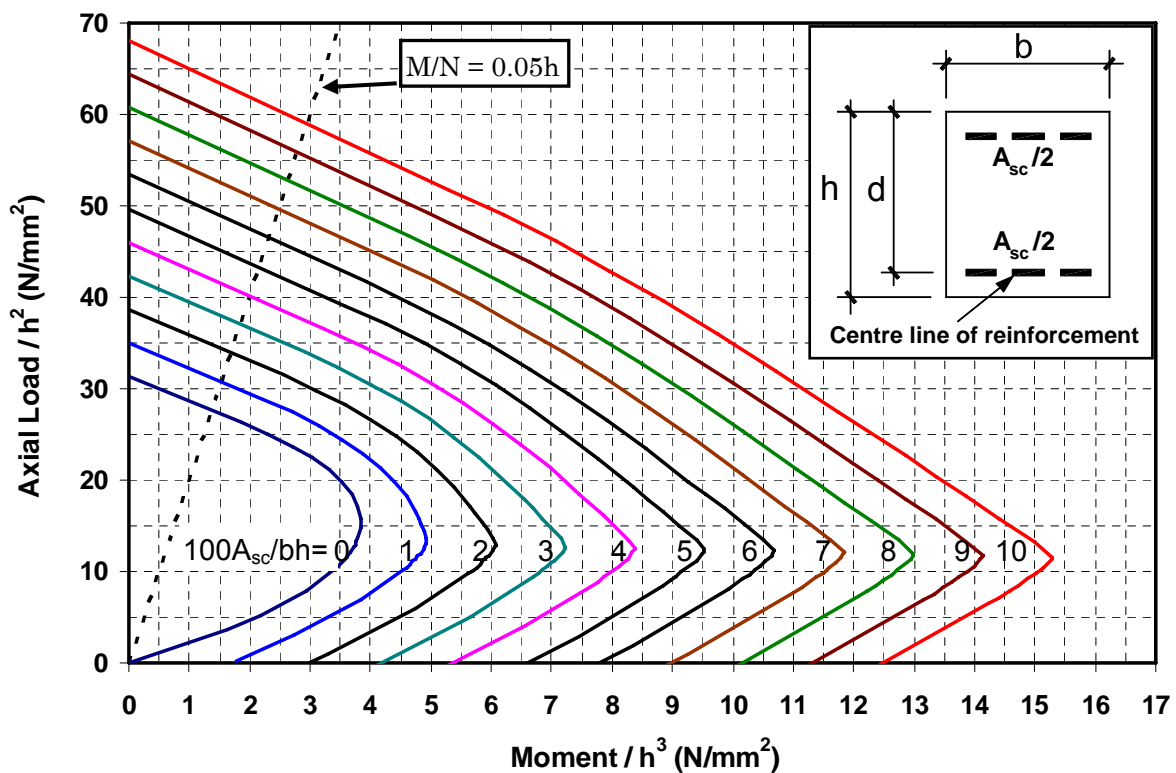


Chart 6.13

$f_{cu} = 70 \text{ N/mm}^2$
 $d/h = 0.85$

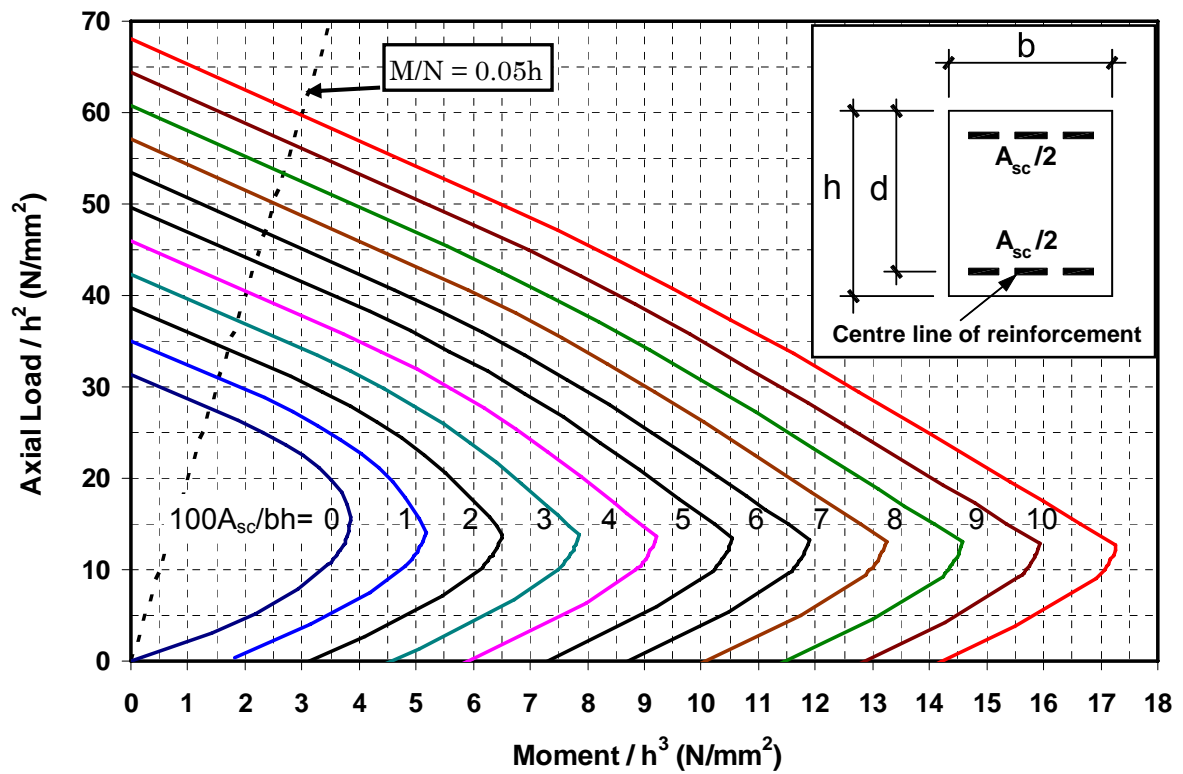


Chart 6.14

$f_{cu} = 70 \text{ N/mm}^2$
 $d/h = 0.90$

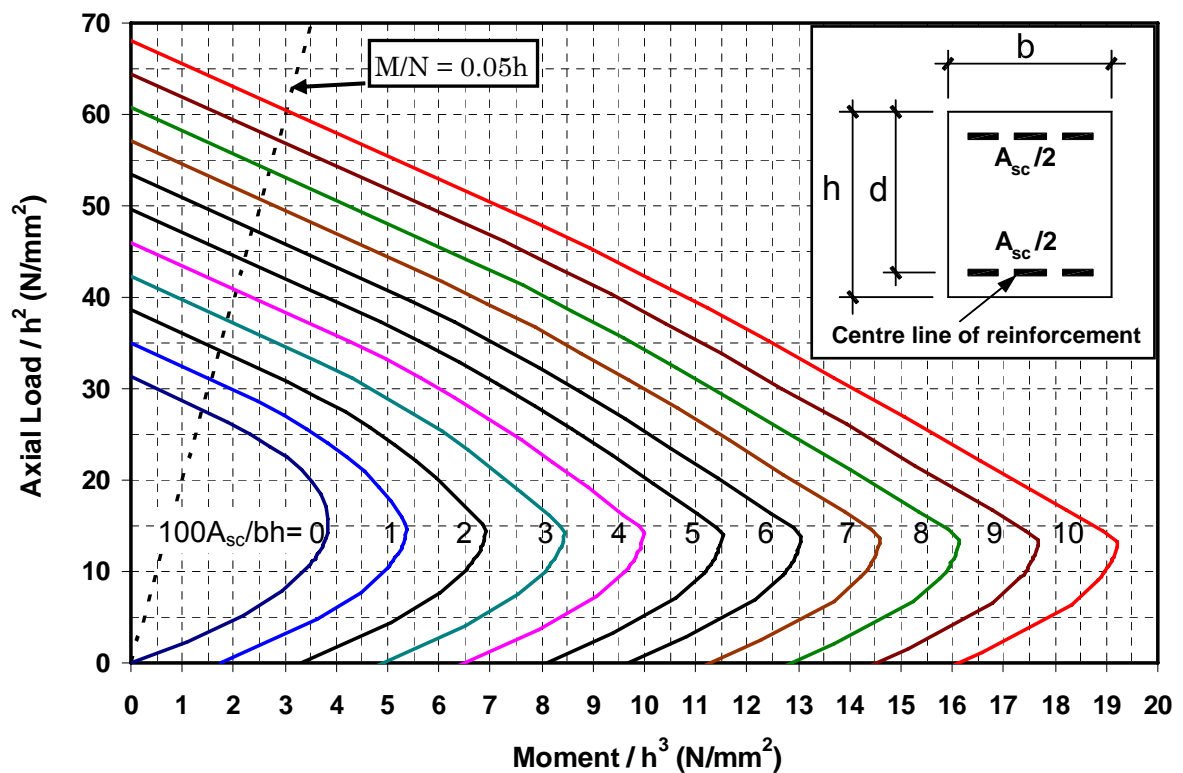


Chart 6.15

$f_{cu} = 70 \text{ N/mm}^2$
 $d/h = 0.95$

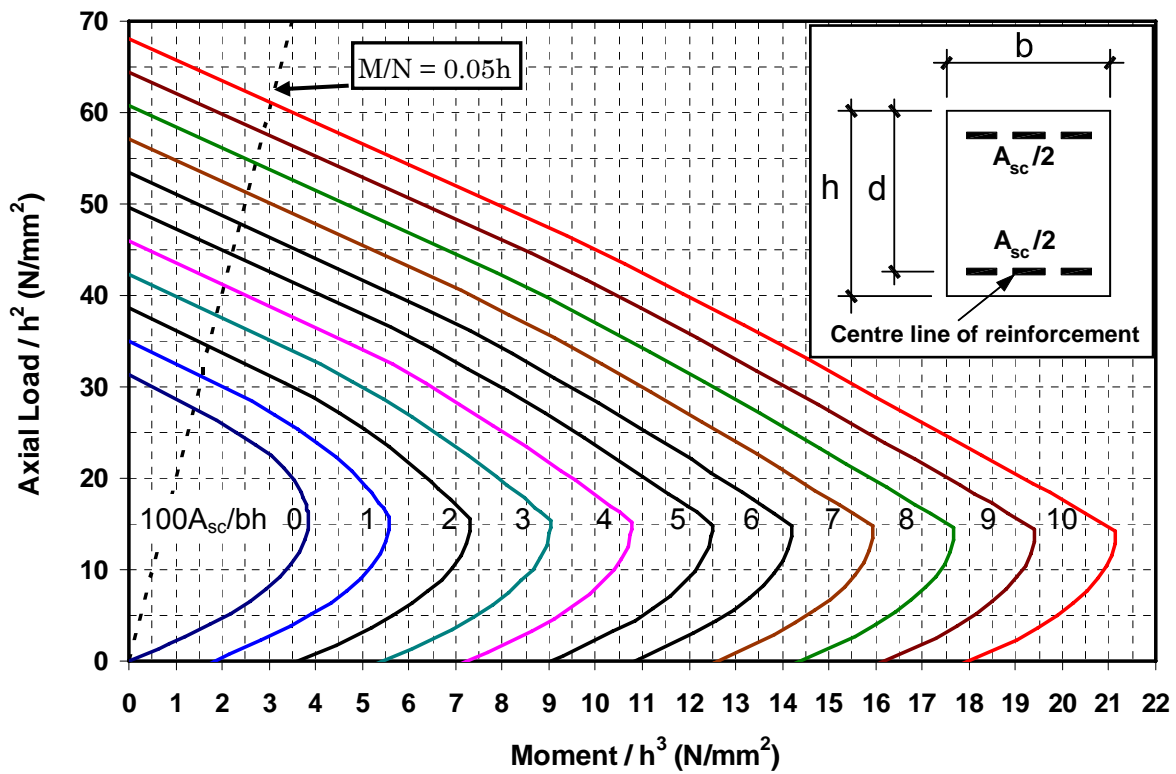


Chart 6.16

$f_{cu} = 75 \text{ N/mm}^2$
 $d/h = 0.75$

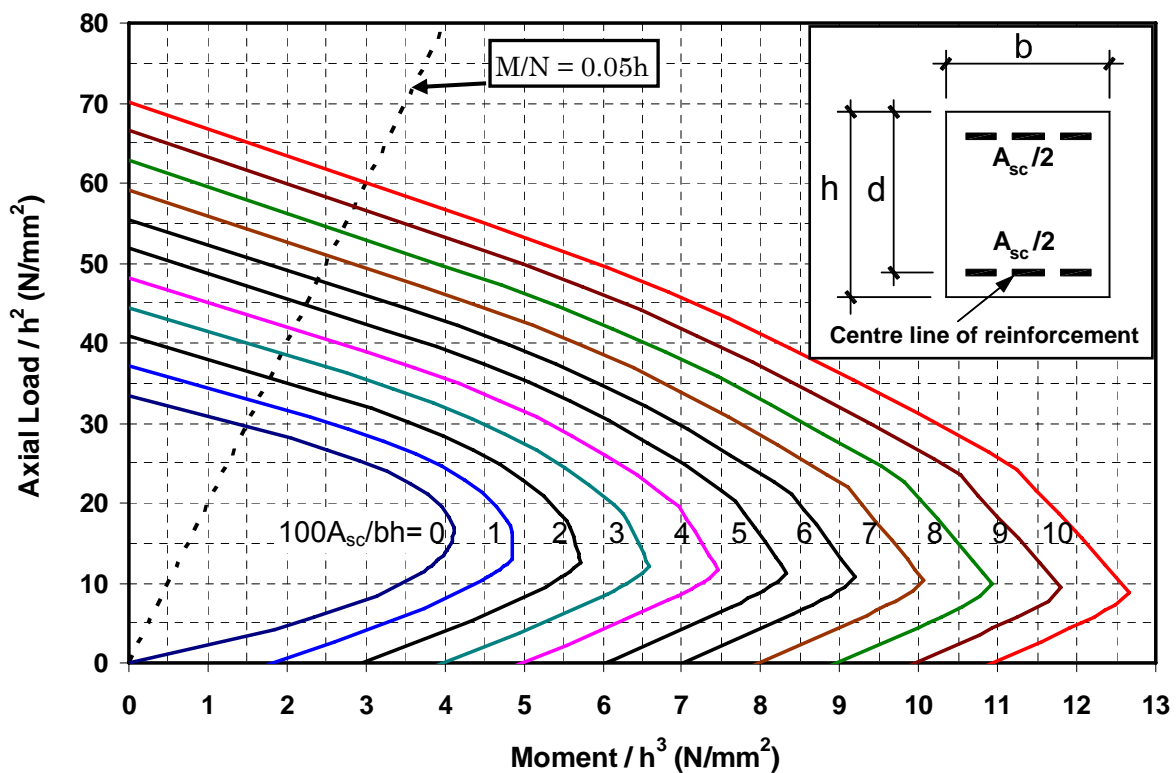


Chart 6.17

$f_{cu} = 75 \text{ N/mm}^2$
 $d/h = 0.80$

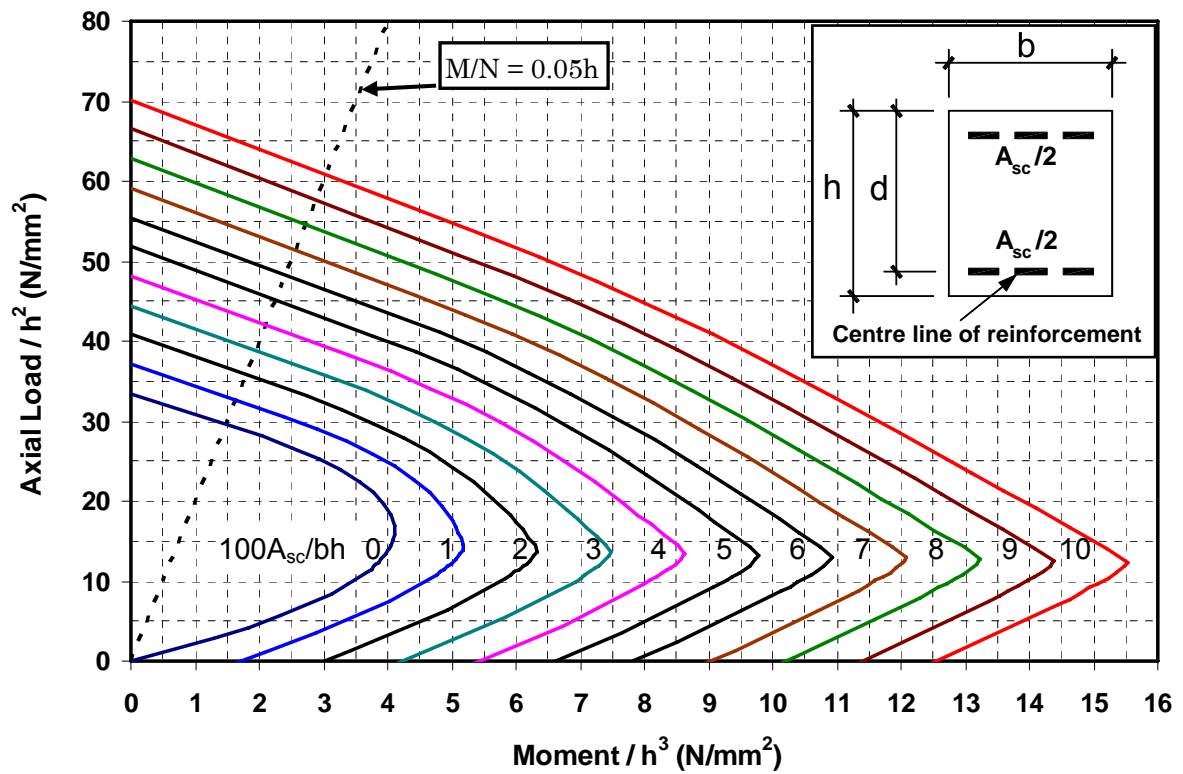


Chart 6.18

$f_{cu} = 75 \text{ N/mm}^2$
 $d/h = 0.85$

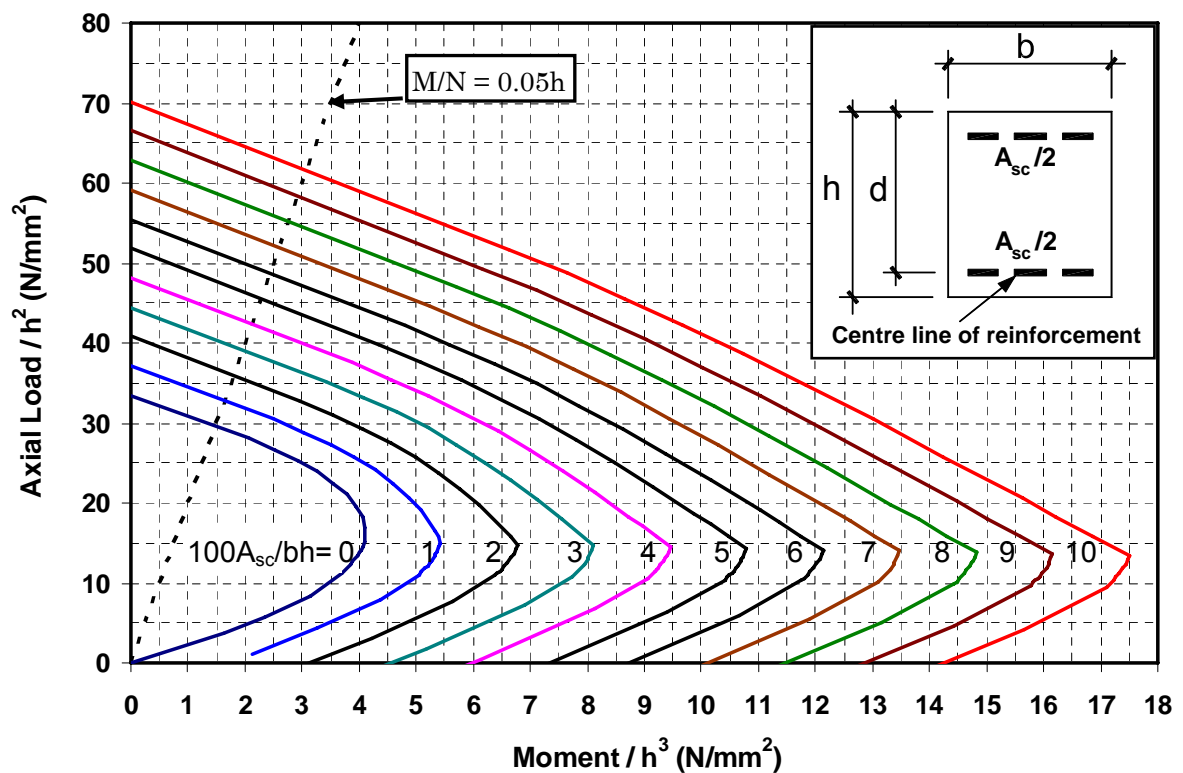


Chart 6.19

$f_{cu} = 75 \text{ N/mm}^2$
 $d/h = 0.90$

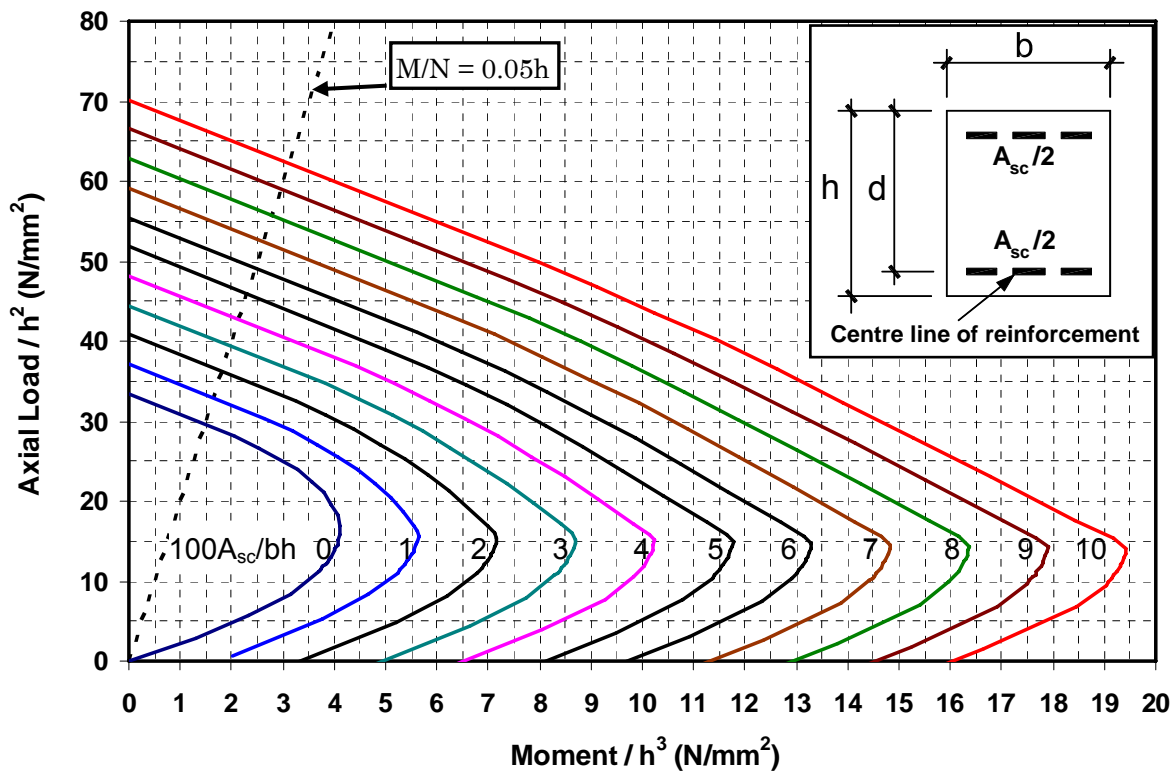


Chart 6.20

$f_{cu} = 75 \text{ N/mm}^2$
 $d/h = 0.95$

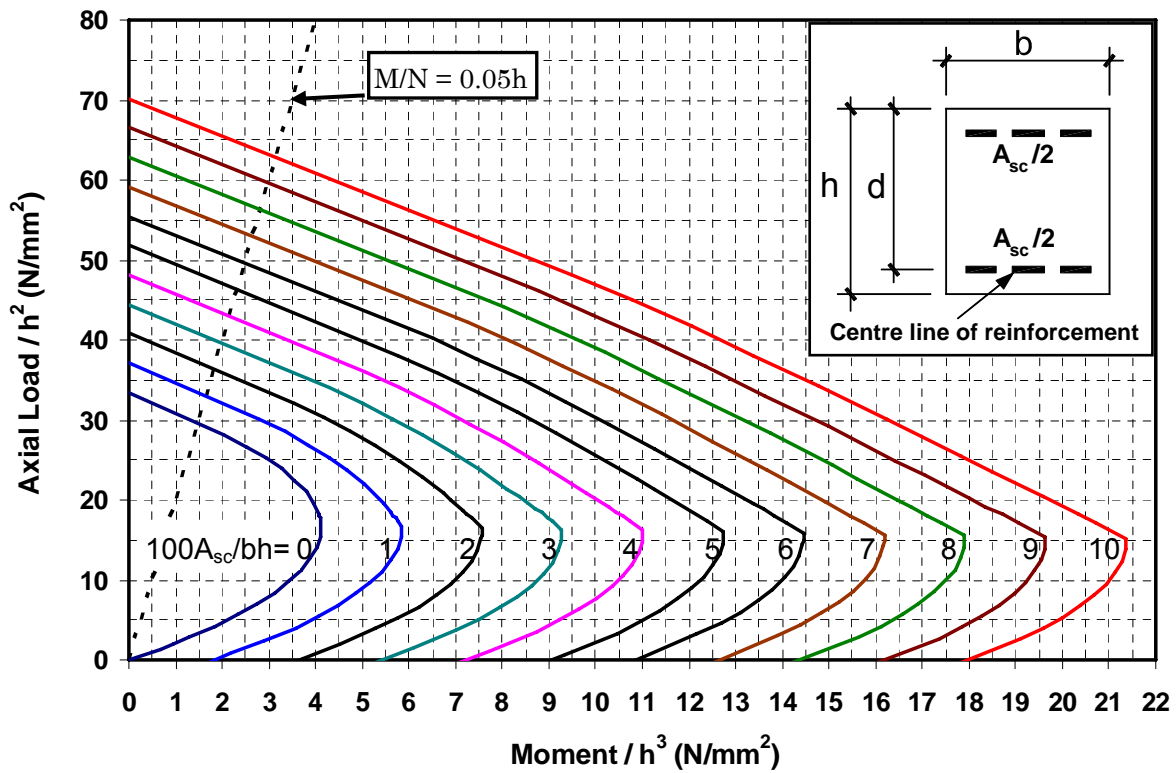


Chart 6.21

$f_{cu} = 80 \text{ N/mm}^2$
 $d/h = 0.75$

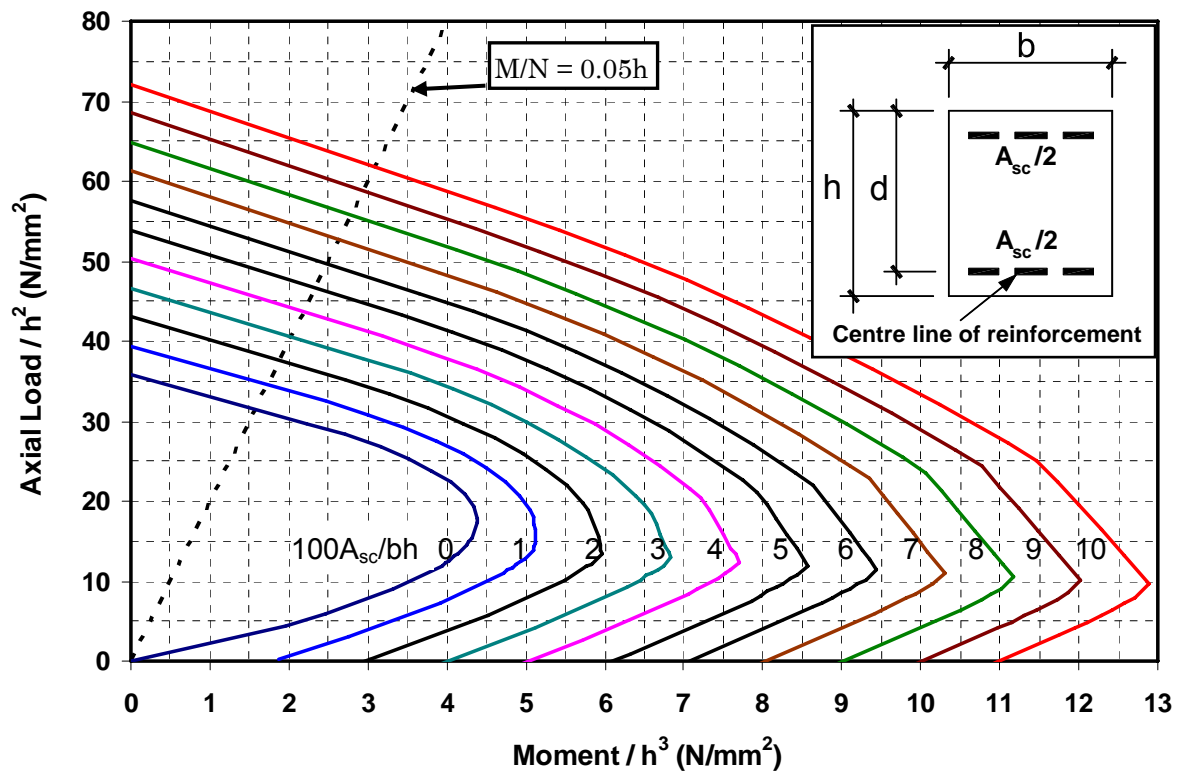


Chart 6.22

$f_{cu} = 80 \text{ N/mm}^2$
 $d/h = 0.80$

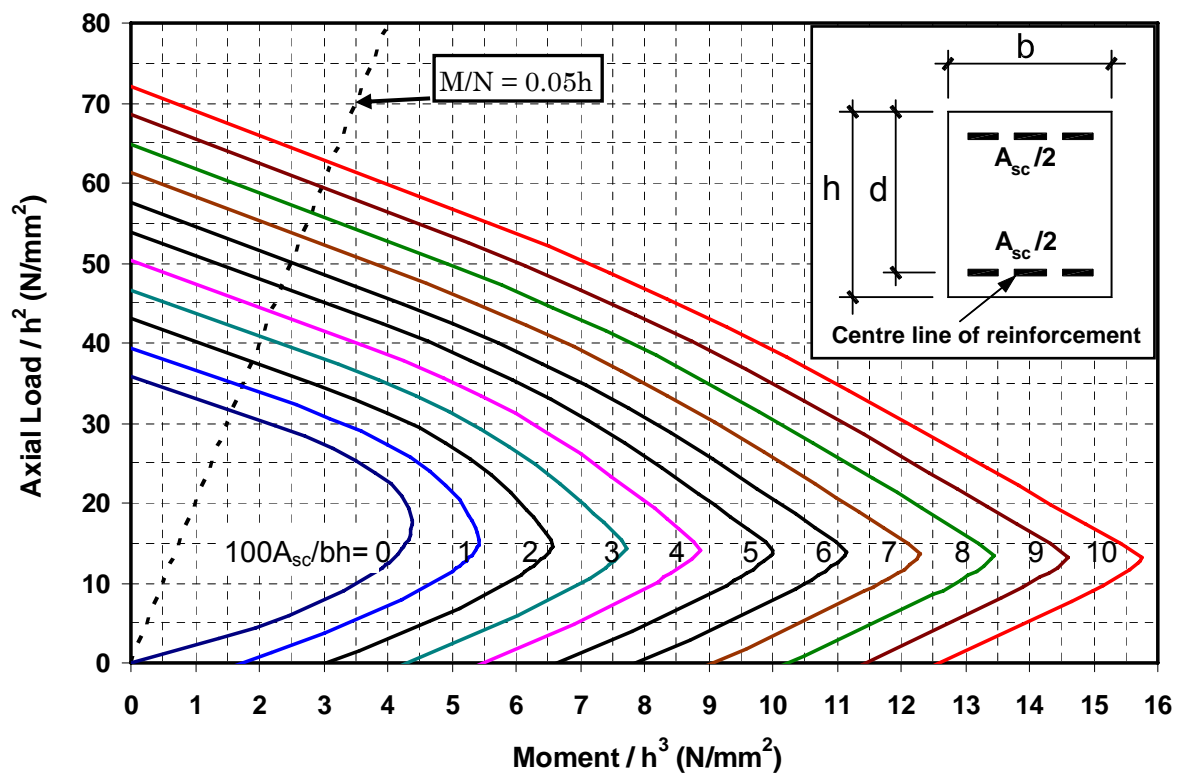


Chart 6.23

$f_{cu} = 80 \text{ N/mm}^2$
 $d/h = 0.85$

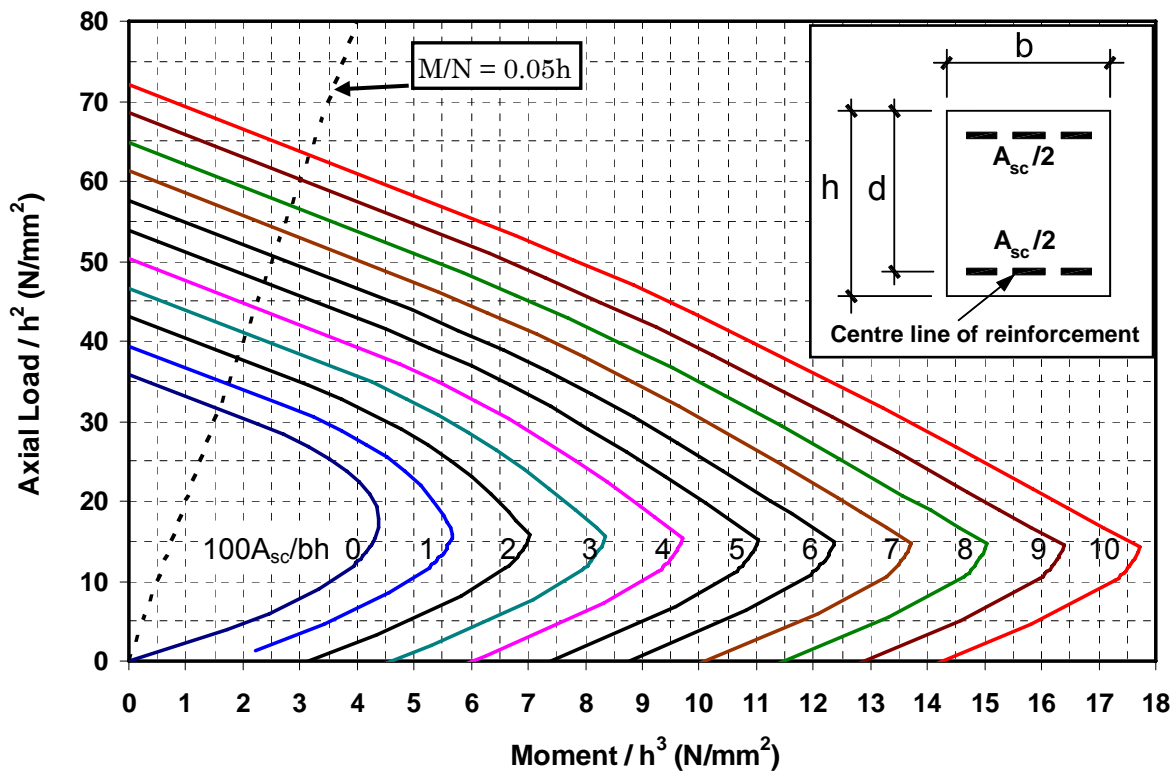


Chart 6.24

$f_{cu} = 80 \text{ N/mm}^2$
 $d/h = 0.90$

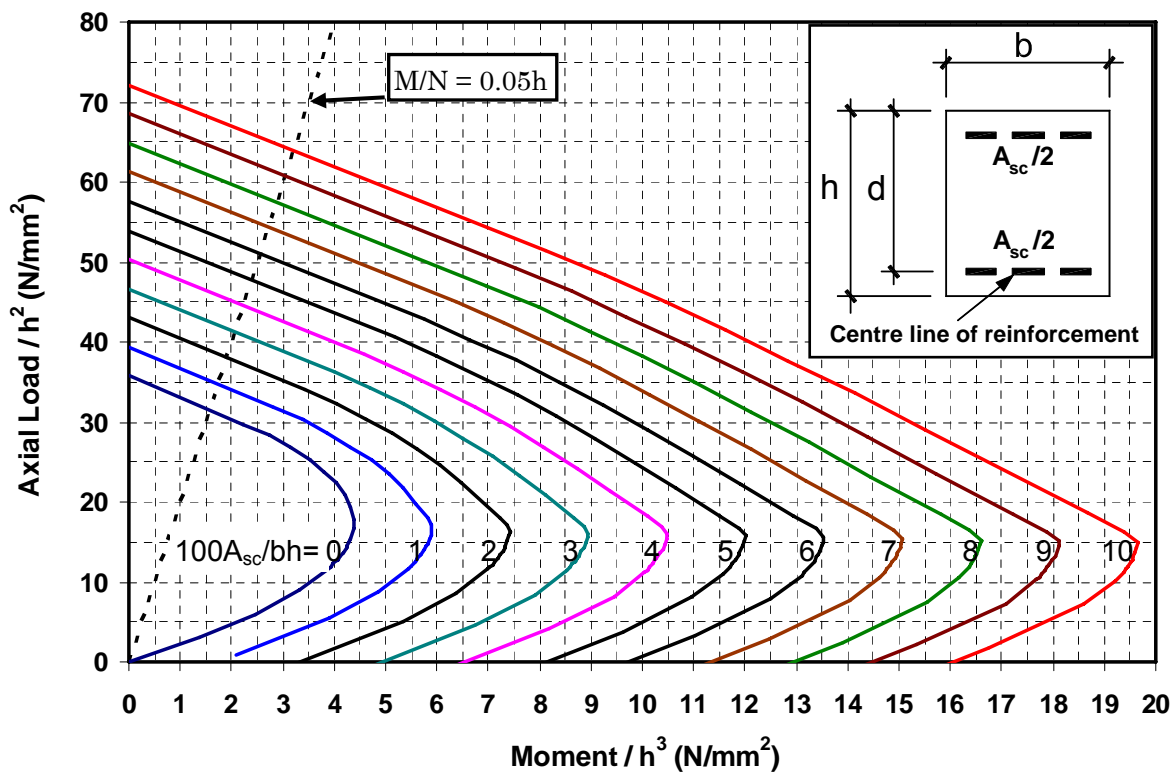


Chart 6.25

$f_{cu} = 80 \text{ N/mm}^2$
 $d/h = 0.95$

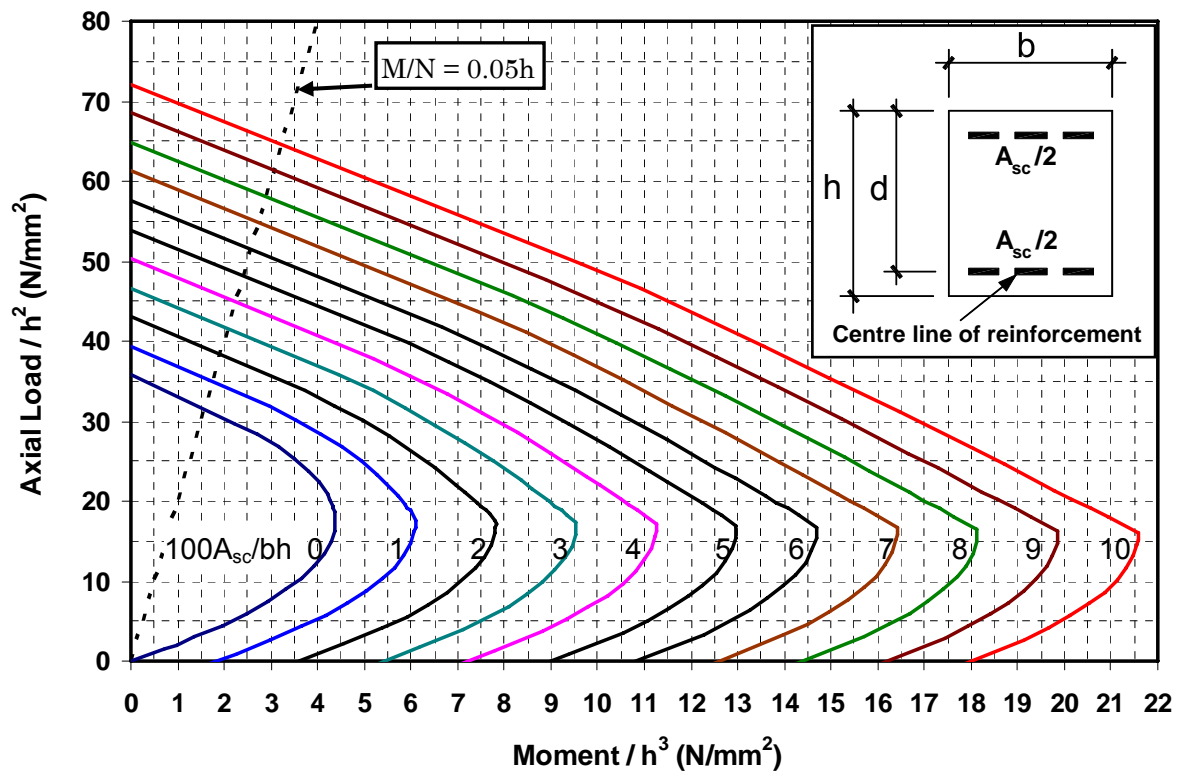


Chart 6.26

$f_{cu} = 85 \text{ N/mm}^2$
 $d/h = 0.75$

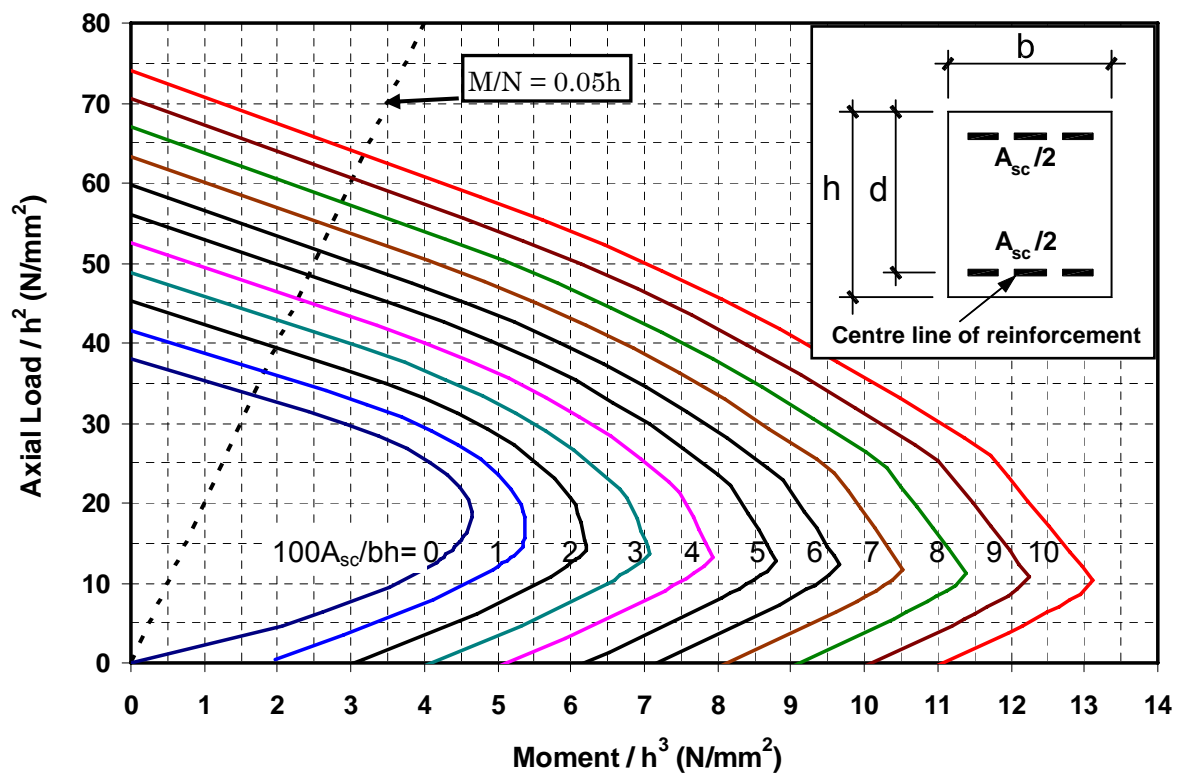


Chart 6.27

$f_{cu} = 85 \text{ N/mm}^2$
 $d/h = 0.80$

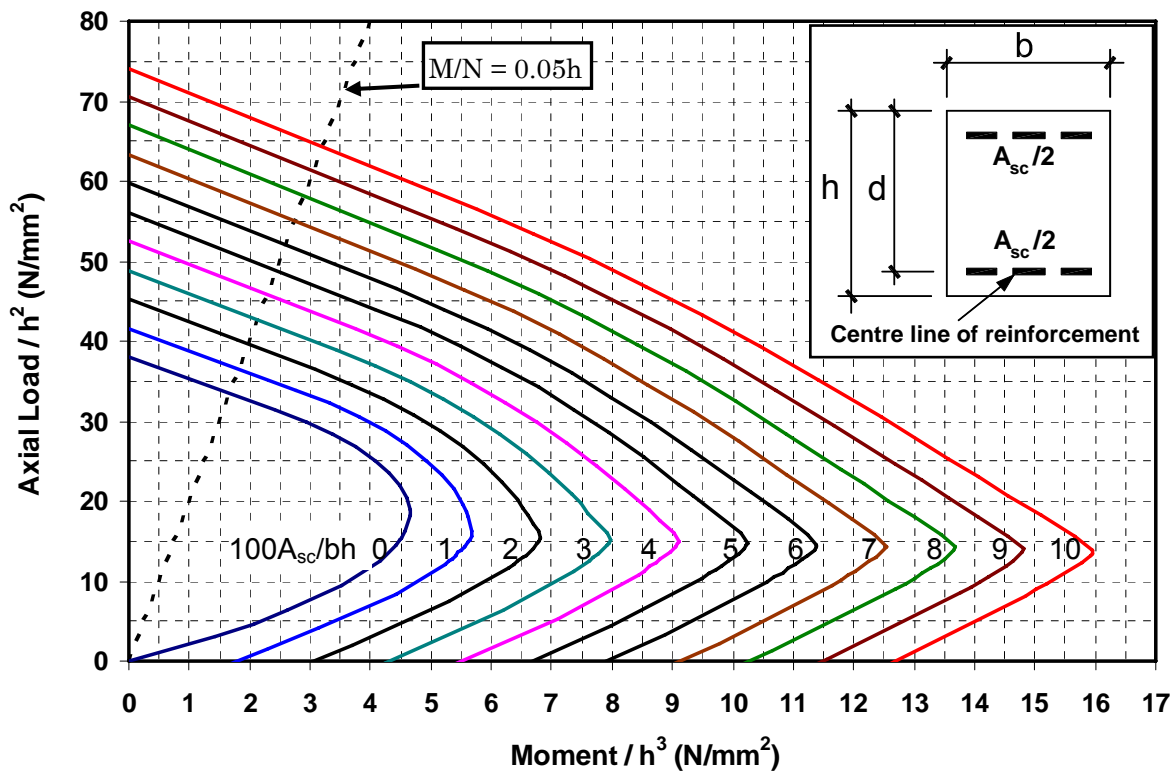


Chart 6.28

$f_{cu} = 85 \text{ N/mm}^2$
 $d/h = 0.85$

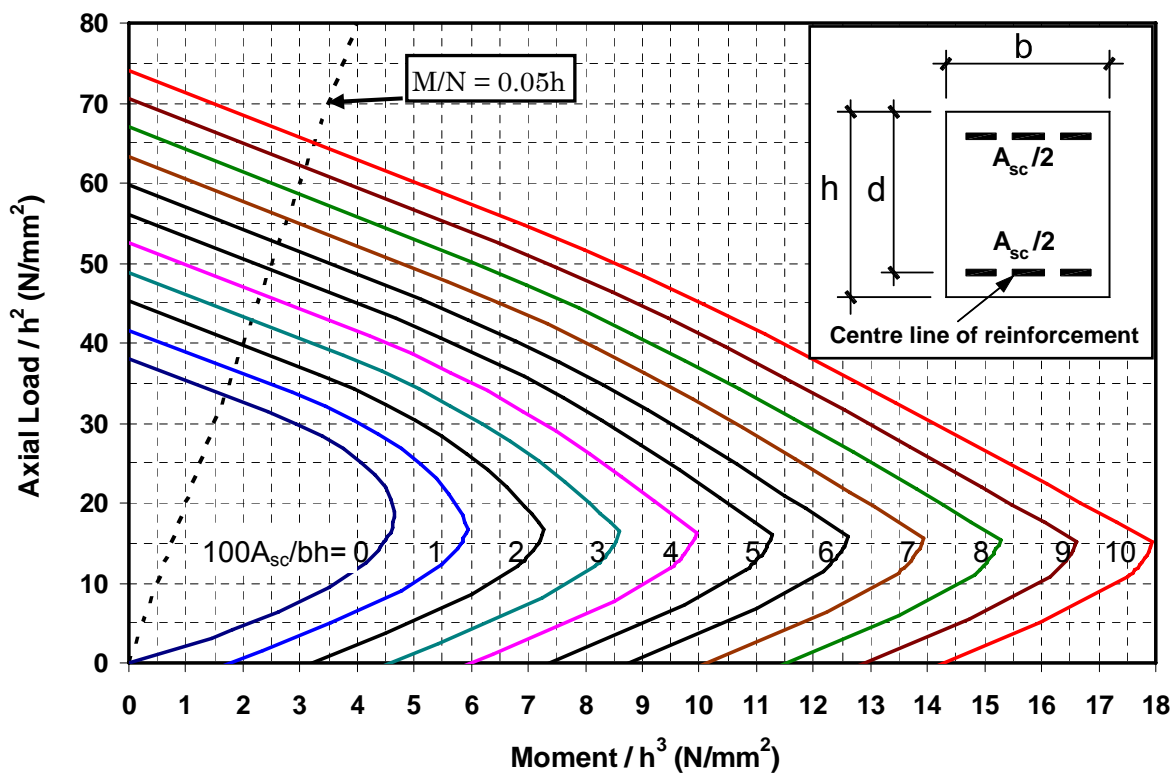


Chart 6.29

$f_{cu} = 85 \text{ N/mm}^2$
 $d/h = 0.90$

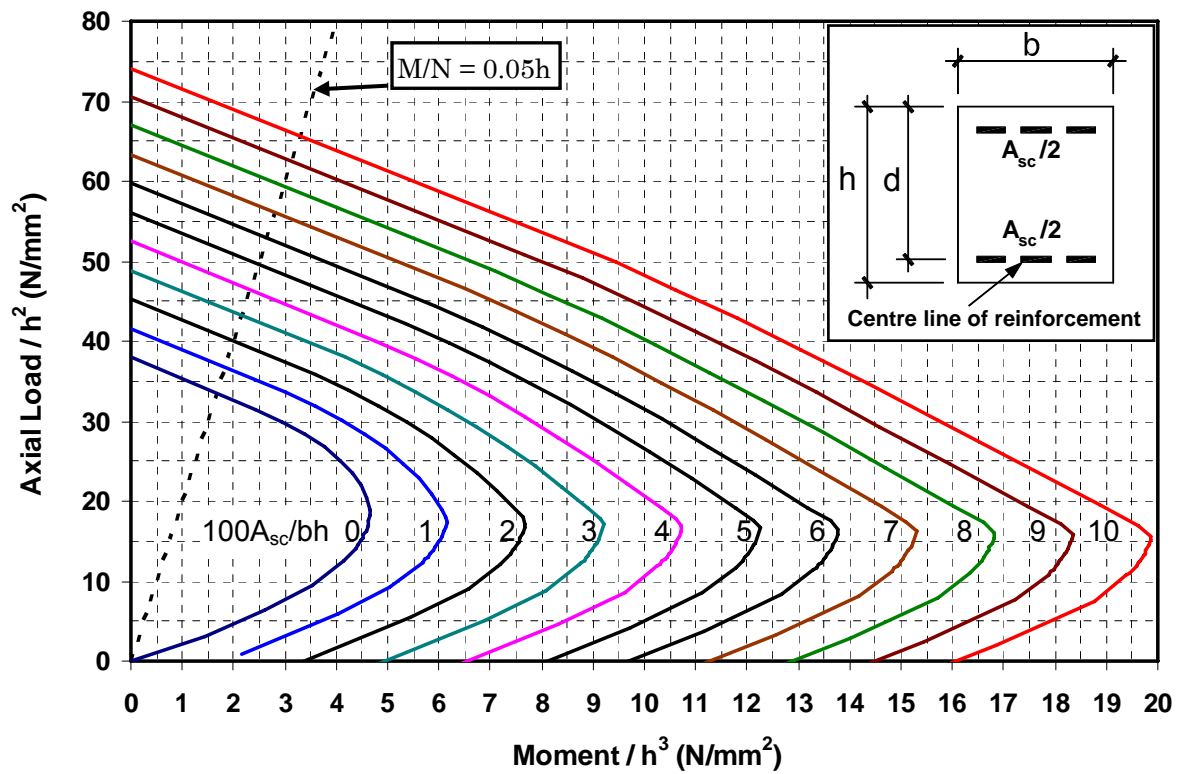


Chart 6.30

$f_{cu} = 85 \text{ N/mm}^2$
 $d/h = 0.95$

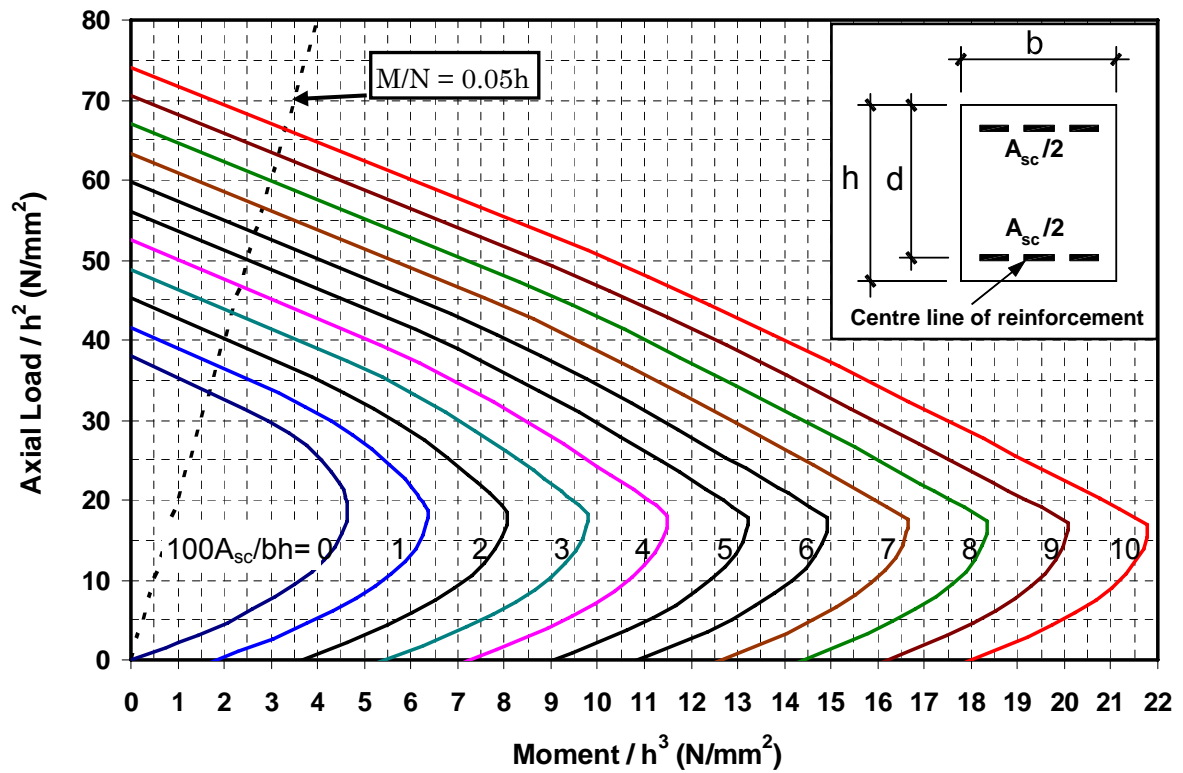


Chart 6.31

$f_{cu} = 90 \text{ N/mm}^2$
 $d/h = 0.75$

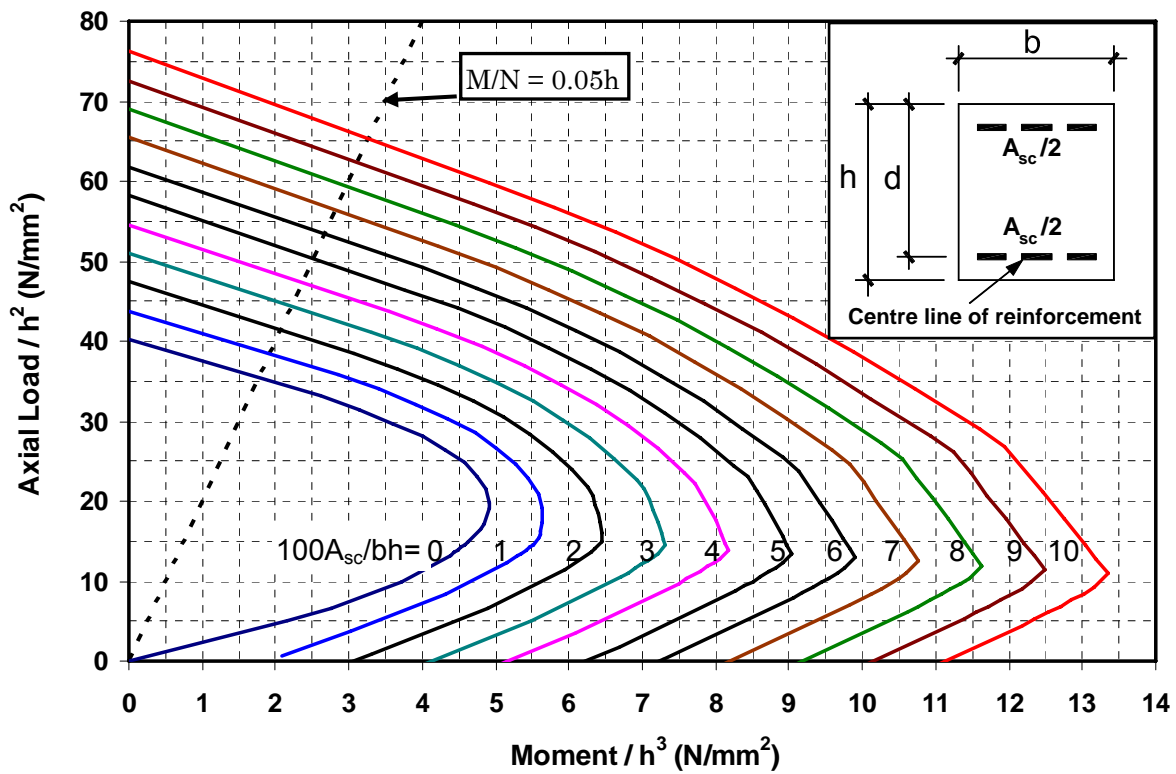


Chart 6.32

$f_{cu} = 90 \text{ N/mm}^2$
 $d/h = 0.80$

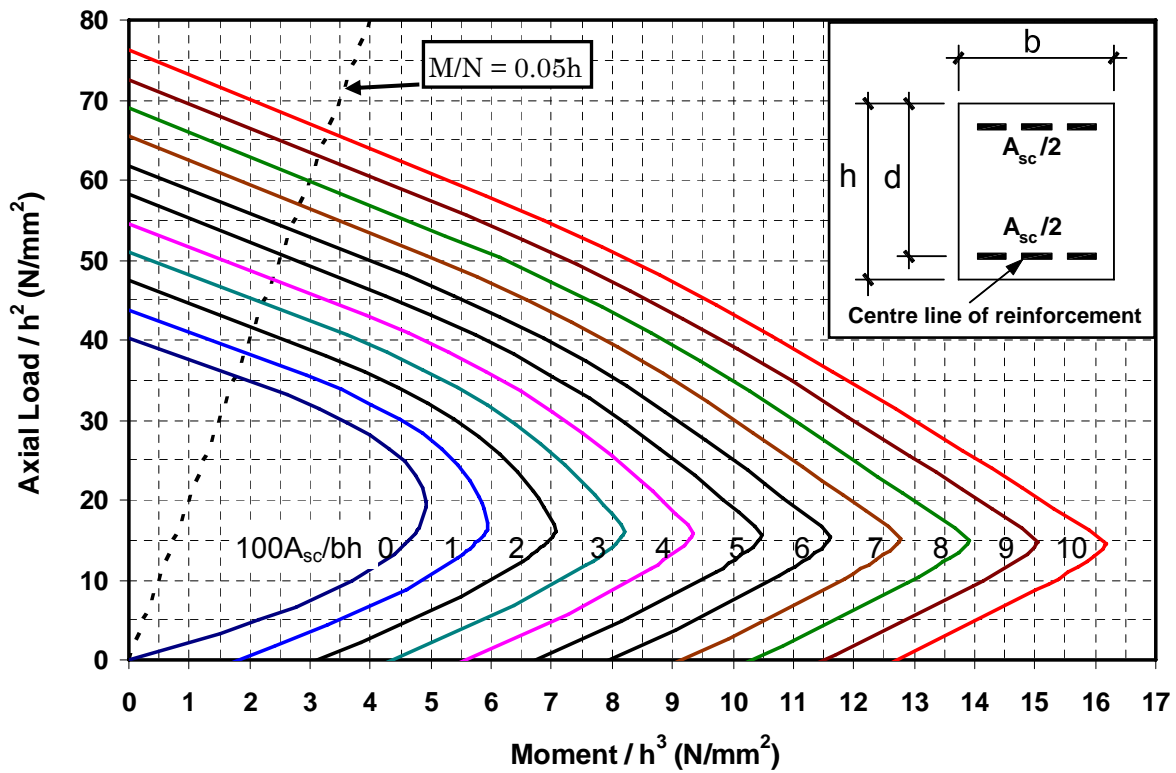


Chart 6.33

$f_{cu} = 90 \text{ N/mm}^2$
 $d/h = 0.85$

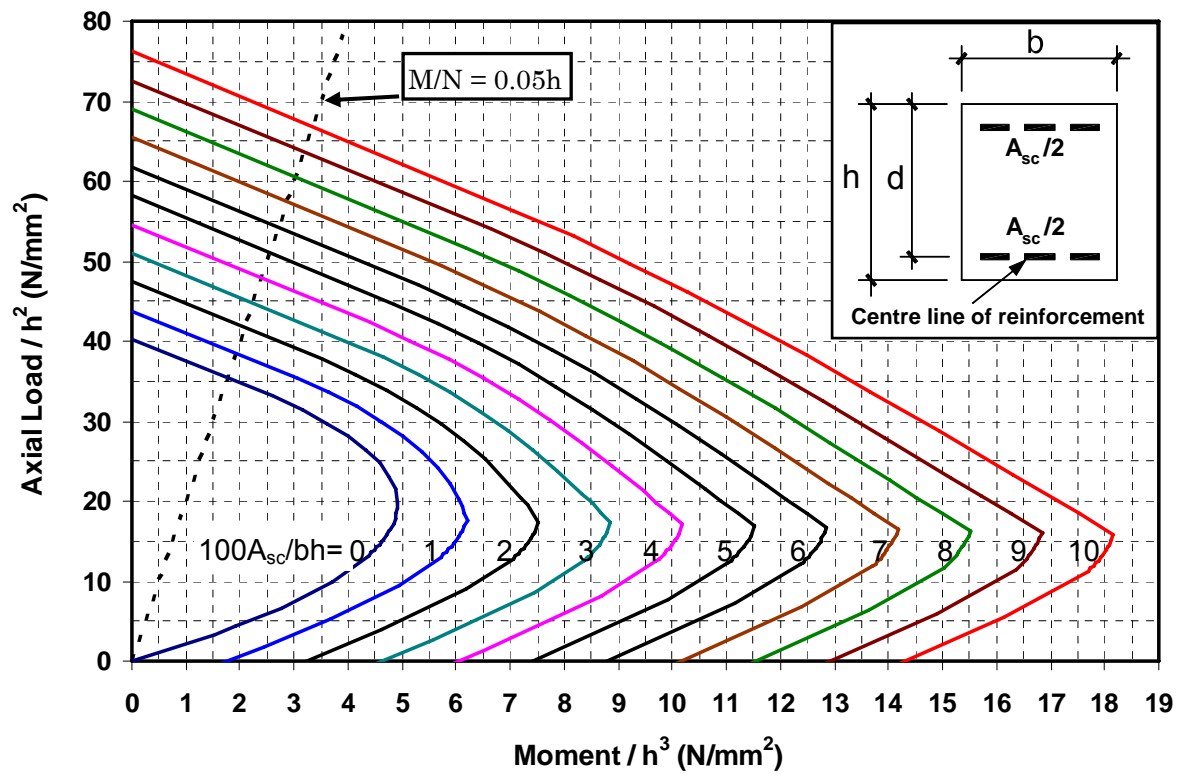


Chart 6.34

$f_{cu} = 90 \text{ N/mm}^2$
 $d/h = 0.90$

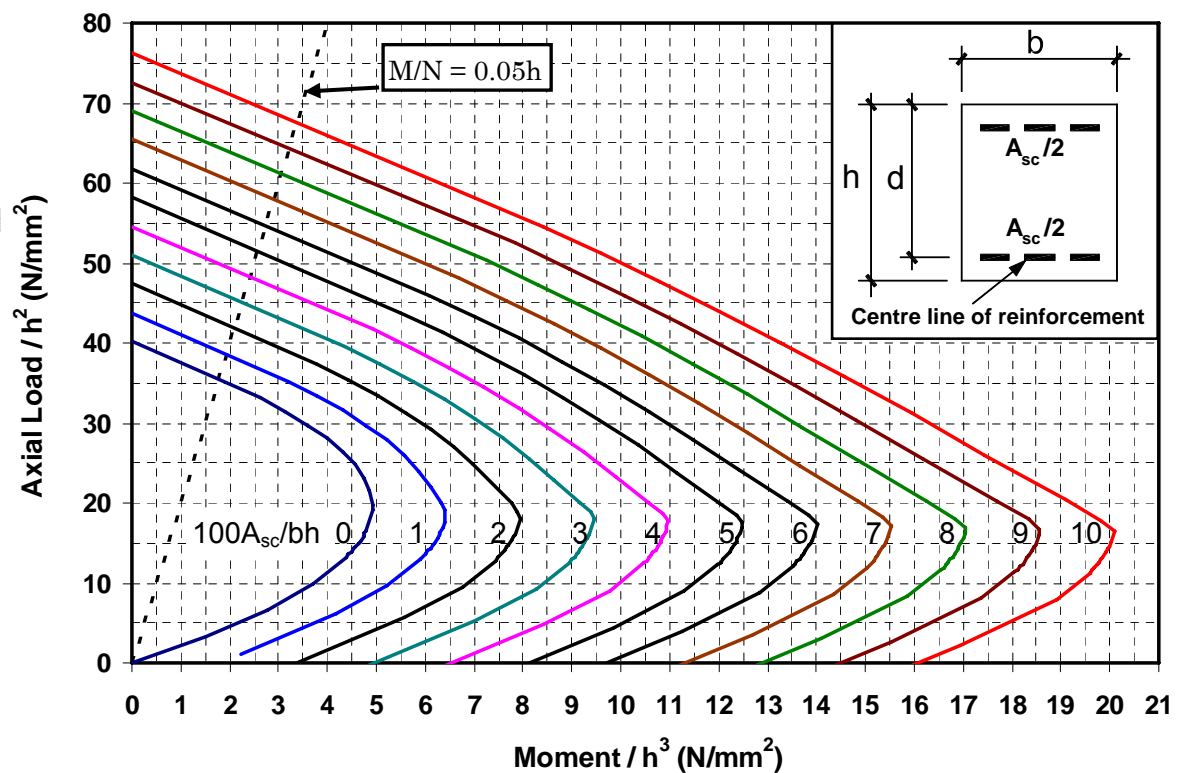


Chart 6.35

$f_{cu} = 90 \text{ N/mm}^2$
 $d/h = 0.95$

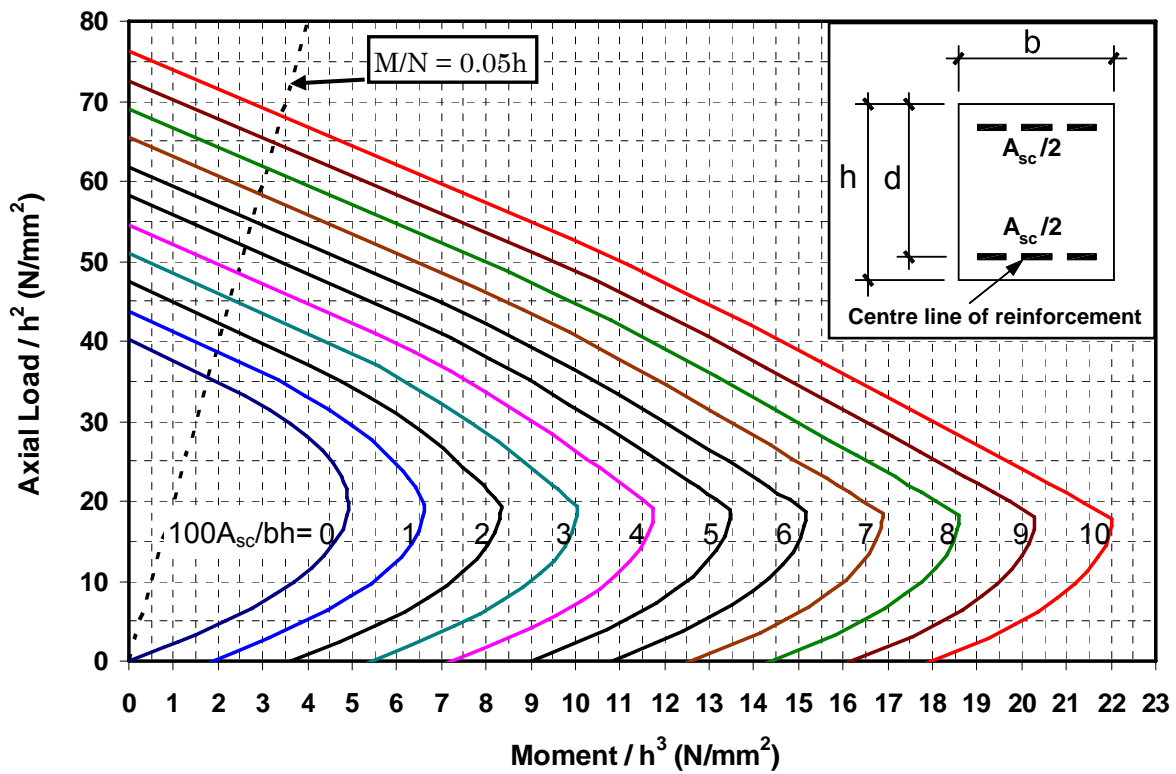


Chart 6.36

$f_{cu} = 95 \text{ N/mm}^2$
 $d/h = 0.75$

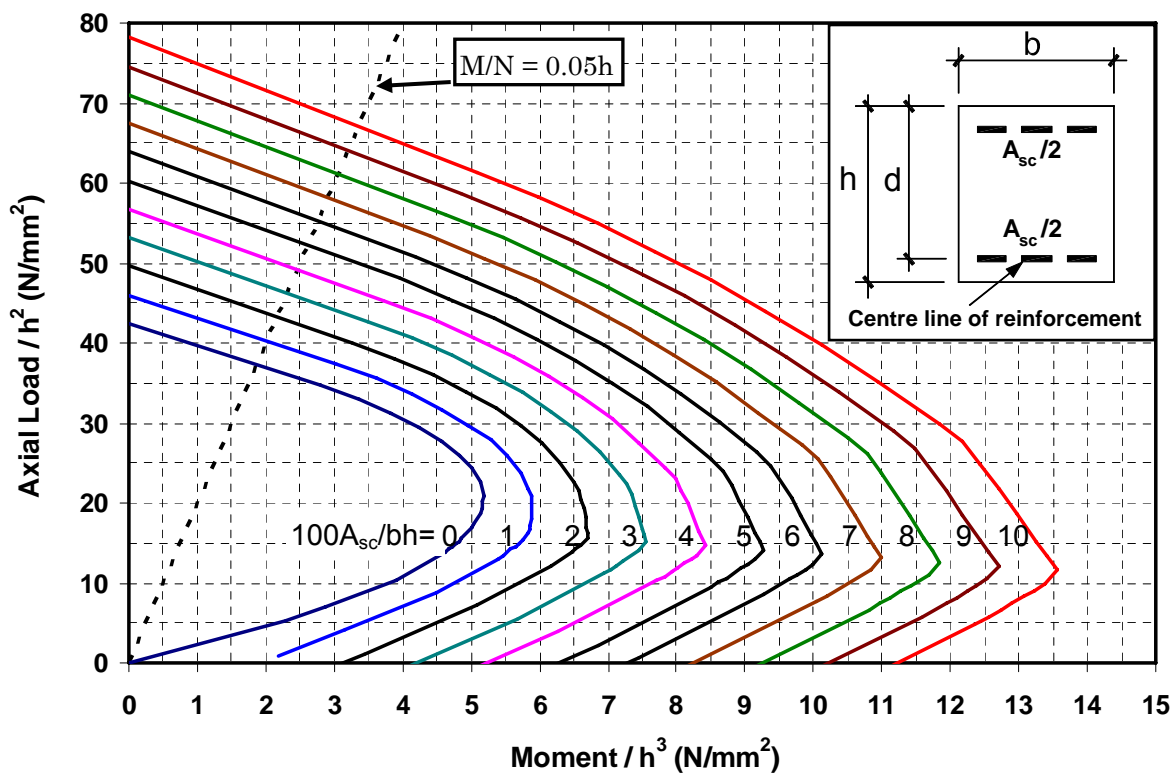


Chart 6.37

$f_{cu} = 95 \text{ N/mm}^2$
 $d/h = 0.80$

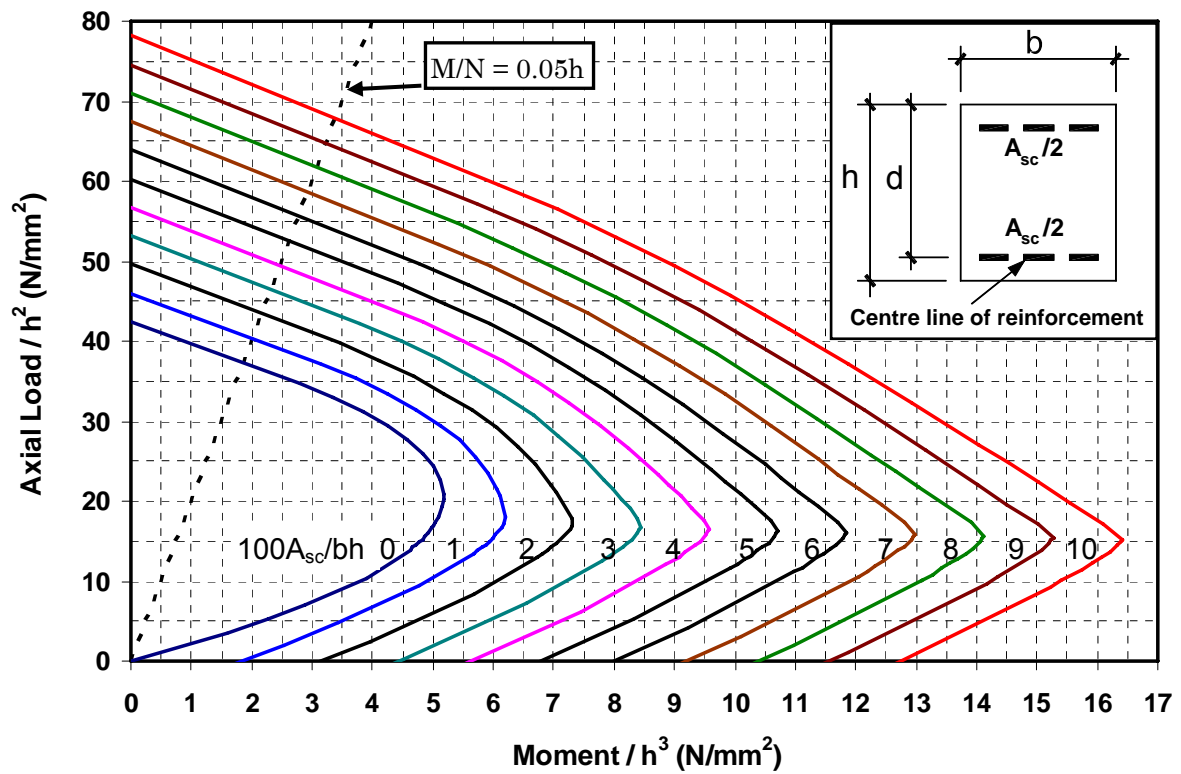


Chart 6.38

$f_{cu} = 95 \text{ N/mm}^2$
 $d/h = 0.85$

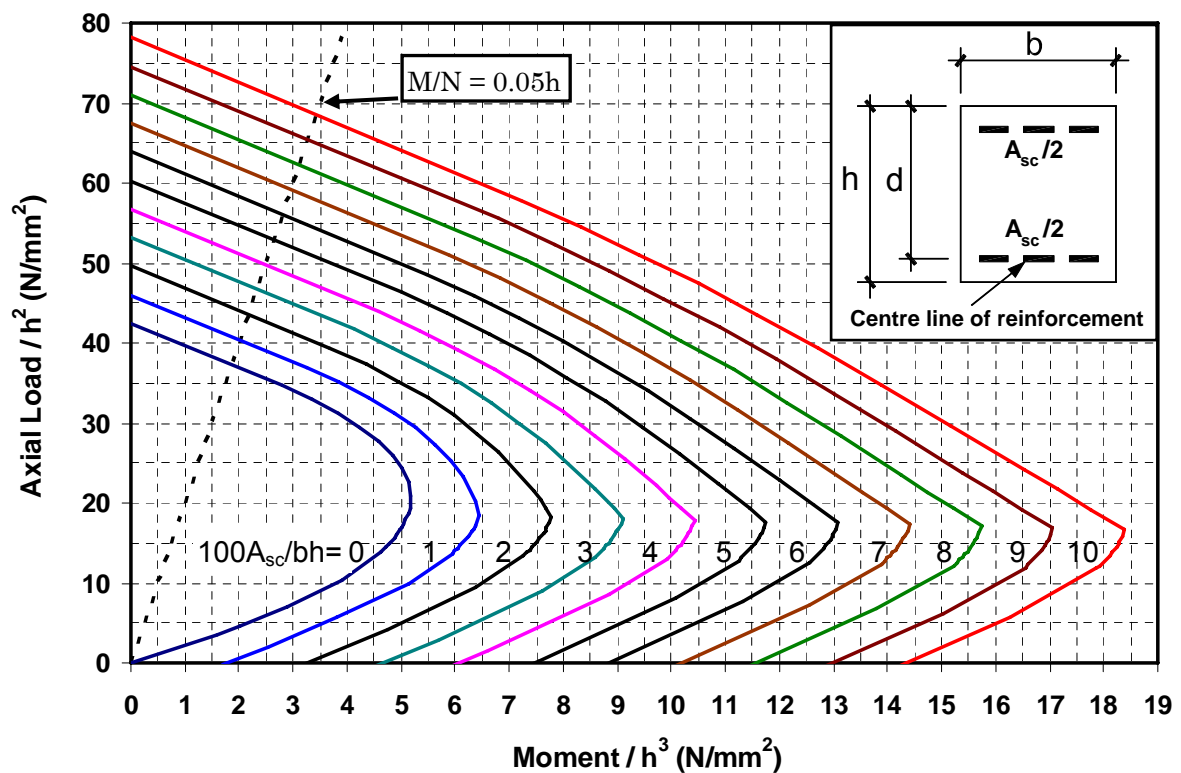


Chart 6.39

$f_{cu} = 95 \text{ N/mm}^2$
 $d/h = 0.90$

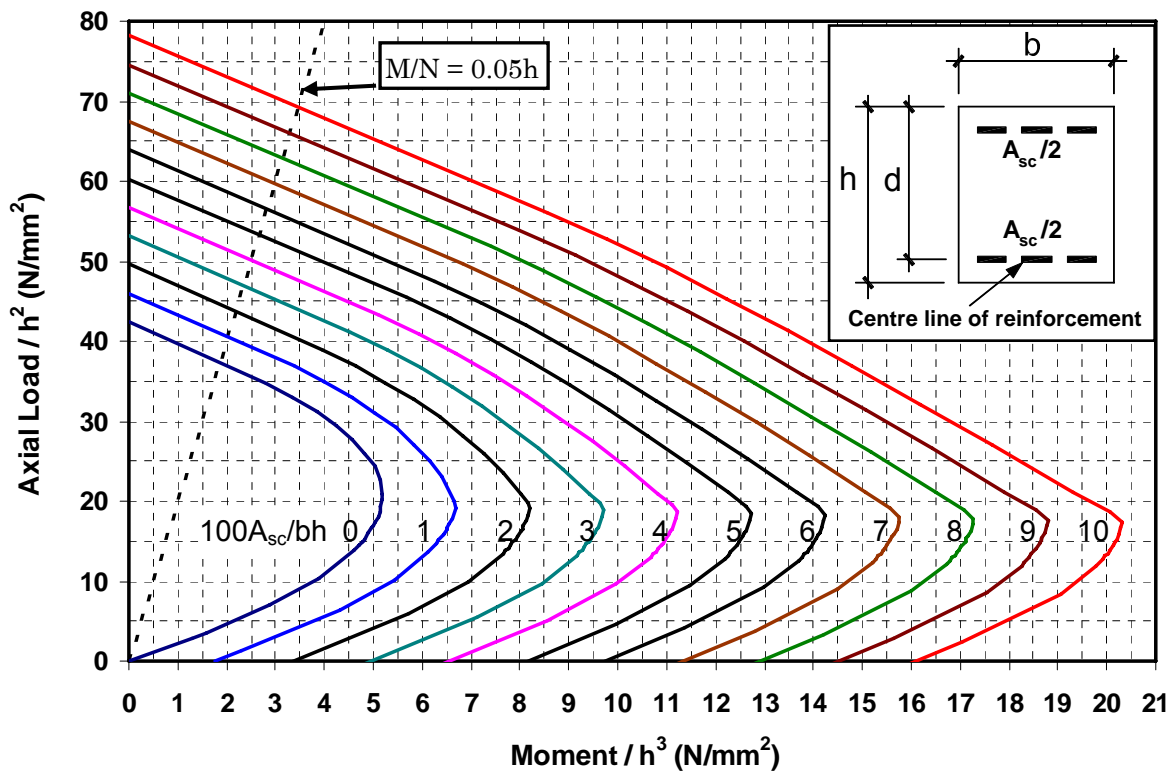


Chart 6.40

$f_{cu} = 95 \text{ N/mm}^2$
 $d/h = 0.95$

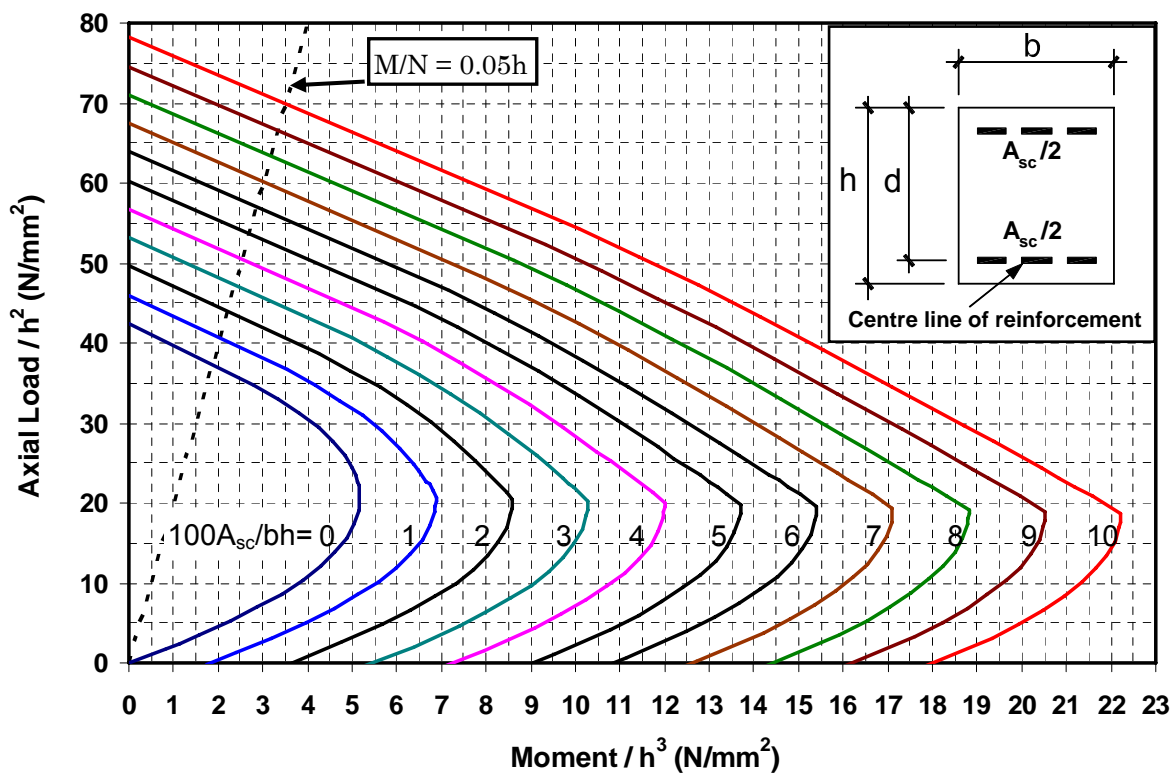


Chart 6.41

$f_{cu} = 100 \text{ N/mm}^2$
 $d/h = 0.75$

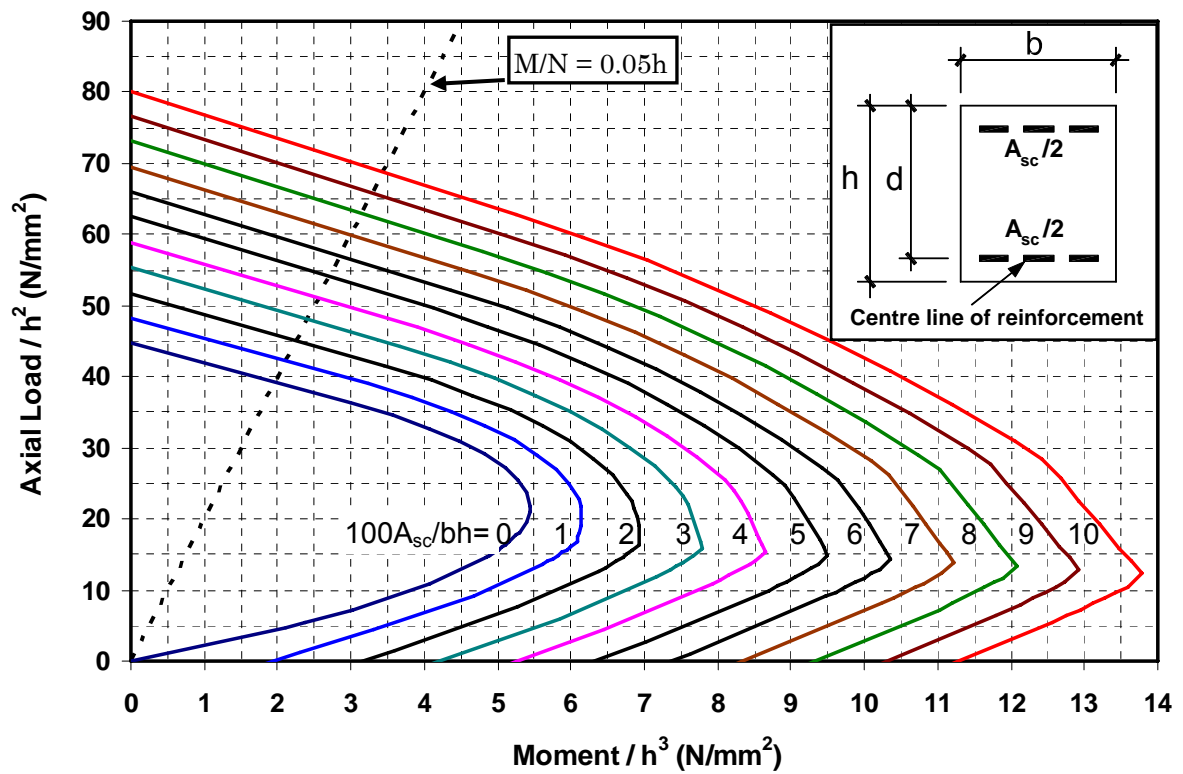


Chart 6.42

$f_{cu} = 100 \text{ N/mm}^2$
 $d/h = 0.80$

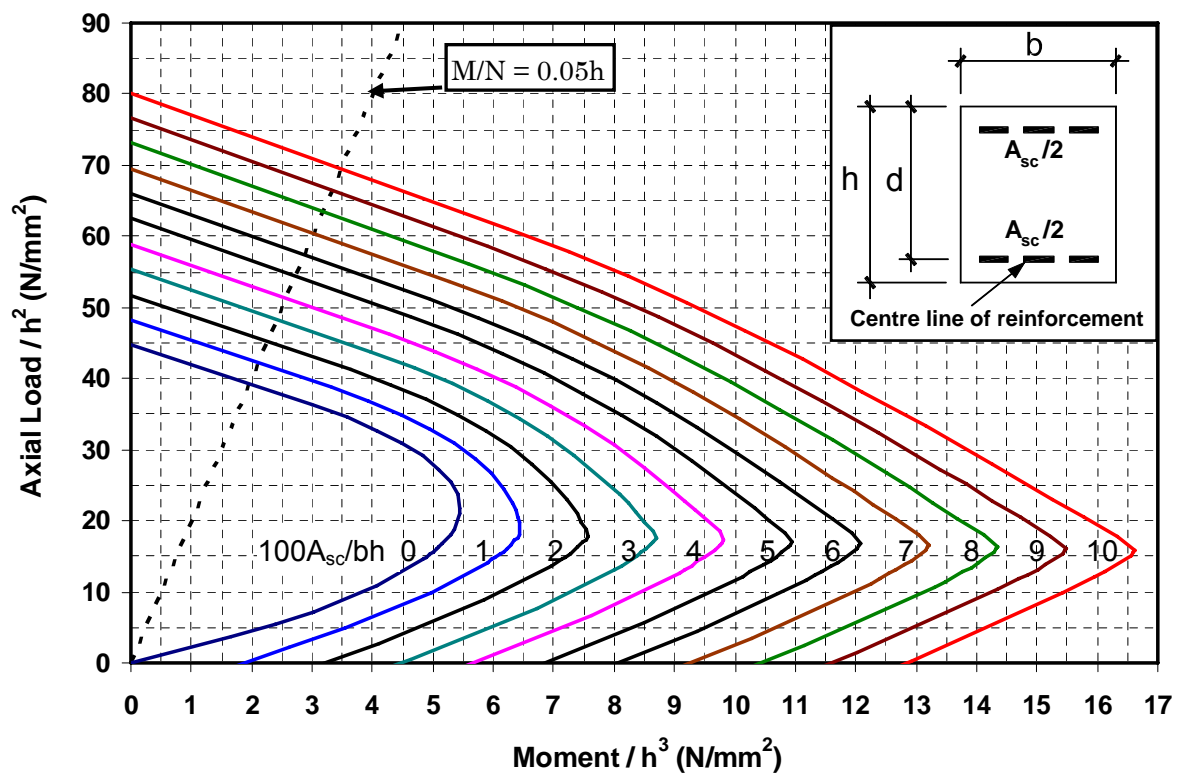


Chart 6.43

$f_{cu} = 100 \text{ N/mm}^2$
 $d/h = 0.85$

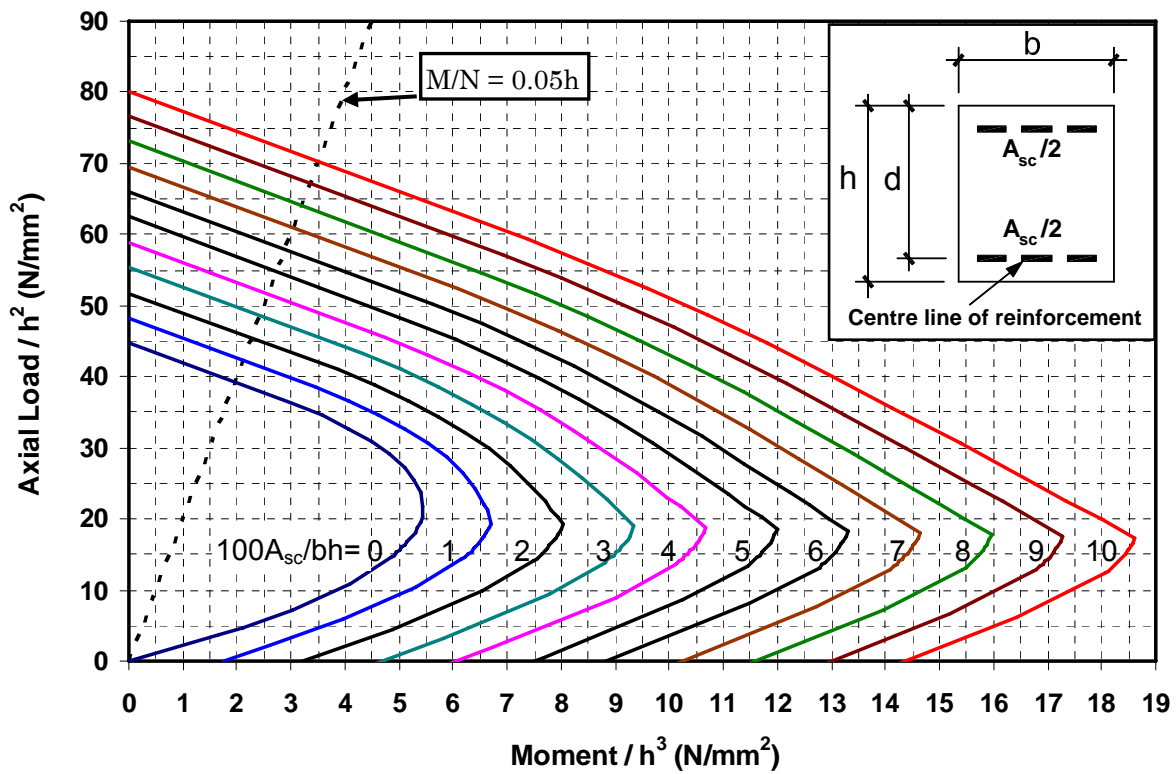


Chart 6.44

$f_{cu} = 100 \text{ N/mm}^2$
 $d/h = 0.90$

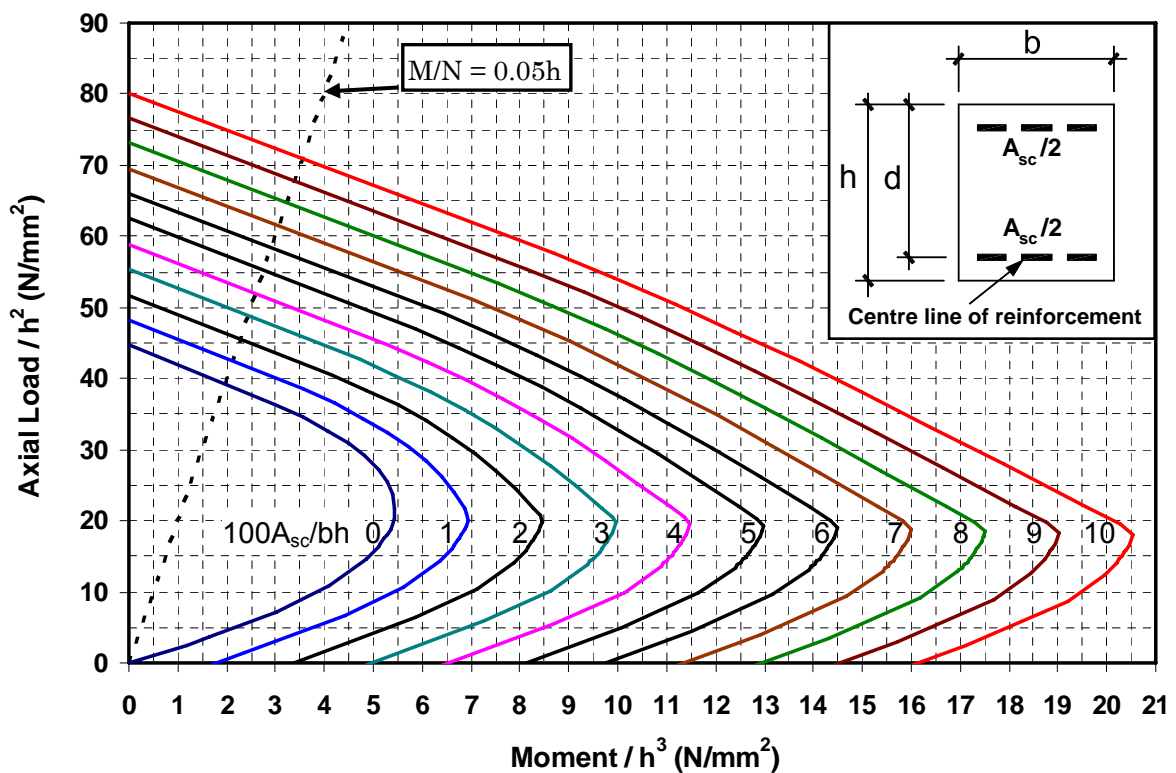


Chart 6.45

$f_{cu} = 100 \text{ N/mm}^2$
 $d/h = 0.95$

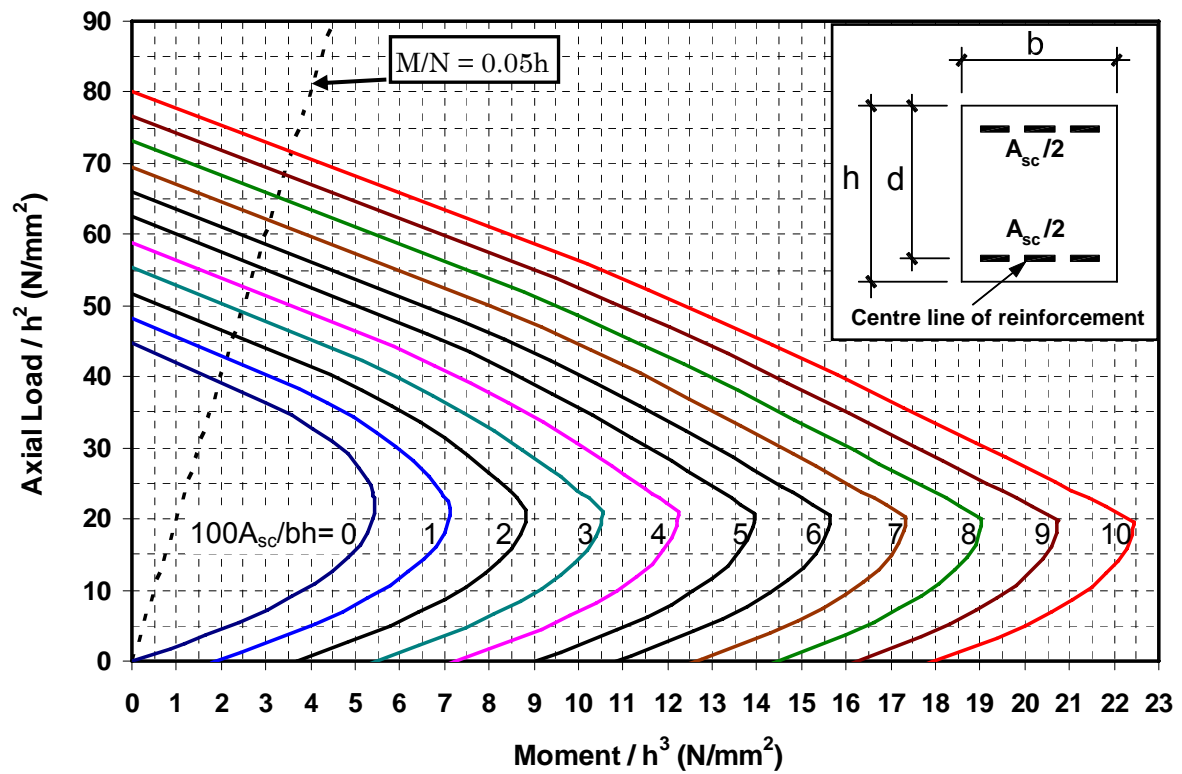


Chart 6.46

$f_{cu} = 105 \text{ N/mm}^2$
 $d/h = 0.75$

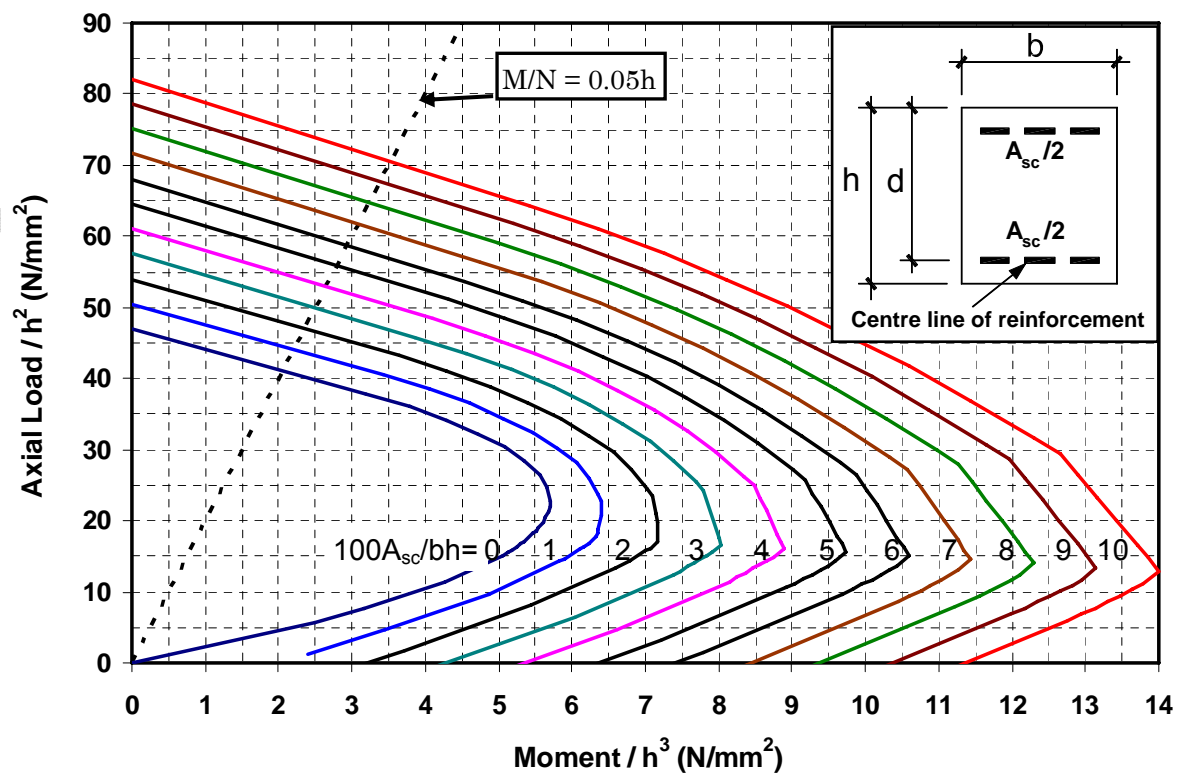


Chart 6.47

$f_{cu} = 105 \text{ N/mm}^2$
 $d/h = 0.80$

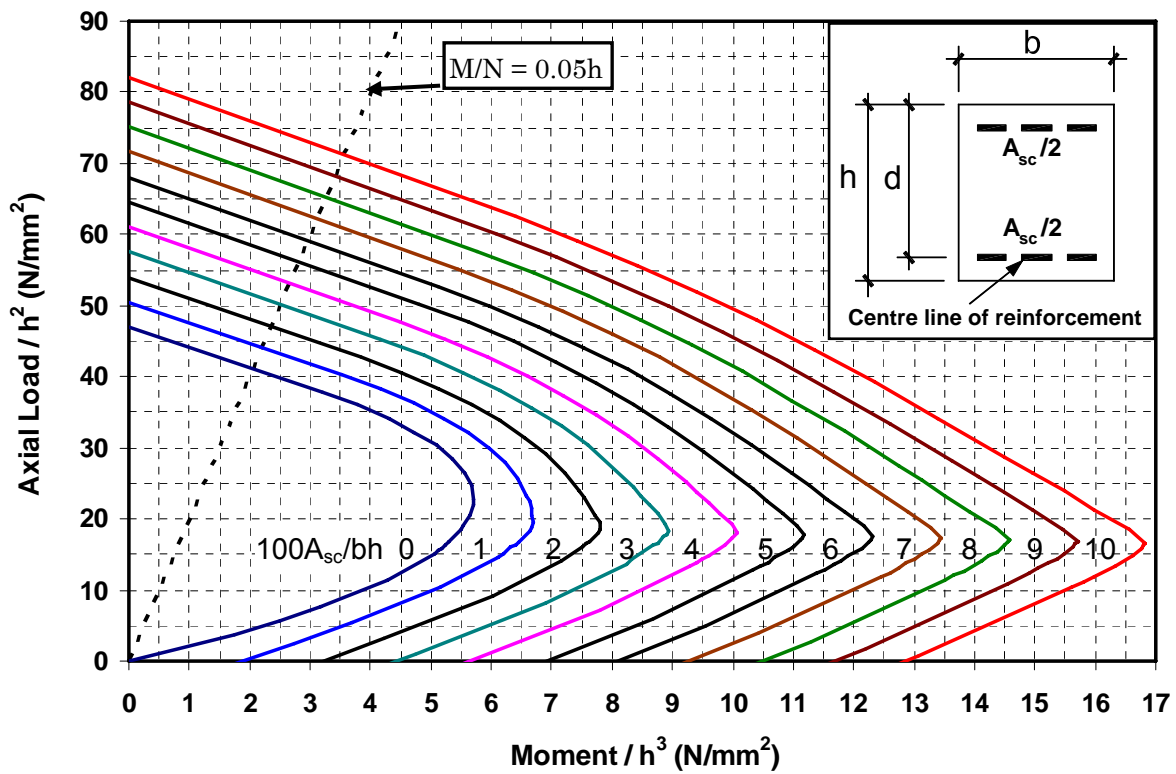


Chart 6.48

$f_{cu} = 105 \text{ N/mm}^2$
 $d/h = 0.85$

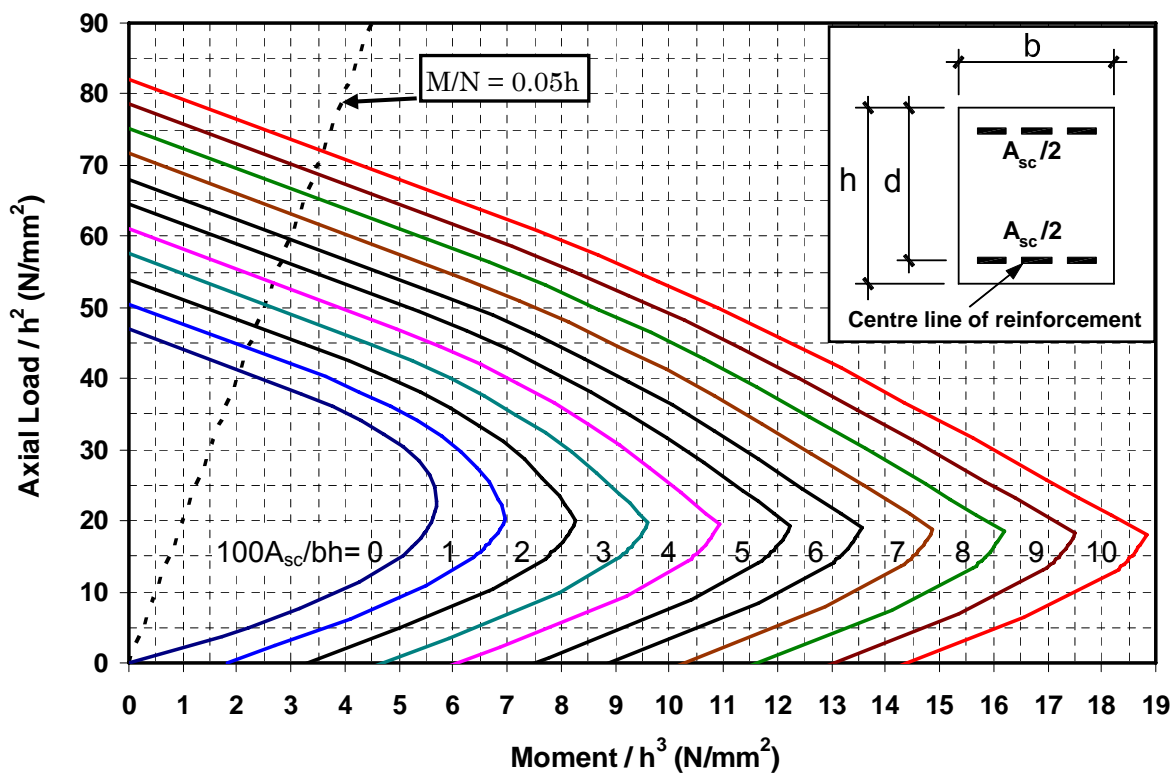


Chart 6.49

$f_{cu} = 105 \text{ N/mm}^2$
 $d/h = 0.90$

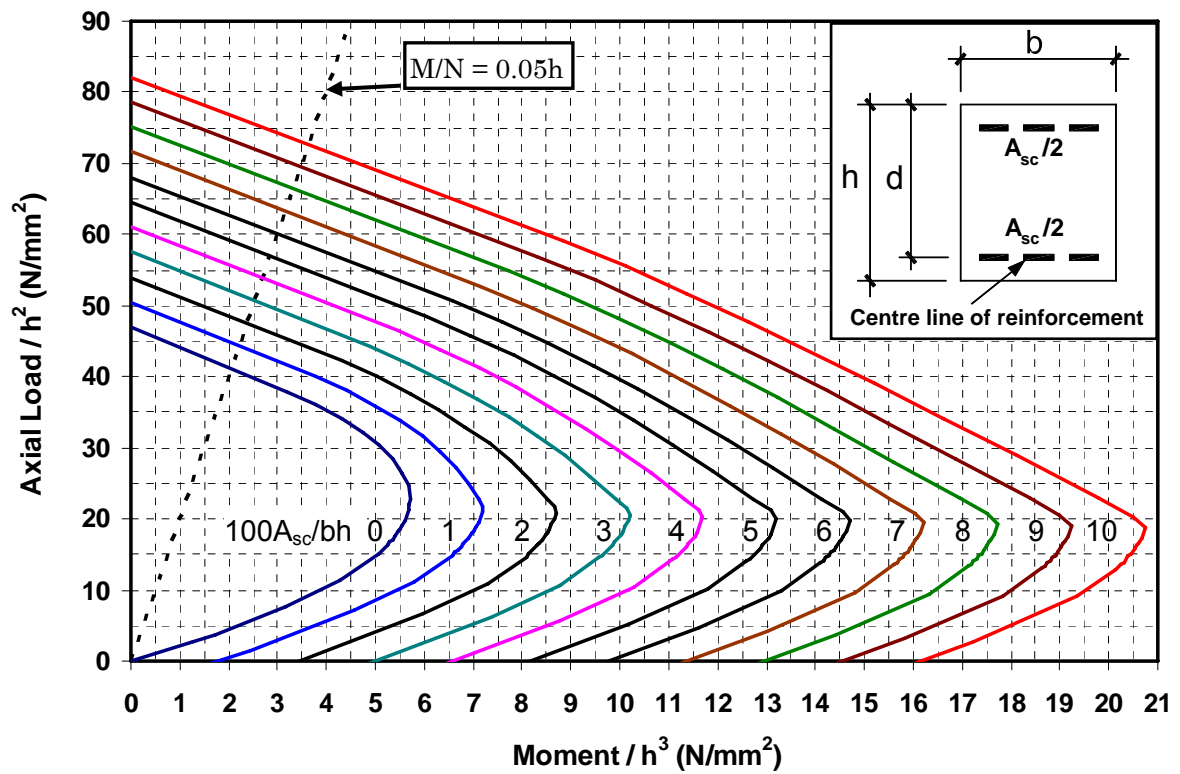
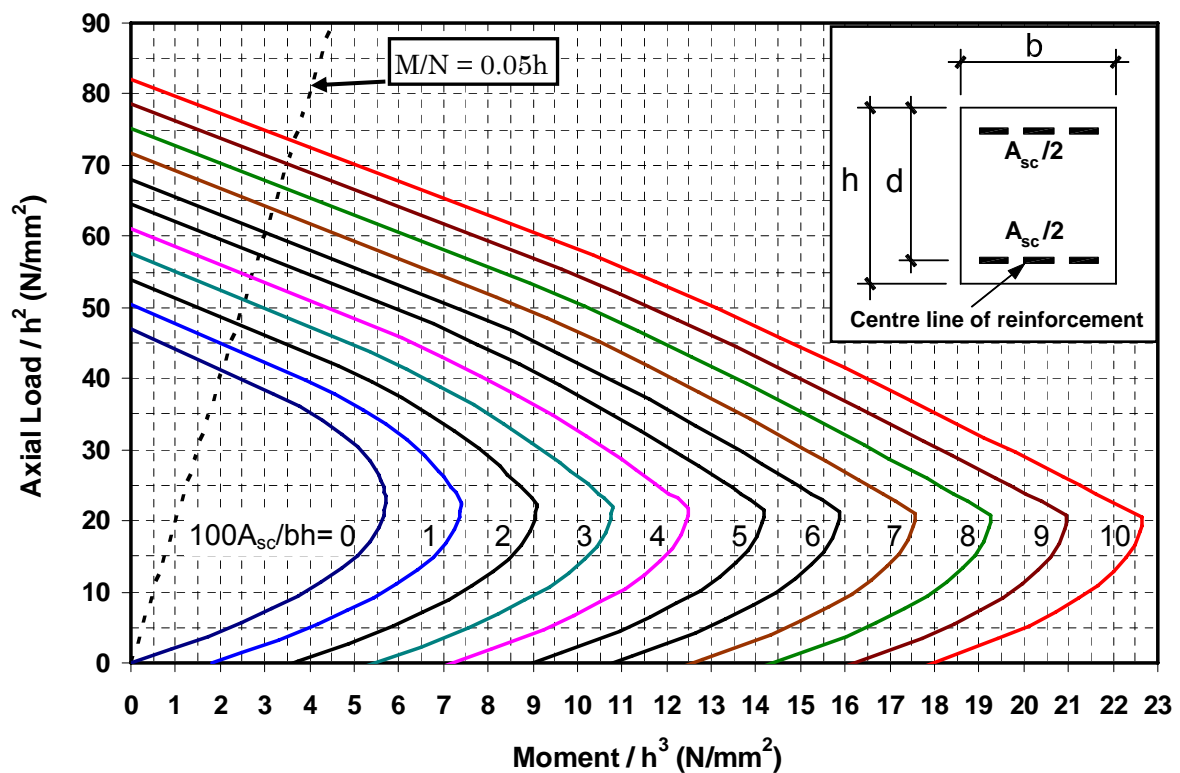


Chart 6.50

$f_{cu} = 105 \text{ N/mm}^2$
 $d/h = 0.95$



6.4 Design Charts for Circular Columns

A total of forty design charts for circular columns have been prepared for concrete strengths from 60 N/mm² to 100 N/mm² in steps of 5 N/mm². The value of h_s/h have been chosen between 0.6 and 0.9 in steps of 0.1. Reinforcement ratios have been taken from 1.0 to 10% in steps of 1%. Two kinds of reinforcement arrangement are assumed: 16-bar equally distributed arrangement for reinforcement ratios between 1% and 5%, and 32-bar equally distributed arrangement for reinforcement ratios between 6% and 10%.

For all charts f_y is taken as 460 N/mm² and the partial safety factor for reinforcement is $\gamma_m = 1.15$. Modulus of elasticity of reinforcement steel has been taken as 200,000 N/mm².

The parabolic-rectangle stress block was used for concrete stress distribution. A four-point numerical integration is employed in the calculation of the contribution of concrete force and moment.

Symbols used in the charts are:

M: moment about the centroid of the cross-section

N: axial load

A_{sc} : total cross-sectional area of longitudinal reinforcement

A_c : cross-sectional area of the column

Chart 6.51

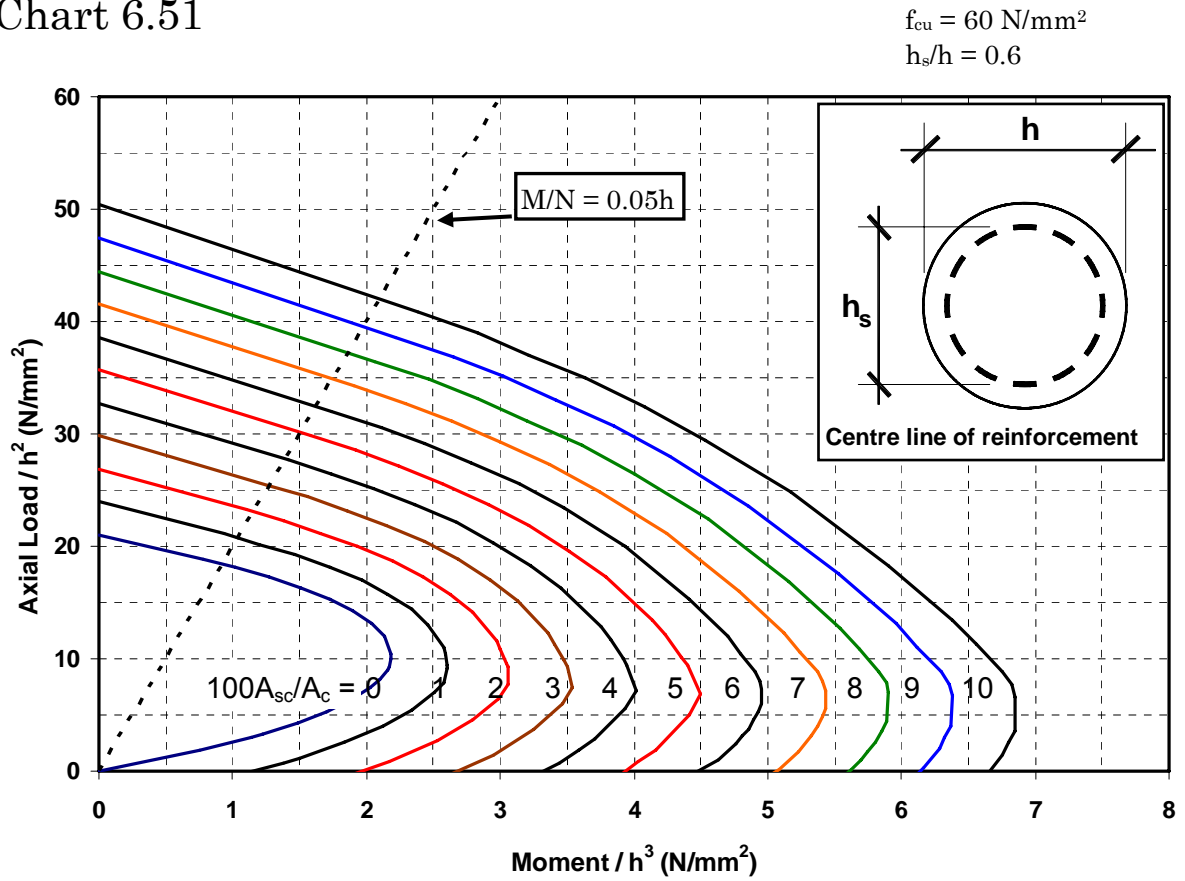


Chart 6.52

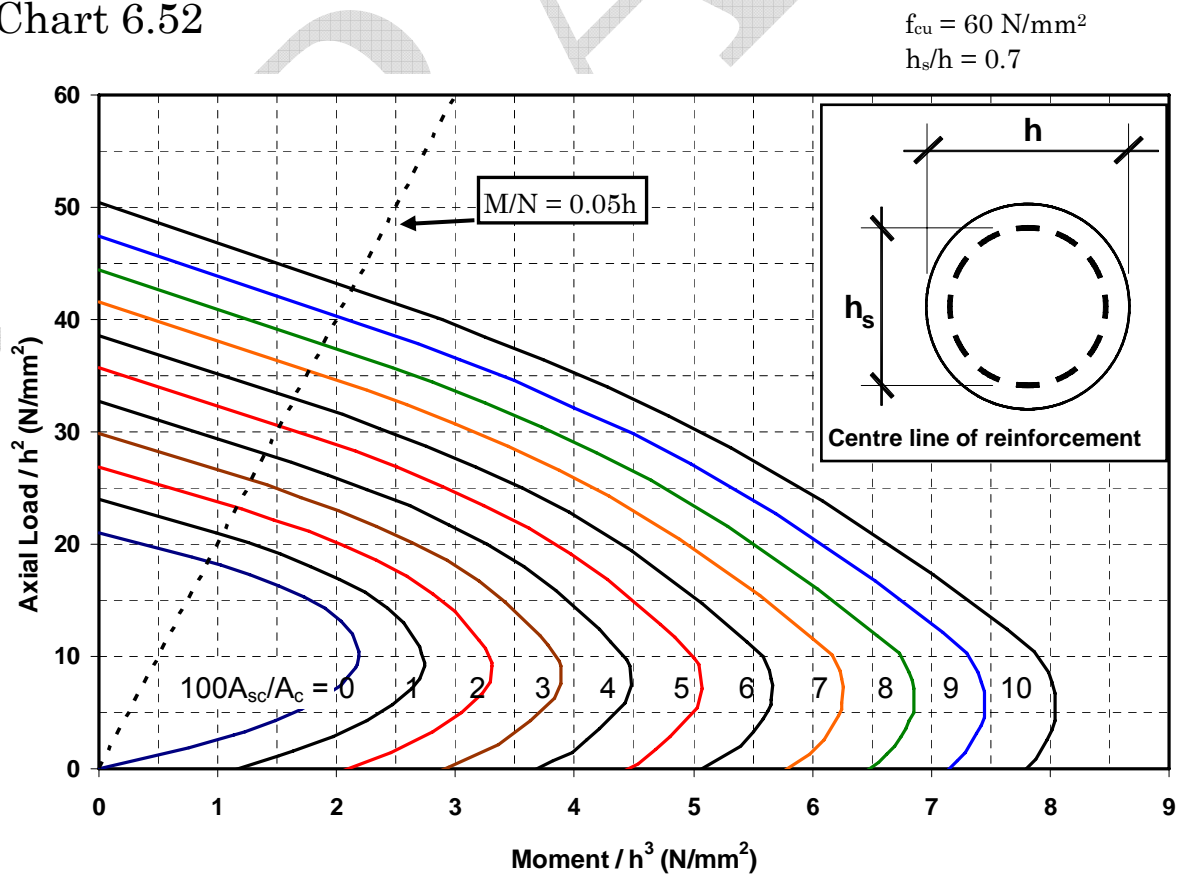


Chart 6.53

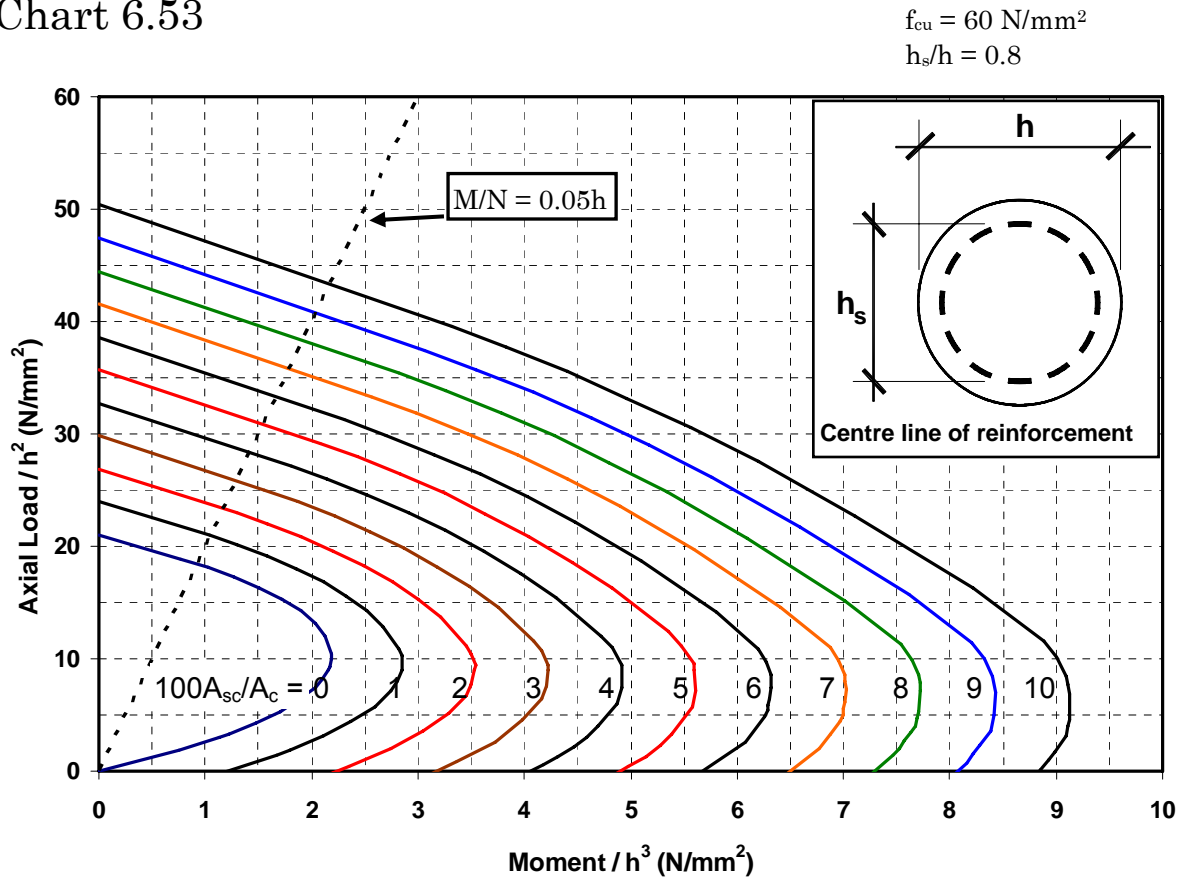


Chart 6.54

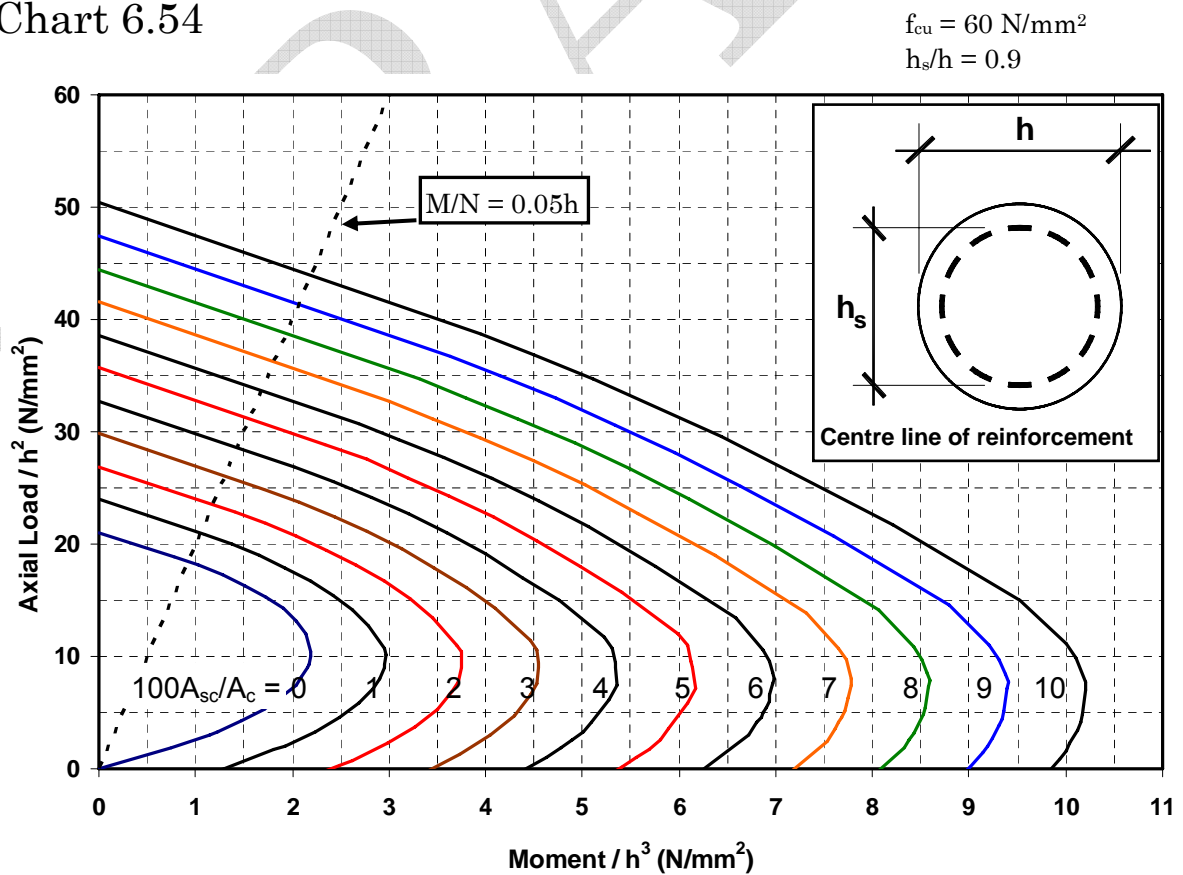


Chart 6.55

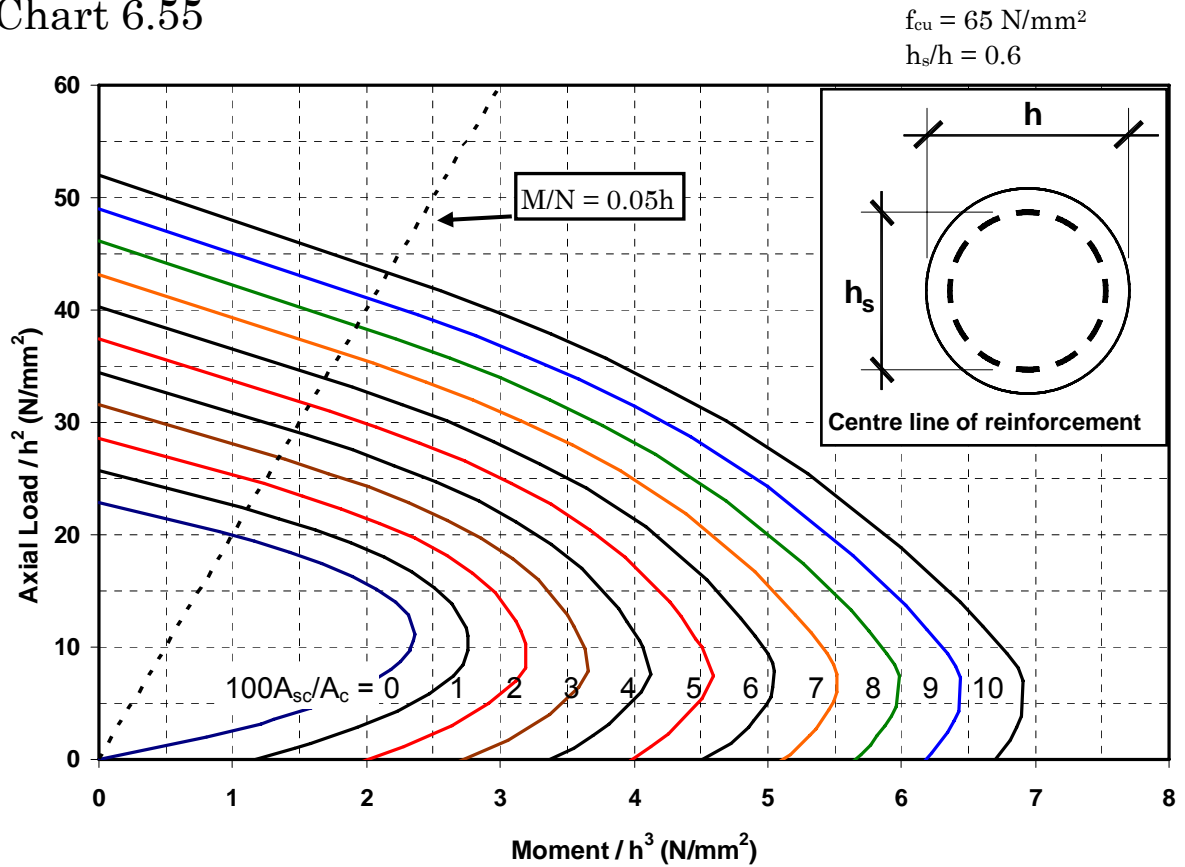


Chart 6.56

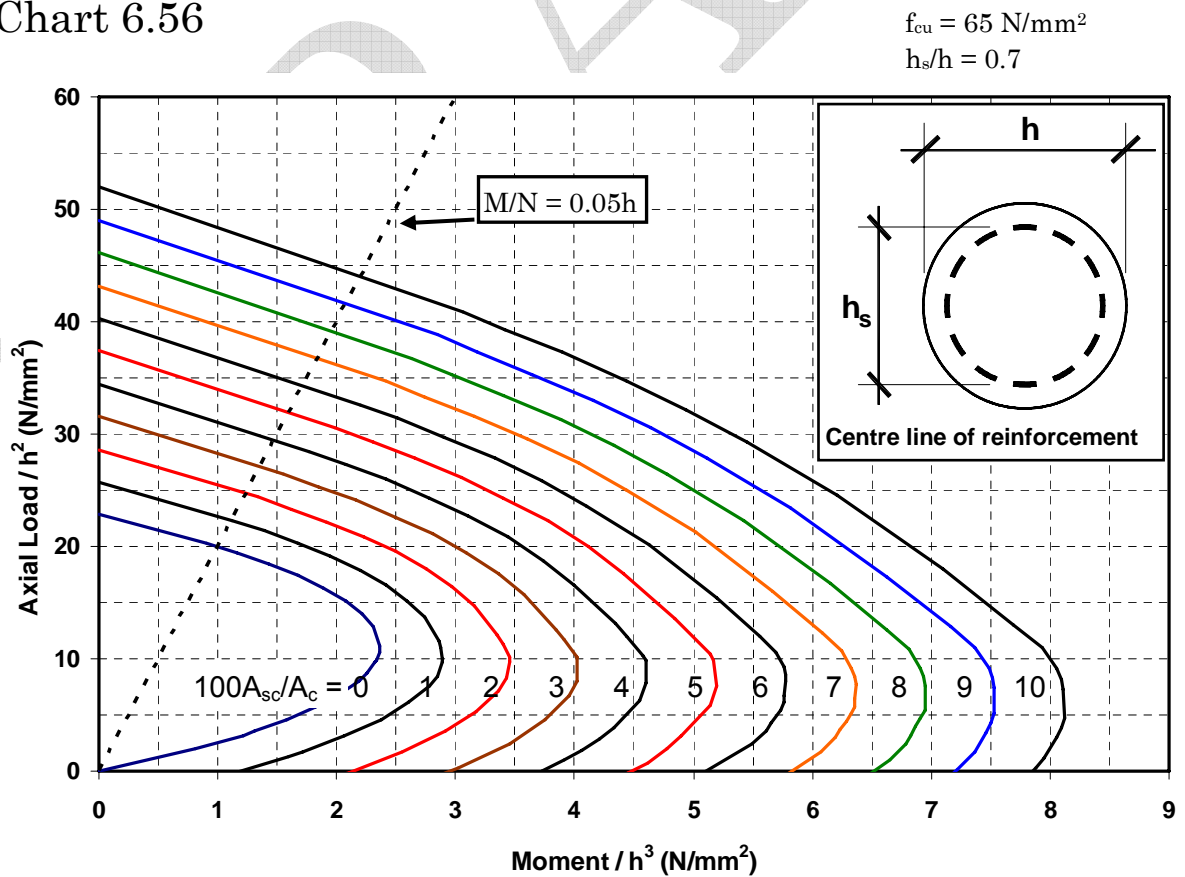


Chart 6.57

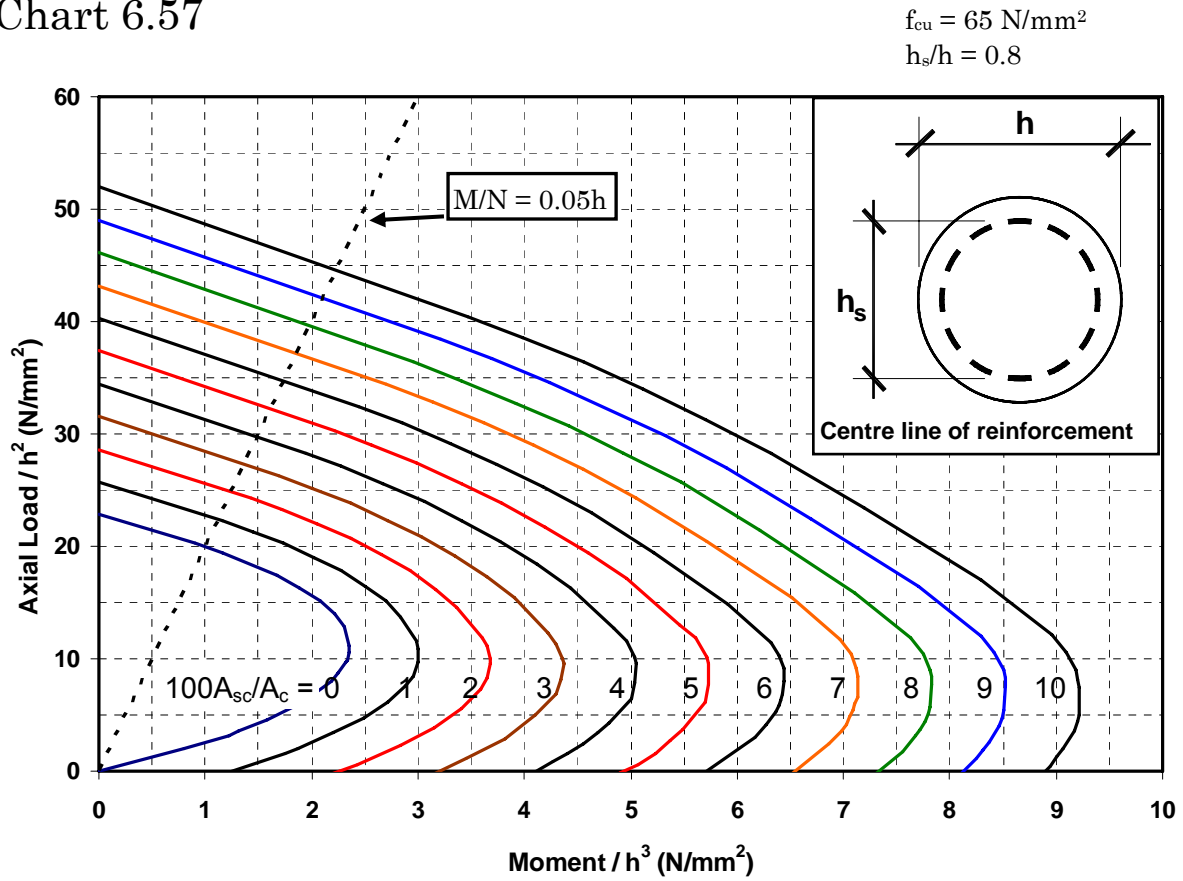


Chart 6.58

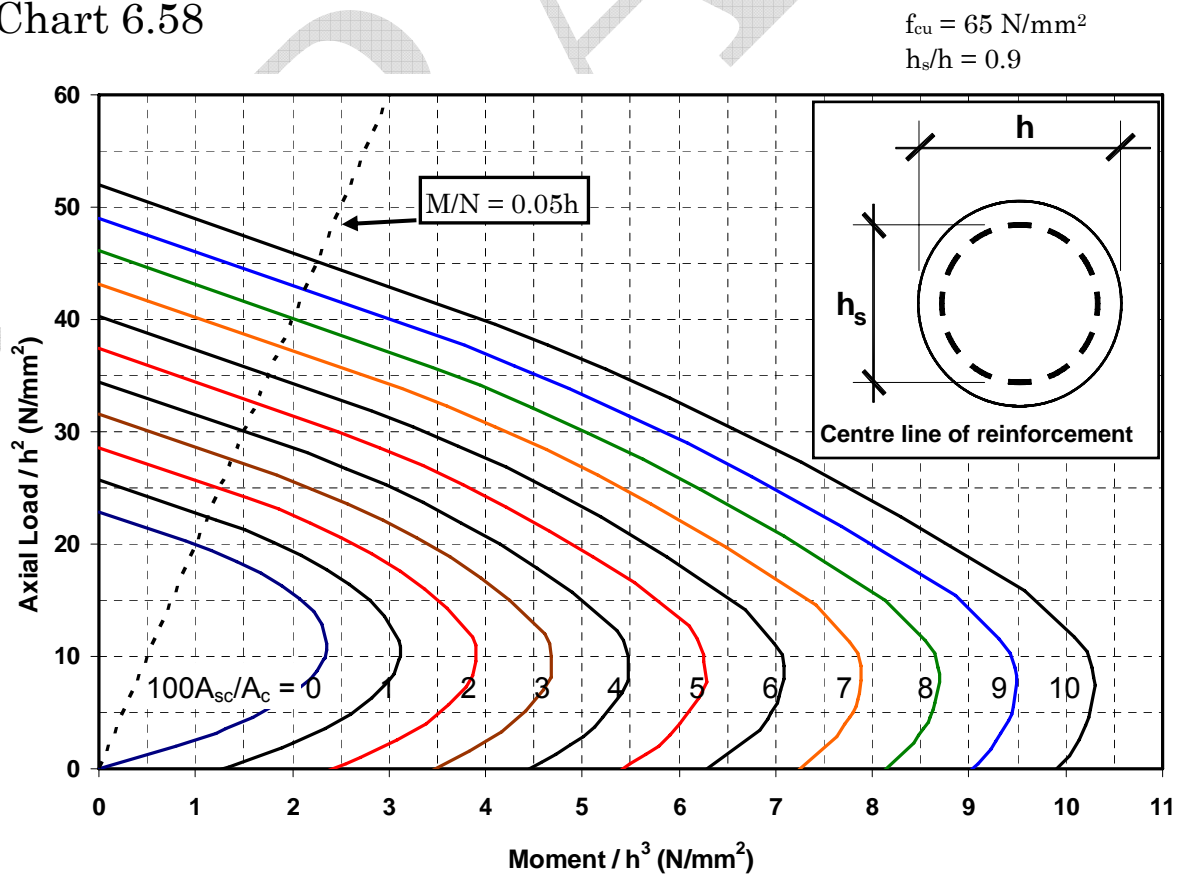


Chart 6.59

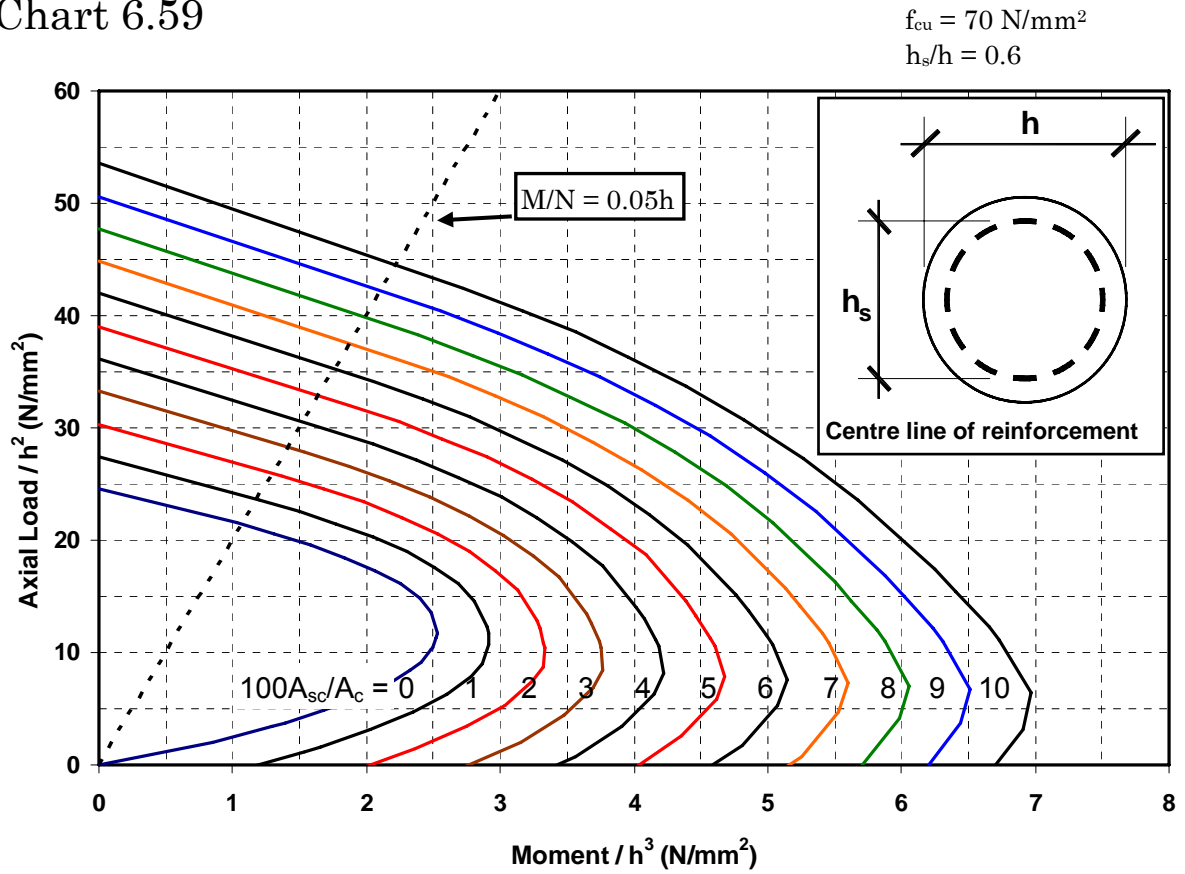


Chart 6.60

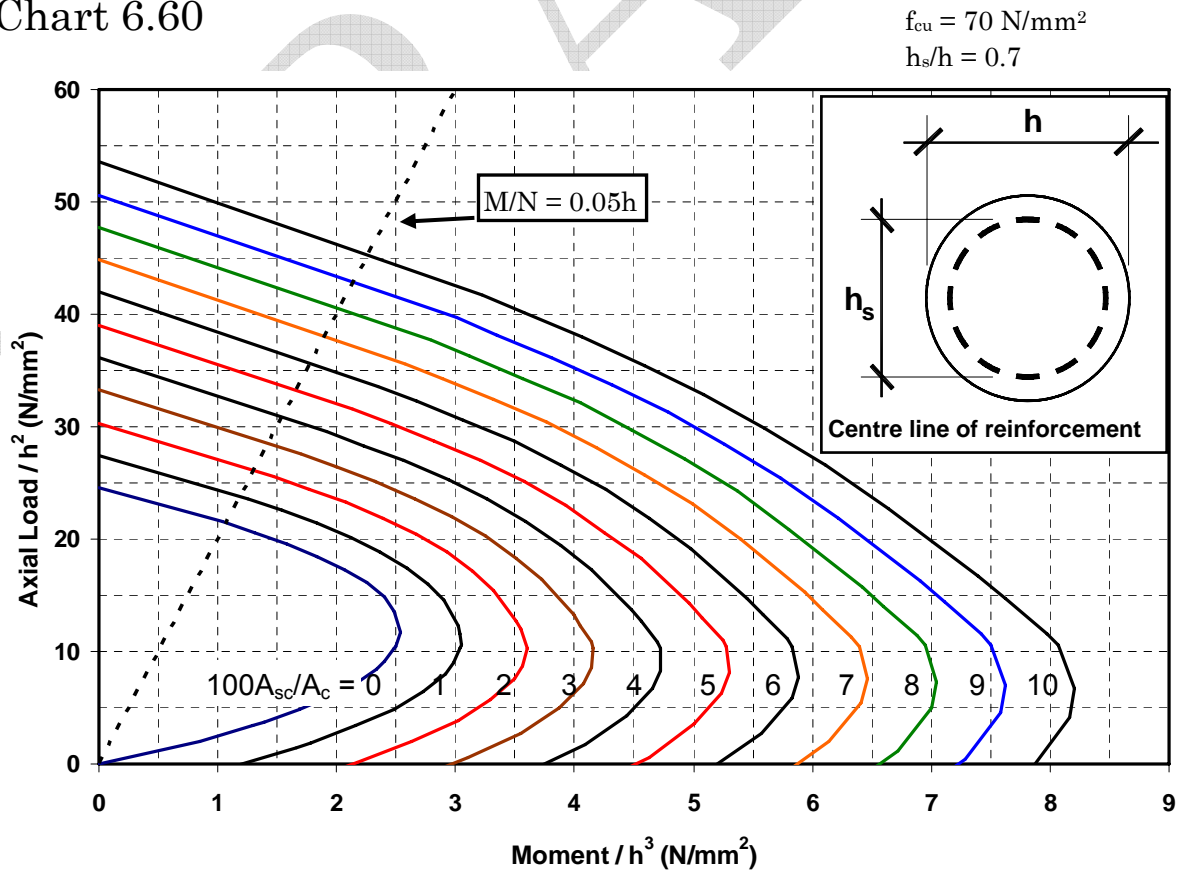


Chart 6.61

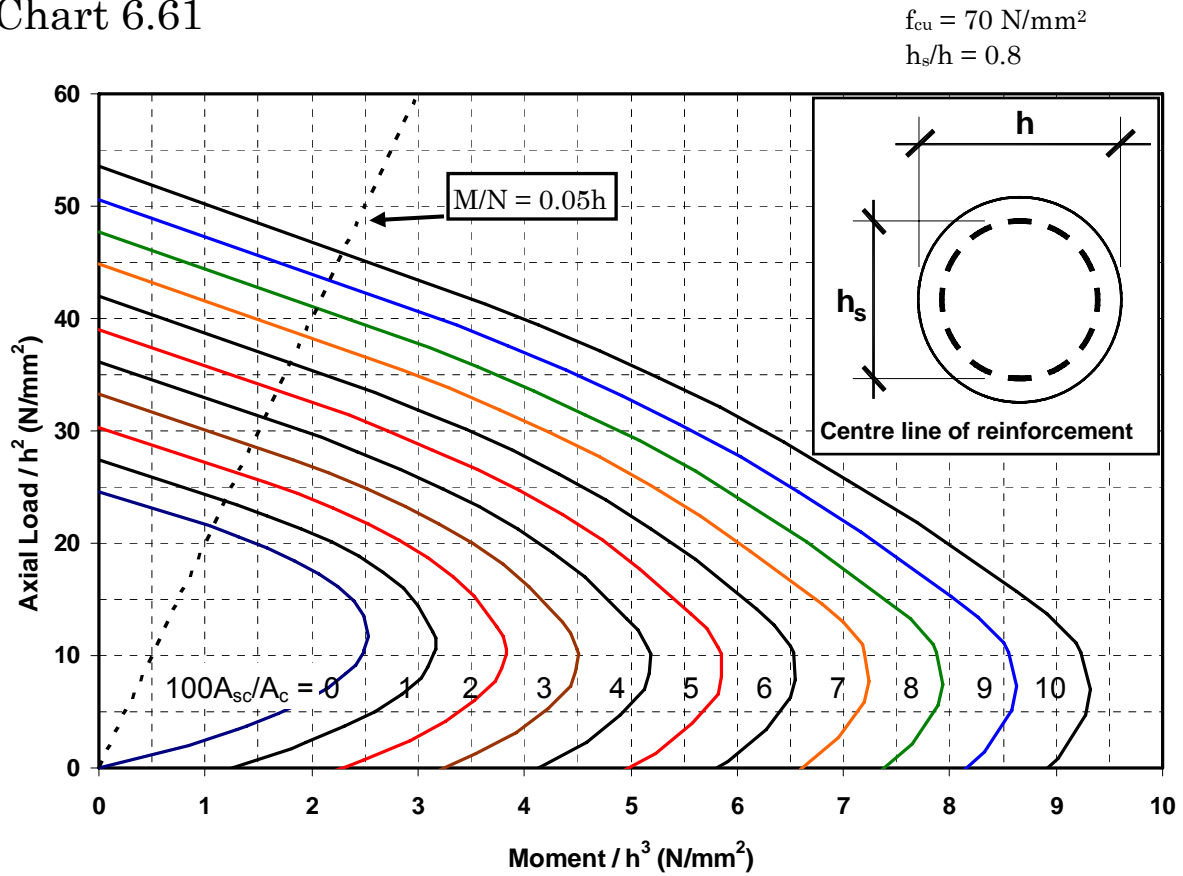


Chart 6.62

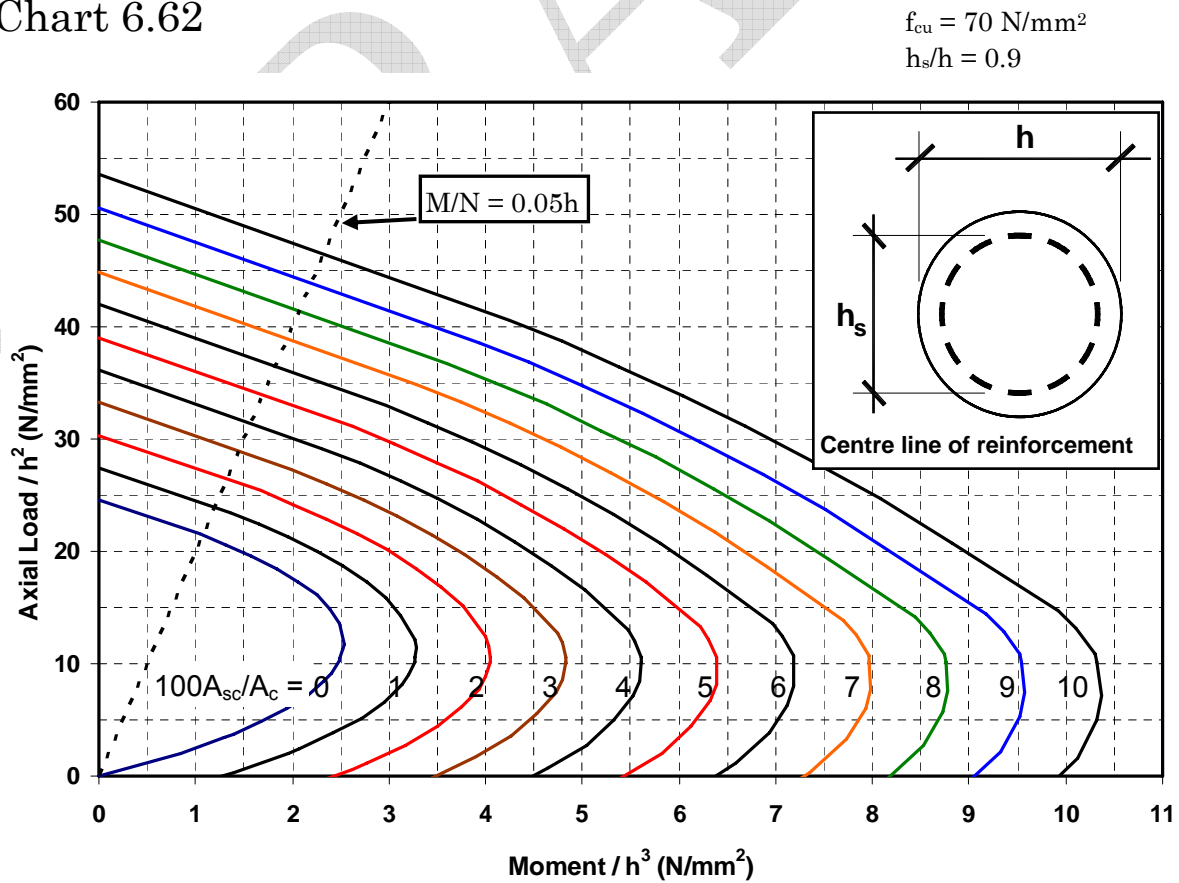


Chart 6.63

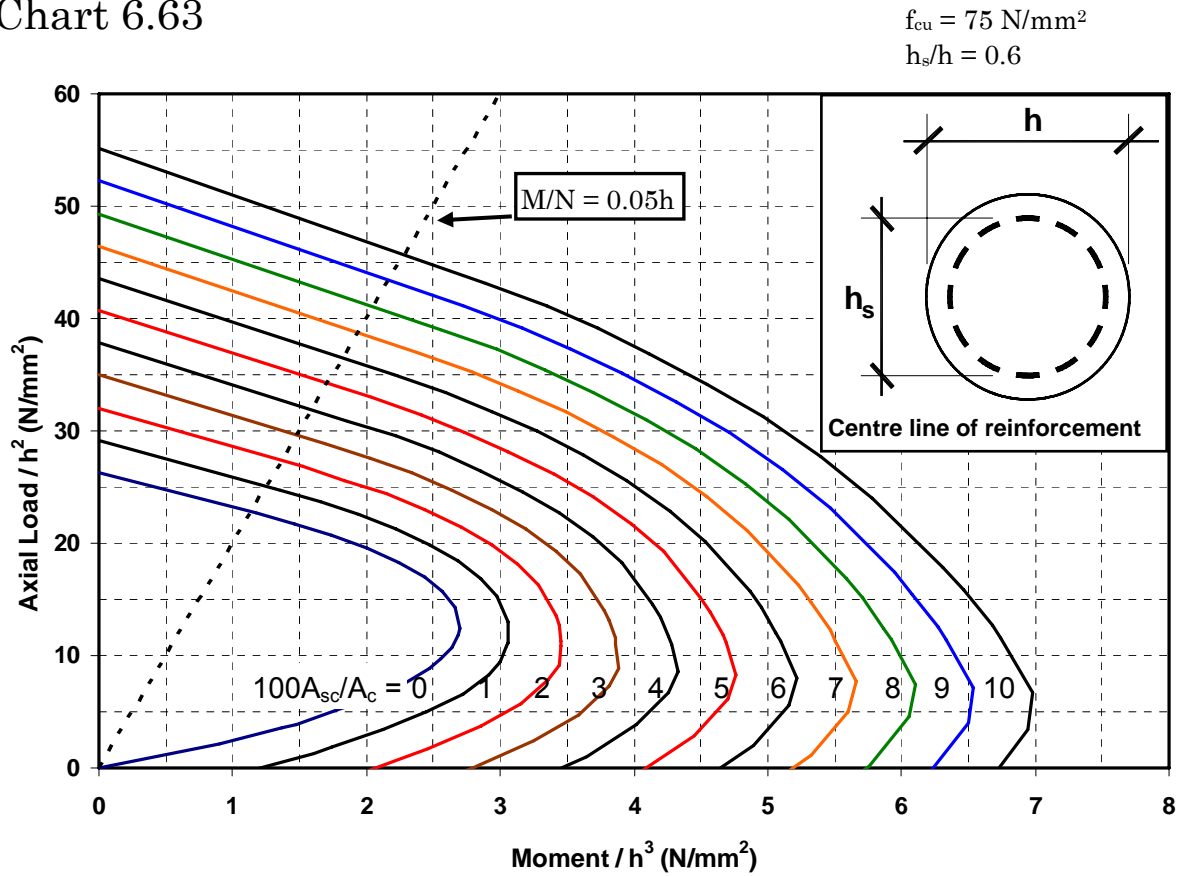


Chart 6.64

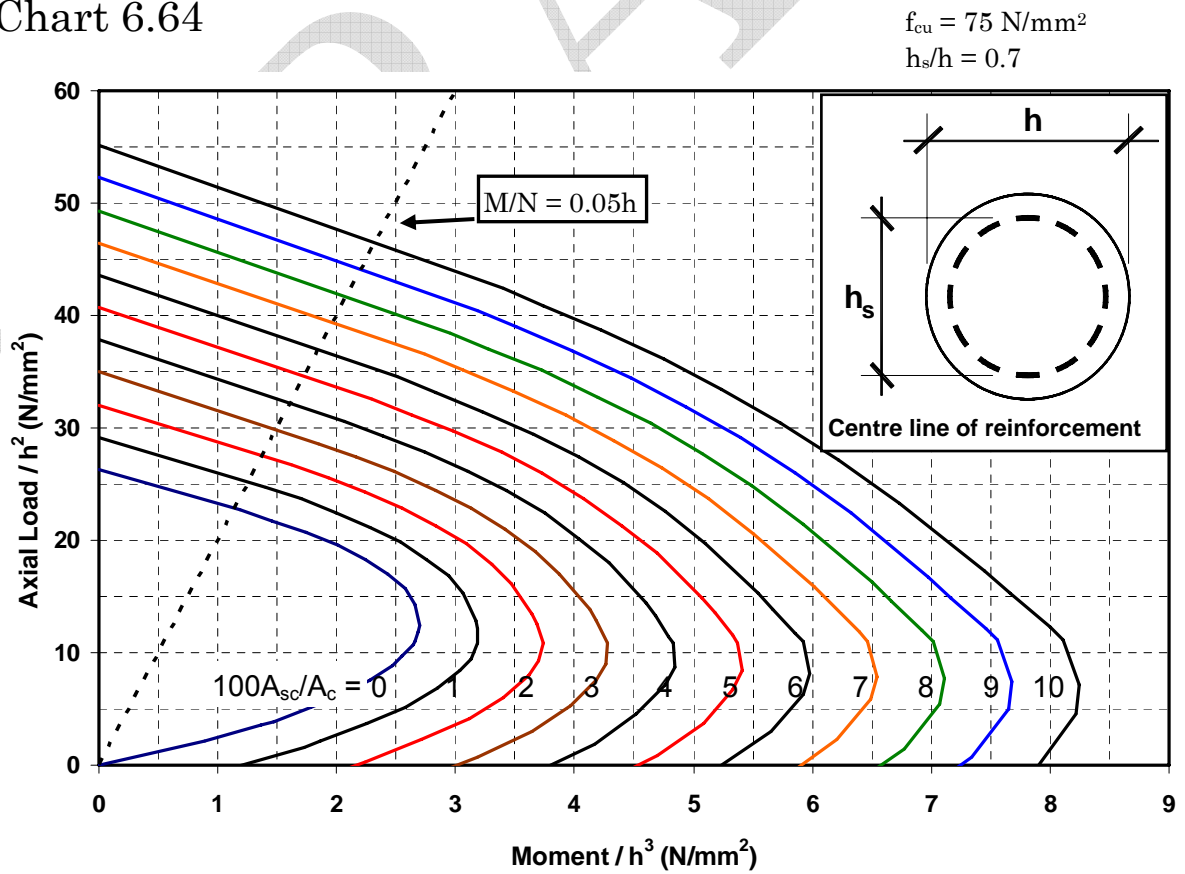


Chart 6.65

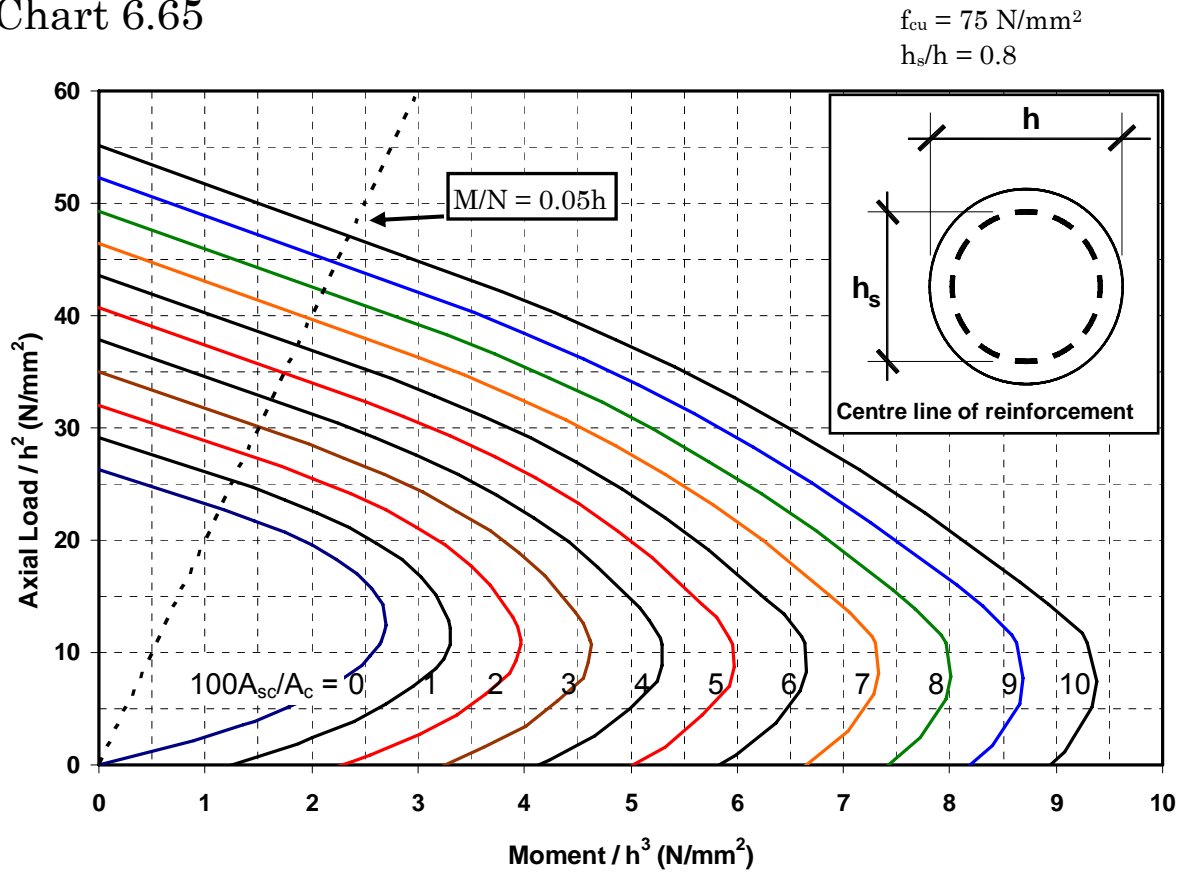


Chart 6.66

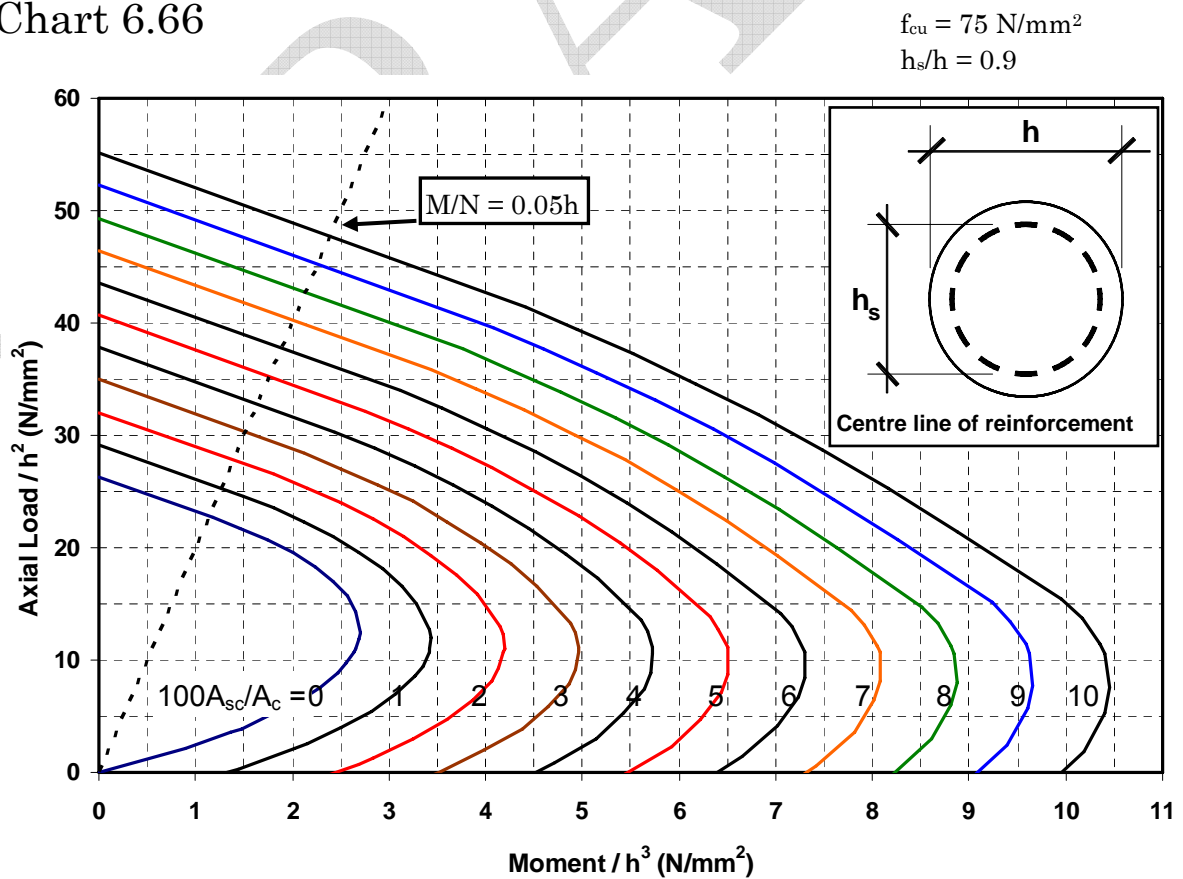


Chart 6.67

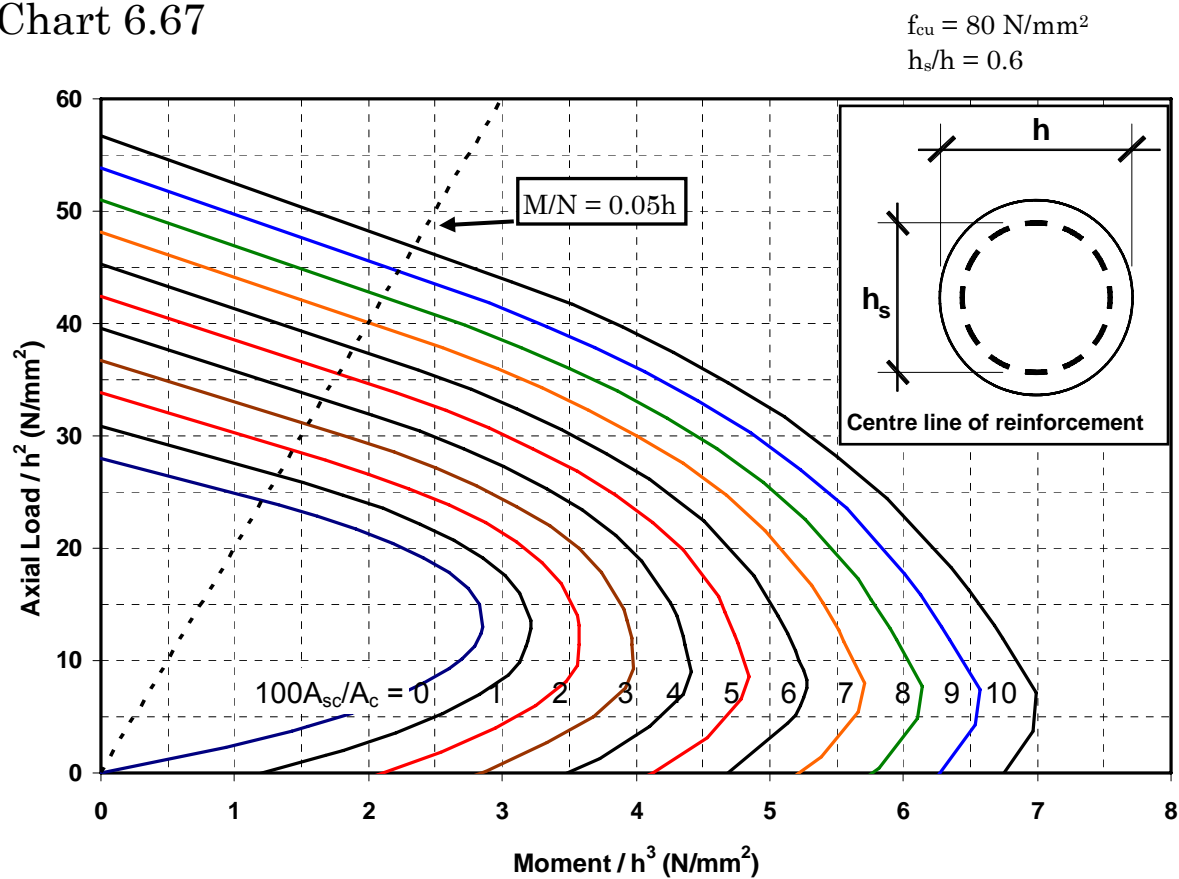


Chart 6.68

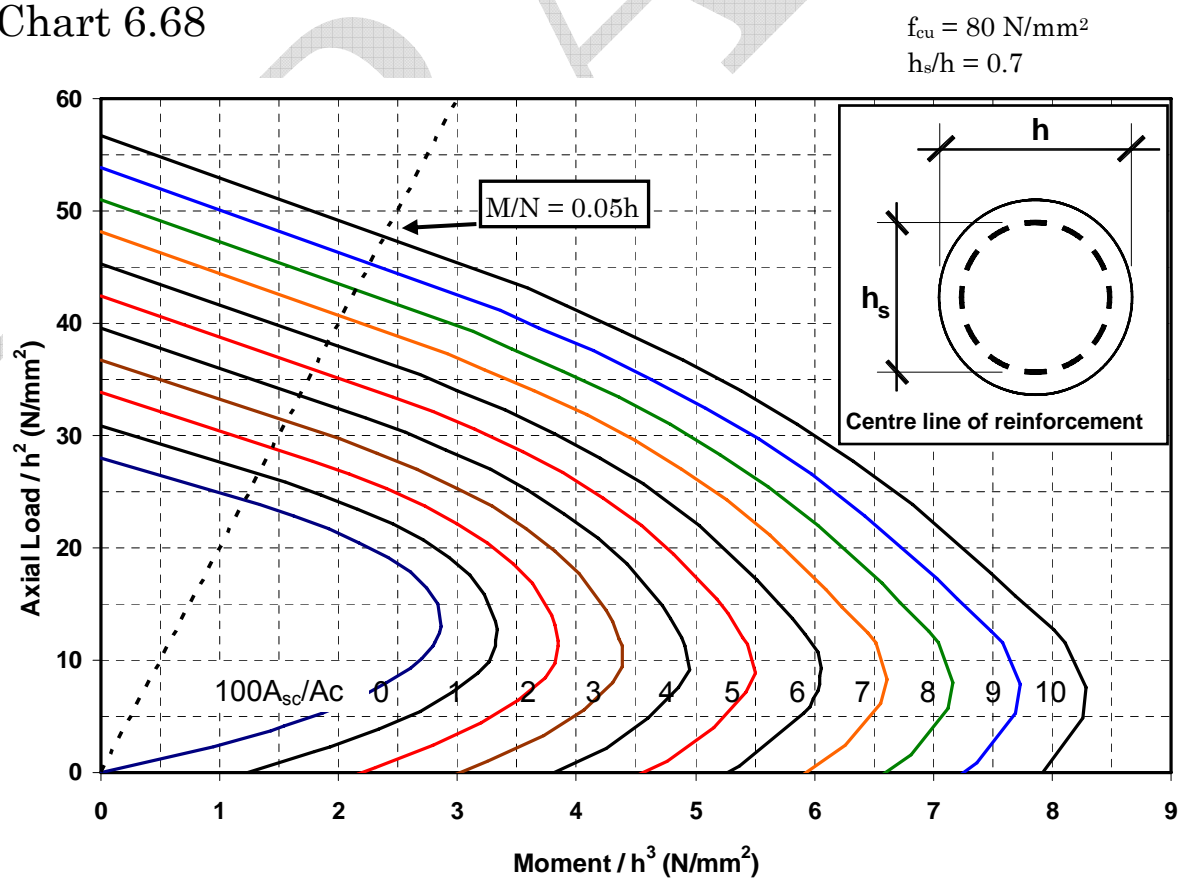


Chart 6.69

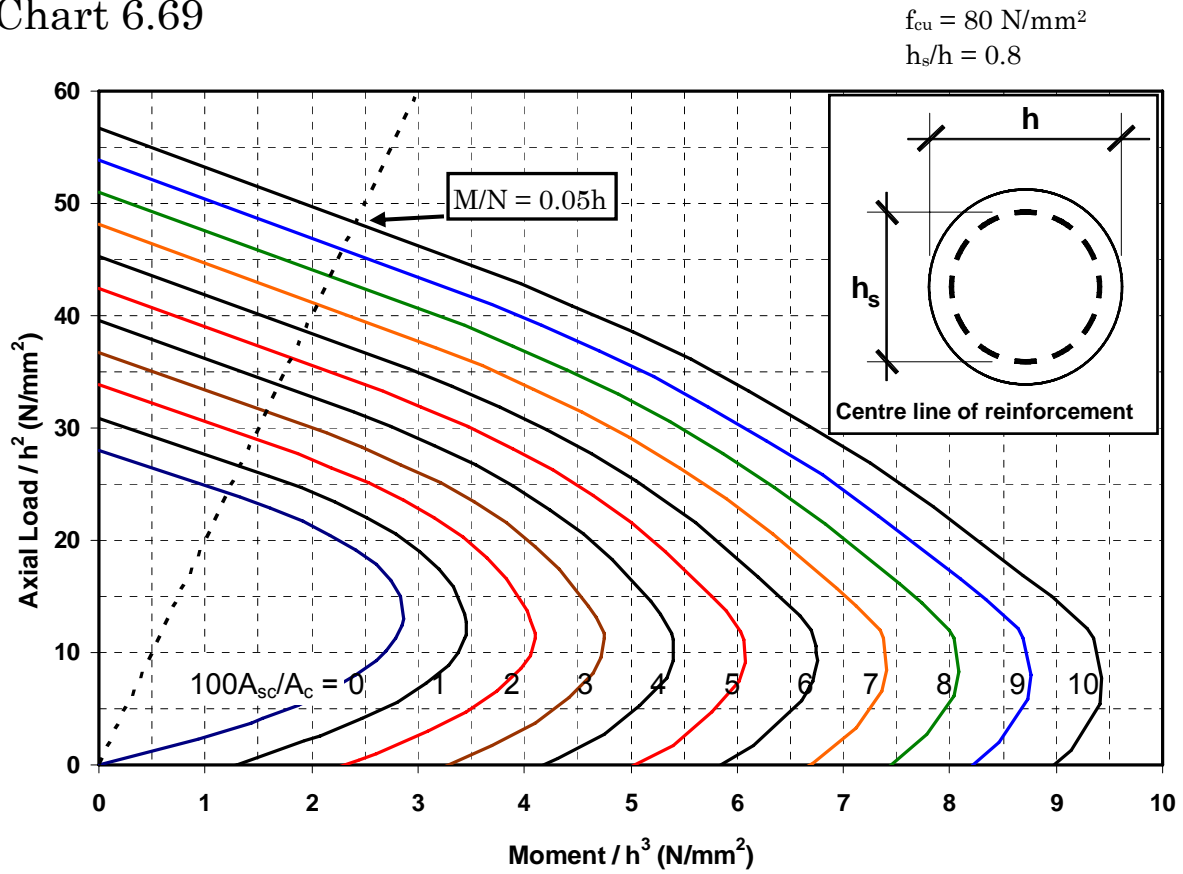


Chart 6.70

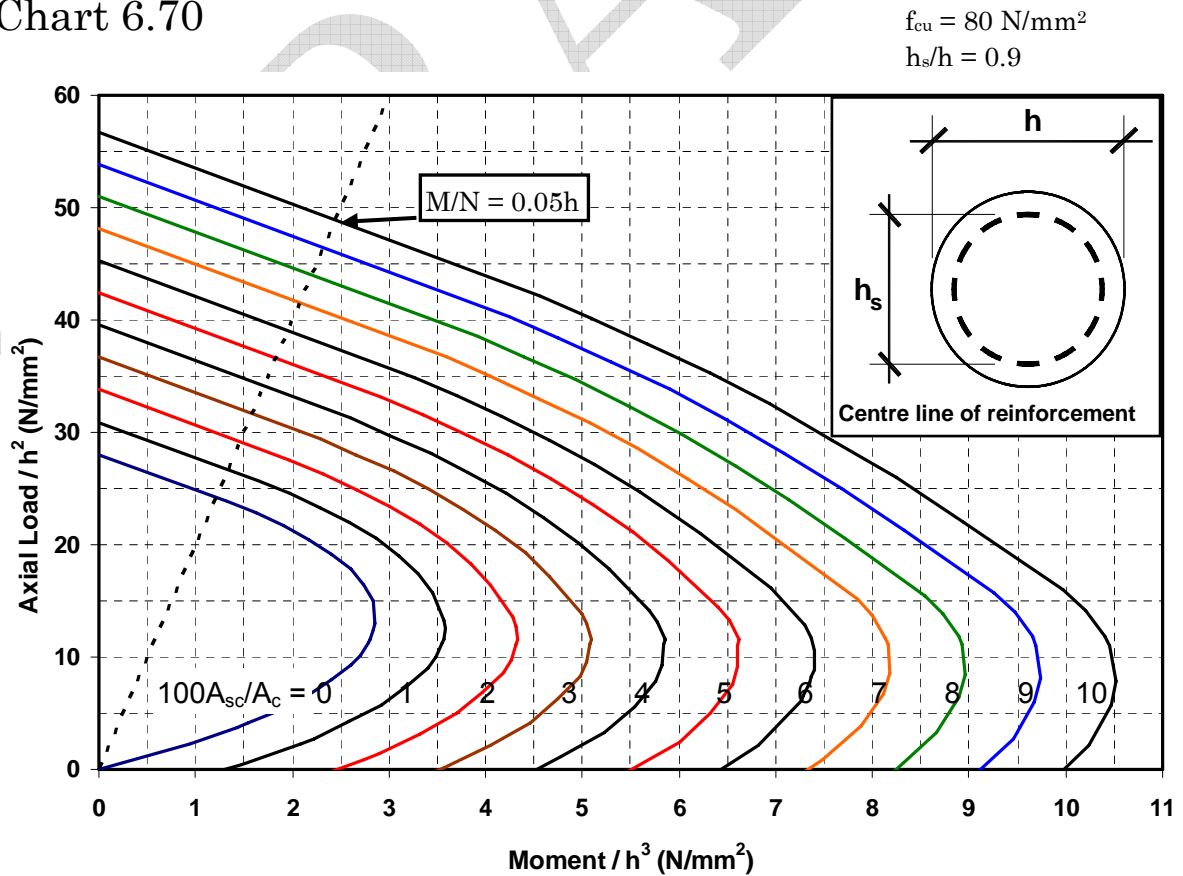


Chart 6.71

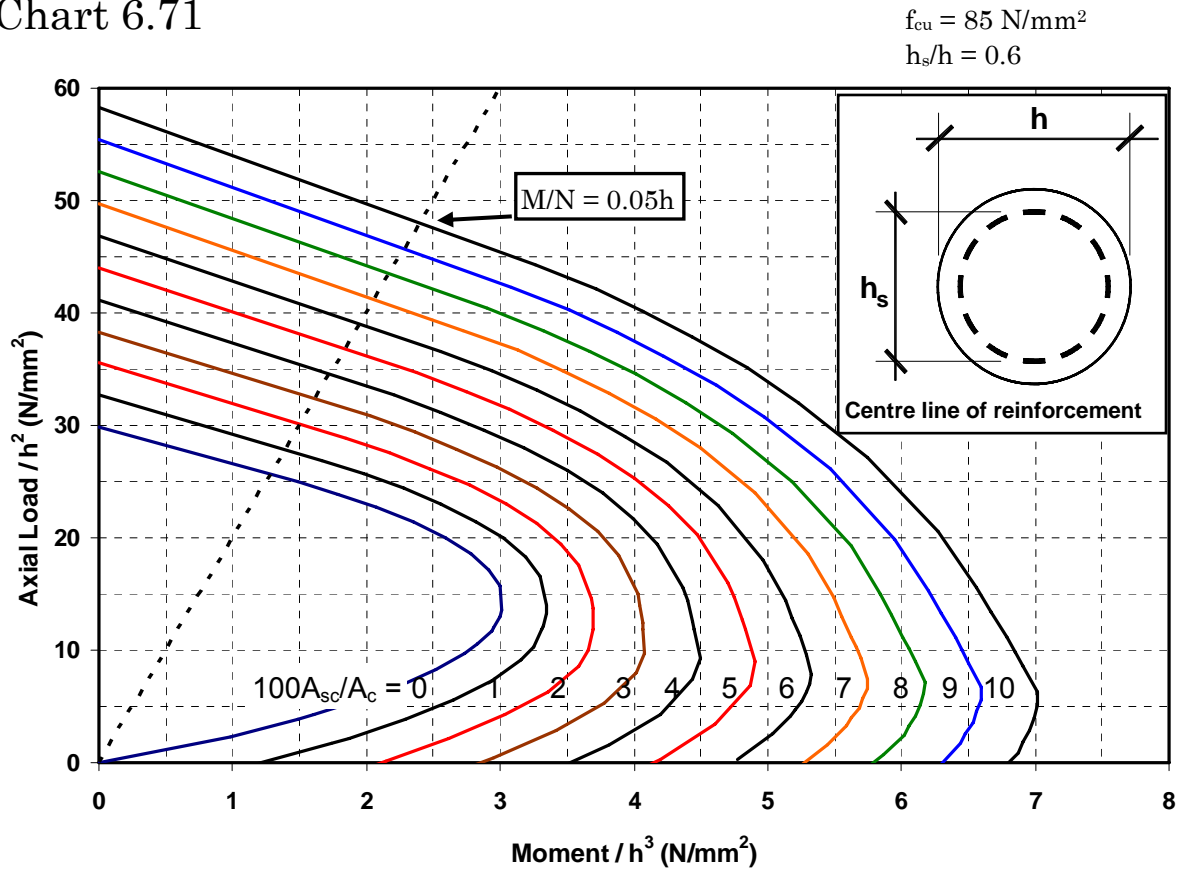


Chart 6.72

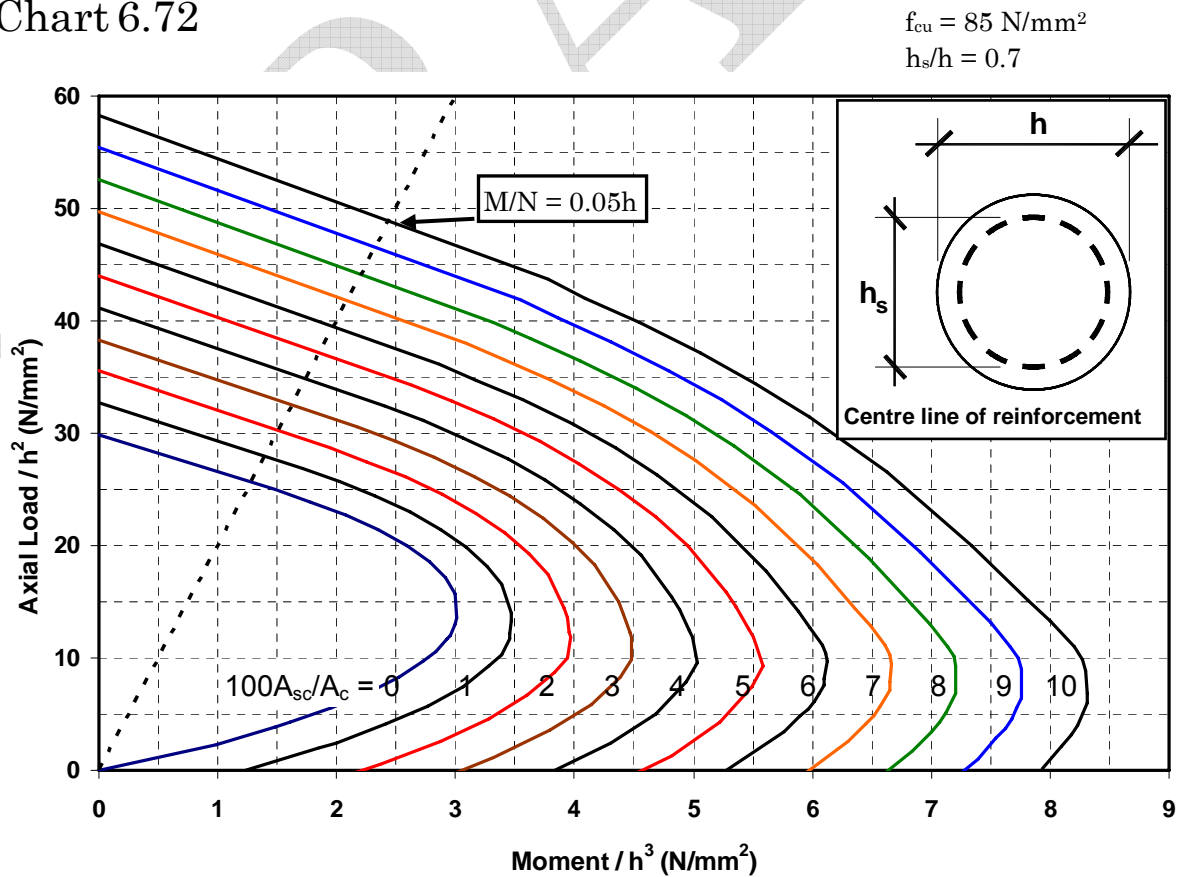


Chart 6.73

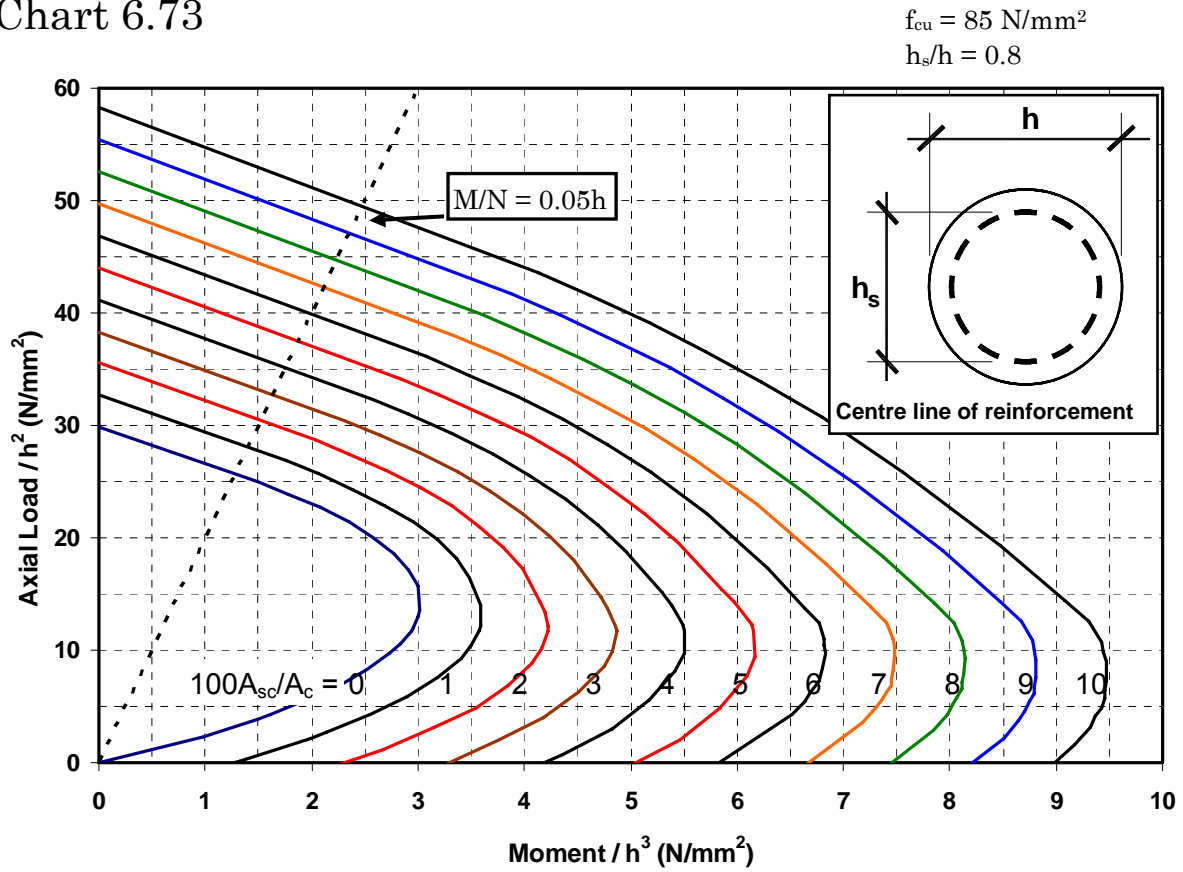


Chart 6.74

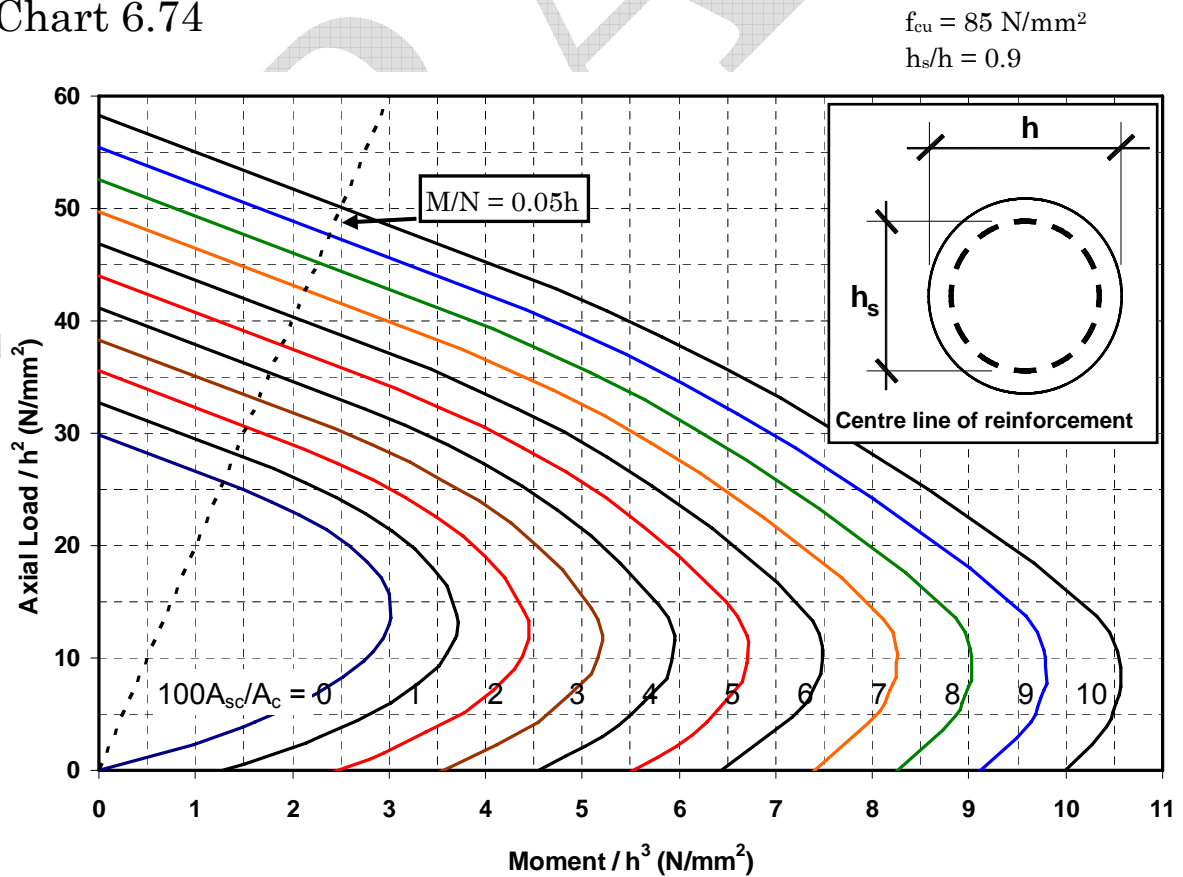


Chart 6.75

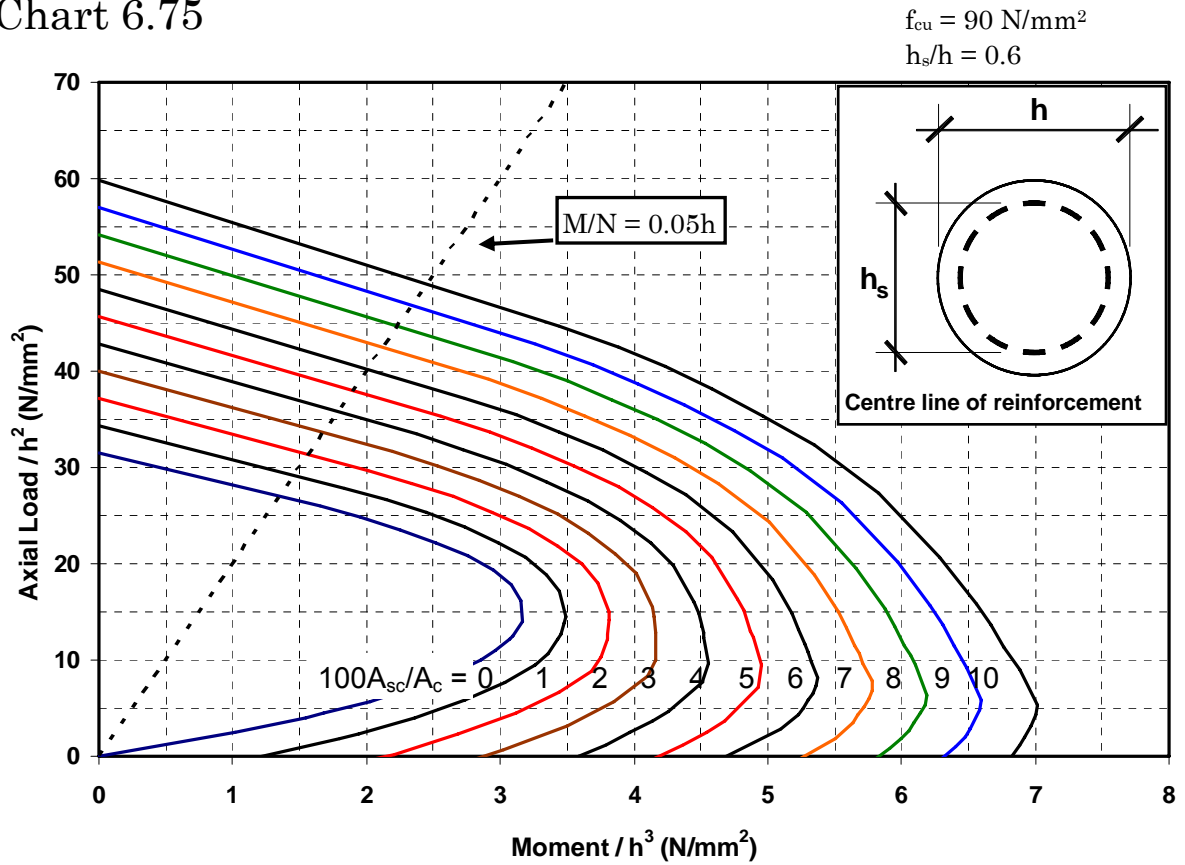


Chart 6.76

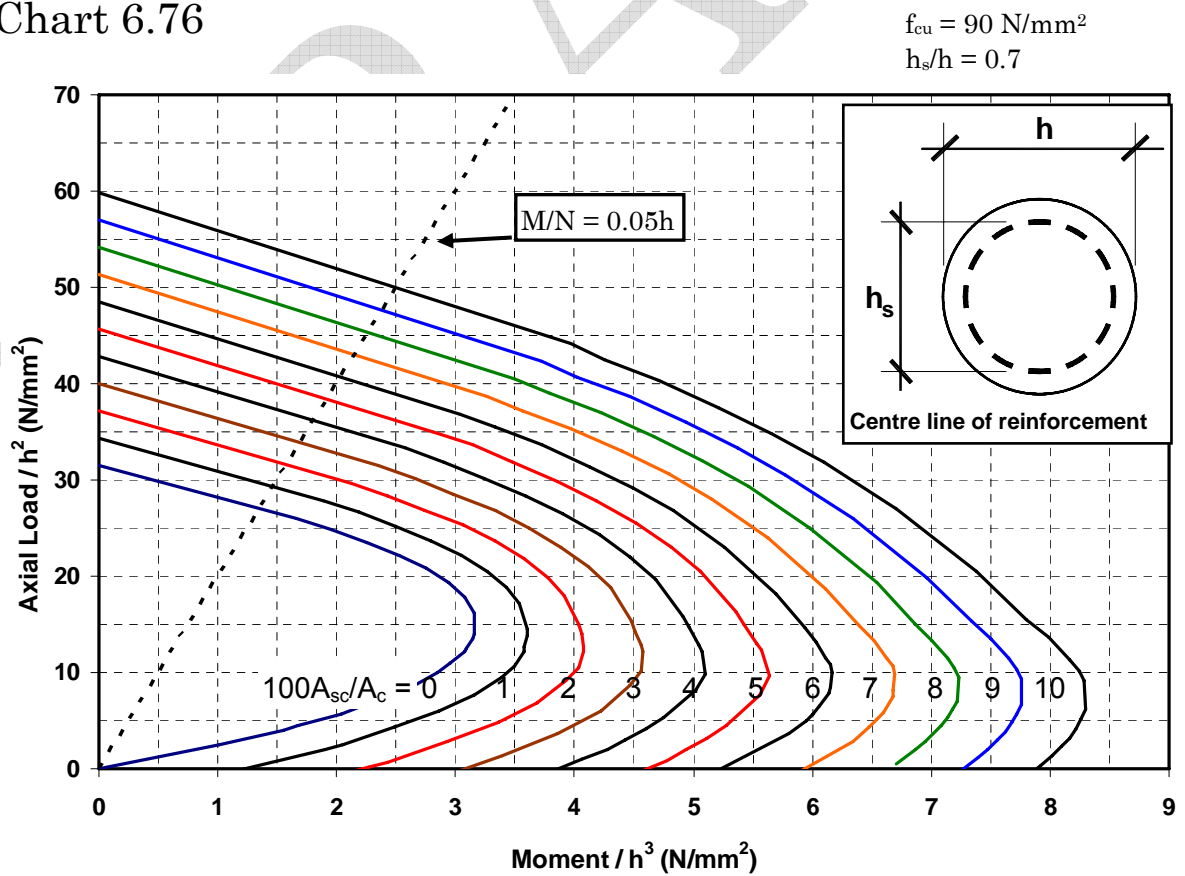


Chart 6.77

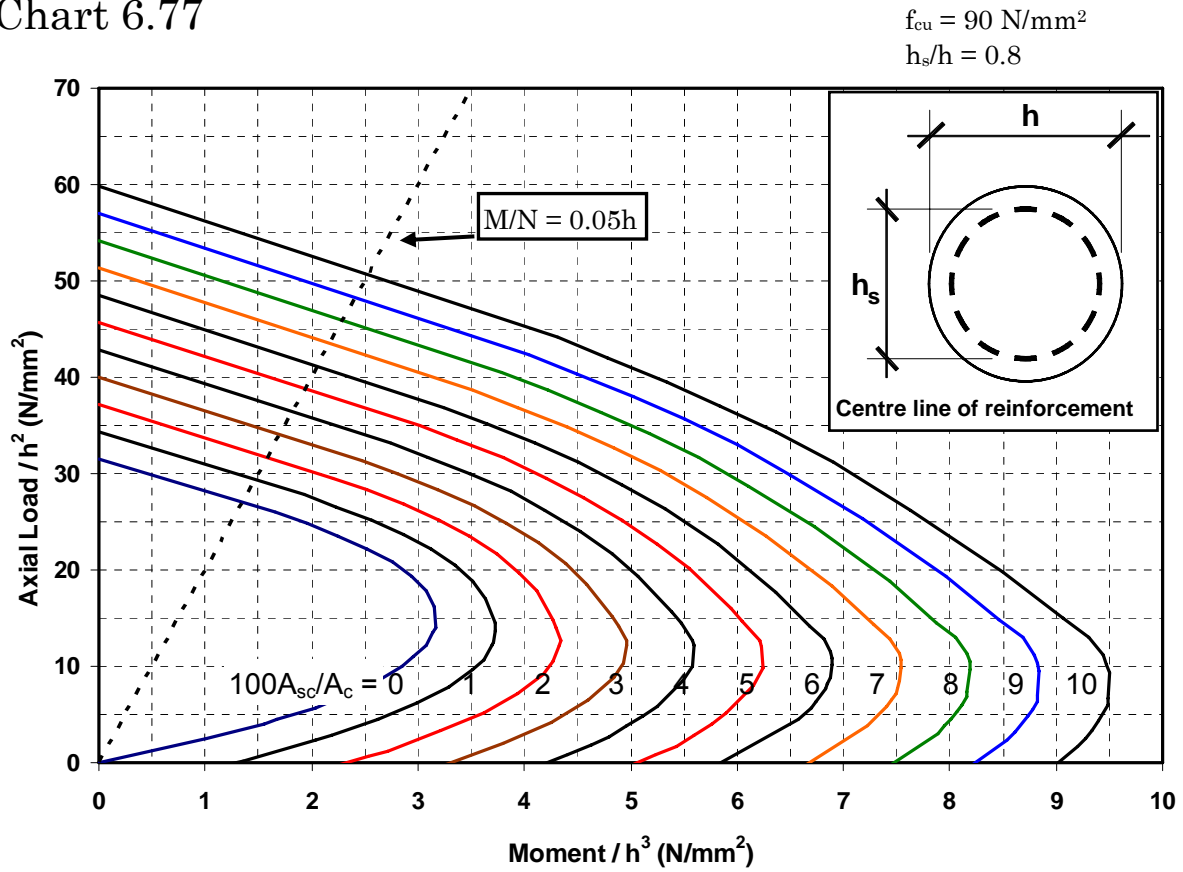


Chart 6.78

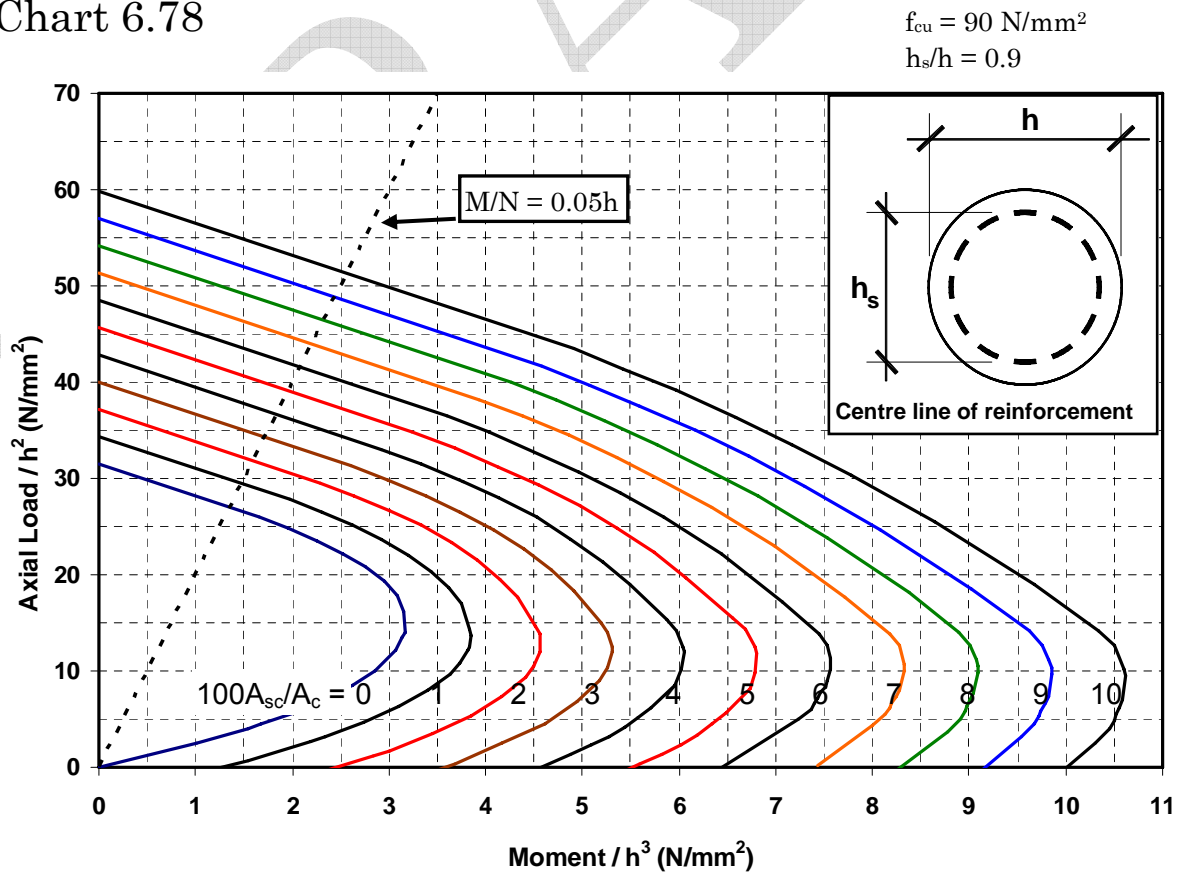


Chart 6.79

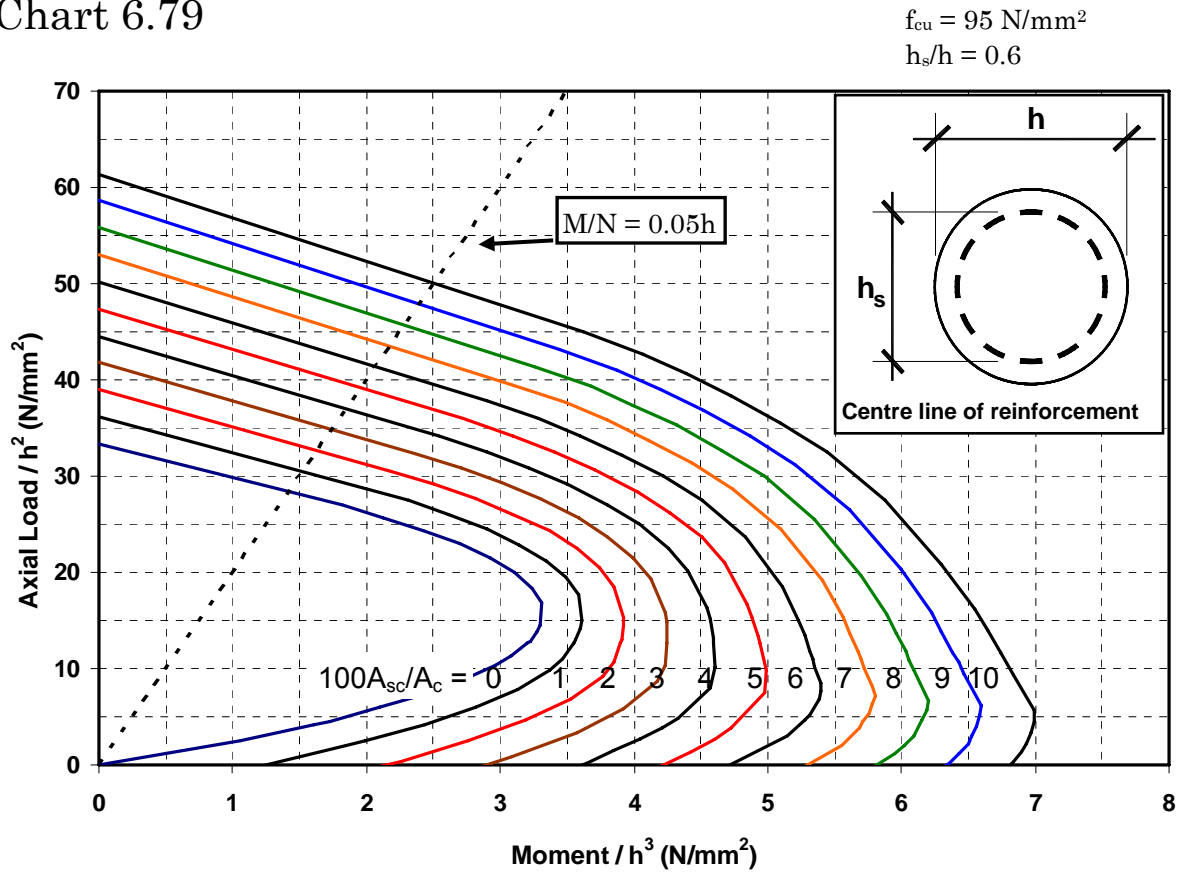


Chart 6.80

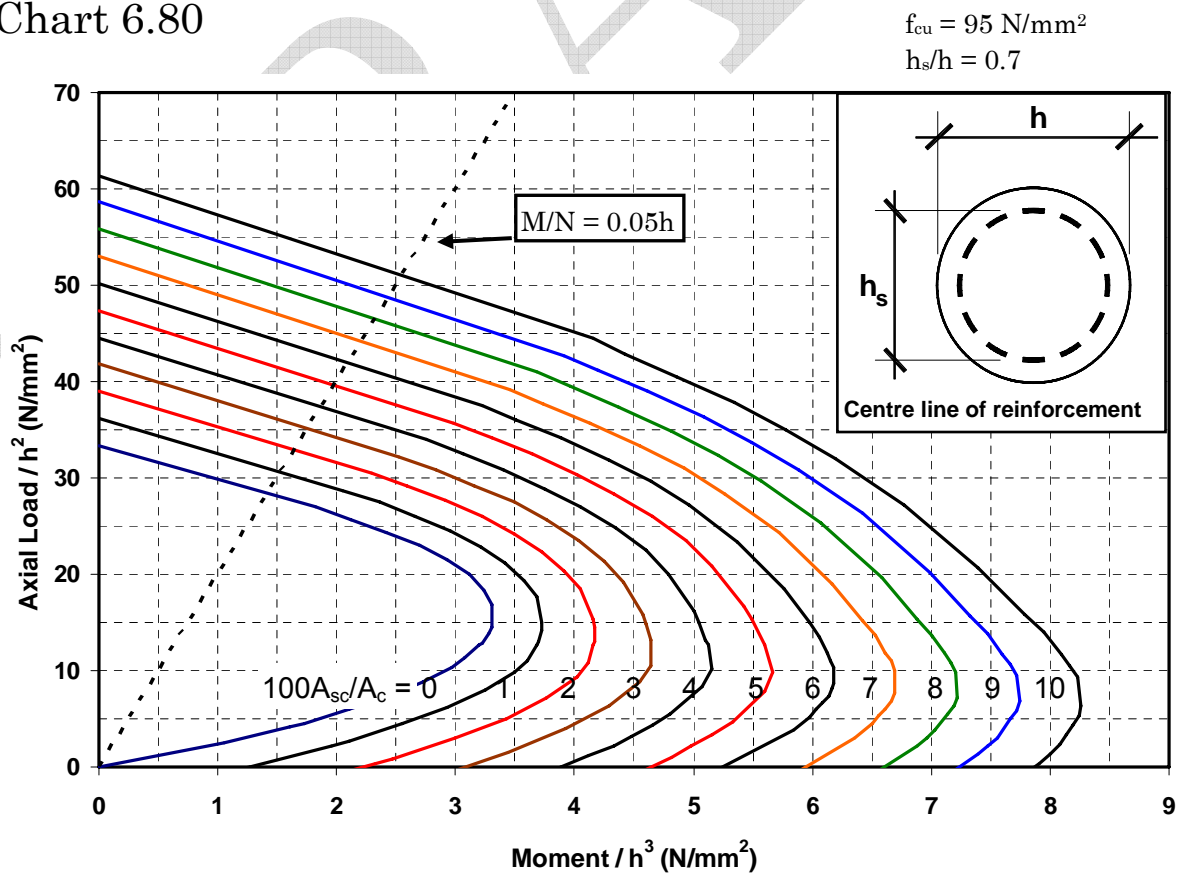


Chart 6.81

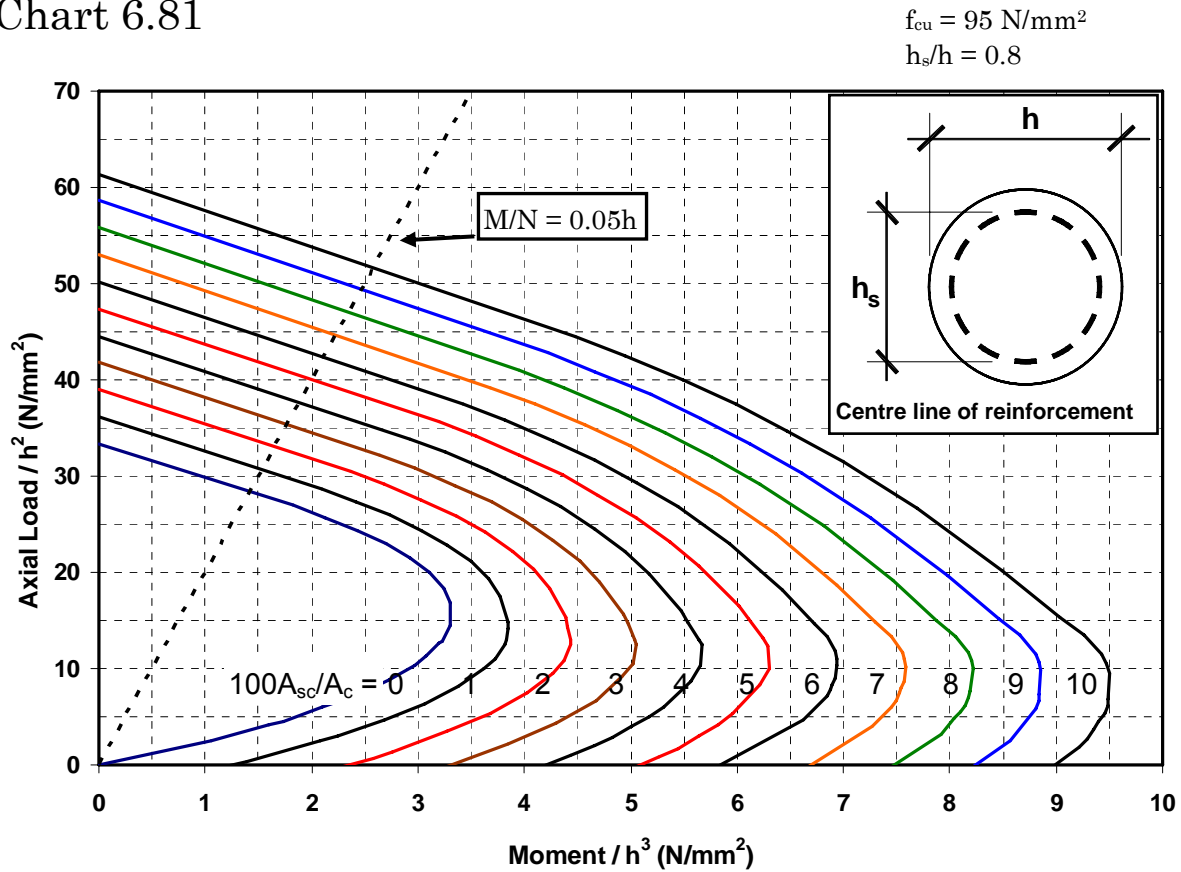


Chart 6.82

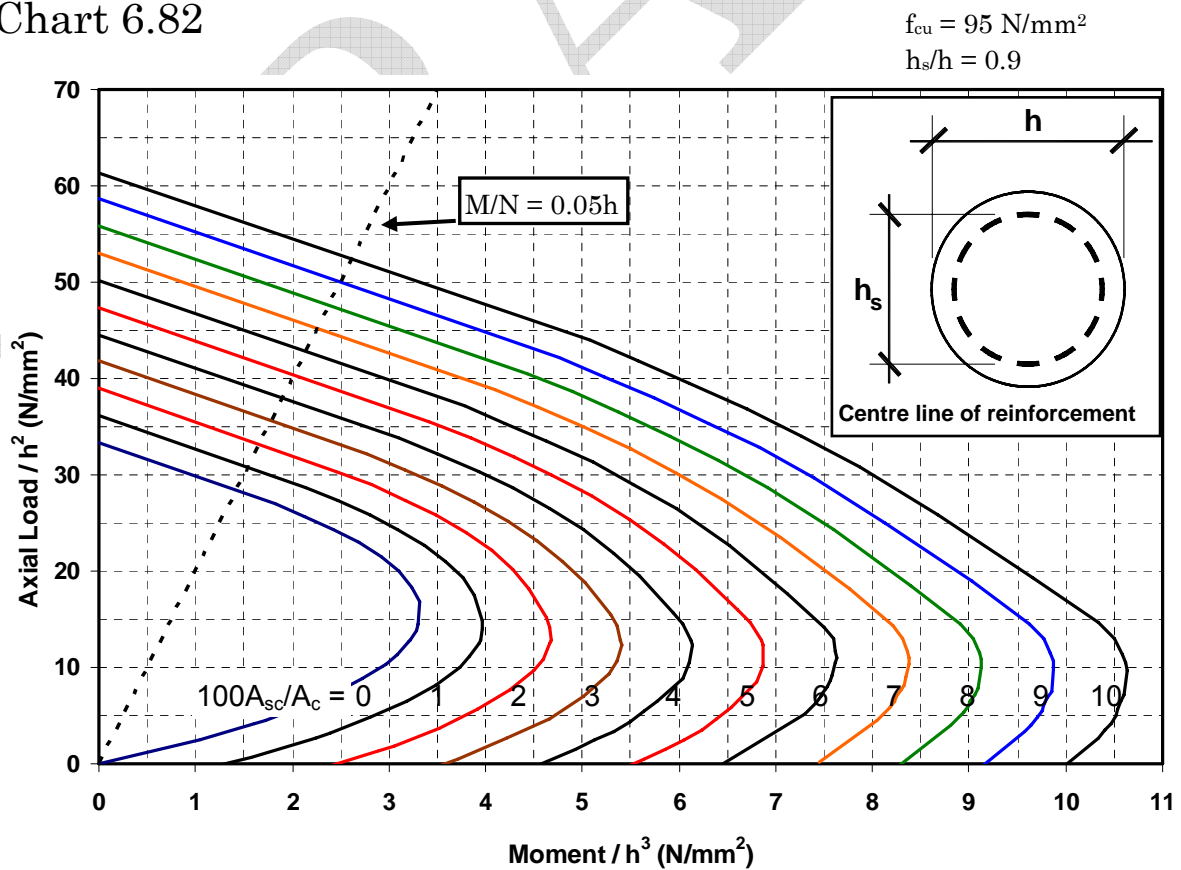


Chart 6.83

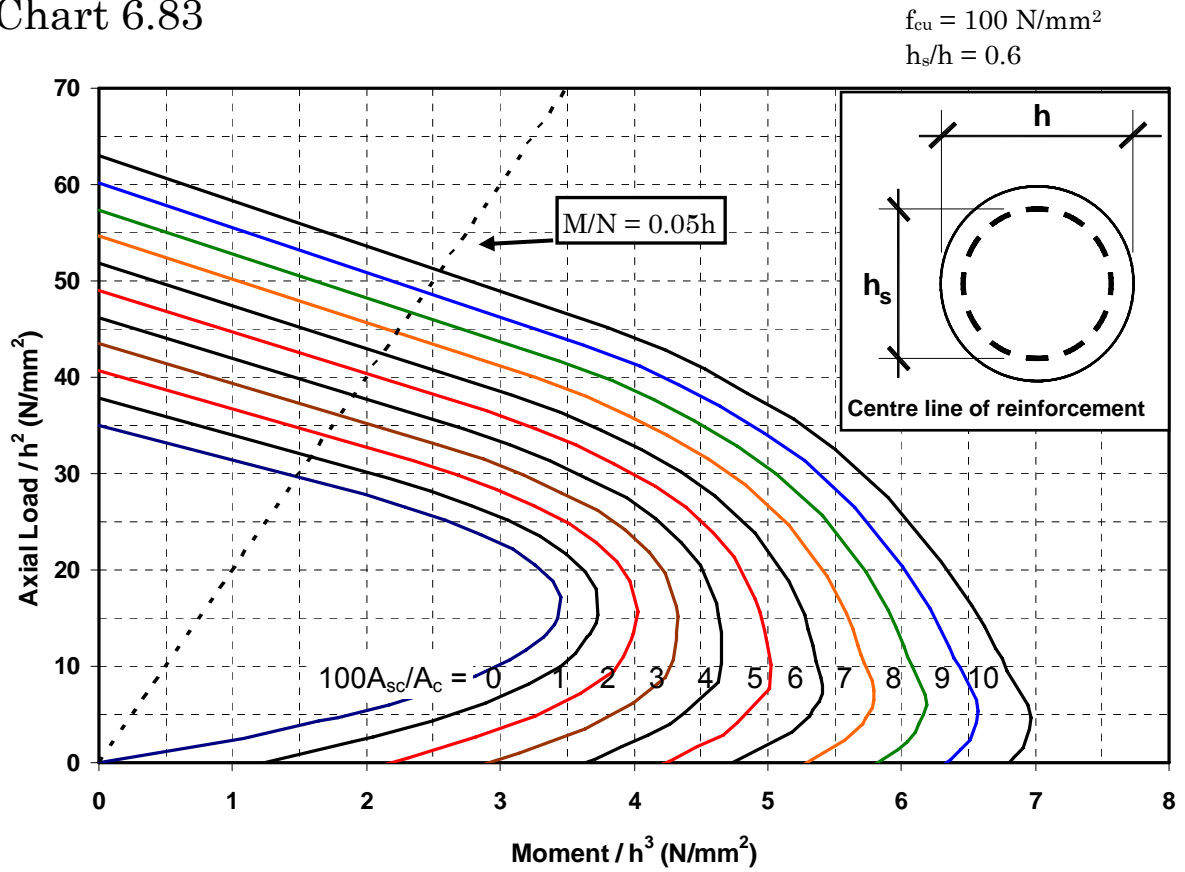


Chart 6.84

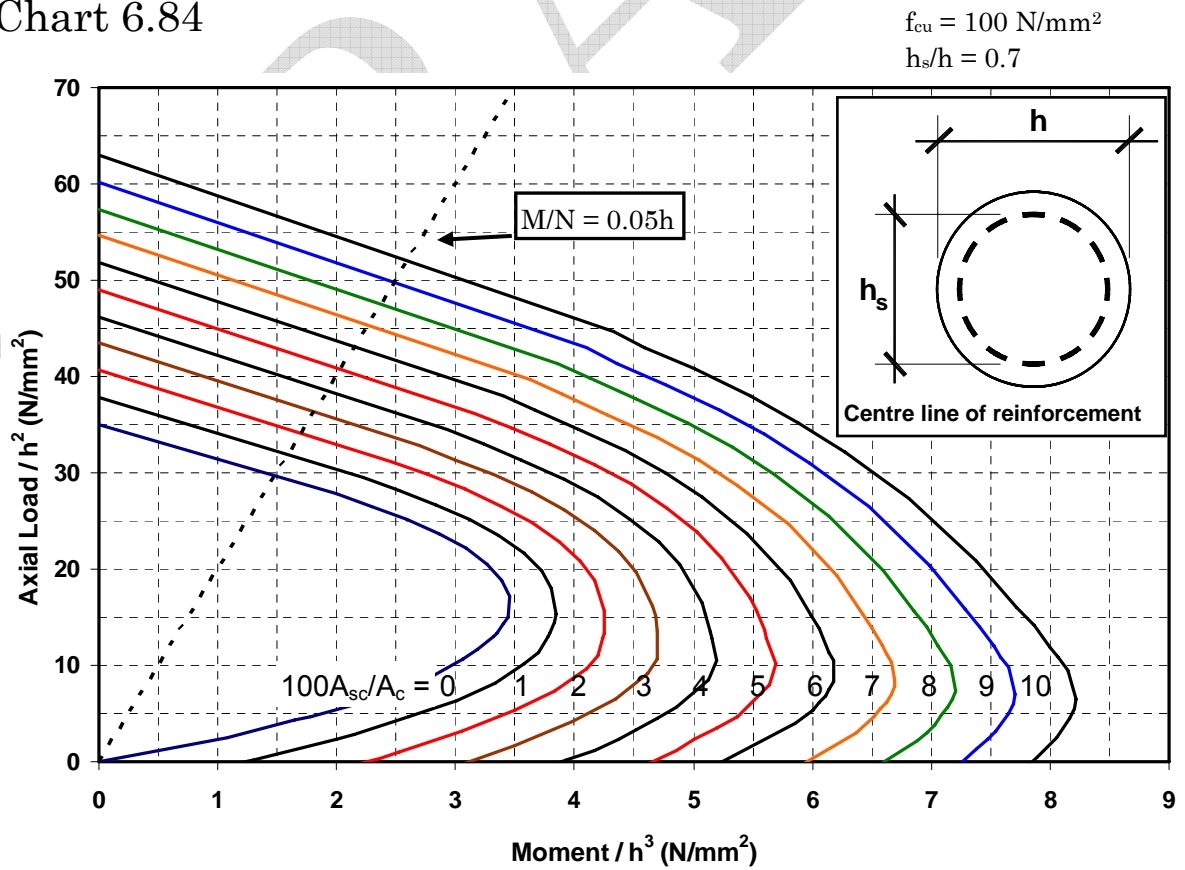


Chart 6.85

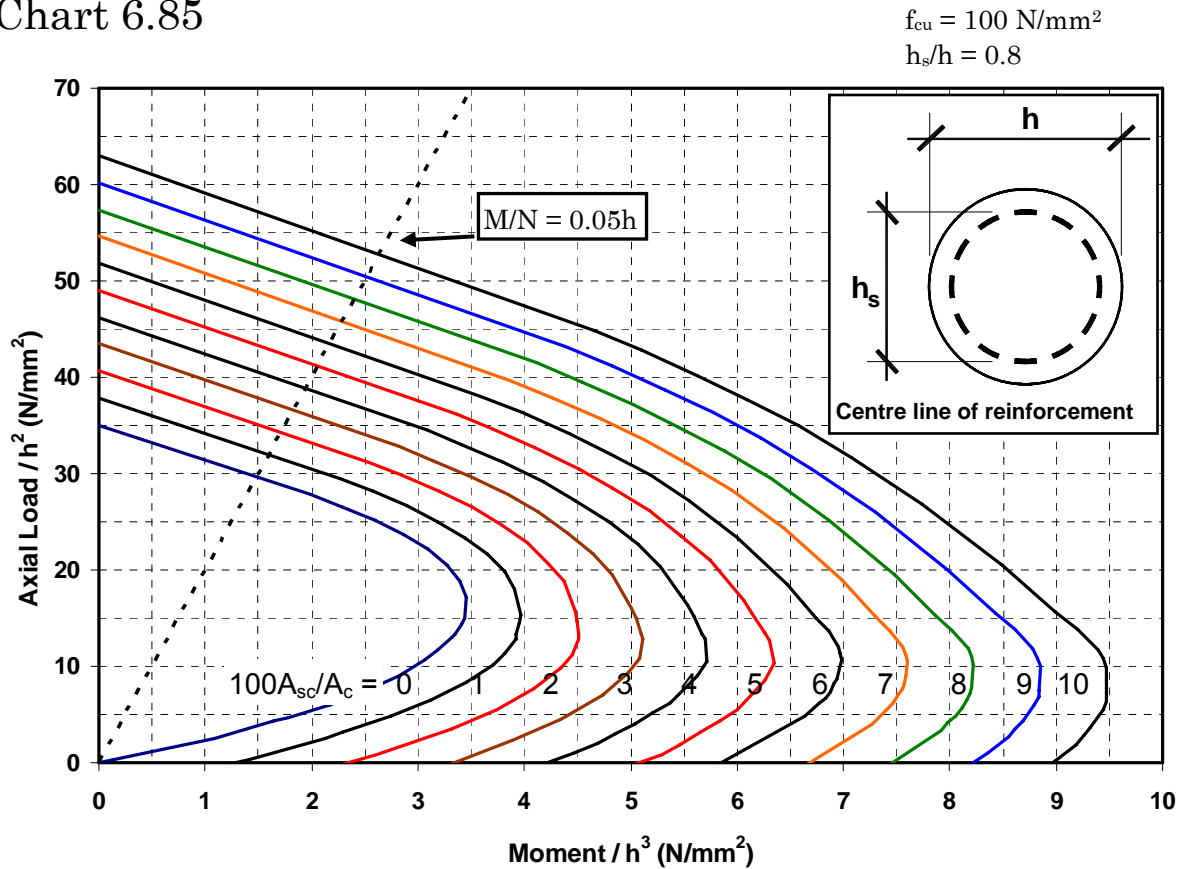


Chart 6.86

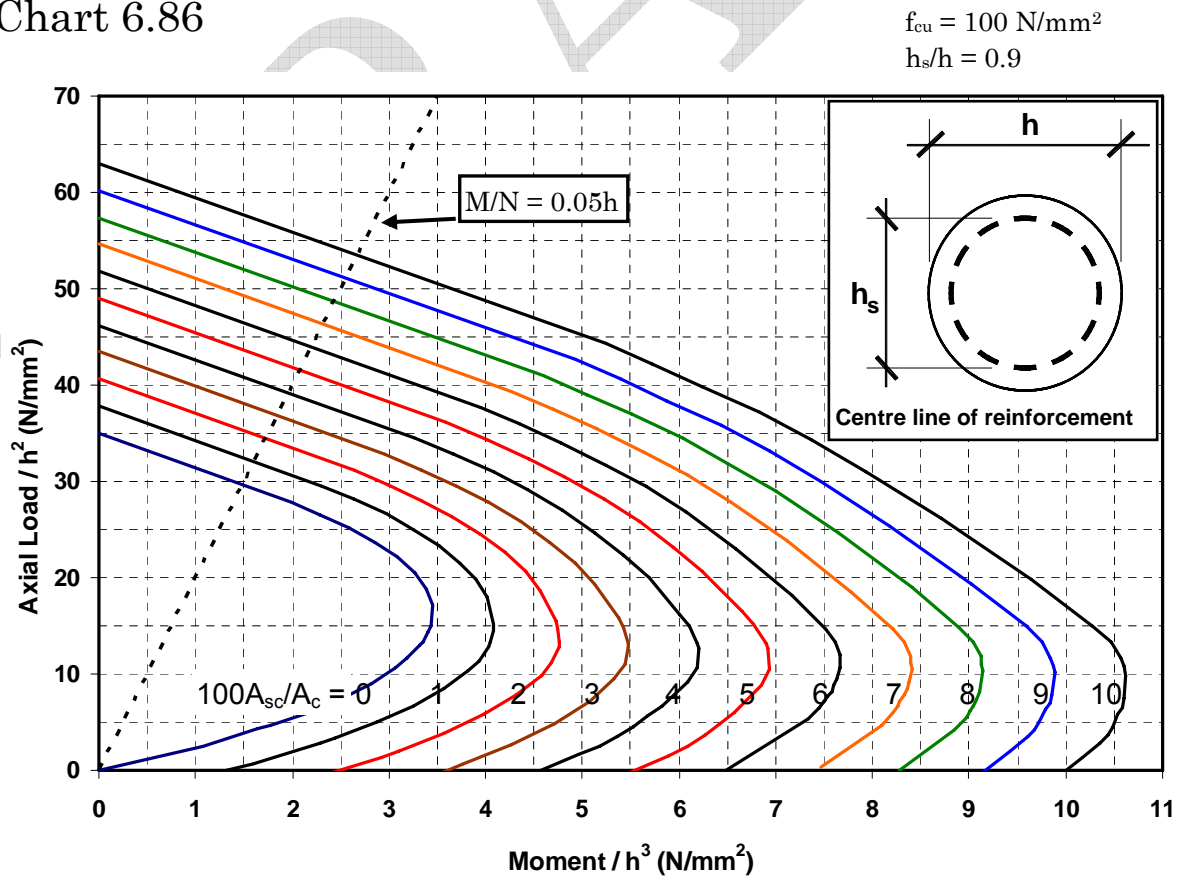


Chart 6.87

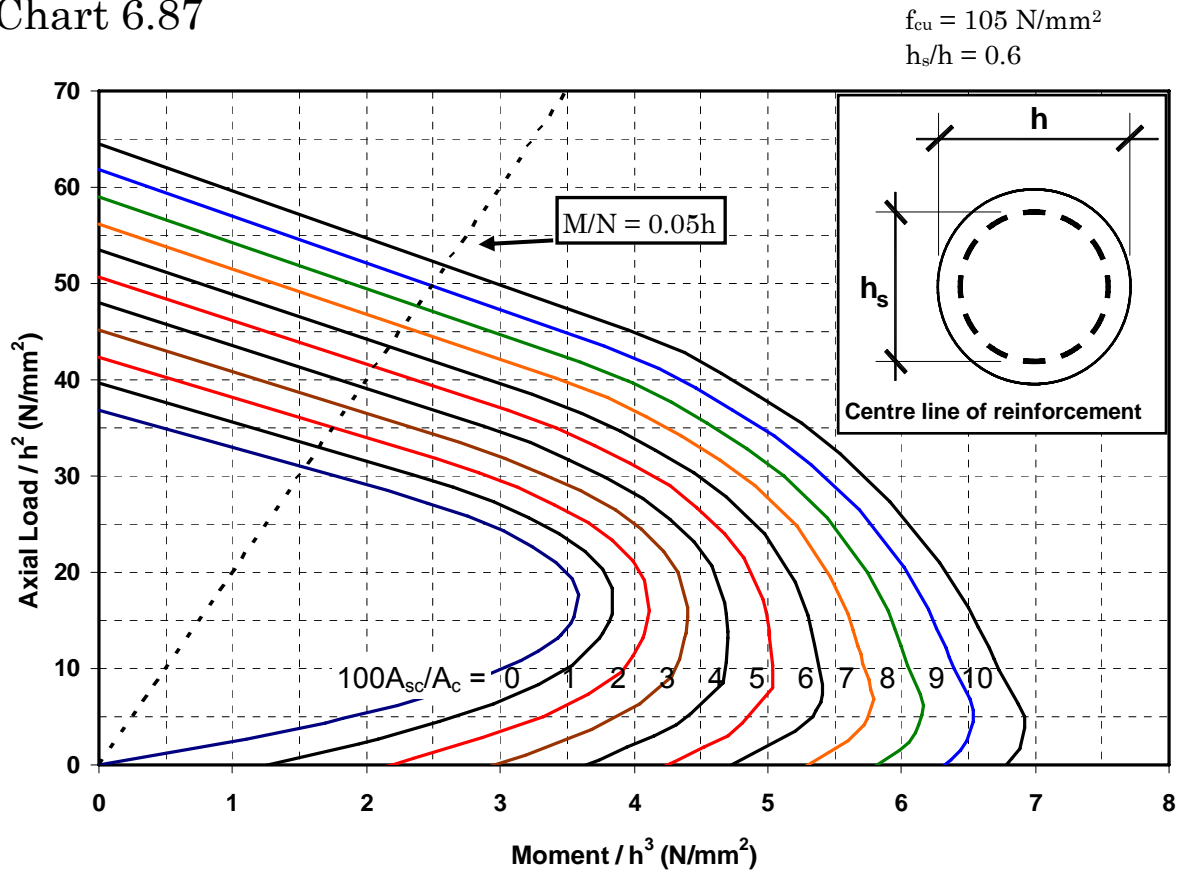


Chart 6.88

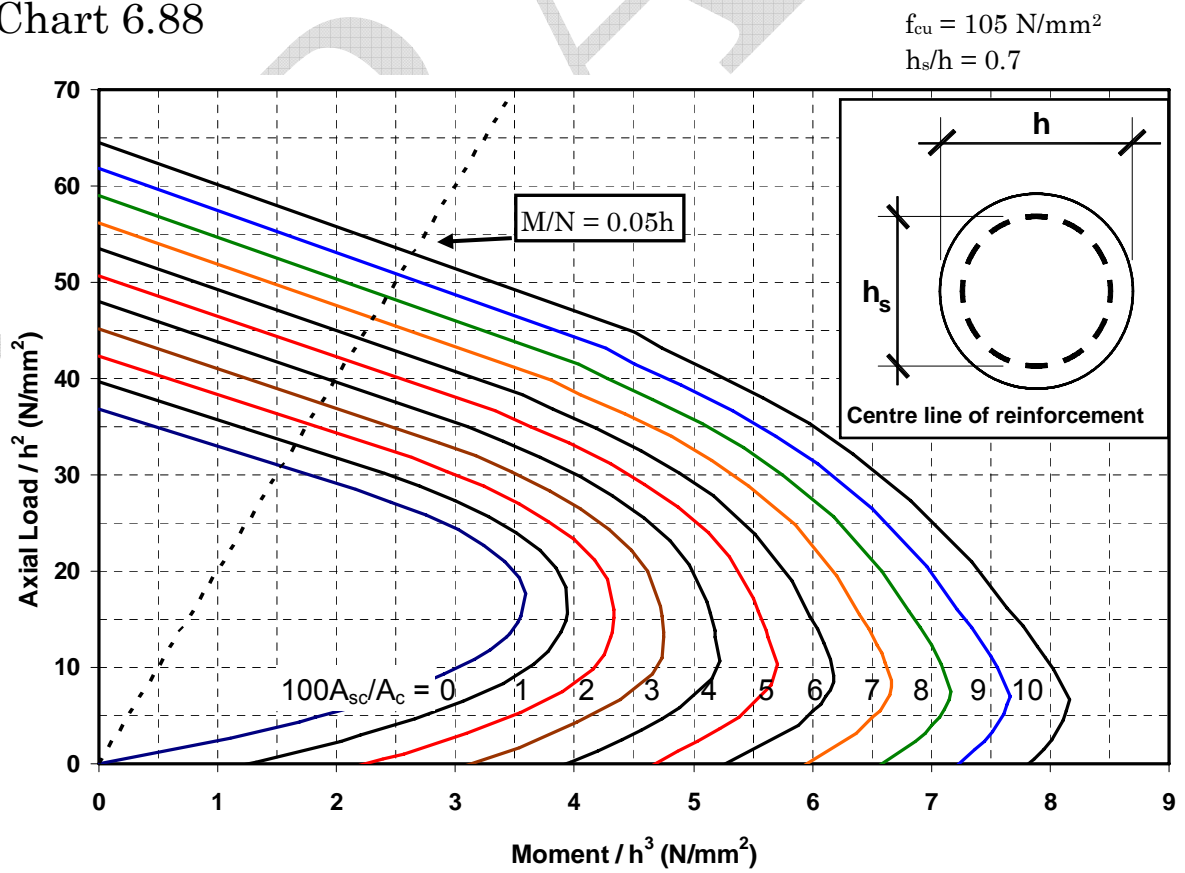


Chart 6.89

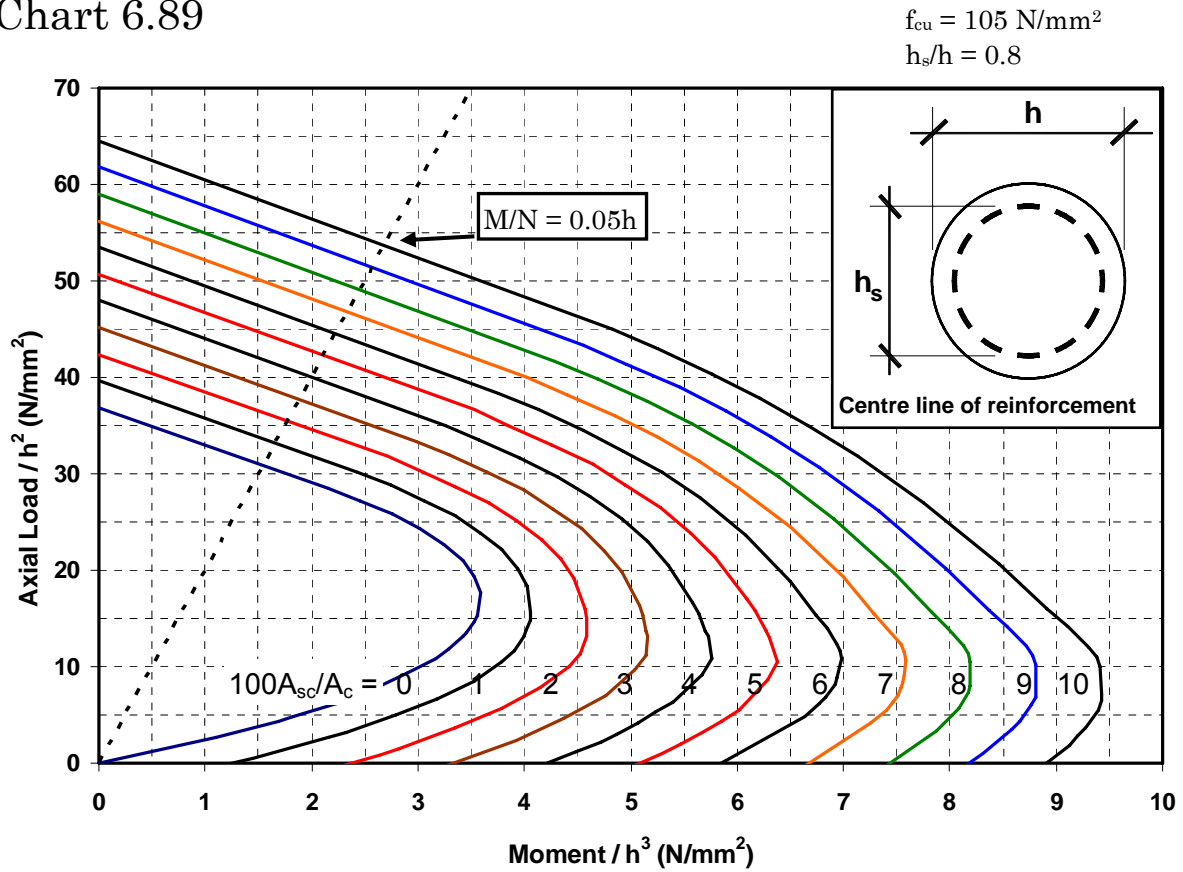
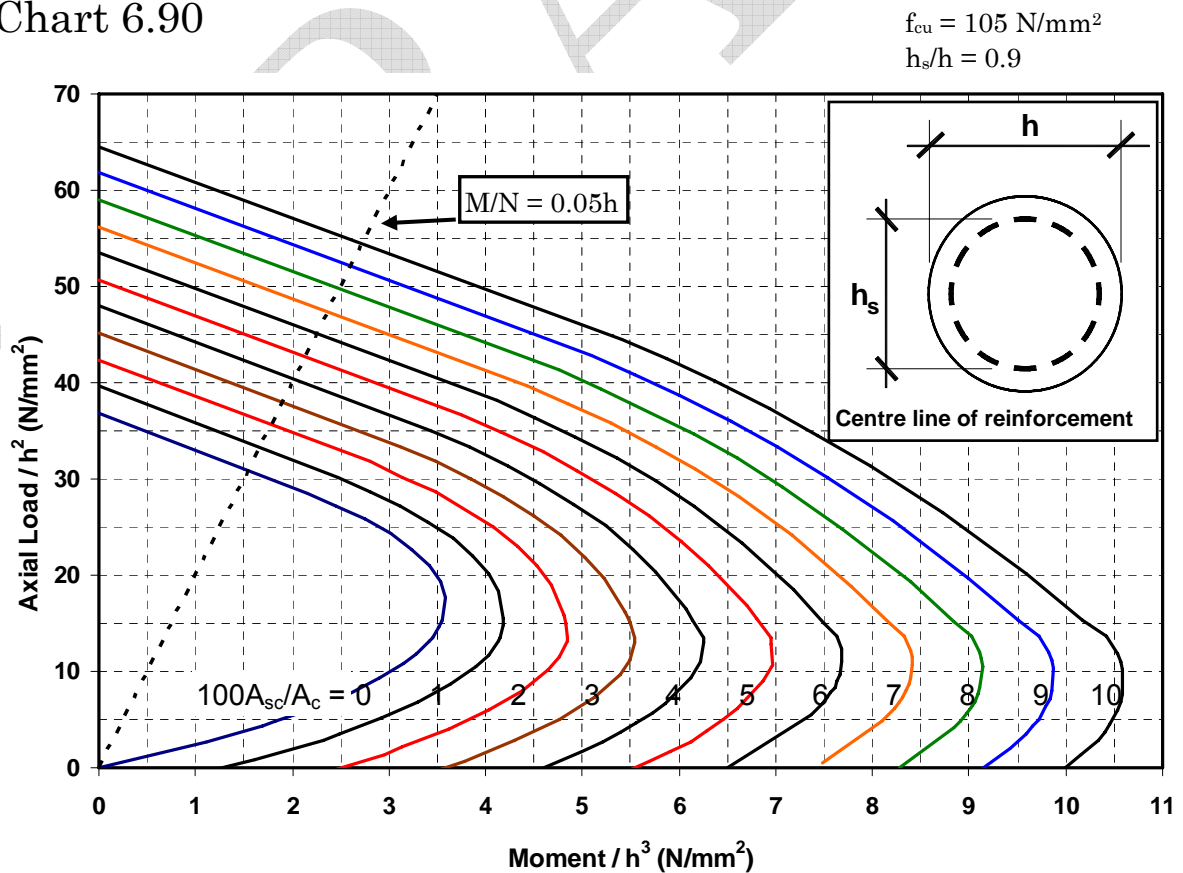


Chart 6.90



Section 7. Reference

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