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MORGAN JOHANSSON, RASMUS REMPLING

Department of Civil and Environmental Engineering
Division of Structural Engineering
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Chalmers University of Technology

SE-412 96 Göteborg

Sweden

Telephone: + 46 (0)31-772 1000

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Abstract

The Swedish Fortifications Agency has a long history in the design and maintenance of military fortifications and recently published a revised version of their design regulations; FKR 2011. These regulations can be said to represent the traditional Swedish view of the design of impulse loaded concrete structures and differs from the regulations normally used (i.e. Eurocode) for static design in Sweden today. Further, even though many parts of the content of FKR 2011 are similar to that of corresponding regulations in other countries it is not identical.

The purpose of this project was to assess different regulations for the design of reinforced concrete structures subjected to impulse loading. The focus was on FKR 2011 and its applicability for some common design criteria. One aim was to compare FKR 2011 with other similar regulations in order to identify similarities and differences to these; and if necessary, recommend possible improvements. Another aim was to provide an improved basis in order to give general recommendations of further investigations that is deemed necessary.

The main subjects compared were how the different regulations treated material strength, bending moment, shear and spalling/breaching. The comparisons were made based on the concept/expressions used in the respective regulations, and using several case studies of a simply supported slab strip of different geometry, concrete strength and reinforcement amount. Based on this it was concluded that the concept used for bending stiffness and moment capacity was similar in all the recommendations compared. Further, the method used in FKR for plastic deformation capacity is based on an older, today non-existing, reinforcement type and there is a need of further comparisons of the method used. The concept used in FKR for shear differs much compared with the regulations compared and it is suggested that further development of it should be made. Finally, the concept used for spalling and breaching is deemed to be okay to use.

Key words: impulse load, reinforced concrete structure, bending moment, plastic deformation capacity, shear, spalling and breaching, regulation comparison

Contents

Preface

The work presented in this report has been performed in the research project: “Utvärdering av Fortifikationsverkets regler” that was financed by the Swedish Fortifications Agency. Rolf Dalenius represented the Swedish Fortifications Agency. His fruitful comments and recommendations have been of significant value for the project results. Special thanks are also due to Johan Magnusson and Göran Svedbjörk, both at Sweco, for valuable comments on the report.

The project group consisted of: Adjunct Professor Morgan Johansson and Assistant Professor Rasmus Rempling .

1 Introduction

1.1 Background

It is of growing interest to innovate the structural design of concrete structures subjected to impulse loading; not only in military context, but also in civil applications. The Swedish Fortifications Agency has a long history in the design and maintenance of military fortifications and recently published a revised version of their design regulations; FKR 2011, Fortifikationsverket (2011). These regulations are to a large extent based on Swedish knowledge gathered during the 1970s and can be said to represent the traditional Swedish view of the design of impulse loaded concrete structures.

Due to its background and aim of use the regulations in FKR 2011 differs from the regulations normally used (i.e. Eurocode) for static design in Sweden today. In respect to ease-of-use, though, there would be an advantage if the conceptual difference of these regulations could be minimised as much as possible. Further, even though many parts of the content of FKR 2011 are similar to that of corresponding regulations in other countries it is not identical. Hence, a comparison of such regulations is of interest.

There is also ongoing research in e.g. Sweden on concrete structures subjected to impulse loading. The development of materials and innovation in application introduce needs for change in the design regulations used for designing fortified structures. On the basis, that there is a growing interest in using new types of concrete, such as fibre reinforced and/or high strength concrete, the design regulations could be outdated. This further motivates an assessment of the concurrent regulations.

1.2 Purpose and aim

The purpose of this project was to assess different regulations for the design of reinforced concrete structures subjected to impulse loading. The focus was on the Swedish design regulation FKR 2011, Fortifikationsverket (2011), and its applicability for some common design criteria. One aim was to compare FKR 2011 with other similar regulations in order to identify similarities and differences to these; and if necessary, recommend possible improvements. Another aim was to provide an improved basis in order to give general recommendations of further investigations that is deemed necessary, e.g. to incorporate the effect of new types of material such as fibre reinforced concrete and high strength concrete, which might lead to changes in the regulations.

1.3 Method

The project was carried out by a comparison of different common design criteria. As subject for this study a simply supported, reinforced concrete slab-strip, subjected to an evenly distributed impulse load was used. The expressions given in the regulations compared are presented, and when deemed possible, also physically explained in the report. Case studies are then carried out in order to illustrate the effect of different parameters and the results are compared and discussed. Based on this the accuracy and functionality of FKR 2011 is commented and recommendations are given for possible improvements

The regulations compared in this report are briefly presented in Section 2.1:

- FKR 2011, Fortifikationsverket (2011): Swedish regulation, impulse loading.
- Eurocode 2, SIS (2008): European regulation, static loading.
- UFC 3-340-02, DOD (2008): American regulation, impulse loading.
- Cormie *et al.* (2009): British regulation, impulse loading.

1.4 Limitations

This report is limited to the comparison of the structural response of reinforced concrete structures only. Hence, background for the resulting load from an explosion is not treated, and neither is dynamic analyses or equivalent methods (e.g. pressure-impulse relations) used here for such a load. The report is limited to the comparison of a strip in a one-way, simply supported slab. Hence, the expressions presented in the report are adapted to this case

2 General overview of report

2.1 Design regulations compared

In this report four different regulations are compared. These regulations are briefly described below in order to give a basic background for them being included in the comparison. Of these all but Eurocode 2 are design regulations specialised to be used for impulse loaded structures.

- **FKR 2011, Fortifikationsverket (2011):** This is the present design regulations of Swedish Fortifications Agency (*Fortifikationsverket*) for buildings and facilities that requires physical protection. FKR 2011 consists of three parts: FortLast, FortMaterial and FortSkydd (Load, Material and Protection), of which mainly FortSkydd, and some parts of FortMaterial, are treated in this report. These regulations are a compilation of a large number of reports, particularly from the Swedish Fortifications Agency and Swedish Defence Research Agency, and their various predecessors. The methodology described in FKR 2011 can be said to represent the traditional Swedish view of how to determine, and design against, the effect of an impulse load from an explosion. The regulation is in this report referred to as FKR.
- **Eurocode 2, SIS (2008):** This code is used in large parts of Europe and regulates the design of concrete structures; normally subjected to static loads. Hence, it is of interest to clarify in what way this code agrees and disagrees with the methodology used for impulse loaded structures. Since Eurocode 2 is used in several European countries there are some parameters that may be chosen individually by each country. If nothing else is mentioned the Swedish national choice for these parameters have been used. The regulation is in this report referred to as Eurocode 2.
- **UFC 3-340-02, DOD (2008):** This reference is published by the Department of Defence in US and contains a very large amount of information on both impulse loads and the structural response of different type of situations. This reference is considered here to represent the American approach to how an impulse-loaded structure should be handled and are hereafter referred to as UFC.
- **Cormie *et al.* (2012):** This reference is a book composed of independent chapters written by a little over ten different individuals with recognized expertise within the field of explosion loads and structural response due to impulse loading. The diversity of fields treated is larger than in UFC, but with a much more limited extent. This reference is considered here to represent the British approach to how an impulse-loaded structure should be handled and are hereafter referred to as Cormie *et al.*

A somewhat similar comparison of the references mentioned above has also been made in Johansson (2015a). Material from this reference has also partly been used in this report.

2.2 Studied topics

Based on the different design regulations presented in Section 2.1 a comparison is made regarding the following topics:

- Material strength
  - Influence of protection level
  - Design strength
- Bending moment
  - Moment capacity
  - Plastic deformation capacity
  - Elastic stiffness
  - Reinforcement requirements
- Shear
  - Design shear force
  - Shear capacity
  - Reinforcement requirements
  - Direct shear
- Spalling and breaching

How these topics are handled in different design regulations are described in Chapter 3 to 6. The effect of these regulations is then presented and compared in Chapter 7, using a case study of a simply supported strip of a one-way slab subjected to an evenly distributed impulse load.

2.3 Equivalent static load

The term equivalent static load is in this report used to denote the static load that corresponds to the situation that the dynamically loaded structure experiences at the moment when its maximum load capacity is reached. For a structure with a linear elastic response, see Figure 2.1a, this means that the maximum displacement obtained is the same for the case of a dynamic load and an equivalent static load. For a structure with an elastoplastic response, as schematically illustrated in Figure 2.1b, the equivalent plastic load will be the same as the load capacity R_{Rd}. For such a structure it is the combination of load capacity and plastic deformation capacity that together governs the final value of the equivalent static load.

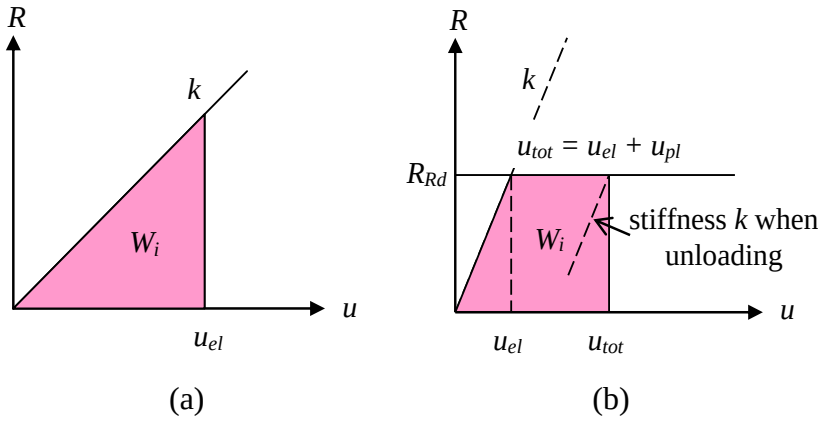

Figure 2.1 Schematic structural response of structure: (a) linear elastic response; (b) elastoplastic response.

The resulting equivalent static load of a given dynamic load depends on the mass, stiffness, load capacity and plastic deformation capacity of the loaded structure. Consequently, it is not possible to determine an equivalent static load just based on the dynamic load; the response of the structure is also of essential importance.

For a structure with an elastoplastic response that obtains plastic deformation, the equivalent static load q_{eq} corresponds to the design strength q_{Rd} of the structure. Hence, for such a case the equivalent static load can be defined as a function of the load capacity with regard to bending moment M_{Rd} and plastic deformation capacity u_{Rd}. Hence, if the deformation capacity is sufficient, i.e. $u_{pl,1} \leq u_{Rd}$, the equivalent static load q_{eq} can, for a simply supported slab strip of a one-way slab subjected to an evenly distributed load, be determined as

$$q_{eq} = q_{Rd} = \frac{8 \cdot M_{Rd}}{b \cdot l^2} \tag{2.1}$$

where b and l are the width and length, respectively, of the slab strip. For a simply supported beam subjected to an evenly distributed load q_{eq} this means that the total load capacity R_{Rd} can be determined as

$$R_{Rd} = q_{Rd} \cdot b \cdot l = \frac{8 \cdot M_{Rd}}{l} \tag{2.2}$$

What is here referred to as equivalent static load q_{eq} is in FKR referred to as design strength q_{Rd}. However, since the practical meaning of these two terms is the same in regard to what is discussed in this document the term equivalent load is used throughout the report.

3 Material

3.1 Orientation

None of the regulations compared in this report include any instructions of how to handle fibre reinforced concrete. Hence, in the comparison made here only ordinary concrete is treated. In the literature, though, there exist different regulations of how to design concrete structures using fibre reinforced concrete, e.g. SIS (2014). As in Eurocode 2, these regulations assume static loading and static response of the structure, and even though there should be good opportunities to use such guidelines even for impulse loaded structures there may still be areas that are unclear how they are affected to such load situations.

3.2 Influence of protection level

3.2.1 FKR

In FKR the material design strength and design coefficients are affected by the function availability and protection level for the structure studied. The function availability is defined according to Table 3.1 and the protection level according to Table 3.2. The highest level of protection of a structure corresponds to protection level A; this level more or less indicates that only elastic structural response is accepted. Protection level C, on the other hand, allows the largest damage on the structure and can be interpreted that the structure is fully utilised just prior to failure; i.e. it is assumed that there is no remaining capacity to withstand any more type of impulse loading.

Table 3.1 Definition of function availability. Based on Fortifikationsverket (2011).

Function availability	Accepted time for loss of function
Level 5	None
Level 4	< 30 min
Level 3	< 6 hours
Level 2	< 7 days
Level 1	> 7 days

Table 3.2 Definition of protection level. Based on Fortifikationsverket (2011).

Protection level	Number of load occasions	Description
A	> 5	The damages after each loading are assumed to be so small that they do not affect the function of the facility.
B3	3	The damage obtained shall be limited so that demands in the service limit state are still fulfilled.
B2	2	The damage obtained shall be limited so that demands in the service limit state are still fulfilled.
B1	1	The damage obtained shall be limited so that demands in the service limit state are still fulfilled.
C	1	Large plastic deformations are accepted and the ultimate deformation capacity is utilised. It is assumed that there is no remaining capacity in the structure to withstand extra loading.

3.2.2 Eurocode 2

In Eurocode 2 there is no distinction due to different types of functionality or protection level for structures subjected to impulse loading. In the load combinations for static loading there are different load coefficients, due to different safety, depending of what type of structure is studied. However, for an accidental load, which is the case for an explosion, these coefficients are all the same regardless of what type of structure is studied. However, the partial coefficients used to determine the material design strength is somewhat different compared to normal static loading, allowing higher utilisation of the material strength.

3.2.3 UFC

In UFC there is a definition of four different protection categories:

- **Protection category 1:** Protection of personnel against, among all, blast pressures and structural motion, and to shield them from the effects of primary and secondary fragments and falling portions of the structure.
- **Protection category 2:** Protect equipment, supplies and stored explosives from fragment impact, blast pressures and structural response.
- **Protection category 3:** Prevent communication of detonation by fragments, high-blast pressures, and structural response.
- **Protection category 4:** Prevent mass detonation of explosives as a result of subsequent detonations produced by communication of detonation between two adjoining areas and/or structures. This category is similar to Category 3.

The protection category affects what type of cross section is used to determine the bending moment capacity and plastic deformation capacity of the structure as described in Section 4.3.3 and Section 4.4.4, respectively.

3.2.4 Cormie *et al.*

In Cormie *et al.* there is a definition of two different protection categories:

- **Protection category 1:** Protection of personnel and equipment through the attenuation of blast pressure and to shield them from the effects of primary and secondary fragments and falling portions of the structure.
- **Protection category 2:** Protection of structural elements themselves from collapse under the action of blast loading.

Comparing these categories with those defined in UFC, see Section 3.2.3, it can be concluded that category 1 in Cormie *et al.* resembles that of category 1 and 2 in UFC, and that category 2 in Cormie *et al.* resembles that of category 3 and 4 in UFC.

3.3 Design strength of concrete and reinforcement

3.3.1 FKR

In FKR the concrete strength is limited to $f_{ck} \leq$ 50 MPa.

The design strength f_d of concrete and reinforcement is in FKR determined as

$$f_d = \frac{f_k}{\gamma_{fmn}} \tag{3.1}$$

where f_k is the characteristic strength and γ_{fmn} is a partial safety factor that takes into account the function availability and protection level according to Table 3.3. The definition of function availability and protection level is presented in Section 3.2.1.

Table 3.3 Partial coefficient γ_{fmn} for reinforcement due to protection level and function availability. Based on Fortifikationsverket (2011).

Function availability	Protection level		
	C	B	A
1-2	1.0	1.05	1.05
3-4	1.05	1.05	1.05
5	1.05	1.05	1.1

3.3.2 Eurocode 2

In Eurocode 2 the concrete strength is limited to $f_{ck} \leq$ 90 MPa. For concrete strength f_{ck} > 50 MPa, though, the expressions to determine the moment capacity in bending and plastic deformation capacity are affected.

The design strength of concrete is in Eurocode 2 determined as

$$f_d = \alpha_c \cdot \frac{f_k}{\gamma_c} \tag{3.2}$$

and for reinforcement

$$f_d = \frac{f_k}{\gamma_s} \tag{3.3}$$

where f_k is the characteristic strength and α_c is a coefficient taking into account long term effects and of unfavourable effects resulting from the way the load is applied. This coefficient can be individually chosen by different nations and in Sweden α_c = 1.0 for all cases. Further, γ_c = 1.2 and γ_s = 1.0 are partial coefficients used for accidental loads for concrete and reinforcement, respectively.

3.3.3 UFC

In UFC there is no mentioning of an upper limitation of the concrete strength. However, it is recommended that a concrete strength $f_{ck} \geq$ 28 MPa is used for blast resistant structures, and under no circumstances should a concrete of strength f_{ck} < 21 MPa be used.

The concept of safety used in UFC is different compared to that in FKR or Eurocode 2 and there are no partial safety factors. Instead UFC is based on the American concrete code ACI 318-11, ACI (2011), in which the design strength R_d is determined as

$$R_d = \phi \cdot R_{nom} \tag{3.4}$$

where R_{nom} is the nominal strength according to given expressions and ϕ is a strength reduction factor. In ACI 318-11 this reduction factor depends on which accuracy different capacities can be calculated. Consequently, for structures subjected to conventional static loading the strength reduction factor for e.g. bending moment is ϕ_M = 0.90 for bending moment while it for shear is ϕ_V = 0.75 in order to reflect that the former is easier to predict correctly than the latter. In UFC, though, ϕ = 1.0 for all type of capacity controls; i.e. no reduction of the load capacity is made for structures subjected to impulse loading.

In UFC the effect of high strain rates, i.e. the increase in strength due to intense dynamic loading, is taken into account. This is done by determining the dynamic strength f_{dyn} as

$$f_{dyn} = DIF \cdot f_{sta} \tag{3.5}$$

where *DIF* is the dynamic increase factor and f_{sta} is the static strength.

The DIF listed in UFC for different types of responses are presented in Table 3.4 for far and close design range. These DIF values can also be more accurately estimated by determining the strain rate $\dot{\varepsilon}$ for the actual situation and using relations given in UFC for concrete and reinforcement.

Table 3.4 Dynamic increase factor DIF for concrete and reinforcement used in equation (3.5). Based on DOD (2011).

Type of stress	Far design range [1]			Close in design range [2]		
	Concrete	Reinforcement		Concrete	Reinforcement	
	DIF_c	$DIF_{s,y}$	$DIF_{s,u}$	DIF_c	$DIF_{s,y}$	$DIF_{s,u}$
Bending	1.19	1.17	1.05	1.25	1.23	1.05
Diagonal tension	1.00	1.00	-	1.00	1.10	1.00
Direct shear	1.10	1.10	1.00	1.10	1.10	1.00
Bond	1.00	1.17	1.05	1.00	1.23	1.05
Compression	1.12	1.10	-	1.16	1.13	-

[1] Defined as a scaled distance $Z = r / W^{1/3} > 3.0$ m/kg$^{1/3}$; r = range, W =charge weight in TNT.

[2] Defined as a scaled distance $Z = r / W^{1/3} \leq 3.0$ m/kg$^{1/3}$; r = range, W =charge weight in TNT.

In UFC the strain hardening of the reinforcement is taken into account depending on what type of cross section is relevant. There are three different types of cross sections used to determine the bending moment capacity, see schematic illustration in Figure 3.1:

- **Type I:** The concrete is active and contribute to the moment capacity of the cross section. The concrete cover on both surfaces of the element remains intact.
- **Type II:** The concrete in compression is assumed to be crushed and hence does not contribute to the moment capacity of the cross section. Compression reinforcement, tied with stirrups, of equal amount to the tensile reinforcement is required to resist the moment. However, the crushed concrete is still present and hence contributes to the mass of the cross section.
- **Type III:** The concrete cover over the reinforcement on both surfaces of the element is completely disengaged, due to a combination of crushing, scabbing and spalling, and contributes with no mass. Compression reinforcement, tied with stirrups, of equal amount to the tensile reinforcement is required to resist the moment.

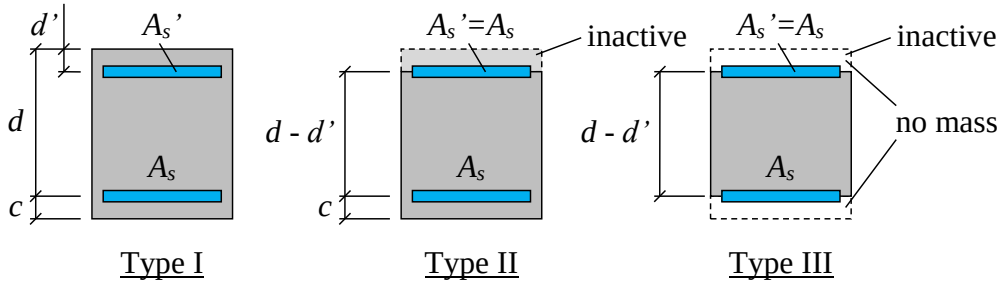

Figure 3.1 Definition of different cross sections used in UFC.

The difference between cross section of Type II and III is that the concrete cover of the latter has disengaged, and thus the mass of a Type III cross section will be smaller than that of a Type II cross section. This does not affect the moment resistance of the structure but will decrease its effective mass, and thereby also increase the external energy adopted to the structure from an impulse load.

Cross section of Type I is valid for protection category 1-2 while cross sections of Type II and III are valid for protection category 1-4, see Section 3.2.3.

For a Type I cross section no strain hardening is used; i.e. the reinforcement capacity is determined as

$$f_{s,I} = f_y \tag{3.6}$$

where f_y is the yield strength of the reinforcement. However, for a Type II section the reinforcement capacity is determined as

$$f_{s,II} = f_y + \frac{f_u - f_y}{4} \tag{3.7}$$

where f_u is the ultimate strength of the reinforcement, and for a Type III section the reinforcement capacity is determined as

$$f_{s,III} = \frac{f_y + f_u}{2} \tag{3.8}$$

3.3.4 Cormie *et al.*

The instructions given in Cormie *et al.* are based on UFC but also adapted to Eurocode 2. Therefore, a dynamic material capacity and the effect of strain hardening in the reinforcement is determined, as in UFC, but using the concept of design strength according to Eurocode 2.

In Cormie *et al.* there are no special information regarding the concrete strength and it is therefore interpreted here that the same regulations as those given in Eurocode 2 is valid; i.e. that the concrete strength is limited to $f_{ck} \leq$ 90 MPa, see Section 3.3.2.

The design strength of concrete is in Cormie *et al.* determined as

$$f_d = \alpha_c \cdot \frac{f_k}{\gamma_c} \tag{3.9}$$

and for reinforcement

$$f_d = \frac{f_k}{\gamma_s} \tag{3.10}$$

where f_k is the characteristic strength and α_c is a coefficient taking into account long term effects and of unfavourable effects resulting from the way the load is applied. In the UK this coefficient is choosen differently compared to Sweden; for bending moment α_c = 0.85 while it for shear is α_c = 1.0. Further, γ_c = 1.2 and γ_s = 1.0 are partial coefficients used for concrete and reinforcement, respectively, for accidental loads.

As in UFC the effect of high strain rates is taken into account in Cormie *et al* and the dynamic strength f_{dyn} is determined as

$$f_{dyn} = DIF \cdot f_{sta} \tag{3.11}$$

where *DIF* is the dynamic increase factor and f_{sta} is the static strength. The DIF for different types of responses is presented in Table 3.5. In contrast to UFC, here only one set of values is given and no values are given for bond[1]. Further, these DIF values are somewhat different to those used in UFC, listed in Table 3.4, and there are no instructions of how more accurate values of DIF can be determined. However, the concept is still the same as in UFC.

Table 3.5 Dynamic increase factor DIF for concrete and reinforcement used in equation (3.5). Based on Cormie et al (2009).

Type of stress	Far design range [1)]		
	Concrete	Reinforcement	
	DIF_c	$DIF_{s,y}$	$DIF_{s,u}$
Bending	1.25	1.20	1.05
Shear	1.00	1.10	-
Compression	1.15	1.10	-

In Cormie *et al.* the strain hardening of the reinforcement is taken into account in a way similar to UFC. However, in Cormie *et al.* only cross sections of Type I and II are used, see Figure 3.1 in Section 3.3.3. For a Type I cross section no strain hardening is used; i.e.

$$f_{s,I} = f_y \tag{3.12}$$

where f_y is the yield strength of the reinforcement, and for a Type II section the reinforcement capacity is determined as

$$f_{s,II} = f_y + \frac{f_u - f_y}{4} \tag{3.13}$$

where f_u is the ultimate strength of the reinforcement.

[1] It is here assumed that the values given for shear is also valid for direct shear.

4 Bending moment

4.1 Reinforcement requirements

4.1.1 Orientation

The reinforcement amount for bending is defined as

$$\rho = \frac{A_s}{b \cdot d} \tag{4.1}$$

where A_s is the reinforcement area on the tensile side, and b and d are the width and effective height of the cross section, respectively, see cross section in Figure 4.1

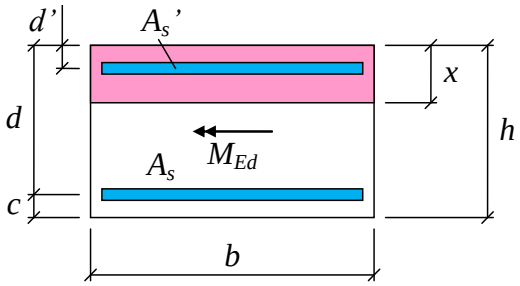

Figure 4.1 Geometry of cross section subjected to bending.

In concrete structures there is often a requirement of a minimum reinforcement amount and there may also be a limit for a maximum amount. The overall purpose to set limits on the minimum and/or maximum reinforcement amount is to make sure that the response of the concrete structure does not become too brittle.

For bending moment the main purpose of the minimum reinforcement amount is to insure that the bending moment capacity M_{crack} of the uncracked cross section does not exceed the design moment capacity M_{Rd} of the reinforced section; i.e.

$$M_{crack} \leq M_{Rd} \tag{4.2}$$

In the regulations compared in this report, though, this requirement is set using a minimum reinforcement amount for the main reinforcement in bending.

4.1.2 FKR

In FKR the minimum reinforcement amount is determined as

$$\rho_{\min,FKR} = \frac{f_{ck,cube} + 30}{f_{yk} + 100} \tag{4.3}$$

where $f_{ck,cube}$[2] and f_{yk} are the characteristic concrete cube strength and characteristic reinforcement yield strength, respectively, given in [MPa]. According to Svedbjörk (2016) the background for this minimum reinforcement amount is to prevent local failure modes; i.e. to make sure that the yield line failure modes assumed appear in the concrete slab.

[2] In FKR the parameter f_{ck}, i.e. the characteristic concrete cylinder strength, is incorrectly given in the equation instead of $f_{ck,cube}$.

In order to make sure that the full potential of the plastic deformation capacity is not limited by concrete compressive failure the maximum reinforcement amount is limited to

$$\rho_{\max,FKR} = 0{,}50\,\% \tag{4.4}$$

4.1.3 Eurocode 2

In Eurocode 2 the minimum reinforcement amount is determined by

$$\rho_{\min,EC} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \;\geq\; 0.013 \tag{4.5}$$

where f_{ctm} is the concrete tensile mean strength and f_{yk} is the characteristic yield strength of the reinforcement.

In Eurocode 2 (Swedish version) there is no upper limit of allowed reinforcement amount for bending reinforcement. However, there is an upper limit in order to use the plastic redistribution of a structure and for concrete with $f_{ck} \leq$ 50 MPa this limit can be expressed as

$$\frac{x}{d} \leq 0.45 \tag{4.6}$$

where x is the height of the compressive zone and d is the effective height.

It can be shown that

$$\frac{x}{d} = 0.8 \cdot \omega_s = 0.8 \cdot \frac{f_y}{f_c} \cdot \rho \tag{4.7}$$

for a rectangular cross section subjected to no normal forces and where $A_s' = 0$, see Johansson and Laine (2012). Combining equation (4.6) and (4.7) gives an expression for allowed reinforcement amount as

$$\rho_{\max,EC} = 0.56 \cdot \frac{f_c}{f_y} \tag{4.8}$$

where f_c and f_y are the concrete compressive strength and reinforcement yield strength, respectively. If positive effect of compressed reinforcement A_s' is to be included in the moment capacity of the cross section (put a demand on stirrups available) this reinforcement amount may increase further.

4.1.4 UFC

In UFC the minimum reinforcement amount is determined by

$$\rho_{\min,UFC} = 0.1557 \cdot \frac{f_{ck}^{0.5}}{f_{yk}} \tag{4.9}$$

where f_{ck} and f_{yk} are the characteristic concrete compressive strength and reinforcement yield strength, respectively, given in [MPa].

The maximum reinforcement amount is determined by

$$\rho_{\max} = 0.75 \cdot \rho_{bal} \tag{4.10}$$

where the reinforcement amount is defined as

$$\rho = \frac{A_s}{b \cdot d} \tag{4.11}$$

for a Type I cross section (moderate plastic deformations $\theta \leq 2°$) and

$$\rho = \frac{A_s}{b \cdot (d - d')} \tag{4.12}$$

for a Type II cross section (large plastic deformations $\theta > 2°$), see Section 3.3.3 and 4.4.4. The limit value ρ_{bal} corresponds to the reinforcement amount which gives a so called balanced cross section; i.e. a cross section in which crushing of the concrete and yielding of the reinforcement occurs at the same time. In UFC the balanced reinforcement amount is determined as

$$\rho_{bal} = 0.85 \cdot k_1 \cdot \left(\frac{600}{600 + f_y} \right) \cdot \frac{f_c}{f_y} \tag{4.13}$$

where

$$k_1 = 0.85 - 0.05 \cdot \left(\frac{f_c - 28}{7} \right) \geq 0.85 \tag{4.14}$$

and f_c and f_y are given in [MPa].

Equation (4.9), (4.13) and (4.14) have been recalculated from imperial units to SI units using the conversion factor 1 psi = 6.895 kPa.

4.1.5 Cormie *et al.*

In Cormie *et al.* there are no special instruction regarding minimum or maximum reinforcement amount and it is here therefore interpreted that the demands given in Eurocode 2 is valid, see Section 4.1.3.

4.2 Stiffness

4.2.1 Cracked and uncracked cross section

The stiffness of a simply supported concrete beam subjected to an evenly distributed load can be determined as

$$k = \frac{384}{5} \cdot \frac{E_c \cdot I_c}{l^3} \tag{4.15}$$

where E_c is the concrete Young's modulus, I_c is the moment of inertia of the concrete cross section and l is the span length of the beam.

The moment of inertia of a concrete cross section depends on whether the concrete is cracked or not. For an uncracked cross section, denoted State I, the moment of inertia can, for a rectangular cross section, be approximated as

$$I_I = \frac{b \cdot h^3}{12} \tag{4.16}$$

For a cracked cross section the moment of inertia I_{II} is reduced; how much depends on the cross section dimensions, material properties and the reinforcement amount. If the effect of the reinforcement A_s' on the compressive side is neglected the moment of inertia I_{II} for a cracked concrete section can, according to Al-Emrani *et al.* (2011), be determined as

$$I_{II} = \frac{b \cdot x_{II}^{\ 3}}{12} + b \cdot x_{II} \cdot \left(\frac{x_{II}}{2} - x_{cp} \right)^2 + \alpha \cdot A_s \cdot \left(d - x_{cp} \right)^2 \tag{4.17}$$

Here x_{II} is the height of the compressive zone in state II, x_{cp} is the distance to the centre point of the equivalent cross section,

$$\alpha = \frac{E_s}{E_c} \tag{4.18}$$

is the ratio between the Young's modulus of reinforcement and concrete, respectively, A_s is the tensile reinforcement area, and b and d are the width and effective height of the cross section, respectively.

For a case of pure bending, i.e. no normal force acting on the cross section, it can be shown that the distance from the concrete edge to the centre point x_{cp} of the equivalent cross section is the same as the height of the compressive zone; i.e. $x_{cp} = x_{II}$ which also means that equation (4.17) instead can be expressed as

$$I_{II} = \frac{b \cdot x_{II}^{\ 3}}{3} + \alpha \cdot A_s \cdot \left(d - x_{cp} \right)^2 \tag{4.19}$$

The compressive zone is then determined from the expression for the location of the centre point in the equivalent cross section; i.e.

$$x_{cp} = x_{II} = \frac{b \cdot \frac{x_{II}^{\ 2}}{2} + \alpha \cdot A_s \cdot d}{b \cdot x_{II} + \alpha \cdot A_s} \tag{4.20}$$

This can be rewritten as

$$x_{II}^{\,2} + \frac{2 \cdot \alpha \cdot A_s}{b} \cdot \left(x_{II} - d\right) = 0 \tag{4.21}$$

from which the height of the compressive zone in state II can be determined as

$$x_{II} = -\frac{\alpha \cdot A_s}{b} + \sqrt{\left(\frac{\alpha \cdot A_s}{b}\right)^2 + \frac{2 \cdot \alpha \cdot A_s \cdot d}{b}} \tag{4.22}$$

The final stiffness will be somewhere in between the stiffness obtained for a fully uncracked (stiffness k_I) and fully cracked (stiffness k_{II}) beam. This can be analytically determined but can, depending on the case, be relatively complex. For an impulse loaded structure, though, it is normally conservative to assume a stiffness corresponding to that of a fully cracked beam; i.e. $k = k_{II}$.

4.2.2 FKR

In FKR the moment of inertia I_{II} for a cracked concrete section is determined as

$$I_{c,FKR} = I_{II,FKR} = \left(5.4 \cdot \rho + 0.016\right) \cdot b \cdot d^3 \tag{4.23}$$

where ρ is the reinforcement amount, and b and d are the width and effective height of the cross section, respectively.

4.2.3 Eurocode 2

In Eurocode 2 there is no explicit description of how the moment of inertia I_{II} for a cracked concrete section is to be determined. Therefore,

$$I_{c,EC} = I_{II,EC} = \frac{b \cdot x_{II}^{\,3}}{3} + \alpha \cdot A_s \cdot \left(d - x_{II}\right)^2 \tag{4.24}$$

is used here; i.e. the same expression as in equation (4.19). Further, in accordance with Johansson and Laine (2012), it is also deemed reasonable to assume a fully cracked beam and thus use I_{II} to represent the moment of inertia in the whole beam.

4.2.4 UFC

In UFC the moment of inertia is determined as the average of the uncracked (State I) and cracked (State II) cross section; i.e.

$$I_{c,UFC} = \frac{I_I + I_{II,UFC}}{2} \tag{4.25}$$

where I_I is determined according to equation (4.16) and

$$I_{II,UFC} = F \cdot b \cdot d^3 \tag{4.26}$$

where F is a coefficient according to Figure 4.2. In UFC the value of F is only presented using these graphs; i.e. no equations. However, a control strongly indicates that the relations presented in Figure 4.2a have been determined using equation (4.24); i.e. the same expression as used in Eurocode 2.

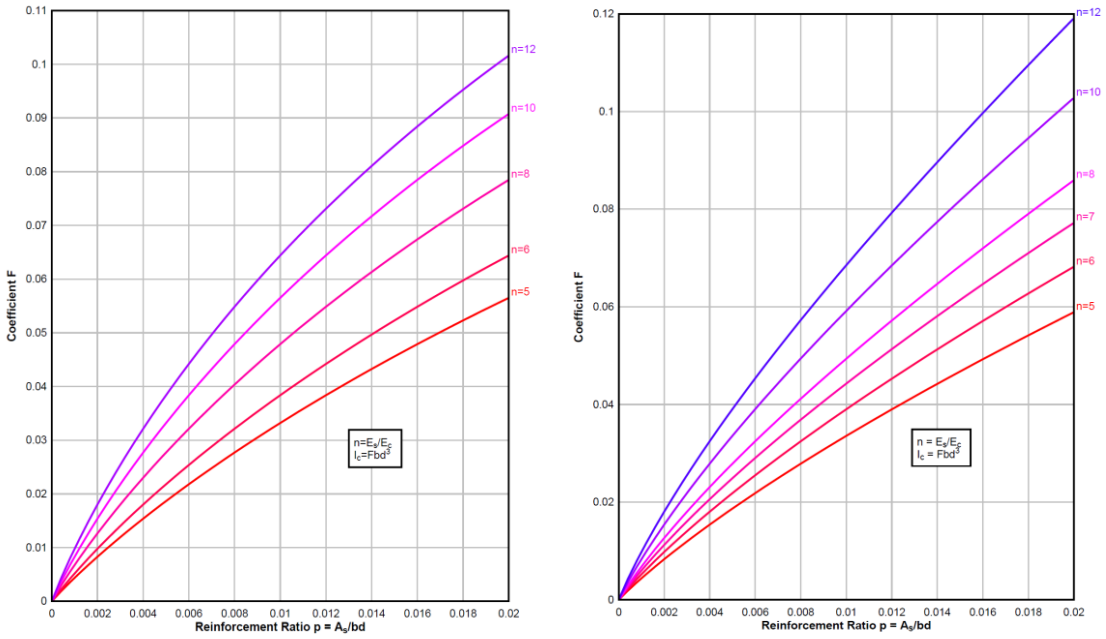

Figure 4.2 *Coefficient for moment of inertia of cracked concrete cross section: (a) tensile reinforcement only, (b) equal reinforcement amount on both sides. Based on UFC, DOD (2008).*

4.2.5 Cormie *et al.*

In Cormie *et al.* the moment of inertia used is based on a cracked section only; i.e. no average value on the moment of inertia, as is the case in UFC in equation (4.25), is used.

For a Type I cross section the cracked moment of inertia is determined as

$$I_{c,Co,I} = I_{II,Co,I} = F_I \cdot b \cdot d^3 \tag{4.27}$$

where F_I is a coefficient according to Figure 4.2a. Based on the comment regarding this figure, given in Section 4.2.4, this means that the moment of inertia used in Cormie *et al.* for a Type I cross section is the same as that used for a cracked section in Eurocode 2.

For a cross section Type II, though, the moment of inertia is determined as

$$I_{c,Co,II} = I_{II,Co,II} = F_{II} \cdot b \cdot (d - d')^3 \tag{4.28}$$

where F_{II} is a coefficient according to Figure 4.2b.

4.3 Moment capacity

4.3.1 FKR

In FKR the moment capacity is determined as

$$M_{Rd,FKR} = 0.95 \cdot f_y \cdot A_s \cdot d \tag{4.29}$$

where f_y is reinforcement yield strength, A_s is reinforcement area on the tensile side of the cross section and d is effective height.

This is an approximation to the expression used in Eurocode 2, see Section 4.3.2, and works well for small reinforcement amounts.

4.3.2 Eurocode 2

In Eurocode 2 the moment capacity can, for a rectangular cross section according to Figure 4.3, be determined as

$$M_{Rd,EC} = F_s \cdot z = f_y \cdot A_s \cdot (d - 0.4x) \tag{4.30}$$

where f_y is reinforcement yield strength, A_s is reinforcement area on the tensile side of the cross section, d is effective height and x is the height of the compressive zone.

The latter may be determined as

$$x = \frac{f_y \cdot A_s}{0.8 \cdot f_c \cdot b} \tag{4.31}$$

where f_c is the concrete compressive strength and b is the width of the cross section. Potential influence of the reinforcement A_s' on the compressive side is not included here. If the configuration of stirrups is satisfactory, though, the effect of A_s' may also be taken into account when determining x and M_{Rd}.

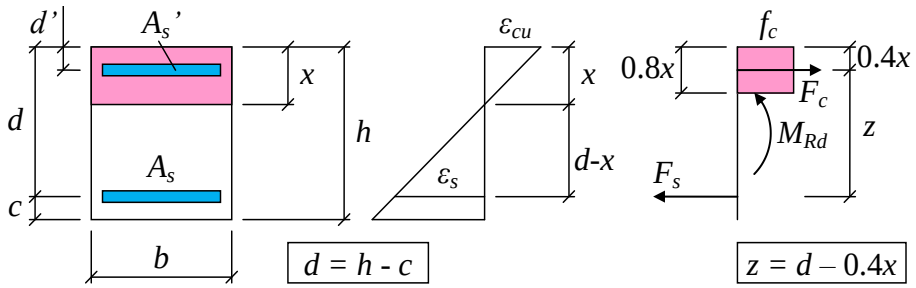

Figure 4.3 Analysis of concrete cross section subjected to bending.

4.3.3 UFC

In UFC the bending moment capacity is determined in two different ways, depending on which type of cross section that is assumed, see Section 3.3.3. For cross section of Type I, see Section 3.3.3, the moment capacity is determined as

$$M_{Rd,I,UFC} = f_s \cdot A_s \cdot (d - 0.5x) \tag{4.32}$$

where f_s is the reinforcement strength according to Section 3.3.3 and the height of the compressive zone x is calculated as

$$x = \frac{f_s \cdot A_s}{0.85 \cdot f_c \cdot b} \tag{4.33}$$

In this report, the influence of reinforcement A_s' on the compressive side is approximately neglected when determining both $M_{Rd,I,UFC}$ and x.

However, for cross section of Type II or III, see Section 3.3.3, the concrete cover is assumed to be inactive, which also affects the internal lever arm z. Such a response requires that $A_s' = A_s$, and that there are enough amount of stirrups embracing the compressive reinforcement, thus hindering it to buckle. If this requisite is met the moment capacity can instead be determines as

$$M_{Rd,II,UFC} = f_s \cdot A_s \cdot (d - d') \tag{4.34}$$

4.3.4 Cormie *et al.*

The instructions given in Cormie *et al.* are based on UFC but also adapted to Eurocode 2. The moment capacity is therefore, as in UFC, determined in two ways, as described in Section 4.3.3, but using a concept according to Eurocode 2. This means that the moment capacity, for a cross section of Type I, is determined according to equation (4.30), and for a cross section of Type II or III according to equation (4.34).

4.4 Deformation capacity

4.4.1 Orientation

The plastic deformation capacity u_{rd} is based on the plastic rotation capacity θ_{Rd} as is schematically shown in Figure 4.4. From this the deformation capacity can for a simply supported beam be determined as

$$u_{Rd} = \frac{\theta_{Rd} \cdot l}{2} \tag{4.35}$$

where l is the length of the beam. To simplify the comparison of different structures the ratio l / u_{Rd} is used here.

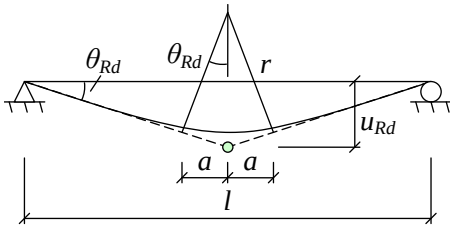

Figure 4.4 Relation between plastic rotation θ_{Rd} and plastic deformation u_{Rd} in a simply supported beam.

4.4.2 FKR

The method used in FKR to determine the plastic deformation capacity of a concrete structure is based on Bk 25, Fortifikationsförvaltningen (1973a, b). The derivation of the expressions used is thoroughly treated in Johansson and Laine (2012), and is not described in detail here. The rotational capacity is originally derived to be the minimum due to rupture of reinforcement or concrete crushing. However, in FKR only the expression based on ruptured reinforcement is given. Here though, the limitation due to concrete crushing, from Bk 25, is also presented.

To determine what type of failure is obtained a reinforced concrete section according to Figure 4.3, Section 4.3.2, is used in which the mechanical reinforcement ratio can be determined as

$$\omega_s = \rho \cdot \frac{f_y}{f_c} = \frac{A_s}{b \cdot d} \cdot \frac{f_y}{f_c} \tag{4.36}$$

A balanced value of the mechanical reinforcement ratio can be defined as

$$\omega_{s,bal} = \frac{0.8 \cdot \varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{su}} \tag{4.37}$$

where ε_{cu} and ε_{su}[3] are the ultimate strain at failure in concrete and reinforcement, respectively. This corresponds to the cross section with balanced reinforcement ratio described in equation (4.13) in Section 4.1.4. If $\omega_s < \omega_{s,bal}$ the maximum plastic rotation capacity will be limited due to rupture of the reinforcement, while if $\omega_s > \omega_{s,bal}$ failure will be reached due to concrete crushing.

[3] Here, ε_{su} is not the maximum tensile strain at failure for a single bar; instead it indicates the average reinforcement strain over the plastic hinge with length $2a$ in Figure 4.4.

In FKR the plastic deformation capacity, due to rupture of reinforcement, is given as

$$u_{Rd,s,FKR} = 0.26 \cdot \varepsilon_{su} \cdot \left(1 + 0.3 \cdot \frac{l}{d}\right) \cdot l \tag{4.38}$$

where ε_{su} is the average reinforcement strain over the plastic hinge (length $2a$ in Figure 4.4), l is the length of the beam and d is the effective height. No corresponding value is given for concrete crushing, though.

From Johansson and Laine (2012) the original expressions in Bk 25 can be found. Assuming failure due to concrete crushing, i.e. $\omega_s > \omega_{s,bal}$, the rotational capacity in the mid span can be determined as

$$\theta_{Rd,c,Bk} = \frac{0.4 \cdot \varepsilon_{cu}}{\omega_s} \cdot \left(1 + 0.3 \cdot \frac{l}{d}\right) \tag{4.39}$$

and assuming failure due to rupture of the reinforcement, i.e. $\omega_s < \omega_{s,bal}$, the rotational capacity θ_f in the mid span can be determined as

$$\theta_{Rd,s,Bk} = \frac{0.4 \cdot \varepsilon_{su}}{0.8 - \omega_s} \cdot \left(1 + 0.3 \cdot \frac{l}{d}\right) \tag{4.40}$$

These expressions are also used in this report to represent the rotational capacity of FKR.

Combining equation (4.35) and (4.40) gives

$$u_{Rd,s,Bk} = \frac{0.2 \cdot \varepsilon_{su}}{0.8 - \omega_s} \cdot \left(1 + 0.3 \cdot \frac{l}{d}\right) \cdot l \tag{4.41}$$

and comparing this with equation (4.38) gives

$$0.26 = \frac{0.2}{0.8 - \omega_s} \quad \rightarrow \quad \omega_s = 0.031 \tag{4.42}$$

which means that it in the expression in equation (4.38) is assumed a cross section with a mechanical reinforcement ratio $\omega_s = 0.031$. Using equation (4.36) and assuming $f_y = 500$ MPa and $f_c = 25$ MPa or 50 MPa this gives that the reinforcement ratio for such a case corresponds to $\rho = 0{,}16$ % and 0,31 %, respectively; i.e. a rather small ratio.

The value for the ultimate concrete strain can, for structures mainly subjected to bending, be determined as $\varepsilon_{cu} = 3.5$ ‰. However, for structures mainly subjected to compression, $\varepsilon_{cu} = 2.4$ ‰ should be used instead. If the structure should be able to withstand more than one load occasion, compare influence of protection level in Section 3.2.1, a modified concrete strain should be determined as

$$\varepsilon_{cu,\mathrm{mod}} = \varepsilon_{c0} + \frac{\varepsilon_{cu} - \varepsilon_{c0}}{n} \tag{4.43}$$

where

$$\varepsilon_{c0} = \frac{f_{cd}}{0.8 \cdot E_{ck}} \tag{4.44}$$

and n is the number of load occasions.

The average reinforcement strain ε_{su} used here is in FKR set to the ultimate strain at failure of a single reinforcement. For reinforcement of class C this means that ε_{su} = 75 ‰, which can be compared with the strain value of 80 ‰ proposed in Bk 25. However, this latter value is related to an older, and more ductile, type of reinforcement (Ks 40) that is no longer used in Sweden. Hence, this value is judged to be too liberal for the reinforcement types used in Sweden today (K500). If using a reinforcement of class C (the most ductile type available) a more realistic value to use would be ε_{su} = 30 ‰, see Johansson and Laine (2012). This recommendation is based on an extensive experimental test series carried out at KTH on reinforced concrete slab strips subjected to static tests, Ansell and Svedbjörk (2000, 2003, 2005).

If the structure should be able to withstand more than one load occasion a modified average steel strain should be determined as

$$\varepsilon_{su,\mathrm{mod}} = \varepsilon_{s0} + \frac{\varepsilon_{su} - \varepsilon_{s0}}{n} \tag{4.45}$$

where

$$\varepsilon_{s0} = \frac{f_k}{E_{sk}} \tag{4.46}$$

and n is the number of load occasions.

It can also be pointed out that in Bk 25 there was a requirement used for protection level B that $l / u \geq 33$, which is no longer included in FKR 2011. This requirement, though, was related to functional requirements (e.g. the possibility to open internal doors) and hence not a requirement related to the ultimate load capacity.

4.4.3 Eurocode 2

In Eurocode 2 the rotational capacity is determined based on the relations for θ_{pl} given in Figure 4.5, which depends on the concrete strength and class of reinforcement as defined in Table 4.1. The rotational capacity[4] is determined as

$$\theta_{Rd,EC} = \frac{k_\lambda}{2} \cdot \theta_{pl} \tag{4.47}$$

where

$$k_\lambda = \sqrt{\frac{\lambda}{3}} \tag{4.48}$$

is a coefficient taken into account the shear slenderness λ. This, in turn, is defined as

$$\lambda = \frac{l_0}{d} \tag{4.49}$$

where l_0 is the distance from zero moment and the plastic hinge, and d is the effective height. For a simply supported beam, as shown in Figure 4.4, $l_0 = l / 2$.

[4] In Eurcode 2 the definition of θ_{pl} differs compared to that defined in Figure 4.4. To adjust for this difference the denominator 2 is introduced in equation (4.47). Hence, the expression given here is adjusted to correspond to the definition given in Figure 4.4; see also Section 4.4.6.

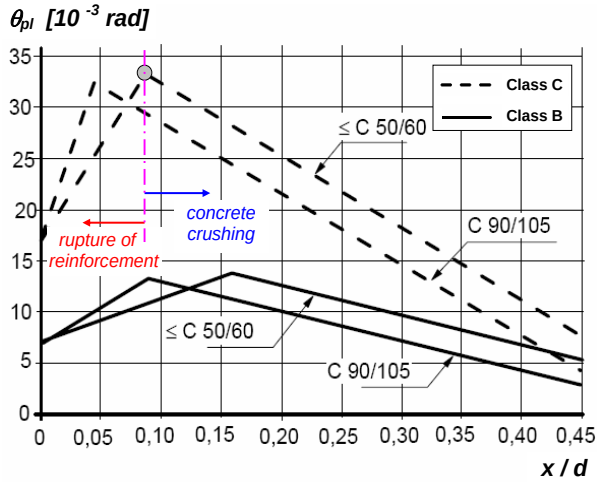

Figure 4.5 *Relations to determine the plastic rotation capacity θ_{pl} for different types of concrete and reinforcement. The cause of failure is marked. Based on Eurocode 2, SIS (2008).*

The limitations to use the relations in Figure 4.5 is for concrete ≤ C 50/60 that

$$\frac{x}{d} \leq 0.45 \tag{4.50}$$

and for concrete ≥ C 55/67 that

$$\frac{x}{d} \leq 0.35 \tag{4.51}$$

where x is the height of the compressive zone.

Table 4.1 *Definition of reinforcement classes according to Eurocode 2, SIS (2008).*

Class	f_{yk} [MPa]	$\gamma = f_{uk} / f_{yk}$ [-]	$\varepsilon_{s,fu}$ [%]
B	400 - 600	≥ 1.08	≥ 5.0
C	400 - 600	≥ 1.15 < 1.35	≥ 7.5

In order to take plastic rotation capacity into account the reinforcement has to be of class B or C according to Table 4.1. Reinforcement of class A (cold worked; e.g. pre-stress strands) is not applicable for plastic redistribution.

As is shown in Figure 4.5 the ratio x / d is used in Eurocode 2 to describe the property of the reinforced concrete cross section. In FKR, this is instead based on the mechanical reinforcement ratio ω_s, see Section 4.4.2. However, it can be shown, see Johansson and Laine (2012), that the relation between the ratio x / d and ω_s can be expressed as

$$\frac{x}{d} = \frac{\omega_s}{0{,}8} \tag{4.52}$$

if there are no normal forces and there is no reinforcement on the compressive side; i.e. $A_s' = 0$.

4.4.4 UFC

In UFC only a very general description of the plastic deformation capacity is given. It is stated, though, that the maximal deformation capacity is a function of structure height and span width, as well as reinforcement amount and configuration, but any more detailed information than so is not given. Given instructions are instead based on a schematic load-deformation relation for a reinforced concrete beam. In Figure 4.6 the conceptual response is illustrated, where the response strongly depends on whether stirrups are present or not and of what type those stirrups are used. In summary, a reasoning is presented which means that concrete compressive failure is reached when the plastic rotation reaches $\theta = 2°$. If no stirrups are present this means that failure is reached. However, if there are stirrups present in the beam it is assumed that the compressive reinforcement will replace the effect of concrete in compression, and thus prolong the deformation capacity. Although the moment capacity, due to reduced internal lever arm, is somewhat reduced as explained in Section 4.1.4, the plastic rotation capacity still increases to $\theta = 6°$. If so called lacing reinforcement is used, see Figure 4.7, the plastic rotation capacity can be increased even more to $\theta = 12°$.

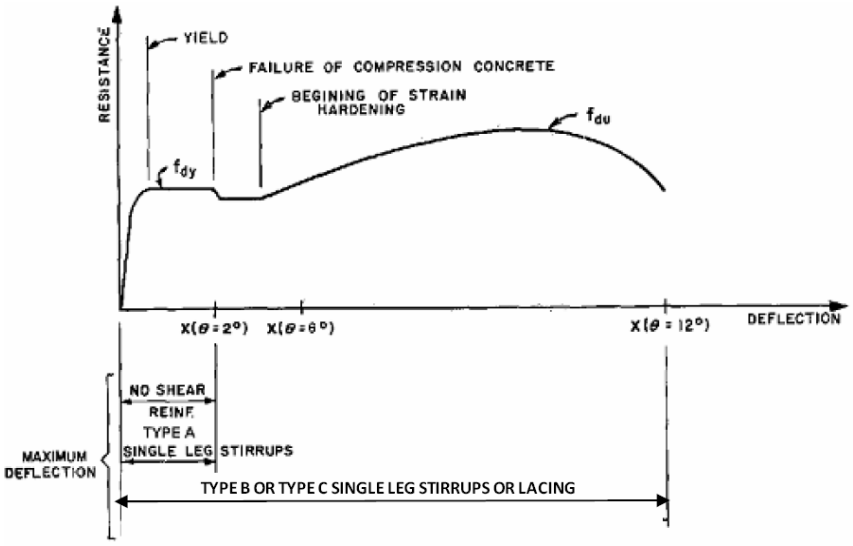

Figure 4.6 Schematic load-deformation relation for a reinforced concrete beam. From UFC, DOD (2008).

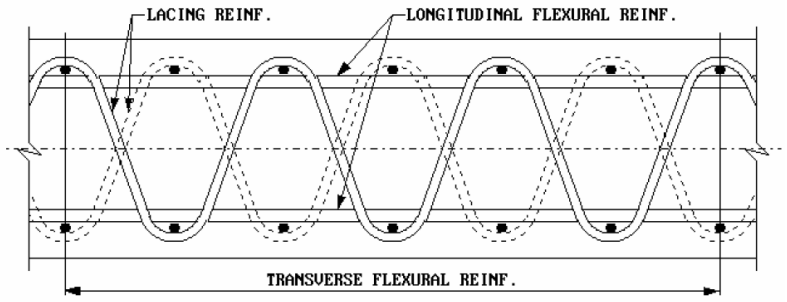

Figure 4.7 Special type of stirrups, so called lacing, which is used in order to increase the deformation capacity of a reinforced concrete structure. From UFC, DOD (2008).

An effect of this concept, i.e. that the presence of stirrups increase the plastic rotational capacity, is that the failure always is assumed to be reached due to concrete crushing. Hence, this is an important difference to FKR and Eurocode 2, where the rupture of reinforcement also is a possible cause of failure; see Section 4.4.2 and Section 4.4.3, respectively.

The general correctness of the load-deformation relation given in Figure 4.6 can perhaps be questioned since such a relation will depend on the type, amount and configuration of reinforcement and concrete. Hence, the relation is rather rough and it is suggested, by the authors of this report, that it is only used as an approximate rule of thumb of allowable values for the plastic rotations. It is probable that the recommendations given in UFC for the plastic deformation capacity are valid for a certain span of combinations of reinforcement amount and concrete strength but since no such spans are explicitly given[5] it is here suggested that the recommendations given in UFC is used with care; at least in structures with a large reinforcement ratio.

Based on Figure 4.6 allowable plastic rotation capacity θ and the length/deformation ratio l / u is summarised in Table 4.2. Here, the plastic rotation capacity allowed is also linked to the protection category and type of cross section used.

Table 4.2 Plastic rotation capacity θ and length/deformation ratio l / u according to UFC, DOD (2008).

Type of stirrups	Protection category	Type of cross section	θ [°]	θ [10^{-3} rad]	l / u [-]
None	1	I	1	17	115
None	2	I	2	35	57
Normal	2	II	6 [1)]	105	19
Figure 4.7	2	III	12 [1)]	210	10

[1)] Reduced inner lever arm, compression force balanced by reinforcement only, see Section 4.3.3.

The material properties of the reinforcement used in UFC is listed in Table 4.3 and from this it can be concluded that the ductility ratio γ is higher than that demanded in Eurocode 2, see Table 4.1. In UFC no demands on the ultimate strain is given but it is stated that the ultimate strain is larger in reinforcement of type A 706, which hence compensates for the lower value of the ductility ratio η, compared to type A 615.

Table 4.3 Definition of reinforcement types in UFC, DOD (2008).

Reinforcement type	f_{yk} [MPa]	f_{uk} [MPa]	$\eta = f_{uk} / f_{yk}$ [-]
ASTM A 615 Grade 60	455	620	1.36
ASTM A 706 Grade 60	455	550	1.21

[5] Instead the reinforcement amount allowed varies between about 0.2-1.5 % for f_{ck} = 25 MPa and 0.2-3.0 % for f_{ck} = 50 MPa, see Section 7.3.4.1.

4.4.5 Cormie *et al.*

The instructions given in Cormie *et al.* for the plastic deformation capacity is similar to those in UFC, described in Section 4.4.4. As in UFC, a schematic load-deformation relation as that shown in Figure 4.6 is used. Here, though, the allowable rotations are somewhat reduced compared to that in UFC, see Table 4.4.

Table 4.4 Plastic rotation capacity θ and length/deformation ratio l / u according to Cormie et al. (2009).

Type of stirrups	Protection category	Type of cross section	θ [°]	θ [10^{-3} rad]	l / u [-]
None	1	I	1	17	115
Normal	1	I	2	35	57
Normal	2	II	4 [1)]	70	29
Normal	2	II	8 [1), 2)]	141	14

1) Reduced inner lever arm, compression force balanced by reinforcement only, see Section 4.3.3.

2) This case is permitted if the structure can develop membrane action.

4.4.6 MSB

In the documentation of impulse loaded structures provided by the Swedish Civil Contingencies (MSB) it is recommended to base the plastic deformation capacity on Eurocode 2, e.g. Johansson and Laine (2012) and Johansson (2015b). However, in MSB's documentation the expression for the plastic rotation capacity in equation (4.47) is replaced by

$$\theta_{Rd,MSB} = 2 \cdot \theta_{Rd,EC} = k_{\lambda} \cdot \theta_{pl} \tag{4.53}$$

as argued for in Johansson (2015a). Consequently, the resulting plastic deformation capacity used by MSB is twice as large as that obtained when using the expressions given in Eurocode 2.

The impact on the plastic deformation capacity due to this suggested change is further compared in the case study presented in Section 7.3.2.

5 Shear

5.1 Reinforcement requirements

5.1.1 Orientation

The shear reinforcement amount is defined as

$$\rho_w = \frac{A_{sw}}{s \cdot b} \tag{5.1}$$

where A_{sw} is the shear reinforcement amount, s is the spacing used for the stirrups and b is the width of the cross section. For the expressions given in this report all stirrups are assumed to be fully vertical, i.e. perpendicular to the longitudinal axis of the beam.

The minimum shear reinforcement can be divided into two types: a minimum reinforcement amount and maximum stirrup spacing. The former is believed to make sure that the shear reinforcement becomes statically active prior to concrete shear failure (i.e. making sure that the stirrups are not torn off) and the latter is to make sure that the critical inclined shear crack does not fully appear in between two stirrups.

There is also a practical maximum amount of the shear reinforcement when extra amount will no longer provide higher shear capacity of the cross section. This amount corresponds to the amount when crushing of the concrete in inclined struts cause shear failure of the cross section, see Section 5.3.1 and Section 5.3.2.

5.1.2 FKR

In FKR the minimum amount of shear reinforcement shall fulfil[6]

$$A_{sw,\min,FKR} \geq \frac{1.6 \cdot a_\tau}{b \cdot d} \cdot \frac{V_{Ed}}{f_{yw}} \tag{5.2}$$

where a_τ is the shear span, b and d are the width and effective height of the cross section, V_{Ed} is the design shear load and f_{yw} is the yield strength of the shear reinforcement.

In FKR the contribution from the shear reinforcement depends on which phase is studied: the initial elastic deformation phase or the later plastic deformation phase. In the initial phase a larger amount of shear reinforcement is distributed over a shorter length $l_{\tau,el}$ compared to the length $l_{\tau,pl}$ that is used in the plastic phase, see Section 5.3.1. It is not explicitly mentioned in FKR but this means that the length l_τ, over which the shear reinforcement is distributed, corresponds to the length of the inclined shear crack. Hence, in practice this means that the shear crack angle varies from case to case.

[6] The form of this expression is uncertain since the resulting unit of $A_{sv,min,FKR}$ will be [m^2/m]. In Bk 25 this expression cannot be found. Further, this current expression means that the minimum shear reinforcement amount will be very large; if $a_\tau / d = 1.0$ the shear capacity has to be 1.6 times larger than the design shear force V_{Ed}. Hence, it seems that there may be something wrong with the current expression.

The maximum spacing of the shear reinforcement is limited to

$$s_{\max,el,FKR} \le 0.5 \cdot d \tag{5.3}$$

over the length $l_{\tau,el}$ in the elastic deformation phase and to

$$s_{\max,pl,FKR} \le 0.75 \cdot d \tag{5.4}$$

over the length $l_{\tau,pl}$ in the plastic deformation phase

5.1.3 Eurocode 2

In Eurocode 2 the minimum amount of shear reinforcement shall fulfil

$$\rho_{w,\min,EC} \ge \frac{0.08 \cdot f_{ck}^{0.5}}{f_{ywk}} 0.75 \cdot d \tag{5.5}$$

and the maximum spacing of shear reinforcement is set to

$$s_{\max,EC} \le 0.75 \cdot d \tag{5.6}$$

According to Eurocode 2 the minimum shear reinforcement should always be provided in beams, even though the concrete shear capacity $V_{Rd,c,EC}$ is larger than the design shear force $V_{Ed,EC}$. This requirement, though, is not valid in slabs; here shear reinforcement only needs to be used if $V_{Rd,c,EC} < V_{Ed,EC}$.

5.1.4 UFC

In UFC the minimum strength of the shear reinforcement is listed in Table 5.1. When stirrups are provided the required amount is determined in the critical section and this quantity is then uniformly distributed over the whole length of the structure.

Table 5.1 Minimum design shear strength $V_{Rd,s,UFC}$ of shear reinforcement.

Design range [m/kg$^{1/3}$]	Type of cross section	$V_{ED} \le V_{Rd,c}$	$V_{Rd,c} \le V_{ED} \le 1.85 \cdot V_{Rd,c}$	$V_{ED} > 1.85 \cdot V_{Rd,c}$
Z ≥ 3.0	I	0	$0.85 \cdot V_{Rd,c}$	$V_{Ed,c} - V_{Rd,c}$
	II, III	$0.85 \cdot V_{Rd,c}$	$0.85 \cdot V_{Rd,c}$	$V_{Ed,c} - V_{Rd,c}$
Z < 3.0	I, II, III	$0.85 \cdot V_{Rd,c}$	$0.85 \cdot V_{Rd,c}$	$V_{Ed,c} - V_{Rd,c}$

The maximum spacing of shear reinforcement depends on the type of cross section:

$$s_{\max,UFC} \le \min \begin{cases} 0.5 \cdot d \\ 610\,\text{mm} \end{cases} \quad \text{Type I} \tag{5.7}$$

$$s_{\max,UFC} \le \min \begin{cases} 0.5 \cdot (d - d') \\ 610\,\text{mm} \end{cases} \quad \text{Type II and III} \tag{5.8}$$

5.1.5 Cormie *et al.*

The requirements for shear reinforcement in Cormie *et al.* are based on Eurocode 2 and hence follow the description given in Section 5.1.3. However, the maximum spacing of the shear reinforcement is set to

$$s_{\max,Co} \leq 0.5 \cdot d \tag{5.9}$$

5.2 Design shear force

5.2.1 FKR

In FKR the design shear force is determined for two different phases: during the initial (elastic) deformation phase and the later (plastic) deformation phase. The design shear force is determined as

$$V_{Ed,FKR} = 0.5 \cdot k_v \cdot R_{\sup} \tag{5.10}$$

where k_v is a factor that takes into account the load distribution (k_v = 0.5 for a simply supported beam) and R_{sup} is the total dynamic support reaction. This support reaction is conceptually determined as

$$R_{\sup}(t) = \alpha_{FKR} \cdot R(t) + \beta_{FKR} \cdot F(t) \tag{5.11}$$

where R is the resisting force, F is the external load and

$$\alpha_{FKR} = \frac{\kappa_F{}^2}{\kappa_m} \tag{5.12}$$

$$\beta_{FKR} = \left(1 - \frac{\kappa_F{}^2}{\kappa_m}\right) \tag{5.13}$$

are load factors based on the transformation factors κ_F and κ_m for load and mass, respectively. This relation is based on dynamic force equilibrium as shown in Johansson (2015a). The transformation factors are different in the elastic and plastic deformation phase, see Table 5.2.

Table 5.2 Transformation factors and load factors for a simply supported beam subjected to evenly distributed load. Transformation factors are taken from Johansson and Laine (2012).

Deformation phase	P_1 / q [1)] [-]	κ_m [-]	κ_F [-]	α_{FKR} [-]	β_{FKR} [-]
Elastic	≤ 2	0.504	0.640	0.812	0.188
Plastic	> 2	0.333	0.500	0.750	0.250

1) Condition used in FKR to determine what deformation phase is to be used: P_1 = peak pressure of external load, q = equivalent static load according to equation (2.1).

From FKR it is not clear how to determine $R_{sup,pl}$, i.e. the support reaction in the plastic deformation phase. Based on the information given in Bk 25, Fortifikationsverket (1973a) and discussion with Svedbjörk (2016), though, it is concluded that these parameters are determined as stated in equation (5.11) but with the exception that the external load *F(t)* is set equal to the characteristic pressure load F_k for the structure; i.e.

$$F(t) = F_k \tag{5.14}$$

In FKR F_k is determined as

$$F_k = k \cdot R_{Rd} \tag{5.15}$$

where *k* is defined as

$$k = \frac{0.5 \cdot u_{el} + u_{pl}}{u_{el} + u_{pl}} \tag{5.16}$$

where u_{el}, u_{pl} and R_{Rd} are elastic deformation, plastic deformation and load capacity as illustrated in Figure 5.1.

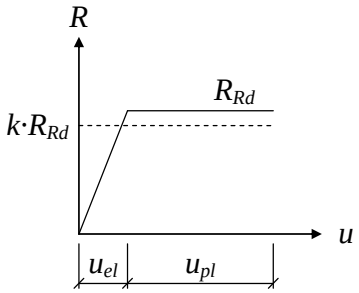

Figure 5.1 Elastoplastic structural response and definition of equivalent plastic respons.

Hence, this means that the support reaction for elastic and plastic deformation phases can be determined as

$$R_{\sup el}(t) = \alpha_{FKR,el} \cdot R(t) + \beta_{FKR,el} \cdot F(t) \tag{5.17}$$

$$R_{\sup,pl}(t) = \alpha_{FKR,pl} \cdot R(t) + \beta_{FKR,pl} \cdot k \cdot R_{Rd} \tag{5.18}$$

The design shear force is assumed to act at a distance a_τ (denoted: shear span) from the support, and depends on the support boundary condition. For a moment free support the shear span can be determined as

$$a_\tau = \left(0.025 + 0.25 \cdot \sqrt{\frac{q_{eq}}{P_1}}\right) \cdot l \qquad a_\tau \le 0.25 \cdot l \tag{5.19}$$

and for a fixed support it can be determined as

$$a_\tau = \left(0.010 + 0.35 \cdot \sqrt{\frac{q_{eq}}{P_1}}\right) \cdot l \qquad a_\tau \le 0.25 \cdot l \tag{5.20}$$

where q_{eq} is the equivalent static load, P_1 is the peak pressure of the external load and *l* is the span length of the structure.

5.2.2 Eurocode 2

In Eurocode 2 the critical section is determined at a distance

$$a_{crit} = z \cdot \cot\theta \tag{5.21}$$

from the support edge as shown in Figure 5.2. Here, z is the internal lever arm, as shown in Figure 4.3 ($z = 0.9 \cdot d$ is often used as an approximation), and θ is the shear crack angle.

For a structure with no stirrups

$$a_{crit} = d \tag{5.22}$$

which approximately corresponds to a case where $\theta = 45^{o}$, and for a structure with stirrups

$$1.0 \le \cot\theta \le 2.5 \tag{5.23}$$

which corresponds to a shear crack angle of $22^{o} \le \theta \le 45^{o}$.

From this the design shear force can, for a simply supported beam subjected to an evenly distributed load, be determined as

$$V_{Ed,EC} = q \cdot \left(\frac{l}{2} - \left(a_{crit} + \frac{a_{sup}}{2} \right) \right) \tag{5.24}$$

where q is the evenly distributed static load and a_{sup} is the width of the support

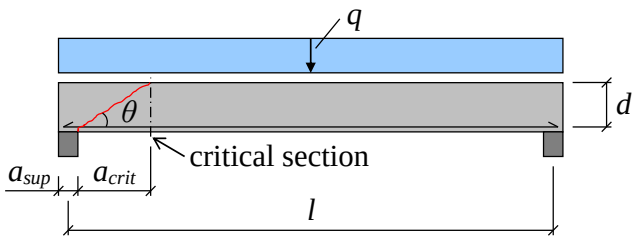

Figure 5.2 Critical section for the design force when the load is applied on top of the loaded beam.

5.2.3 UFC

In UFC the design shear force is based on an equivalent static load q_{eq}. The critical section is determined using a similar concept as in Eurocode 2, see Section 5.2.2. However, in UFC the shear crack angle is assumed to be constant, $\theta \approx 45^{o}$, and the same critical section is used regardless of whether stirrups are used or not.

Accordingly,

$$a_{crit} = d \tag{5.25}$$

and the design shear force is hence determined as

$$V_{Ed,UFC} = q_{eq} \cdot \left(\frac{l}{2} - \left(d + \frac{a_{sup}}{2} \right) \right) \tag{5.26}$$

5.2.4 Cormie *et al.*

In Cormie *et al.* the design shear force is, as in UFC, based on an equivalent static load q_{eq}. However, since Eurocode 2 is used in the design the instructions given there are also used in Cormie *et al*. Hence, the design shear force is determined as

$$V_{Ed,Co} = q_{eq} \cdot \left(\frac{l}{2} - \left(a_{crit} + \frac{a_{\sup}}{2} \right) \right) \tag{5.27}$$

where a_{crit} is determined as described in Section 5.2.2.

5.3 Shear capacity

5.3.1 FKR

In FKR there is some confusion of how the design of shear forces is meant to be carried out. In the initial part of the description in FKR there is a reference to an older Swedish code (BBK 04) of which approach should be used depending on the ratio between shear span a_τ and effective height d. If

$$\frac{a_\tau}{d} \leq 1.5 \tag{5.28}$$

an approach based on a deep beam should be used (Section 6.6 in BBK 04) and if

$$\frac{a_\tau}{d} > 1.5 \tag{5.29}$$

an approach suitable for normal beam theory should be used (Section 3.7 in BBK 04). However, after this initial statement FKR still includes a detailed description of how the shear capacity should be determined. Hence, it is not clear how these somewhat contradictory instructions should be treated. In this document the latter concept, described in detail in FKR, is used and described below.

In FKR the concrete shear capacity is determined as

$$V_{Rd,c,FKR} = k_c \cdot b \cdot d \tag{5.30}$$

where

$$k_c = k_\tau \cdot \frac{k_\rho}{s} \tag{5.31}$$

is a factor determined depending on the shear span a_τ , reinforcement amount ρ and protection level s.

Here

$$k_\tau = \frac{0.45}{a_\tau / d} \cdot 0.25 \cdot f_{ck} \qquad k_\tau \leq 0{,}25 \cdot f_{ck} \tag{5.32}$$

where a_τ is the shear span according to equation (5.19) or (5.20), and f_{ck} is the characteristic concrete compressive strength,

$$k_\rho = 0.7 + \frac{\rho - 0.1}{3} \tag{5.33}$$

where $0.1 \leq \rho \leq 0.5$ % is the reinforcement amount ρ, and

$$s = \begin{cases} 1.2 & \text{for protection level A and B} \\ 1.0 & \text{for protection level C} \end{cases} \tag{5.34}$$

is a safety factor taken into account the chosen protection level, see Section 3.2.1.

If the concrete shear capacity is less than the design shear strength, i.e. $V_{Rd,c,FKR} < V_{Ed,FKR}$, shear reinforcement is needed. The shear force taken by the shear reinforcement depends on what deformation phase is studied. In the elastic deformation phase the required shear reinforcement capacity is determined as [7]

$$V_{Rd,s,FKR,el} = k_v \cdot R_{\sup el} \cdot \left(1 - \sqrt{\frac{V_{Rd,c,FKR}}{V_{Ed,FKR,el}}}\right) \tag{5.35}$$

where $R_{sup,el}$ is the total reaction force according to equation (5.17), $V_{Rd,c,FKR}$ is the concrete shear strength according to equation (5.30) and $V_{Ed,FKR,el}$ is the design shear strength according to equation (5.10) when setting $R_{sup} = R_{sup,el}$.

In the plastic deformation shape the required shear reinforcement capacity is determined as

$$V_{Rd,s,FKR,pl} = k_v \cdot R_{\sup,pl} \cdot \left(1 - \sqrt{\frac{V_{Rd,c,FKR}}{2 \cdot V_{Ed,FKR,pl}} \cdot \frac{8 \cdot a_\tau}{l}}\right) \tag{5.36}$$

where $R_{sup,pl}$ is the total reaction force according to equation (5.18), $V_{Rd,c,FKR}$ is the concrete shear strength according to equation (5.30) and $V_{Ed,FKR,el}$ is the design shear strength according to equation (5.10) when setting $R_{sup} = R_{sup,pl}$.

The shear force in equation (5.35) and (5.36) determine a shear reinforcement area

$$A_{sw,FKR} = \frac{V_{Rd,s,FKR}}{f_{yw}} \tag{5.37}$$

where f_{yw} is the yield strength of the shear reinforcement. This reinforcement amount is evenly distributed over a length l_τ. For the elastic deformation phase this length is determined as

$$l_{\tau,el} = a_\tau \cdot \left(1 - \sqrt{\frac{V_{Rd,c,FKR}}{V_{Ed,FKR,el}}}\right) \tag{5.38}$$

and for the plastic deformation phase this length is determined as

$$l_{\tau,pl} = a_\tau \cdot \left(1 - \sqrt{\frac{V_{Rd,c,FKR}}{2 \cdot V_{Ed,FKR,\min}} \cdot \frac{8 \cdot a_\tau}{l}}\right) \tag{5.39}$$

[7] A factor k_v is included in equation (5.35) and (5.36) to take into account that the shear force is determined at only one support and not as the total shear force, which is the case according to the expression used in FKR. Such a change is also recommended to be included in FKR.

Regardless of what shear reinforcement amount is used the shear force in the plastic deformation phase is limited by

$$V_{Ed,FKR,\min} \le 0.25 \cdot f_c \cdot b \cdot d \tag{5.40}$$

which indicates crushing of the compressive strut in the inclined shear crack.

5.3.2 Eurocode 2

In Eurocode 2 the concrete shear capacity is determined as

$$V_{Rd,c,EC} = v_{Rd,c,EC} \cdot b \cdot d \tag{5.41}$$

where

$$v_{Rd,c,EC} = \max \begin{cases} \dfrac{0.18}{\gamma_c} \cdot k \cdot (100 \cdot \rho \cdot f_{ck})^{1/3} \\ 0.035 \cdot k^{3/2} \cdot f_{ck}^{\,0.5} \end{cases} \tag{5.42}$$

where $\gamma_c = 1.2$ is the partial coefficient factor for concrete at accidental loading,

$$k = 1 + \sqrt{\frac{200}{d}} \qquad k \le 2.0 \ \ (d \text{ in [mm]}) \tag{5.43}$$

is a factor taking into account the size effect, ρ is the reinforcement amount and f_{ck} is the characteristic concrete compression strength. The concrete shear capacity is also limited by

$$V_{Rd,c,EC} \le 0.30 \cdot \left(1 - \frac{f_{ck}}{250}\right) \cdot f_c \cdot b \cdot d \tag{5.44}$$

which indicates crushing of the compressive strut in the inclined shear crack.

If the concrete shear capacity is less than the design shear force, the whole shear force has to be taken by the shear reinforcement; i.e.

$$V_{Rd,s,EC} \ge V_{Ed,EC} \tag{5.45}$$

Based on this the shear reinforcement area needed is determined as

$$\frac{A_{sw,EC}}{s} = \frac{V_{Rd,s,EC}}{z \cdot \cot\theta \cdot f_{yw}} \tag{5.46}$$

where s is the stirrup spacing and $z \cdot \cot\theta = a_{crit}$ is the length of the inclined shear crack as shown in Figure 5.2. The reinforcement shear capacity is also limited to

$$V_{Rd,s,EC} \le \frac{0.60}{\cot\theta + \tan\theta} \cdot \left(1 - \frac{f_{ck}}{250}\right) \cdot f_c \cdot b \cdot d \tag{5.47}$$

due to crushing of the compressive strut in the inclined shear crack.

5.3.3 UFC

In UFC the concrete shear capacity is determined as

$$V_{Rd,c,UFC} = v_{Rd,c,UFC} \cdot b \cdot d \tag{5.48}$$

for a cross section of Type I and

$$V_{Rd,c,UFC} = v_{Rd,c,UFC} \cdot b \cdot (d - d') \tag{5.49}$$

for a cross section of Type II or III. Here

$$v_{Rd,c,UFC} = \max \begin{cases} 2 \cdot f_c^{\,0.5} \\ 1.9 \cdot f_c^{\,0.5} + 2500 \cdot \rho \end{cases} \quad \leq 3.5 \cdot f_c^{\,0.5} \text{ (units in [psi])} \tag{5.50}$$

where f_c is the concrete compressive strength and ρ is the reinforcement amount. In Equation (5.42) $v_{Rd,c,UFC}$ and f_c is given in the unit [psi] and hence need to be recalculated to [Pa] in order to make calculations using SI-units[8].

If the concrete shear capacity is less than the design shear force, shear reinforcement must be provided to carry the excess; i.e.

$$V_{Rd,s,UFC} = V_{Ed,UFC} - V_{Rd,c,UFC} \tag{5.51}$$

However, depending on among all what type of cross section is used, there is also a minimum value that the shear force $V_{Rd,s,UFC}$, needs to fulfil, see Section 5.1.4.

Based on this the shear reinforcement area needed is determined as

$$\frac{A_{sw,UFC}}{s} = \frac{V_{Rd,s,EC}}{0.85 \cdot d \cdot f_{yw}} \tag{5.52}$$

where $0.85 \cdot d = a_{crit}$ is the length of the inclined shear crack as shown in Figure 5.2.

5.3.4 Cormie *et al.*

The shear capacity in Cormie *et al.* is based on Eurocode 2, and hence follows the description given in Section 5.3.2. The only difference is that equation (5.44) and (5.47) are multiplied by a factor 5/6 to take into account a different choice of a national parameter that is chosen differently in UK than in Sweden.

For a cross section of Type I the design shear force may be resisted by the concrete shear capacity. For a cross section of Type II, though, this is not allowed and the whole shear force always has to be resisted by the shear reinforcement.

[8] This is done using the conversion factor 1 psi = 6.895 kPa.

5.4 Direct shear

5.4.1 Orientation

In both UFC and Cormie *et al.* there is a control of the capacity due to, so called, direct shear cracks. This failure type is due to a straight shear crack that appears close to the support as schematically shown in Figure 5.3. The design against such a failure is in UFC and Cormie *et al.* made using the support reaction obtained when using an equivalent static load q_{eq}, i.e.

$$V_{Ed,ds} = \frac{q_{eq} \cdot l}{2} \tag{5.53}$$

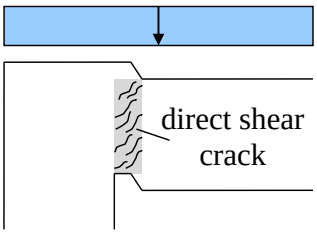

Figure 5.3 Schematic illustration of direct shear crack close to support.

In FKR and Eurocode 2, though, there are no special controls of failure due to direct shear crack; this is further discussed in Section 7.4.5.

5.4.2 FKR

In FKR there is no special control of failure due to direct shear crack.

5.4.3 Eurocode 2

In Eurocode 2 there is no special control of failure due to direct shear crack.

5.4.4 UFC

In UFC the concrete shear capacity due to direct shear is in slabs determined as

$$V_{Rd,c,ds,UFC} = 0.16 \cdot f_c \cdot b \cdot d \tag{5.54}$$

However, this capacity is only valid if the used plastic rotational capacity $\theta \leq 2°$ or if the beam is simply supported. If $\theta > 2°$, or if a cross section (with any rotation) is in net tension the concrete shear capacity is reduced to

$$V_{Rd,c,ds,UFC} = 0 \tag{5.55}$$

If the acting shear force is larger than the concrete shear capacity, diagonal bars must be added to carry the excess shear force, i.e.

$$V_{Rd,s,ds,UFC} = V_{Ed,ds,UFC} - V_{Rd,c,ds,UFC} \tag{5.56}$$

5.4.5 Cormie *et al.*

The regulations for direct shear cracks in Cormie *et al.* is similar to those in UFC, see Section 5.4.4. The concrete shear capacity due to direct shear is determined as

$$V_{Rd,c,ds,Co} = 0.25 \cdot \left(1 - \frac{f_{ck}}{250}\right) \cdot f_c \cdot b \cdot d \tag{5.57}$$

As in UFC, this capacity is only valid if the used plastic rotational capacity $\theta \leq 2°$ or if the beam is simply supported; if $\theta > 2°$ the concrete shear capacity is reduced to

$$V_{Rd,c,ds,Co} = 0 \tag{5.58}$$

In contrast to UFC, though, no information is given of how to treat a situation where a cross section is in net tension. However, it is mentioned that minimum reinforcement (in accordance with Eurocode 2) should be used also for the inclined stirrups even though the concrete capacity against direct shear is sufficient.

6 Spalling and breaching

6.1 Orientation

If the explosion is strong enough there may emerge phenomena such as cratering, spalling and breaching in a concrete structure. These types of damage are associated with what may appear at so called contact detonations; i.e. when the detonating charge is placed in contact with the concrete structure as schematically shown in Figure 6.1. However, provided that the charge is large enough, such damages may also appear when the charge is located at a large distance; e.g. up to several meters.

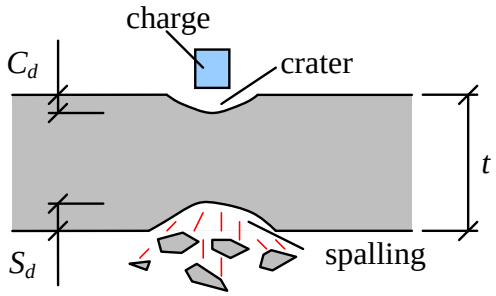

Figure 6.1 *Schematic illustration of cratering and spalling in concrete structure subjected to the load from a contact detonation. Breaching is obtained if the depth of the crater and the spalling reach each other. Concrete thickness t, crater depth C_d and spall depth S_d are marked.*

6.2 FKR

In FKR it is assumed that the charge is placed in contact with the concrete structure for the expressions given related to spalling and breaching. Based on these expressions the minimum thickness of a concrete slab may be determined in order to avoid different types of damage. In Figure 6.2 the type of local damage assumed in FKR, due to a contact detonation, is illustrated for a case of protection level B1. The concrete slab thickness needed for different protection levels is determined according to equation (6.1) to (6.3).

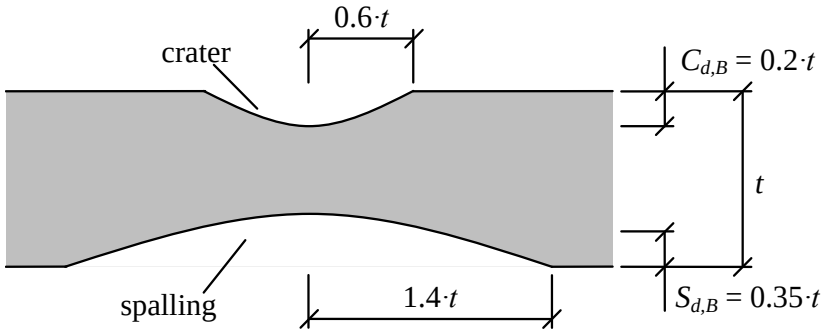

Figure 6.2 *Schematic illustration of local damage due to a contact detonation in protection level B. Based on FKR 2011, Fortifikationsverket (2011).*

Protection level A (no spalling)

$$t_A = t_{spall,FKR} = 0.78 \cdot r_t \cdot W^{1/3} \tag{6.1}$$

Protection level B (spalling is obtained but no breaching; schematic local damage is obtained as shown in Figure 6.2)

$$t_B = 0.31 \cdot r_t \cdot W^{1/3} \tag{6.2}$$

Protection level C (limit for when breaching is obtained)

$$t_C = t_{breach,FKR} = 0.18 \cdot r_t \cdot W^{1/3} \tag{6.3}$$

Here W is the charge weight (equivalent amount of TNT) in [kg] and r_t is a factor taking into account the concrete strength.

In order to use the expressions in equations (6.1) to (6.3) the concrete must fulfil the condition that

$$f_{ck,cube} \geq 40\,\text{MPa} \tag{6.4}$$

If a stronger concrete is used the required concrete thickness t may be reduced using the factor[9]

$$r_t = \sqrt{\frac{40}{f_{ck,cube}}} \qquad f_{ck,cube} \geq 48\,\text{MPa} \tag{6.5}$$

The resulting crater depth C_d and spall depth S_d is obtained for protection level B1 in accordance with Figure 6.2 as

$$C_{d,B} = 0.20 \cdot t_B \tag{6.6}$$

$$S_{d,B} = 0.35 \cdot t_B \tag{6.7}$$

where t_B is the necessary slab thickness according to equation (6.2). No instructions for the crater or spall depth are given for any other protection levels; it is evident from the expressions from the different protection levels, though, that the spall depth decreases with increased slab thickness. Further, it is interpreted here that the expressions for protection level A and C shall be used in order to determine the concrete slab thickness necessary to avoid spalling and breaching, respectively.

It is reasonable to believe that the crater depth will be the same for a given charge regardless of the slab thickness. Hence, based on these equations it is possible to estimate the crater depth of thicker slabs where spalling does not occur. Combining equation (6.1) and (6.2) you get

$$t_B = \frac{0.31}{0.78} \cdot t_A = 0.40 \cdot t_A \tag{6.8}$$

[9] In FKR 2011 the expression for this factor is $r_t = (32 / f_{ck})^{0,5}$ while it at the same time is stated that $f_{ck} \geq 40$ MPa. After a discussion with the founder of the expression, Svedbjörk (2012), it has been confirmed that the correct expression shall be as stated in equation (6.5) and that f_{ck} refer to the characteristic concrete cube strength; i.e. $f_{ck,cube}$.

which together with equation (6.5) gives that the crater depth C_d, for a slab thickness where spalling is avoided (protection level A), can be determined as

$$C_{d,A} = 0.20 \cdot 0.40 \cdot t_A = 0.08 \cdot t_A \tag{6.9}$$

Using the same method it can be shown that the crater depth at breaching (protection level C) can be determined as

$$C_{d,C} = 0.20 \cdot \frac{0.31}{0.18} \cdot t_C = 0.34 \cdot t_C \tag{6.10}$$

However, a similar recalculation of the spall depth S_d is not deemed to be possible.

In the design the concrete slab thickness t_d is determined as

$$t_d = \gamma_{fmn} \cdot t \tag{6.11}$$

where γ_{fmn} is a partial coefficient according to Section 3.3.1.

6.3 Eurocode 2

In Eurocode 2 there is no special control of failure due to spalling or breaching.

6.4 UFC

In UFC there are empirical expressions given of what concrete thickness is required in order to avoid spalling or breaching of the structure. Spalling is avoided if

$$t_{spall,UFC} \geq \frac{r}{-0.02511 + 0.01004 \cdot \psi^{2,5} + 0.13613 \cdot \psi^{0,5}} \tag{6.12}$$

and breaching is avoided if

$$t_{breach,UFC} \geq \frac{r}{0.028205 + 0.144308 \cdot \psi + 0.049265 \cdot \psi^2} \tag{6.13}$$

where ψ is a coefficient that, for a hemispherical noncontact charge without a mantle, can be determines as

$$\psi_{noncontact} \geq r^{0.926} \cdot f_c^{\,0.266} \cdot W^{-0.353} \quad \text{(valid for } 0.5 \leq \psi \leq 14\text{)} \tag{6.14}$$

For a hemispherical contact charge without a mantle it can be determined as

$$\psi_{noncontact} \geq 0{,}527 \cdot r^{0.972} \cdot f_c^{\,0.308} \cdot W^{-0.341} \quad \text{(valid for } 0.5 \leq \psi \leq 14\text{)} \tag{6.15}$$

Here r is the distance (expressed in [ft]) from the charge centre point to the surface of the concrete, f_c is the concrete compressive strength (expressed in [psi]) and W is the weight of the charge (expressed in [lb]). Since equation (6.12) and (6.13) are expressed in imperial units these are also used to determine the factor ψ.

In contrast to FKR the distance r between the charge and the concrete surface is a parameter when determining the required concrete thickness to avoid spalling or breaching. Hence, it is possible to estimate these effects also for cases where the charge is not placed in contact with the concrete structure. In UFC no information is given of how to estimate the crater depth or spall depth.

6.5 Cormie *et al.*

In Cormie *et al.* empirical expressions are given to determine the required concrete thickness in order to avoid spalling or breaching of the structure. Spalling is avoided if

$$t_{spall,Co} \geq 0.07\left(\frac{r}{W^{1/3}}\right)^{-0.62} \cdot W^{1/3} \tag{6.16}$$

and breaching is avoided if

$$t_{breach,Co} \geq 0.03\left(\frac{r}{W^{1/3}}\right)^{-0.62} \cdot W^{1/3} \tag{6.17}$$

In Cormie *et al.* SI units are consistently used and r is the distance (expressed in [m]) between charge and concrete surface, and W denotes the weight (expressed in [kg]) of a spherical charge. To account for a hemispherical charge the charge mass is instead determined as

$$W_{\text{mod}} = \alpha \cdot W \tag{6.18}$$

where α = 1,8 is a factor due to mirroring.

As in UFC it is possible to estimate the risk of spalling and breaching for a case where the charge is not placed in contact with the concrete structure. In Cormie *et al.* no information is given of how to estimate the crater depth or spall depth.

7 Case studies

7.1 Method

In order to compare the regulations treated in this report a simply supported strip in a one-way slab subjected to an evenly distributed impulse load is studied. Load condition, geometry and boundary conditions are shown in Figure 7.1.

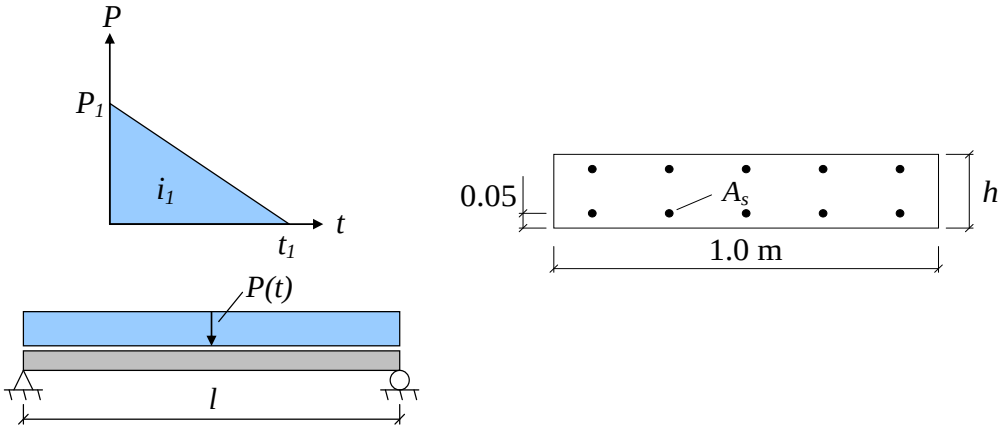

Figure 7.1 Load condition and geometry of simply supported beam studied.

A set of parameters were chosen for a basic case and the comparison was then made by changing one parameter at a time. The parameters varied were (basic values are underlined):

- Concrete quality [<u>C25/40</u>, C50/60]
  - Compressive strength, f_{ck}: [<u>25</u>, 50] MPa
  - Young´s modulus, E_c: [<u>30</u>, 37] GPa
- Slab thickness, h: [<u>250</u>, 500] mm
- Span length of slab strip, l: [<u>2.5</u>, 5.0] m
- Reinforcement amount, ρ: [<u>0.1-0.5</u>] %
- Load peak pressure, P_1: [<u>1000</u>, 2000] kPa

Further, the following reinforcement strengths (class C) were used:

- Yield strength, f_y: 500 MPa
- Ultimate strength, f_u: 575 MPa (η = 1.15)
- Average tensile strain in reinforcement (used in FKR only), ε_{su} = 30 ‰

In the comparisons made here UFC and Cormie *et al.* are both presented for two types of cross sections (I and II) according to Section 3.3.3 and Section 3.3.4, respectively, while FKR and Eurocode are presented by a single type each.

Only those parameters that have any influence of the capacities studied are shown; e.g. the span length l or peak pressure P_1 do not have any influence on the bending moment capacity in any of the regulations studied and are hence also omitted in the comparison made. Further, since the impulse load i_1 does not affect any of the capacities studied it is and hence not included as a parameter.

7.2 Material strength

7.2.1 Comparison

In Section 3.3 it is described how the design strength of concrete and reinforcement is determined in the regulations compared in this report. In Table 7.1 and Table 7.2 a comparison of the coefficients used to determine the design strength of concrete and reinforcement, respectively, in bending is presented.

In order to describe the effect on the final design strength of concrete a coefficient

$$\lambda_c = \frac{\alpha_c \cdot DIF_c}{\gamma_c} \tag{7.1}$$

is introduced, where α_c and γ_c are coefficients used in Eurocode 2 (here γ_{fmn}, used in FKR, is equated as γ_c) and *DIFc* is a coefficient used in UFC and Cormie *et al.* The final design compressive strength f_{cd} can then be determined as

$$f_{cd} = \lambda_c \cdot f_{ck} \tag{7.2}$$

where f_{ck} is the characteristic compressive strength of concrete. In Table 7.1 a comparison of the coefficient λ_c is made for the case studies made in this report; it is also normalised with regard to the value obtained according to Eurocode 2.

Table 7.1 *Comparison of coefficient λ_c for the effective concrete design strength used in bending. For shear capacity λ_c = 1.00 for all regulations compared.*

Regulation	α_c [-]	γ_c [-]	DIF_c [-]	λ_c [-]	$\lambda_c / \lambda_{c,EC}$ [-]
FKR [1)]	1.00	1.0	1.00	1.00	1.20
Eurocode 2	1.00	1.2	1.00	0.83	1.00
UFC [2)]	1.00	1.0	1.19	1.19	1.43
Cormie *et al.*	0.85	1.2	1.25	0.89	1.06

1) $\gamma_c = \gamma_{fmn}$ is used for function availability of level 1 and protection level C.

2) DIF_c is used for far design range.

A similar comparison can be made for the design strength of reinforcement. Here, though, the possible effect of strain hardening also has to be taken into account. As described in Section 3.3 the effect of reinforcement strain hardening, i.e. the reinforcement ultimate strength f_u, is normally not taken into account when determining the bending moment capacity M_{Rd}.; i.e. $f_u = f_y$ is assumed. However, for Type II cross sections in UFC and Cormie *et al.*, this effect is accounted for in the design, and hence, an increased strength is also obtained. Based on equation (3.7) and (3.13) the ratio between the reinforcement capacity f_s and yield strength f_y can be expressed as

$$\frac{f_s}{f_y} = \frac{3 \cdot DIF_s + \eta \cdot DIF_{s,u}}{4} \qquad \geq DIF_s \tag{7.3}$$

where

$$\eta = \frac{f_u}{f_y} \tag{7.4}$$

and DIF_s and $DIF_{s,u}$ is the dynamic increase factor for yield strength f_y and ultimate strength f_u, respectively. Here a reinforcement class C is assumed which means that η = 1.15 for Type II cross sections and η = 1.00 for all other cases.

Based on this the coefficient

$$\lambda_s = \frac{1}{\gamma_s} \cdot \frac{f_s}{f_y} \tag{7.5}$$

is introduced to describe the effect on the final effective design strength of reinforcement, and based on this the final design reinforcement strength f_{sd} for bending capacity can be determined as

$$f_{sd} = \lambda_s \cdot f_y \tag{7.6}$$

where f_y is the yield strength of the reinforcement. In Table 7.2 a comparison of the coefficient λ_s is made for the case studies made in this report. Further, a normalised value of λ_s, with regard to the value obtained according to Eurocode 2, is also presented.

Table 7.2 Comparison of reinforcement design strength for bending moment.

Regulation	γ_s [-]	DIF_s [-]	$DIF_{s,u}$ [-]	η [-]	f_s / f_y [-]	λ_s [-]	$\lambda_s / \lambda_{s,EC}$ [-]
FKR [1)]	1.0	1.00	1.00	1.00	1.00	1.00	1.00
Eurocode 2	1.0	1.00	1.00	1.00	1.00	1.00	1.00
UFC, Type I	1.0	1.17	1.05	1.00	1.17	1.17	1.17
Cormie, Type I	1.0	1.20	1.05	1.00	1.20	1.20	1.20
UFC, Type II	1.0	1.17	1.05	1.15	1.18	1.18	1.18
Cormie, Type II	1.0	1.20	1.05	1.15	1.20	1.20	1.20

1) γ_s is used for function availability of level 1 and protection level C.

2) DIF_s and $DIF_{s,u}$ are used for far design range.

7.2.2 Comments

From Table 7.1 it can be noted that the design material strength of concrete is somewhat higher in FKR than in Eurocode 2 and Cormie *et al.*, but lower than that used in UFC. The differences are due to different values on the partial coefficient γ_c and strain rate effects DIF_c. From Table 7.2 it can be seen that the design material strength for reinforcement is the same in FKR and Eurocode 2, but lower than that used in UFC and Cormie *et al.* The main reason for this is the strain rate effects DIF_s that are accounted for in the latter. The effect of strain hardening, though, is more or less negligible due to different *DIF* values for yield strength f_y and ultimate strength f_u.

The difference in concrete strength has a very minor effect on the moment capacity of a reinforced cross section, and hence this difference is not important. However, the increase in reinforcement strength does have a direct effect since the moment capacity is almost proportional to the reinforcement strength. Hence, an increase in reinforcement strength with 20 % also increases the moment capacity with nearly as much.

Strain rate effects are a well-known phenomenon that will increase the strength of a impulse loaded structure. Historically, this effect has conservatively not been included in the design of Swedish fortification structures; neither by Fortifikationsverket or MSB. Depending on load case, this caution may be sound; however, a general use of strain rate effects according to the *DIF* factors presented in UFC and Cormie *et al.*, is not believed by the authors of this report to be entirely appropriate. However, it may well be argued that this strengthening effect to some extent should be included; at least in such cases where the strain rate will be very high. For which cases it would be suitable to do so, though, have not been further investigated in this report.

However, as described in Section 5.2 an increase in moment capacity also increase the design shear force, regardless which regulation is used. Hence, it can be argued that it is on the unsafe side for the shear force control not to include the strain rate effects when determining the moment capacity of the structure. This stand point, though, opens up an interesting, but difficult, discussion of how the material parameters in the design of structures subjected to impulse loading should be chosen. Based on this it can be argued that an upper characteristic material strength, not a lower characteristic strength (which is currently prescribed to be used in all regulations compared) should be used for f_c and f_y when estimating the moment capacity used to determine the design shear strength. This subject is briefly discussed in Johansson (2014), but even though such a discussion would be of interest, it is not the purpose of this report to further deal with this controversy. Hence, it is here contently assumed that such effects are sufficiently handled by the conservatism present within the method used to determine the shear strength capacity in the respective regulations.

7.3 Bending moment

7.3.1 Moment capacity

7.3.1.1 Comparison

The moment capacity for the basic input data, according to Section 7.1, is shown in Figure 7.2. From this it can be seen that FKR and Eurocode more or less produce identical capacities. For Type I cross sections, though, the moment capacity is higher in both UFC and Cormie *et al.*, while it is lower for Type II cross sections. The former is due to increased material strength, as described in Section 7.2, while the latter is due to decreased cross section height, as described in Section 4.3.3 and Section 4.3.4.

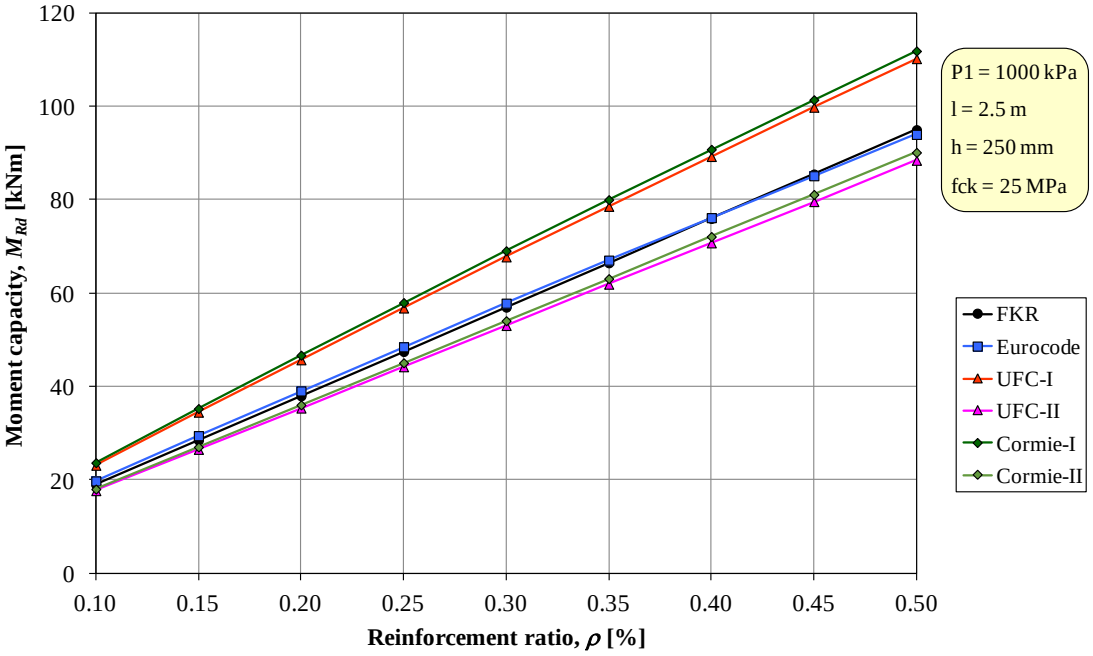

Figure 7.2 Comparison of moment capacity for various reinforcement amount when basic input data according to Section 7.1 is used.

In Figure 7.3 the effect on the moment capacity due to increased concrete compressive strength f_{ck} is shown. An increased concrete strength reduces the height of the compressive zone x in the cross section and thus increase the internal lever arm z as shown in Figure 4.3. This change has a very minor effect on Eurocode and Type I cross sections in UFC and Cormie *et al.* However, in FKR or for Type II cross sections in UFC and Cormie *et al.* there is no effect at all since the height of the compressive zone is not included in these expressions. The overall change in moment capacity is, due to the relatively small reinforcement amounts used in this comparison, very small when changing the concrete compressive strength.

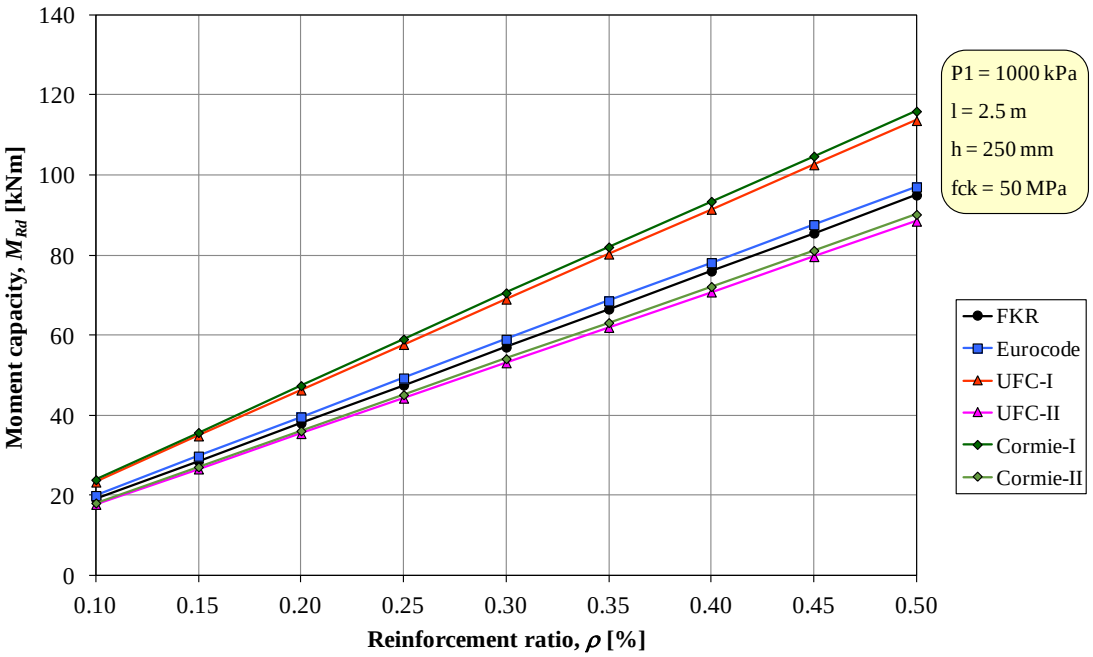

Figure 7.3 Comparison of moment capacity for various reinforcement amount when a concrete compressive strength of f_{ck} = 50 MPa is used.

In Figure 7.4 the effect on the moment capacity due to increased slab thickness *h* is shown. An increased slab thickness increases the internal lever arm *z* and thus also increases the moment capacity in all cases. Further, an increased slab thickness also affects the reinforcement ratio of the cross section. Hence, a kept reinforcement ratio means that the reinforcement amount, and hence the moment capacity, also increase with increased slab thickness. In this case it can also be seen that the moment capacity of Type II cross sections is higher than the moment capacity of FKR and Eurocode. This occurs since the decrease in internal lever arm *z* is overshadowed by the increased material strength obtained in UFC and Cormie *et al.*

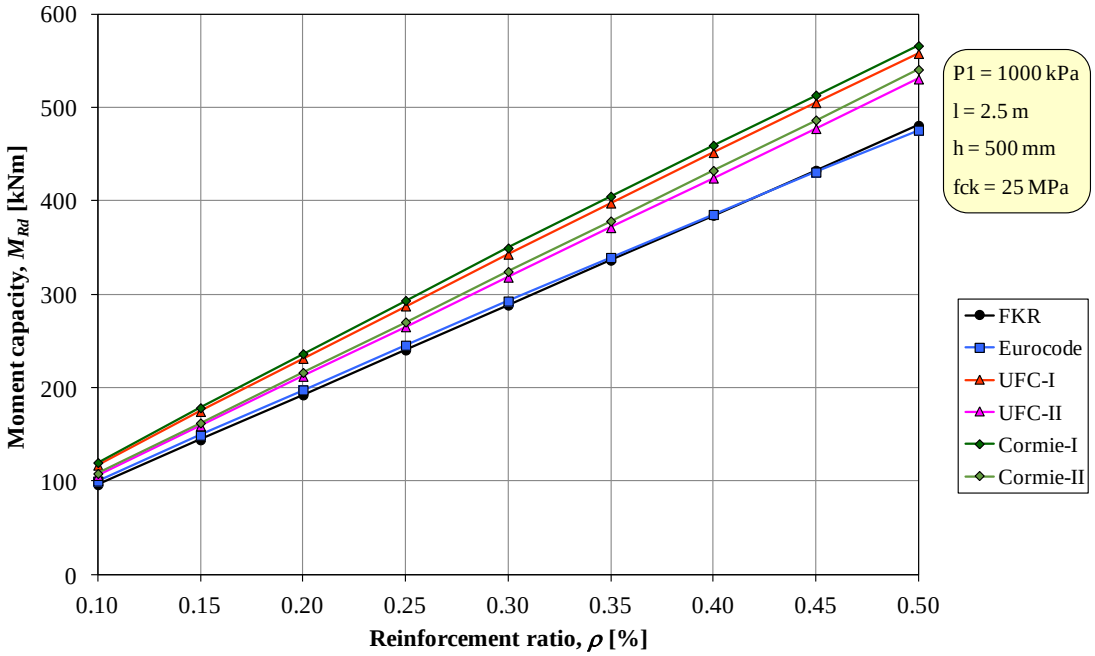

Figure 7.4 *Comparison of moment capacity for various reinforcement amount when a slab thickness of h = 500 mm is used.*

7.3.1.2 Comments

All-in-all the moment capacity is determined using the same conceptual model in all regulations compared; the difference in result is mainly due to different view of the material strength and (for Type II cross sections) due to decrease of the internal lever arm.

The difference between FKR and Eurocode is small but unnecessary for the cases studied here. However, the expressions used in FKR can be used for other type of structures as well (e.g. a beam with a T-shaped cross section) or the possibility to use a higher reinforcement ratio could perhaps be allowed, and it would therefore be worthwhile to base the moment capacity on a more general expression. It is therefore recommended that the expressions in FKR are changed to those used in Eurocode 2 in order to better reflect a more general equilibrium case.

7.3.2 Plastic deformation capacity

7.3.2.1 Comparison

The plastic deformation capacity u_{Rd} depends, among all, on the span length l. Hence, in order to make a direct comparison possible for slab strips of different span lengths the ratio l / u_{Rd} is used here; i.e. a small value on l / u_{Rd} indicates a good plastic deformation capacity.

A new category, denoted MSB, is also included in the comparison made in this section. It corresponds to the method suggested by MSB, see Section 4.4.3, and corresponds to the plastic deformation capacity provided by Eurocode 2 times two; i.e.

$$\frac{l}{u_{Rd,MSB}} = \frac{1}{2} \cdot \frac{l}{u_{Rd,EC}} \quad \rightarrow \quad u_{Rd,MSB} = 2 \cdot u_{Rd,EC} \tag{7.7}$$

The ratio of the plastic deformation capacity for the basic input data, according to Section 7.1, is shown in Figure 7.5. From this it can be seen that there is a large discrepancy in the results between different regulations and different type of cross sections. The results for FKR are close to that of UFC and Cormie *et al.* when assuming a Type II cross section. In the other end of the spectra Eurocode 2 and Cormie *et al*, Type I cross section, show relatively similar deformation capacities.

The change of direction in l / u_{Rd} in Figure 7.5 can clearly be seen for Eurocode 2 and MSB and vaguely also for FKR. This change of direction indicates a change in failure mode from rupture of reinforcement (low ρ) to concrete crushing (large ρ). For UFC and Cormie *et al.*, though, no such indications exist since, for them, failure is defined to always be reached due to concrete crushing, see Section 4.4.4.

In Figure 7.6 the effect on the plastic deformation capacity due to increased concrete compressive strength f_{ck} is shown. This change has a very minor impact on the results obtained when using FKR. However, the change of direction in l / u_{Rd} obtained in Figure 7.5, indicating change of failure mode, has disappeared. This is even clearer for results obtained using Eurocode 2 or MSB; the increase in concrete strength means a decreased chance of obtaining failure due to concrete crushing. In Figure 7.6 there is no longer a change of direction in the graph for Eurocode 2 or MSB; and hence for this case the cause of failure predicted is rupture of the reinforcement.

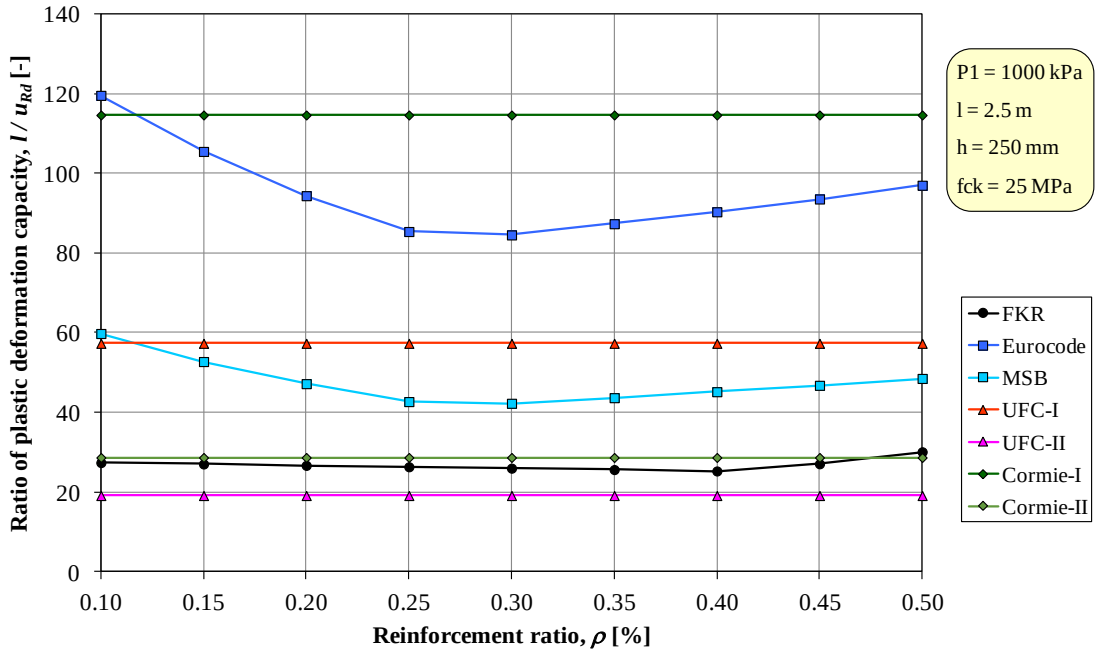

Figure 7.5 *Comparison of the ratio of plastic deformation capacity for various reinforcement amount when basic input data according to Section 7.1 is used.*

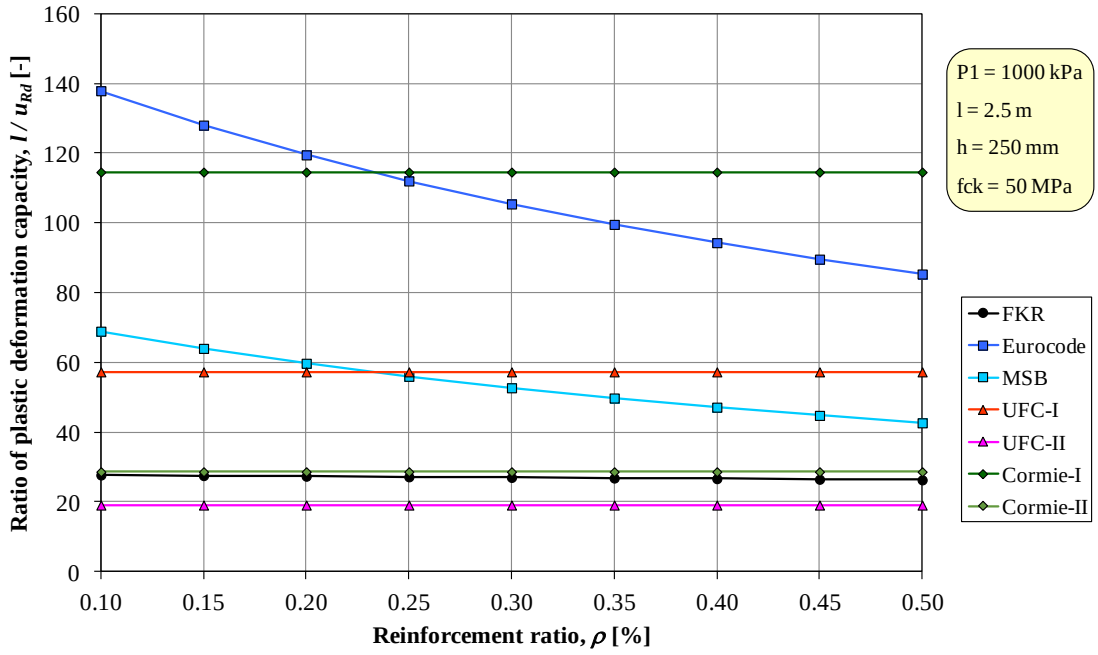

Figure 7.6 *Comparison of the ratio of plastic deformation capacity for various reinforcement amount when a concrete compressive strength of f_{ck} = 50 MPa is used.*

In Figure 7.7 the effect on the plastic deformation capacity due to increased slab thickness *h* is shown. This change reduces the plastic deformation capacity in FKR, Eurocode 2 and MSB; i.e. an increased slab thickness *h* has a negative effect on the plastic deformation capacity. Further, the increased slab thickness makes the result from FKR become more similar to those of UFC when assuming a Type I cross section.

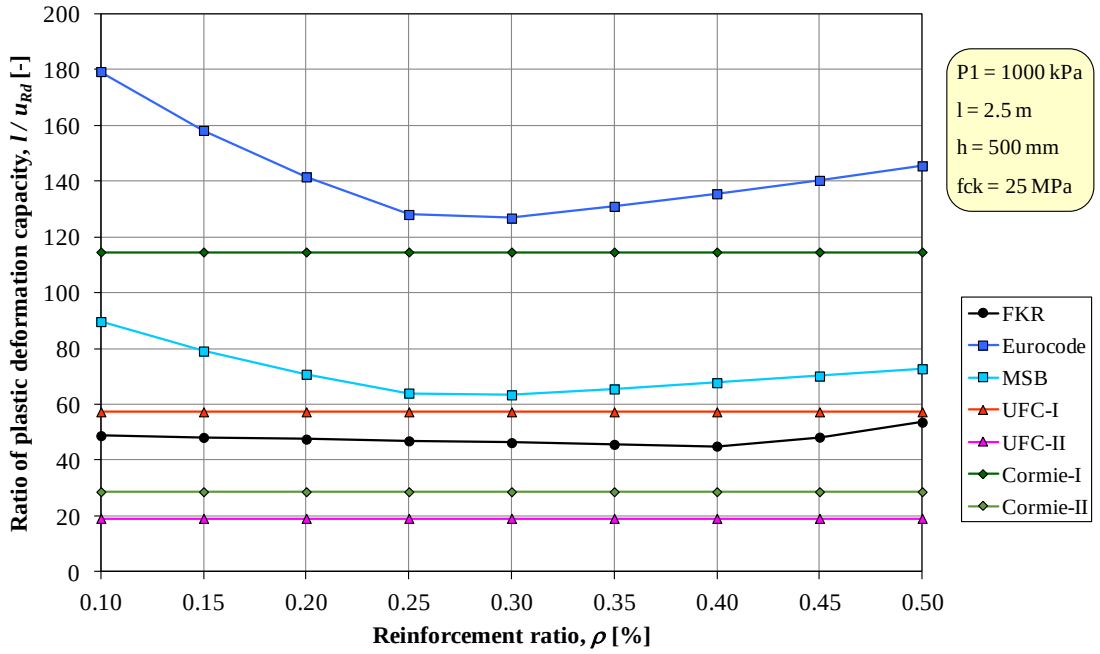

Figure 7.7 *Comparison of the ratio of plastic deformation capacity for various reinforcement amount when a slab thickness of h = 500 mm is used.*

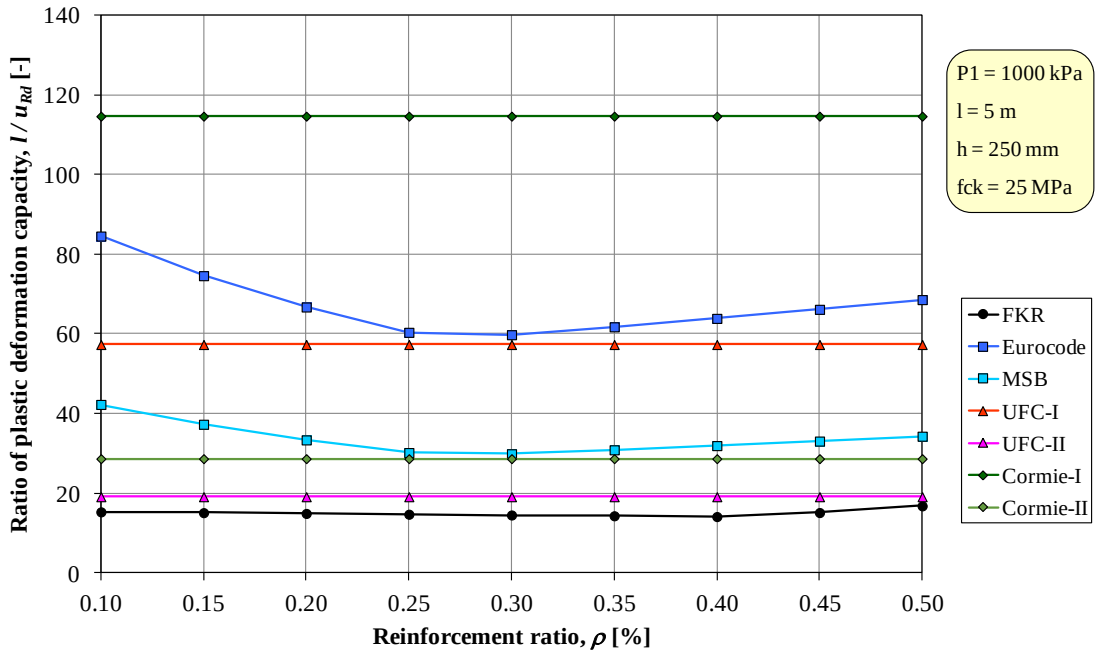

Figure 7.8 *Comparison of the ratio of plastic deformation capacity for various reinforcement amount when a span length of l = 5.0 m is used.*

In Figure 7.8 the effect on the plastic deformation capacity due to increased span length l is shown. This change increases the plastic deformation capacity in FKR, Eurocode 2 and MSB; i.e. an increased span length l has a positive effect on the plastic deformation capacity. Further, the increased span length makes the results from FKR more similar to those of UFC when assuming a Type II cross section.

The effect of an increased slab thickness h and an increased span length l are similar in concept but reversed. A look at the expressions in Section 4.4.4 and Section 4.4.5 for FKR and Eurocode 2, respectively, makes it clear that

$$u_{Rd,FKR} \propto \left(1 + 0.3 \cdot \frac{l}{d}\right) \tag{7.8}$$

$$u_{Rd,EC} \propto \sqrt{\frac{l}{d}} \tag{7.9}$$

for a simply supported strip subjected to an evenly distributed load. Consequently, the effect on u_{Rd}, due to changed ratio l / d, will always be larger for the expressions given in FKR than for those given in Eurocode 2.

7.3.2.2 Comments

Based on the comparison in Section 7.3.2.1 the following observations are made regarding the influence of parameters studied on the ratio of plastic deformation capacity l / u_{Rd}:

- High concrete strength f_{ck} – very small increase in capacity (can be negative in Eurocode, no change in other regulations).
- Large slab thickness h – decreased capacity (larger decrease than in Eurocode, no change in other regulations).
- Increased span length l – increased capacity (larger increase than in Eurocode, no change in other regulations).

The plastic deformation capacity in FKR is larger than that obtained using Eurocode. This is not unrealistic since the safety concept in Eurocode is different compared to that in FKR. However, the expression in FKR is based on an older, more ductile, type of reinforcement. This means that there is a risk that the expressions in FKR are non conservative with respect to the type of reinforcement used in Sweden today. This has also been investigated by Svedbjörk (2014), using an extensive test series carried out at KTH in 2000-2005, see e.g. Ansell and Svedbjörk (2000, 2003, 2005). In this investigation Svedbjörk concluded that the plastic deformation capacity obtained was considerably higher than that proposed in Eurocode 2 but also that there is a risk that the expressions in FKR overestimate the plastic deformation capacity. The latter conclusion was mainly related to the measurements of average plastic reinforcement strain in the plastic hinges; the value of ε_{su} = 75 ‰ that is recommended in FKR for reinforcement of class C was concluded to be too high. The value used in this comparison, i.e. ε_{su} = 30 ‰, is based on the same test series that were used by Svedbjörk and is hence, believed to be an appropriate value.

The plastic deformation capacity obtained using FKR is relatively similar to those obtained when using UFC or Cormie *et al.* when assuming a Type II cross section; i.e. when stirrups are provided in the slab. In Svedbjörk (2014) it was proposed that a variant of the concept used in UFC should be used in FKR as well. However, as briefly stated in Section 4.4.4 the expressions given in UFC and Cormie *et al.* are, by the authors of this report, believed to be very rough and not something to strive for. Further, the ductility of the reinforcement used in the USA and in Europe is not the same; based on Table 4.1 and Table 4.3, more ductile reinforcement is used in USA than in Europe. This might also be a reason why the limit values for plastic rotation capacity used in Cormie *et al.* are somewhat lower than those used in UFC. Therefore, it is here recommended to keep the current concept used in FKR to determine the plastic deformation capacity.

Even though it is not included in this report it should also be mentioned that the plastic rotation capacity, and thus also the plastic deformation capacity, in a strip with fully fixed supports would be considerably smaller in FKR and Eurocode 2 than what is the case for a simply supported beam. However, in UFC and Cormie *et al.* this is not the case; in those regulations the plastic deformation capacity is unaffected by the boundary condition. This is another reason of why it is recommended to not apply the concept of plastic deformation capacity used in UFC and Cormie *et al.*

7.3.3 Stiffness

7.3.3.1 Comparison

In Section 4.2 expressions are given of how the moment of inertia I_c of a concrete slab strip is determined in different regulations. Based on this a stiffness ratio $\eta_{I,FKR}$ can be defined as

$$\eta_{I,FKR} = \frac{I_{c,\#}}{I_{c,FKR}} \tag{7.10}$$

in order to compare how the moment of inertia in regulation # compared to that in FKR. Here $I_{c,\#}$ is the moment of inertia in regulation # and $I_{c,FKR}$ is the moment of inertia in FKR, see equation (4.23).

In Figure 7.9 this stiffness ratio is compared with Eurocode 2 and UFC (Cormie *et al.* is assumed to use the same expression as Eurocode 2). From this it can be seen that there is a considerable difference in stiffness used in the regulations compared. The reason for this is that Eurocode 2 is fully based on the stiffness of a cracked cross section, while UFC is based on an average stiffness of an uncracked and a cracked cross section. The stiffness in FKR is somewhere in between these two cases.

In order to better understand the difference between the stiffness used in FKR, Eurocode 2 and UFC it is of interest to determine how large part of the uncracked and cracked stiffness is assumed when determining the moment of inertia $I_{c,FKR}$. Here it is assumed that the moment of inertia in FKR can be expressed as

$$I_{c,FKR} = \lambda \cdot I_I + (1-\lambda) \cdot I_{II} \tag{7.11}$$

where I_I and I_{II} are the moment of inertia for an uncracked and cracked cross section, respectively, and λ is a stiffness coefficient. This coefficient can then be determined as

$$\lambda = \frac{I_{c,FKR} - I_{II}}{I_I - I_{II}} \tag{7.12}$$

and using this expression a relation according to Figure 7.10 can be determined when concrete strength and slab thickness is varied. From this it can be concluded that $\lambda_{FKR} \approx$ 0.1-0.2, which can be compared to λ_{EC} = 0.0 and λ_{UFC}= 0.5.

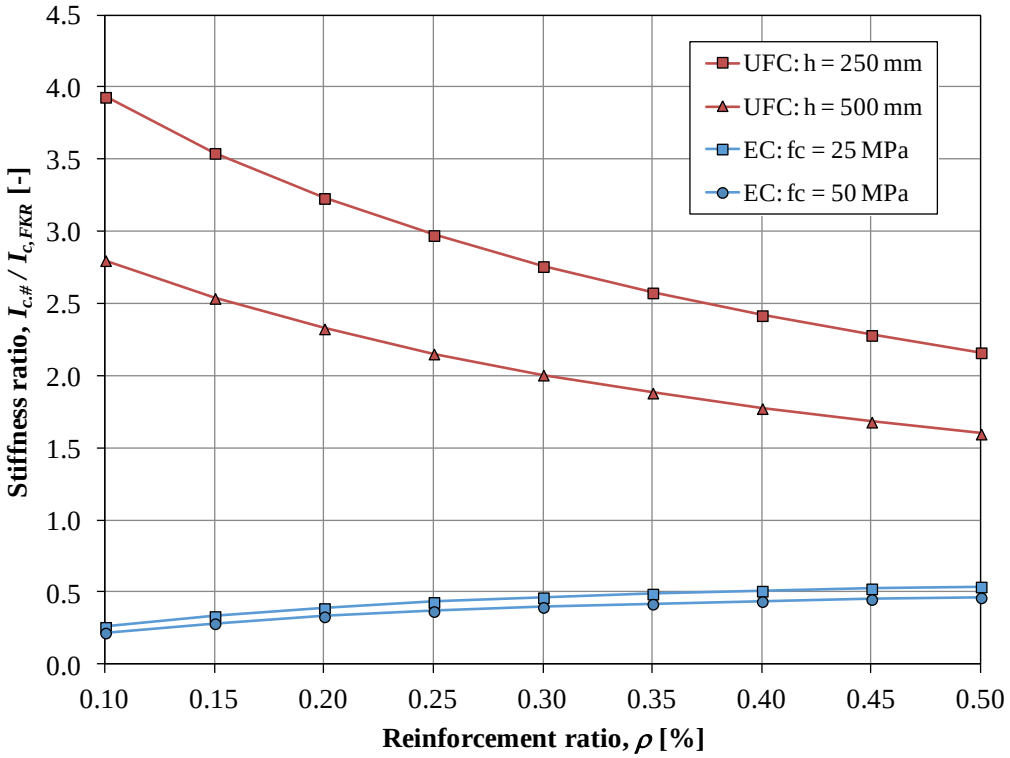

Figure 7.9 *Comparison of the stiffness ratio according to equation (7.10). A ratio of 1.0 corresponds to the stiffness used in FKR.*

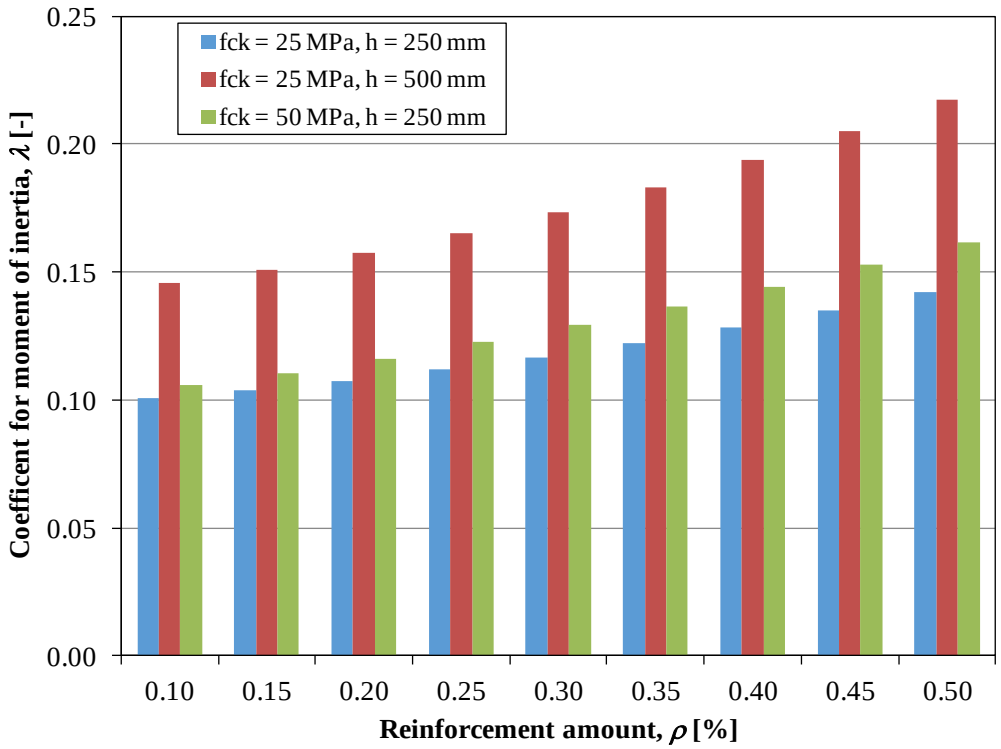

Figure 7.10 *Comparison of the stiffness coefficient λ according to equation (7.10) for moment of inertia in FKR.*

7.3.3.2 Comments

The concept used in FKR to determine the stiffness of a cracked one-way slab is a variant somewhere in between a partially and a fully cracked strip. Depending on the slab thickness and reinforcement amount the resulting stiffness in FKR is about 2-4 times larger than that used for a fully cracked strip and about a factor of 1.5-4 times smaller than the stiffness proposed in UFC. For the cases investigated here it has been found that the stiffness in FKR corresponds to a value of approximately 10-20 % of an uncracked cross section and 80-90 % of a cracked cross section. This is believed to be a rather realistic approximation and it is therefore believed that the expression to determine an effective moment of inertia in FKR is appropriate to use for the type of structure studied in this report. However, in order to further generalise the method to determine the stiffness of the structure it is recommended to base the final moment of inertia on an expression similar to equation (7.11), where the uncracked and cracked moment of inertia is based on the method described in Section 4.2.1. If doing so the stiffness coefficient λ could be set to 0.1-0.2 in accordance with what is currently used in FKR. Such a change would increase the possibility to use FKR for other type of structures as well (e.g. a beam with a T-shaped cross section).

7.3.4 Reinforcement amount

7.3.4.1 Comparison

In Table 7.3 the minimum reinforcement amount for bending moment, according to Section 4.1, is compared for a case when f_{yk} = 500 MPa. From this it can be seen that the minimum reinforcement amount demanded in FKR is substantially smaller than in Eurocode 2, UFC and Cormie *et al*. The amount demanded in Eurocode 2 and Cormie *et al.* are about 45-60 % higher and the amount demanded in UFC is about 65-70 % higher than that demanded in FKR.

Table 7.3 Comparison of minimum reinforcement amount ρ_{min} due to bending moment when f_{yk} = 500 MPa.

f_{ck} [MPa]	$f_{ck,cube}$ [1)] [MPa]	f_{ctm} [2)] [MPa]	$\rho_{min,FKR}$ [%]	$\rho_{min,EC}$ [%]	$\rho_{min,UFC}$ [%]	$\rho_{min,Co}$ [%]
25	30	2.6	0.10	0.13	0.16	0.13
30	37	2.9	0.11	0.15	0.17	0.15
35	45	3.2	0.13	0.17	0.18	0.17
40	50	3.5	0.13	0.18	0.20	0.18
45	55	3.8	0.14	0.20	0.21	0.20
50	60	4.1	0.15	0.21	0.22	0.21

[1)] Compressive cube strength according to Eurocode 2 for the given value of f_{ck}.

[2)] Mean tensile strength according to Eurocode 2 for the given value of f_{ck}.

In Table 7.4 the maximum reinforcement amount for bending moment, according to Section 4.1, is compared for a case when f_{yk} = 500 MPa. From this it can be seen that there is a large difference between FKR and the other regulations compared. This discrepancy also increases with increased concrete strength.

Table 7.4 Comparison of maximum reinforcement amount ρ_{max} due to bending moment when f_{yk} = 500 MPa.

f_{ck} [MPa]	$\rho_{max,FKR}$ [%]	$\rho_{max,EC}$ [%]	$\rho_{max,UFC}$ [%]	$\rho_{max,Co}$ [%]
25	0.5	2.3	1.5	2.1
30	0.5	2.8	1.8	2.5
35	0.5	3.3	2.1	3.0
40	0.5	3.7	2.4	3.4
45	0.5	4.2	2.7	3.8
50	0.5	4.7	3.0	4.2

As a further comparison to the values in Figure 7.10 and Figure 7.11 it can also be mentioned that the reinforcement amount allowed in the Swedish shelter regulations, MSB (2015) are $\rho_{min,MSB}$ = 0.14 % and $\rho_{max,MSB}$ = 1.1 %, respectively.

7.3.4.2 Comments

The minimum reinforcement amounts proposed in FKR are notable smaller than those proposed in Eurocode 2, UFC and Cormie *et al.* The background for the expression used in FKR is not fully clear while it is known for the other regulations. Based on this it is recommended to increase the minimum reinforcement amount in FKR to be in line with that proposed in e.g. Eurocode 2.

It can correctly be argued that it is not realistic to use a reinforcement amount of 2 to 4 % in a slab. However, it is realistic to use an amount that is higher than 0.5 % (i.e. the amount allowed in FKR); e.g. 1.0 to 1.5 %. For a structure subjected to impulse loading, a low reinforcement ratio is positive in many aspects. Hence, if possible it is advantageous to limit the reinforcement amount to a low value; e.g. 0.5 % as is done in FKR. This can probably also be fulfilled in structures, whose main purpose is to withstand extreme loads such as impulse loading from an explosion. However, taking into account the demands present today there is also a need for different type of civil structures to be designed with regard to impulse loading; e.g. road traffic tunnels or buildings in the industry that handles flammable substances. Such structures may contain notably higher reinforcement amounts than ρ = 0.5 %; and hence, there is also a need to incorporate the use of higher reinforcement amounts in a regulation as FKR.

The limitation of maximum reinforcement amount to 0.5 % in FKR is based on the reasoning that concrete crushing should be avoided as cause of failure in the plastic response of a structure. However, theoretically concrete failure is even now obtained when using a concrete of strength f_{ck} = 25 MPa, see e.g. Figure 7.5. Further, there is no need to be too cautious regarding failure due to concrete crushing. This thought is also strongly supported by Table 7.4, in which it is evident that the maximum

reinforcement amount in other regulations is allowed to be considerably higher. Further, in the Swedish shelter regulations, MSB (2015),

Based on this it is recommended that the allowed reinforcement amount in FKR is increased; e.g. to 1.0 %. However, the current limitation of 0.5 % could still be a recommended maximum value in structures whose main purpose is to withstand impulse loading from explosions.

7.4 Shear

7.4.1 Comments on difference in control of shear capacity

The concept used in FKR for the control of shear capacity of an impulse loaded concrete structure differ rather substantially to that used in the other regulations compared in this report. This statement is true for both of how the design shear force and the design shear strength is determined.

The concept applied in FKR when determining the design shear force is to take into account the large shear forces that occurs at a very early initial phase (denoted as the elastic deformation phase in FKR) of the structure that is subjected to the impulse loading. The method used in FKR is developed in order to describe the ability of the concrete structure to withstand this initial loading, and hence also focuses on the structural shear response close to the support. The design shear force can therefore be said to be based upon the support reaction of a simplified dynamic force equilibrium in the structure. Further, the concept used to determine the concrete shear capacity is based on the concrete compressive strength rather, than what is normally the case when determining the shear capacity in statically loaded concrete structures, the concrete tensile strength[10]. This implies that a compressive strut failure, i.e. some type of arch failure, is expected close to the support.

In contrast to FKR the other regulations (Eurocode 2, UFC and Cormie *et al.*) do not make an attempt to describe what happens with the shear force in the initial stage. Instead, in these regulations the design shear force are all based on a concept that depends on the shear forces obtained in the structure at a later phase, corresponding to when the maximum moment capacity is reached in the structure (denoted as the plastic deformation phase in FKR). Hence, this phase corresponds to a load equal to the equivalent static load q, and it can therefore be stated that these regulations use the concept of equivalent static load when determining the design shear force. Due to this similarity it is also natural to use a design shear capacity based on what is used for concrete structures when subjected to ordinary static loading.

It can perhaps be argued that the effect of the initial load stage in FKR and the control of direct shear failure in UFC and Cormie *et al.* have a similar purpose. However, even though there are similarities between these two concepts there are still also some important differences:

- The design force used for direct shear in UFC and Cormie *et al.* still depends on the equivalent static load q at a late stage, i.e. not the initial support reaction as is the case in FKR.

[10] In Eurocode 2 and Cormie *et al.* the design shear strength is proportional to $f_{ck}^{1/3}$, but in reality this expression can be interpreted as just an alternative way to express the concrete tensile strength.

- According to UFC and Cormie *et al.* the shear reinforcement needed to counter a direct shear crack has to be inclined with an angle of 45°. In comparison, the corresponding shear reinforcement needed to handle a shear crack in the initial elastic stage in FKR does not have any such limitations. Such shear reinforcement may instead be placed in the direction of the slab thickness, i.e. more or less parallel to a possible direct shear crack; and consequently not increasing the capacity against such a failure at all.

7.4.2 Concrete shear capacity – Absolute values

7.4.2.1 Comparison

The shear capacity here refers to the shear capacity $V_{Rd,c}$ provided by concrete only; no comparison is made on the effect of shear reinforcement, see Section 7.4.4.

The concrete shear capacity for the basic input data, according to Section 7.1, is shown in Figure 7.11. From this it can be seen that the capacity given by FKR is far higher than that obtained in the regulations compared. Further, the capacity given in UFC is consistently higher than that in Eurocode 2 or Cormie *et al.* Due to decreased effective height a Type II section always results in less capacity than that of a Type II section.

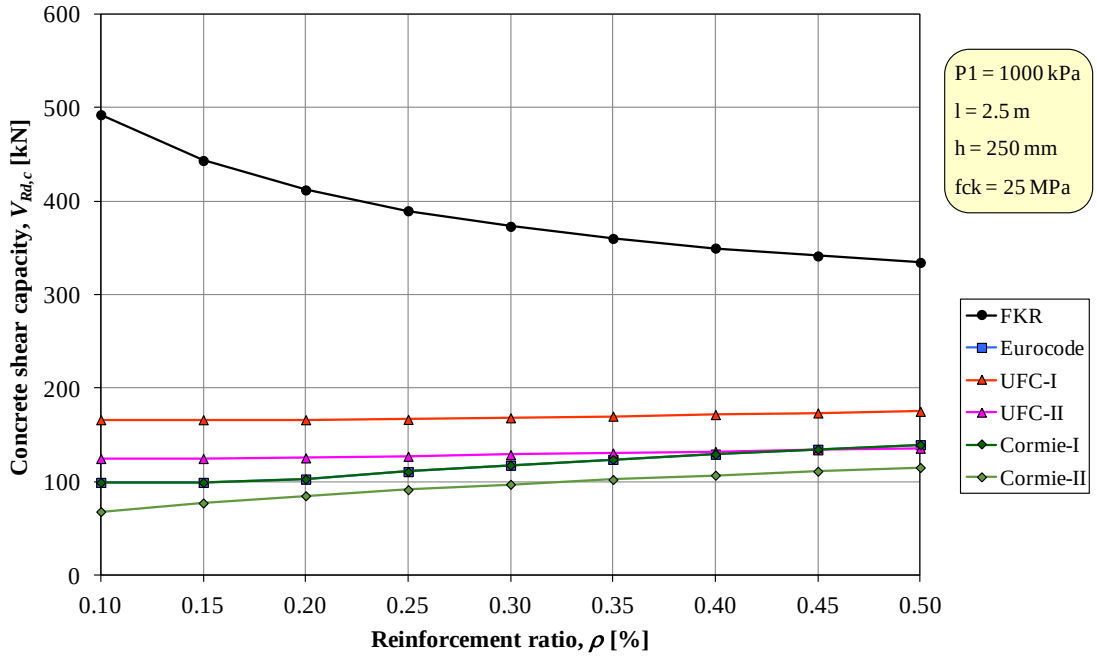

Figure 7.11 Comparison of the concrete shear capacity for various reinforcement amount when basic input data according to Section 7.1 is used.

In Figure 7.12 the effect on the concrete shear capacity due to increased concrete compressive strength f_{ck} is shown. The general observations made in Figure 7.11 are still valid. However, the increase in concrete shear capacity is larger in FKR than in the other regulations; i.e. the deviation between FKR and the regulations compared increase with increased concrete strength.

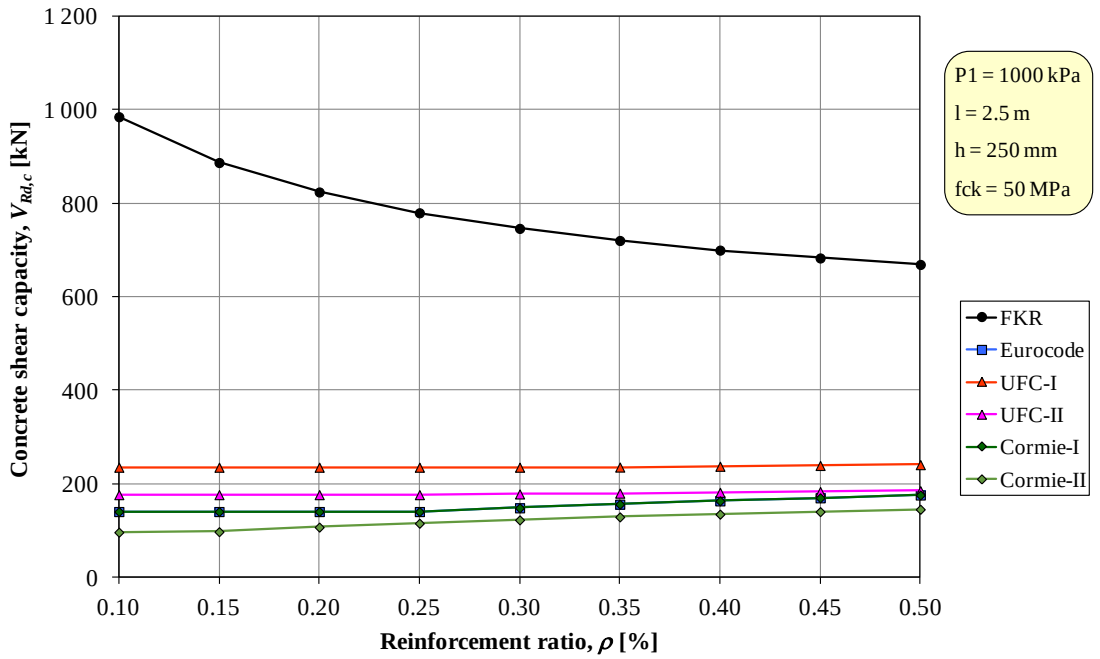

Figure 7.12 Comparison of the concrete shear capacity for various reinforcement amount when a concrete compressive strength of f_{ck} = 50 MPa is used.

In Figure 7.13 the effect on the concrete shear capacity due to increased slab thickness *h* is shown. The general observations made in Figure 7.11 are still valid. However, the increase in concrete shear capacity is larger in the basic case in FKR than in the other regulations; i.e. the deviation between FKR and the regulations compared increase with increased slab thickness.

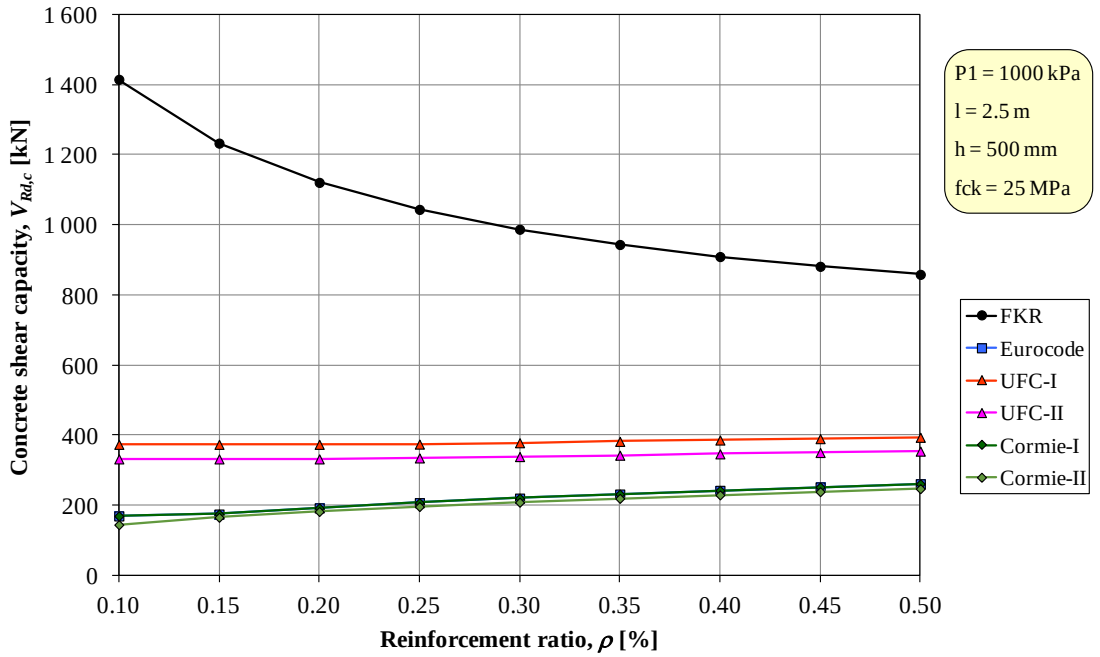

Figure 7.13 Comparison of the concrete shear capacity for various reinforcement amount when a slab thickness of h = 500 mm is used.

In Figure 7.14 the effect on the concrete shear strength due to increased span length l is shown. In FKR this change results in decreased capacity while it has no effect at all in the regulations compared. The reason that the span length affects the shear capacity in FKR is that it influences the length of the shear span a_τ, see equation (5.19); a parameter that is not included in the shear capacity in Eurocode 2, UFC or Cormie *et al.*

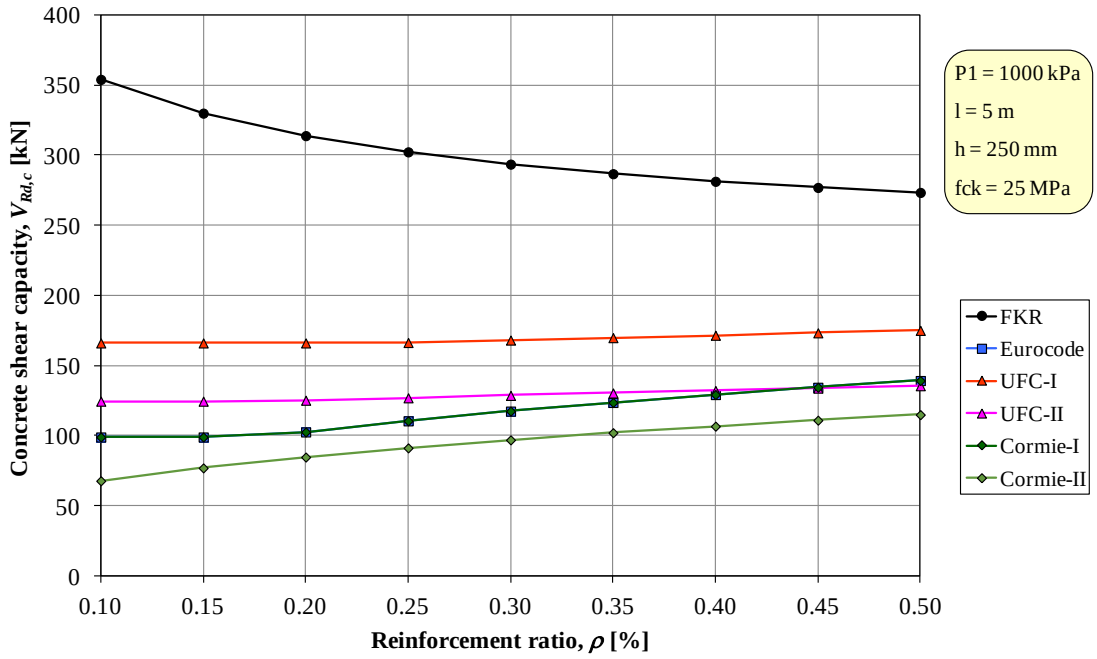

Figure 7.14 Comparison of the concrete shear capacity for various reinforcement amount when a span length of l = 5.0 m is used.

7.4.2.2 Comments

Based on the comparison in Section 7.4.2.1 the following observations are made regarding the influence of parameters studied on the concrete shear capacity $V_{Rd,c}$:

- High concrete strength f_{ck} – increased capacity (larger increase in FKR than in other regulations).
- Large slab thickness h – increased capacity (larger increase in FKR than in other regulations).
- Increased span length l – decreased capacity in FKR, no change in other regulations.

The concrete shear capacity $V_{Rd,c}$ obtained using FKR is considerably larger than the capacity obtained in Eurocode 2, UFC or Cormie *et al.* This difference in capacity, though, does not necessarily mean that the results provided by FKR is incorrect since the design shear force V_{Ed} in FKR is determined differently as well. In the end it is the utility ratio V_{ED} / $V_{Rd,c}$ that is of main interest and this is therefore also compared in Section 7.4.3.

It can be pointed out, though, that the large influence on the concrete shear strength due to varying span length l is a bit strange. It can be argued that this increase in capacity is related to arching effects within the slab close to the support. Similar effects may also be taken into account in Eurocode 2 (not included in the description

in Section 5.2.2 though), but then by reduction of the design shear force instead of increased shear capacity. Further, the effect of this arching effect in Eurocode 2 is then also substantially smaller than what it is in FKR.

The expression used in FKR to determine the concrete shear strength $V_{Rd,c}$ is in practice a scaling of the maximum shear capacity due to shear compression failure; i.e. the crushing of the compressive strut in a cracked concrete structure. Hence, this concept is very different to that of a flexural shear crack, which is used in the other regulations compared.

In FKR the concrete shear capacity is proportional to the parameter k_τ, defined in equation (5.32) in Section 5.3.1 as

$$k_\tau = \frac{0.45}{a_\tau / d} \cdot 0.25 \cdot f_{ck} \qquad k_\tau \leq 0{,}25 \cdot f_{ck} \tag{7.13}$$

and the concrete shear capacity is thus inversely proportional to the ratio a_τ / d. This ratio can be interpreted as the angle θ of a compressive strut following the same direction as the critical shear crack, compare Figure 5.2. For small values of a_τ / d (e.g. < 2) this concept might be realistic in a section close to the support, and in line with so called strut-and-tie models described in e.g. Eurocode 2. However, if this is not the case the conceptual model for the shear force capacity can be questioned. Hence, it can be concluded that the model used in FKR to determine the concrete shear capacity may be reasonable close to the support for control of the initial elastic deformation phase but not in a general section. Further, the large difference in shear capacity, compared to what is obtained in other regulations, make it uncertain whether the model in FKR can be used to determine the design shear capacity in the later plastic deformation phase.

According to Section 5.2.1 $a_\tau \leq 0{,}25$, which in practice means that there will never be any shear reinforcement in at least the middle half of the loaded structure. In a case where the concrete shear capacity is enough this is okay but in a case where shear reinforcement is needed the lack of control in this part of the structure may be a potential problem.

The confusion in FKR, mentioned in Section 5.3.1, regarding which method should be used to determine the concrete shear capacity, may influence the observations made above. The limitation given in equation (5.28), i.e. that an approach related to deep beam theory should be used when $a_\tau / d \leq 1.5$, may indicate that the method used in the comparisons made here is only valid within this limitation. For those cases when $a_\tau / d > 1.5$ another approach should perhaps be used. However, this is unclear in FKR and if this is the intention it has to be clarified which method should be used. Currently, the design shear load $V_{Ed,FKR}$, defined in equation (5.10), is based on a section control located close to the support. Hence, this value may still be used for control of the initial elastic deformation phase but probably not for the later elastic deformation phase.

7.4.3 Concrete shear capacity – Utility ratio

7.4.3.1 Comparison

In this Section the utility ratio of the concrete shear capacity is determined as

$$\eta_{V_{Rd,c}} = \frac{V_{Ed}}{V_{Rd,c}} \tag{7.14}$$

where V_{Ed} is the design shear force and $V_{Rd,c}$ is the concrete shear capacity. In FKR, the design shear force V_{Ed} depends on which deformation phase is checked. Here, only the initial elastic phase (the most critical one), is compared.

The shear utility ratio for the basic input data, according to Section 7.1, is shown in Figure 7.15. From this it can be seen that the shear utility ratio obtained using FKR has a different variation with regard to the reinforcement ratio, compared to the other regulations. The definition of design shear force V_{Ed} is the same in Eurocode 2, UFC and Cormie *et al.* (i.e. based on equivalent static load); and hence, the variation of the utility ratio will be a function of the moment capacity in Figure 7.2 and concrete shear capacity in Figure 7.11. Since those values are fairly well gathered the deviation of the resulting shear utility ratios is also relatively small. In FKR, though, the design shear strength is determined using a different concept and the concrete shear capacity differs considerably, see Section 5.2.1 and Section 7.4.2, respectively. Therefore, it is not surprising that the shear utility ratio for FKR differs to that of the other regulations compared.

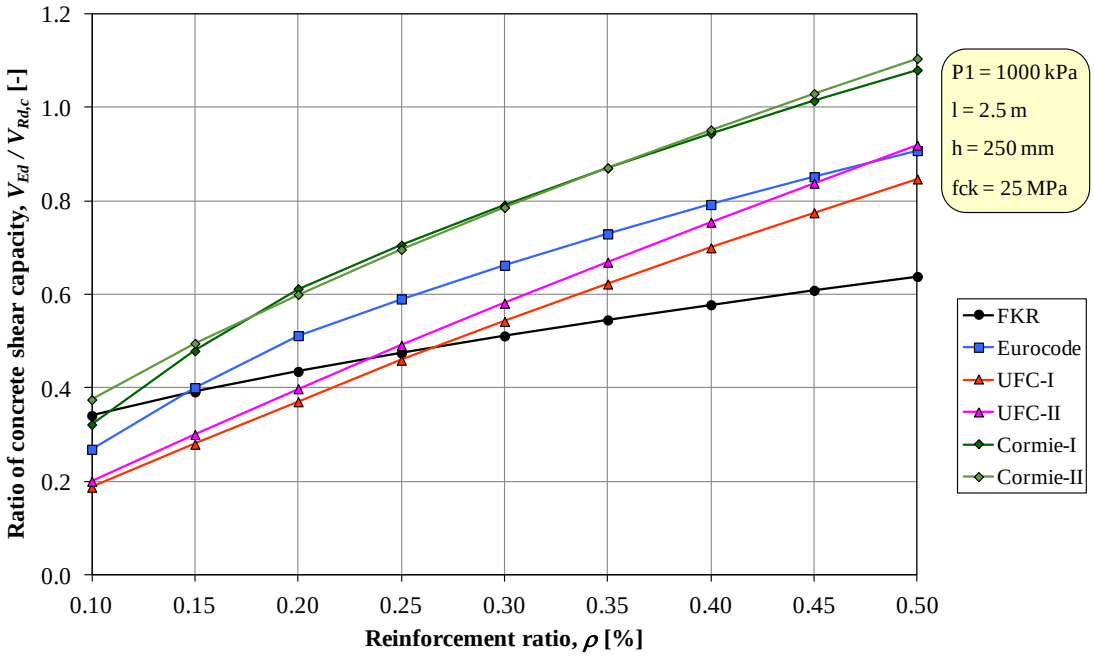

Figure 7.15 Comparison of the utility ratio of concrete shear capacity for various reinforcement amount when basic input data according to Section 7.1 is used.

In Figure 7.16 the effect on the shear utility ratio due to increased concrete compressive strength f_{ck} is shown. From this it can be seen that an increased concrete strength generally also results in decreased shear utility ratios. As concluded in Section 7.4.2 an increased concrete strength leads to higher increase of the concrete

shear strength in FKR than in the other regulations and, therefore, the decrease in the FKR utility ratio is also larger. Accordingly, it now becomes evident that there is a considerable difference in shear utility ratio between FKR and the other regulations compared.

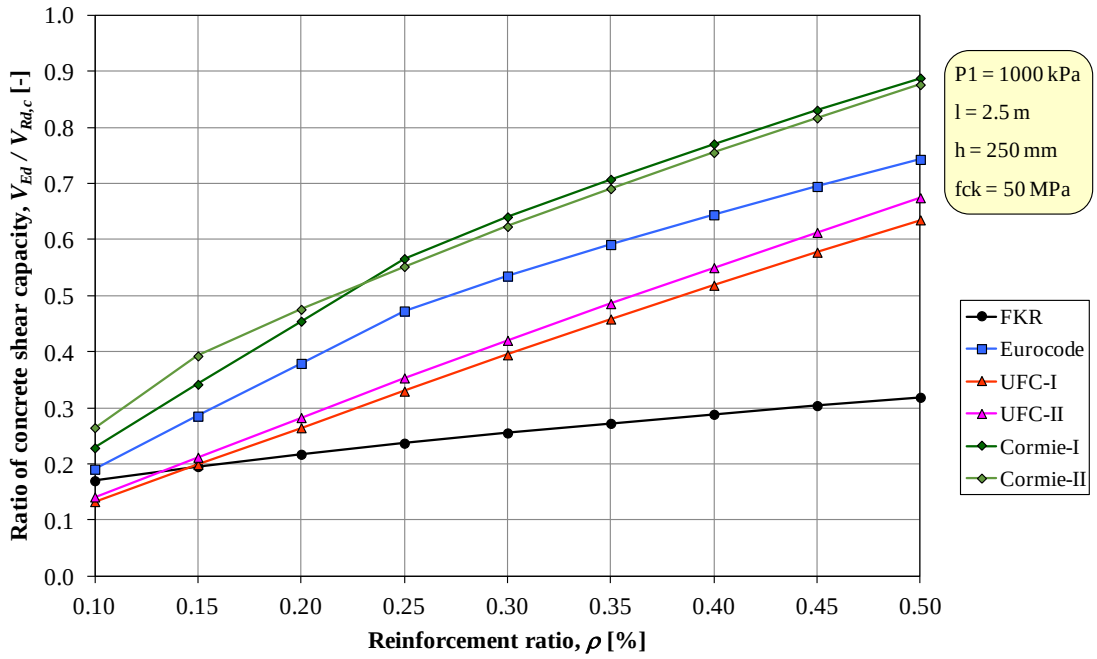

Figure 7.16 Comparison of the utility ratio of concrete shear capacity for various reinforcement amount when a concrete compressive strength of f_{ck} = 50 MPa is used.

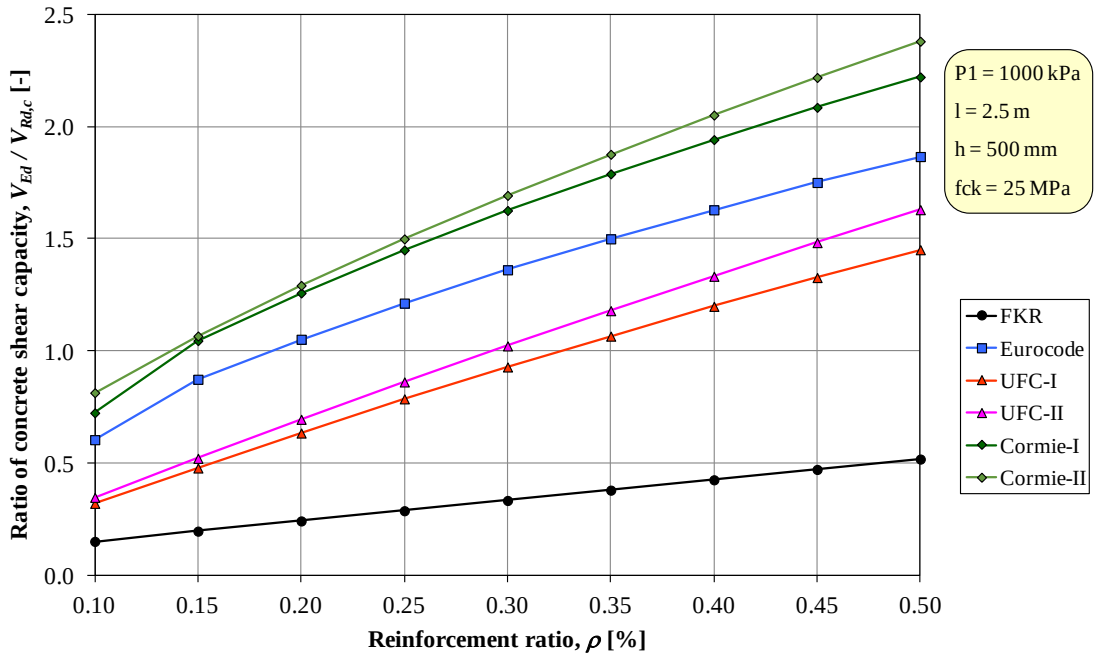

Figure 7.17 Comparison of the utility ratio of concrete shear capacity for various reinforcement amount when a slab thickness of h = 500 mm is used.

In Figure 7.17 the effect on the shear utility ratio due to increased slab thickness *h* is shown. The effect in FKR is that the utility ratio decreases somewhat compared to the basic case in Figure 7.15. For the other regulations, though, the result is dramatically different; the shear utility ratio increase with about a factor of two. Consequently, the effect on the shear utility ratio, due to an increased slab thickness, differs considerably between, on one hand, FKR and, on the other hand, Eurocode 2, UFC and Cormie *et al.*

The reason for this change is that in the latter regulations the design shear strength depends on the bending moment capacity M_{Rd}, which in turn is more or less proportional to the slab thickness *h*. Thus, an increase in slab thickness *h* also results in almost the same increase in design shear force V_{Ed}.

In Figure 7.18 the effect on the shear utility ratio due to increased span length *l* is shown. From this it can be seen that there is a considerable difference in shear utility ratio between FKR and the other regulations compared. This is mainly because the design shear force decreases considerably for the latter when the span length increases, while it, in contrast, increases in FKR. Further, as can be seen in Figure 7.14 the concrete shear strength in FKR decreases with increased span length. Altogether, the difference in shear utility ratio, due to increased span length, is very large between FKR and the other regulations compared.

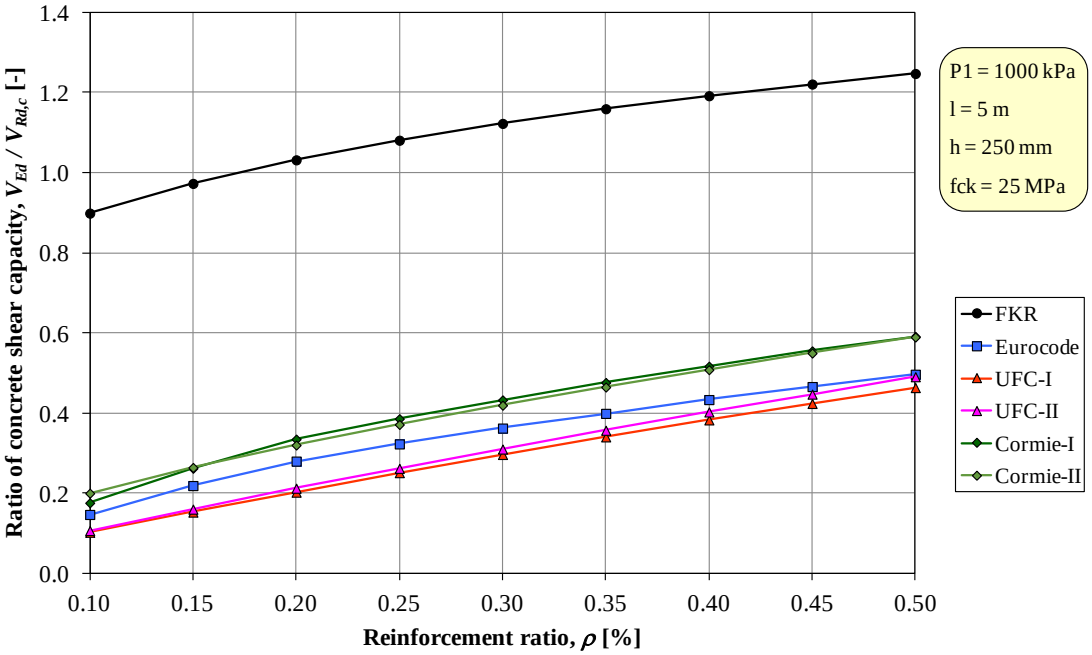

Figure 7.18 Comparison of the utility ratio of concrete shear capacity for various reinforcement amount when a span length of l = 5.0 m is used.

In Figure 7.19 the effect on the shear utility ratio due to increased load peak pressure P_1 is shown. This parameter does not affect the utility ratio for Eurocode 2, UFC or Cormie *et al.* and hence they are identical to the basic case in Figure 7.15. However, in FKR an increased peak pressure results in increased design shear force V_{Ed}, and hence also an increase in the shear utility ratio.

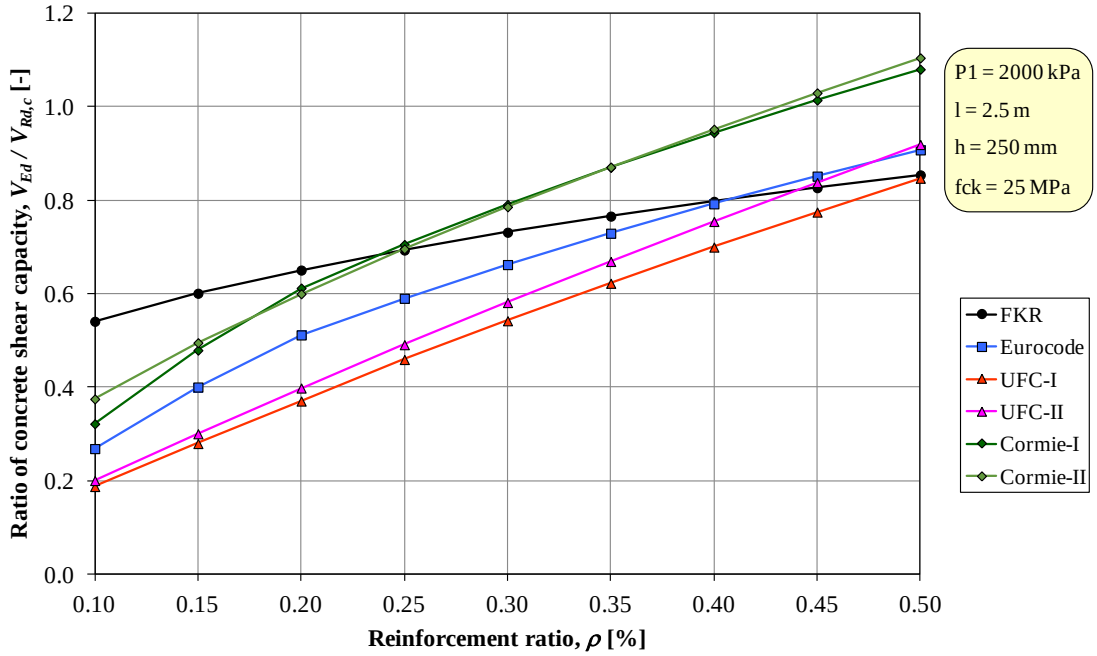

Figure 7.19 Comparison of the utility ratio of concrete shear capacity for various reinforcement amount when a peak pressure of P_1 = 2000 kPa for the impulse load is used.

7.4.3.2 Comments

Based on the comparison in Section 7.4.3.1 the following observations are made regarding the influence of parameters studied on the shear utility ratio V_{Ed} / $V_{Rd,c}$:

- High concrete strength f_{ck} – large decrease in utility ratio in FKR, some decrease in utility ratio in other regulations.
- Large slab thickness h – minor decrease in utility ratio in FKR, large increase in utility ratio in other regulations.
- Increased slab length l – large increase in utility ratio in FKR, large decrease in utility ratio in other regulations.
- Increased load peak pressure P_1 – some increase in utility ratio in FKR, no change in utility ratio in other regulations.

Compared to Eurocode 2, UFC and Cormie *et al.* the shear utility ratio obtained when using FKR is very unstable. Further, the effect on the shear utility ratio is in many cases the opposite in FKR in relation to the other regulations compared. Thus, it can be concluded that the results related to concrete shear failure is very different in FKR compared to Eurocode 2, UFC and Cormie *et al.*

The concept for the design against shear forces in FKR is very different to the other regulations compared. The same holds true for the shear utility ratios obtained in the case study presented in Figure 7.15 to Figure 7.19. Large deviations in the results make it unsure to what extent is it possible to trust the concept used in FKR for control of shear forces.

In this report no comparison with experiments has been made; and hence, it is difficult to draw any strong conclusions regarding these deviations. However, based on the result instability in FKR and the large differences obtained compared to the other regulations compared, it is recommended to replace the method used in FKR for control of shear forces with another alternative. This change can either be to use the concept described in any of the other regulations compared in this report or it can be a combination of that and a newly developed concept as briefly discussed below.

A unique part in FKR, compared to the other regulations treated in this report, is that it strives to take into account the effect on shear due to the initial elastic deformation phase. For unknown reasons this part is not even properly discussed in e.g. UFC. Hence, understandably, it may also be of importance for Fortifikationsverket to keep this part in future editions of FKR. Nevertheless, it is believed by the authors of this report that the present concept in FKR has to be modified. A possible conceptual way to handle shear in future editions of FKR is therefore sketched below:

- As a basis for the control of shear capacity the concept used in Eurocode 2, see Section 5.2.2 and Section 5.3.2, is recommended to be used. This control corresponds to, and would hence fully replace, the control for the phase which FKR denotes as the plastic deformation phase.
- In order to take into account the possible shear effects of the initial loading in the elastic deformation phase a new concept to determine the shear capacity close to the support can be developed. The concept for such a method could perhaps be in line with what is currently used in FKR for barriers (i.e. members with high cross section in relation to its span length), where the internal energy capacity of the compressive strut is regarded and not only its maximum static load capacity. Except for the design shear strength there may also be a need to look over the current concept used to determine the design shear force.

Using an approach as sketched above the main unstable differences of shear capacity and shear utility ratio, between FKR and the other regulations compared with in this report, will disappear. At the same time, the uniqueness of the current edition of FKR, regarding the control of large shear forces close to the support in the initial elastic deformation phase, may be kept.

7.4.4 Reinforcement shear capacity

In this report no comparison is made on the effect of shear reinforcement on the total shear capacity in different regulations. The reason for this is the findings made in Section 7.4.2 and Section 7.4.3 that the concrete shear capacity $V_{Rd,c}$ and shear utility ratio V_{Ed} / $V_{Rd,c}$, respectively, in FKR differs so much compared to that obtained when using the other regulations studied.

Based on the description of the reinforcement shear capacity in Section 5.3.1, though, it can be concluded that the method used in FKR is complex and less intuitive than what is the case in the other regulations compared.

For those cases where shear reinforcement is needed to fulfil the shear capacity in the structure it is recommended that a concept similar to that in Eurocode 2 is used. If not the practicing engineer may encounter different concepts regarding the superposition of shear strength contribution from concrete and steel.

7.4.5 Direct shear

7.4.5.1 Comparison

In FKR there is no special control with regard to direct shear cracks. However, there is a requirement in equation (5.40) that the maximum shear force are not allowed to exceed

$$V_{Ed,FKR,\min} \le 0.25 \cdot f_c \cdot b \cdot d \tag{7.15}$$

This expression corresponds to a compressive shear failure; i.e. that crushing of the inclined compressive strut in the cracked concrete is not reached. A similar expression is also used in Eurocode 2, see equation (5.44), for the same reason:

$$V_{Rd,c,EC} \le 0.30 \cdot \left(1 - \frac{f_{ck}}{250}\right) \cdot f_c \cdot b \cdot d \tag{7.16}$$

In UFC and Cormie et al., direct shear is taken into account in the design. The expressions used to describe the concrete capacity against this type of failure, though, are on the exact same form as those shown in equation (7.15) and (7.16); compare with equation (5.54) for UFC

$$V_{Rd,c,ds,UFC} = 0.16 \cdot f_c \cdot b \cdot d \tag{7.17}$$

and with equation (5.57) for Cormie *et al.*

$$V_{Rd,c,ds,Co} = 0.25 \cdot \left(1 - \frac{f_{ck}}{250}\right) \cdot f_c \cdot b \cdot d \tag{7.18}$$

Consequently, in practice the same type of control is made in all the regulations compared. The difference is that in FKR and Eurocode 2 this control refers to be against compressive shear failure, while it in UFC and Cormie *et al.* refers to be against direct shear.

In UFC and Cormie *et al.*, there are also other requirements of when inclined shear reinforcement is needed in order to handle direct shear; related to e.g. the boundary condition or plastic deformation capacity. In FKR and Eurocode 2, though, there are no requirements of special shear reinforcement in order to use a certain plastic deformation capacity. Consequently, it is here believed to be acceptable to disregard from any special shear reinforcement requirements related to a failure mode such as direct shear.

7.4.5.2 Comments

Based on the similarities of the expressions used in different codes for shear compressive failure (FKR and Eurocode 2) and direct shear (UFC and Cormie *et al.*) it is here interpreted that these checks fill the same purpose. Consequently, this control is already included in the current expressions in both FKR and Eurocode 2; and thus, no special measures has to be taken with regard to direct shear. Neither is it believed that there is a need to put in extra inclined reinforcement close to the support to handle direct shear.

7.5 Spalling and breaching

7.5.1 Charge in contact with slab

7.5.1.1 Comparison

The empirical expression for spalling and breaching given in FKR, see Section 6.2, assumes that the charge is placed in contact with the slab. In contrast the expressions given in UFC and Cormie *et al.* depend on the distance from the charge and slab surface. Nevertheless, it is possible to make a direct comparison of these empirical expressions by assuming a geometric shape of the charge, and based on this determine the distance from the charge centre point and the slab surface. Here, the charge is assumed to be made of a cylinder with height and diameter equal to ϕ_{TNT}, see Figure 7.20. The density of TNT is ρ_{TNT} = 1630 kg/m^3 which means that the geometric size of the charge can be determined as

$$\phi_{TNT} = \left(\frac{\pi}{4} \cdot \frac{W}{\rho_{TNT}} \right)^{1/3} \tag{7.19}$$

where W is the charge weight in kg TNT. The distance r_{TNT} between the charge and the slab can then be determined as

$$r_{TNT} = \frac{\phi_{TNT}}{2} \tag{7.20}$$

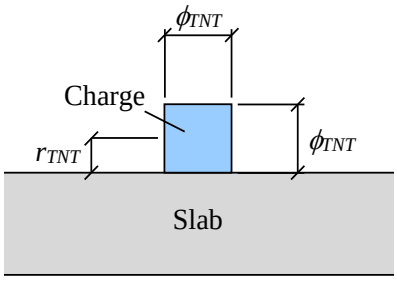

Figure 7.20 Definition of assumed charge geometry and distance r_{TNT} between charge and slab when the charge is placed in contact with the slab.

In Figure 7.21 and Figure 7.22 the required slab thicknesses to avoid spalling and breaching, respectively, are compared in a case where the charge is placed in contact with the slab and the concrete strength is f_{ck} = 50 MPa.

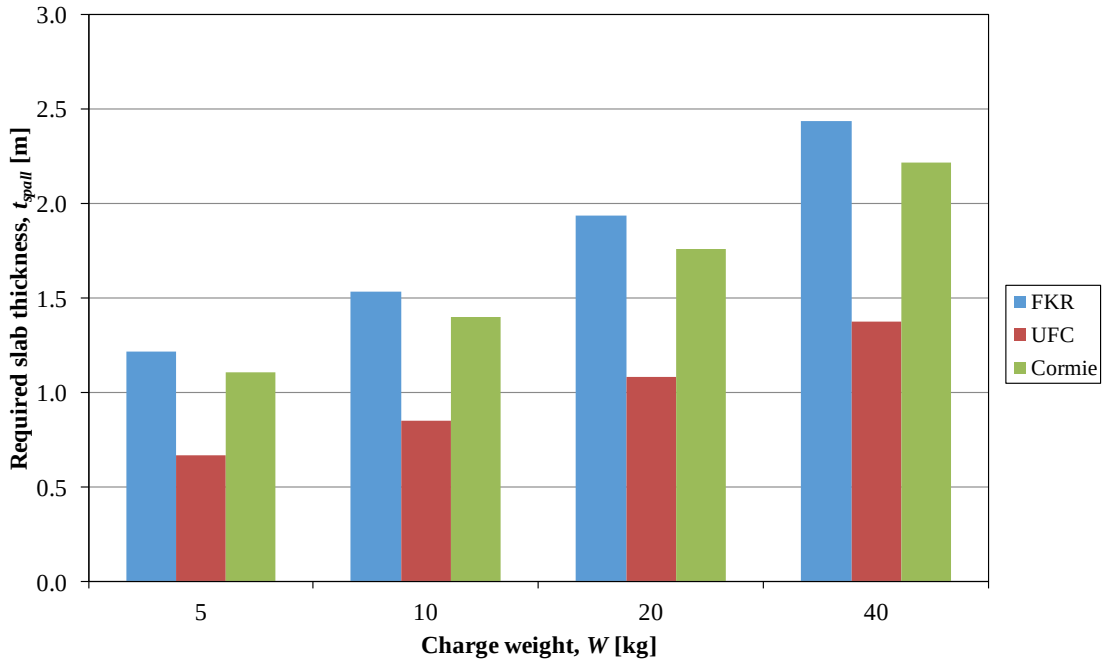

Figure 7.21 *Comparison of required slab thickness to avoid spalling in a case where the charge (W = 5-40 kg TNT) is placed in contact with the slab and f_{ck} = 50 MPa.*

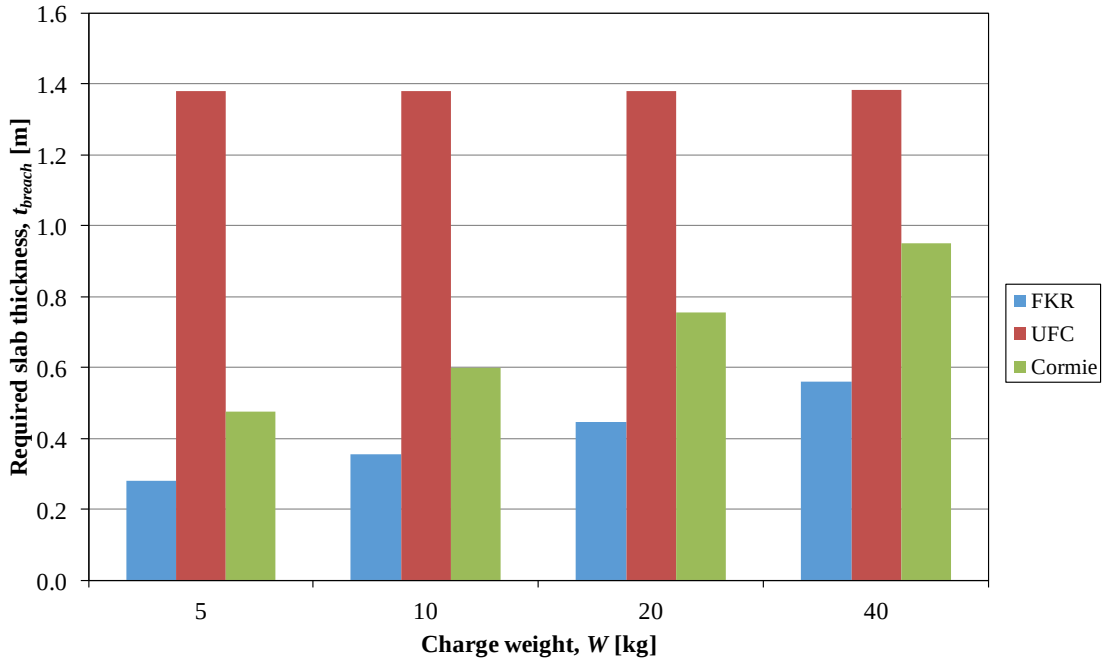

Figure 7.22 *Comparison of required slab thickness to avoid breaching in a case where the charge (W = 5-40 kg TNT) is placed in contact with the slab and f_{ck} = 50 MPa.*

From Figure 7.21 and Figure 7.22 the ratio of required slab thickness for FKR and UFC can be calculated to be

$$\frac{t_{spall,FKR}}{t_{spall,UFC}} = 1.8 \tag{7.21}$$

$$\frac{t_{breach,FKR}}{t_{breach,UFC}} = 0.2 - 0.4 \tag{7.22}$$

while it for FKR and Cormie *et al.* can be calculated to be

$$\frac{t_{spall,FKR}}{t_{spall,Co}} = 1.1 \tag{7.23}$$

$$\frac{t_{breach,FKR}}{t_{breach,Co}} = 0.6 \tag{7.24}$$

Accordingly, it is clear that FKR and Cormie *et al.* produce results that are fairly similar while the results for UFC deviate rather much for the cases studied. Further, comparing the results in UFC for spalling and breaching it is evident that these expressions give strange results since they state that the required slab thickness to avoid breaching is larger than to prevent spalling. Consequently, it seems that the expressions given in UFC are not valid for the studied combination of charge weight W and distance r used in this study (even though the limitations of ψ set in Section 6.4 are fulfilled).

7.5.1.2 Comments

The expressions given in FKR for a charge in contact with concrete slab gives results that are fairly similar to those obtained using the expression presented in Cormie *et al.* Based on this it is believed that the expressions in FKR are appropriate to use.

7.5.2 Charge at a distance from slab

7.5.2.1 Comparison

The expressions given in FKR are based on a case where the charge is placed in contact with the slab. Hence, no comparison with FKR is possible for a case where the charge is placed at a distance from the slab. However, it is still of interest to compare UFC and Cormie *et al.* for such a case, if nothing else since there is such a large deviation between them when the charge is placed in contact with the slab, see Section 7.5.1.

In Figure 7.23 and Figure 7.24 a comparison is made of the expressions presented in UFC and Cormie *et al.* for various charge weights and distances when the concrete strength is f_{ck} = 50 MPa. From this it can be seen that the required slab thickness for the different expressions are relatively similar; much better correspondence here than what is the case in Section 7.5.1.

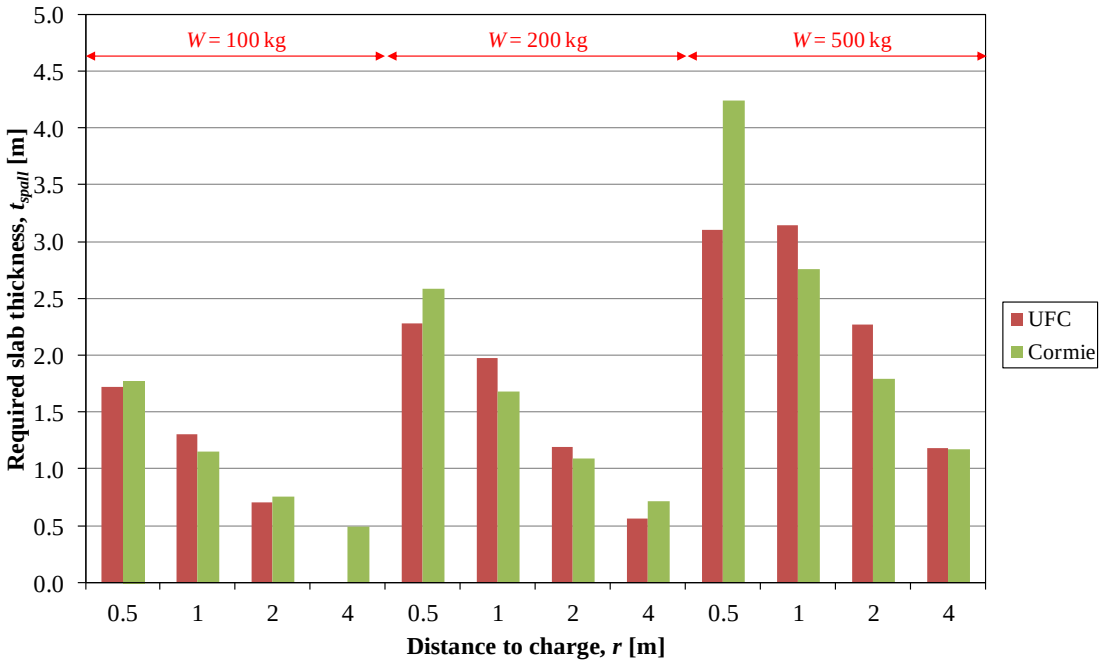

Figure 7.23 *Comparison of required slab thickness to avoid spalling in a case where the charge (W = 100-500 kg TNT) is placed at a distance of r = 0.5-4 m from the slab.*

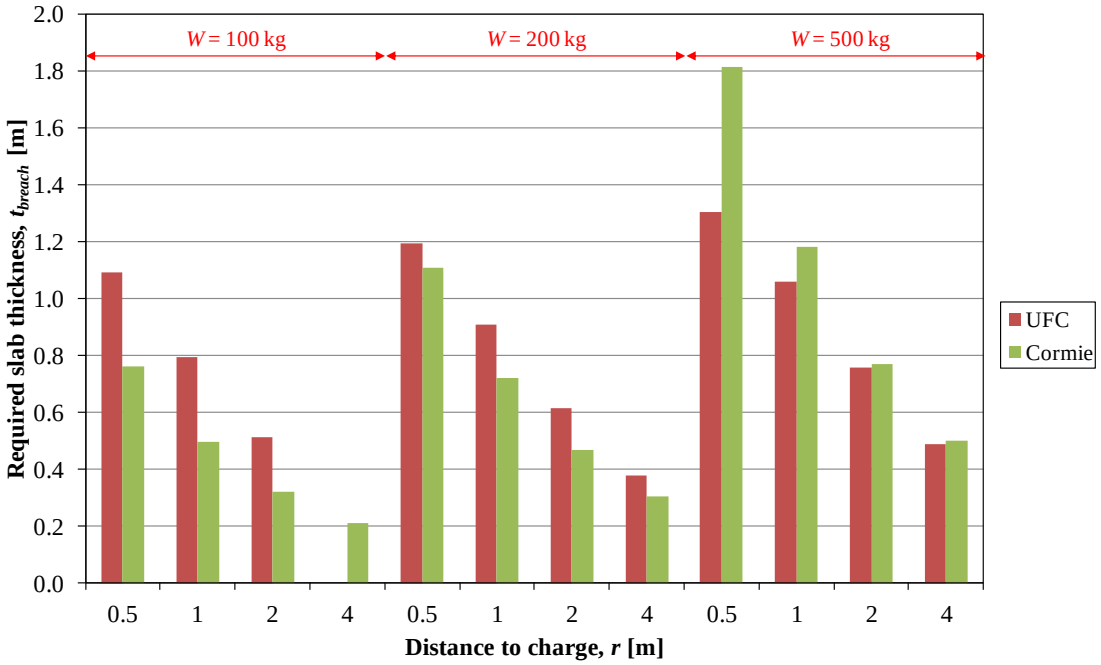

Figure 7.24 *Comparison of required slab thickness to avoid breaching in a case where the charge (W = 100-500 kg TNT) is placed at a distance of r = 0.5-4 m from the slab.*

From Figure 7.23 and Figure 7.24 the ratio of the required slab thickness for Cormie *et al.* and FKR and UFC can be calculated to be

$$\frac{t_{spall,Co}}{t_{spall,UFC}} = 0.8 - 1.4 \tag{7.25}$$

$$\frac{t_{breach,Co}}{t_{breach,UFC}} = 0.6 - 1.4 \tag{7.26}$$

Hence, it can be concluded that the expressions in UFC and Cormie *et al.* corresponds better with each other when the charge is placed with a distance from the slab.

7.5.2.2 Comments

No comparison is possible to make with FKR regarding spalling and breaching for a charge placed at a distance from slab. It can be concluded, though, that the resulting slab thicknesses obtained in the case study are fairly similar in UFC and Cormie *et al.*

8 Conclusions

8.1 Summary of comparisons

8.1.1 General

The Swedish Fortification Agency's design regulation for impulse loaded structures, FKR 2011 (FKR) has been compared with some other design regulations. The main subjects compared were how the different regulations treated material strength, bending moment, shear and spalling/breaching. The comparisons were made based on the concept/expressions used in the respective regulations, and using several case studies of a simply supported slab strip of different geometry, concrete strength and reinforcement amount. In Section 8.1 the observations made, and conclusions drawn from this, are presented, and in Section 8.2 the changes proposed for FKR are summarised.

8.1.2 Material strength

- In none of the regulations compared high strength concrete or fibre reinforced concrete is treated. In FKR the concrete strength is limited to $f_{ck} \leq 50$ MPa and in Eurocode 2 and Cormie *et al.* the limitation is set to 90 MPa; in UFC no upper limit is explicitly mentioned.
- In FKR, no strain rate effects or reinforcement strain hardening effects are taken into account; this is the case, though, in both UFC and Cormie *et al.*
- The influence of different design material strength is negligible for concrete but apparent for reinforcement. The latter is mainly due to strain rate effects.

8.1.3 Bending moment

Moment capacity

- The same concept is used in all the regulations compared and similar results are obtained. The deviation in results obtained is mainly related to different design material strengths.

Plastic deformation capacity

- There is a considerable deviation in plastic deformation capacity in the regulations compared; where the capacities provided in FKR are among the largest. In other regulations, though, there is a requirement of shear reinforcement present in the structure in order to benefit from these large capacities. This, though is not the case in FKR.
- If a one-way slab strip with fixed boundary conditions would have been studied, the plastic deformation capacity in FKR would have been substantially smaller compared to UFC and Cormie *et al.*
- The conceptual model to determine the plastic deformation capacity in FKR is sound. Hence, it is not recommended that the current method is changed in order to use that presented in e.g. UFC.

- Due to less ductile reinforcement used in Sweden today, some caution is appropriate regarding the use of expressions for plastic deformation capacity in FKR. If older type of reinforcement is used (e.g. Ks 40) the expressions given in FKR can probably be used.

Stiffness

- The elastic stiffness used in FKR is based on a mix of an uncracked (10-20 %) and a cracked (80-90 %) cross section. This is believed to be a rather realistic approximation; and hence, appropriate to use.

Reinforcement amount

- The expression used in FKR to determine minimum reinforcement amount is incorrectly based on f_{ck}; this should be replaced with $f_{ck,cube}$. The minimum reinforcement amount in FKR is notably smaller than in the other regulations compared; this deviation also increases with increased concrete strength. The concept used to determine the minimum reinforcement amount is known for the other regulations but not for FKR.
- The maximum reinforcement amount in FKR is considerably lower than in the other regulations compared. The current limitation is based on the reasoning that plastic deformation capacity due to concrete failure should be avoided. However, this is an unnecessary demand and the maximum reinforcement amount could, thus, be increased.
- For a structure subjected to impulse loading it is advantageous to use a small reinforcement amount. For a structure whose main purpose is to withstand impulse loading from an explosion (e.g. fortification or civil defence shelter) it may therefore still be recommended to use the current limitation of 0.5 % in reinforcement amount. However, an increased maximum limitation would make it easier for certain civil structures (e.g. road traffic tunnels), which normally contain larger reinforcement amounts than that, to also make use of the recommendations in FKR.

8.1.4 Shear

Concrete shear strength

- The concrete shear capacity obtained using FKR is considerably larger than the capacity obtained in the regulations compared. In FKR the span length influence the concrete shear strength; a parameter that does not have any influence at all in the other regulations compared.
- The expression used in FKR to determine the concrete shear strength is in practice a scaling of the shear compression failure, and is very different to that of a flexural shear crack, which is used in the other regulations compared. This approach may be reasonable to use close to the support but not in a general section.
- The shear utility ratio obtained when using FKR is very unstable; in many cases the effect of a changed parameter is the opposite in FKR in relation to the other regulations compared.

- Based on the result instability observed when using FKR and the large differences obtained compared to the other regulations compared, it is recommended to modify the method in FKR for the control of shear forces. This change can e.g. be to use the concept described in any of the other regulations compared in this report or it can be a combination of that and a newly developed concept as briefly described below:
    - As a basis use the concept of equivalent static load to determine the design shear force and Eurocode 2 to determine the shear strength in impulse loaded structures. Such an approach would rather well correspond to the control of the plastic deformation phase in FKR.
    - In order to take into account the possible effect of the initial loading close to the support in the elastic deformation phase the concept used in FKR may perhaps still be used. However, it is recommended to investigate whether the current concept can be further developed. Such a development could possibly be in line with what is currently used in FKR for barriers (i.e. members with high cross section in relation to its span length), where the internal energy capacity of the compressive strut is regarded instead of just its maximum static load capacity.

Reinforcement shear strength

- Due to considerable differences in the concept and results obtained for the concrete shear strength the reinforcement shear strength has not been further compared in this report. It can be concluded, though, that the method used in FKR is complex and less intuitive than what is the case in the other regulations compared. This complexity is further enhanced due to unclear explanation and direction in the current version of FKR.

Direct shear

- The concept of direct shear is not treated in FKR. However, the expressions used for this in UFC and Cormie *et al.* is more or less the same as the one used in FKR for compressive shear failure. Hence, it is here interpreted that these expressions fill the same purpose; and consequently, the control for direct shear cracks is indirectly already included in FKR.
- In UFC and Cormie *et al.* there are certain requirements on inclined shear reinforcement close to the support in order to handle shear cracks. However, it is not believed that there is a need to include such requirements in FKR.

8.1.5 Spalling and breaching

Charge in contact with slab

- The expressions given in FKR gives results that are fairly similar to those obtained in Cormie *et al.*; and hence, it is believed that the expressions in FKR are appropriate to use.

Charge at a distance from slab

- The expressions given in FKR are based on a case where the charge is placed in contact with the slab; and hence, no comparison is possible for a case where

the charge is placed at a distance from the slab. A comparison between UFC and Cormie *et al.*, though, shows that they produce fairly similar results.

8.2 Proposed changes to FKR

8.2.1 General

- Since Eurocode 2 is in full use in Sweden today it is generally recommended that an rapprochement of FKR is made with Eurocode 2. Hence, where changes are made in FKR it is recommended to consider if the corresponding method used in Eurocode 2 also can be used in FKR. This is e.g. the case for moment and shear capacity for the plastic deformation phase, minimum reinforcement amount for bending and moment of inertia in a cracked cross section. For shear capacity in the elastic deformation phase another method than those given in Eurocode 2 is still needed.

8.2.2 Material strength

- The increased strength due to high strain rate might be worth including in FKR. If so it is recommended that such a concept is implemented only for cases where it is clear that high strain rates will be obtained. For which cases such an increase would be suitable, though, have not been studied in this report.
- It would be of interest to incorporate the use of higher concrete strength in FKR and also the effect of fibre reinforced concrete.

8.2.3 Bending moment

Moment capacity

- The expression used in FKR to determine the moment capacity is an approximation. Although there is only a minor discrepancy it is still recommended to change to the expressions used in Eurocode 2 in order to better reflect a correct equilibrium case. If an increase in maximum reinforcement amount is implemented this becomes more important.

Plastic deformation capacity

- The average reinforcement strain should be reduced to better reflect the strain observed in experiments. A value of ε_{su} = 30 ‰ is proposed.
- The expression previously used in Bk 25 for control of concrete compression failure should be reintroduced. The general versions of this expression and that for reinforcement rupture are recommended to be used.

Stiffness

- No changes recommended; the current method is appropriate to use.

Reinforcement amount

- In the current expression in FKR to determine minimum bending reinforcement amount, see equation (4.3), f_{ck} should be replaced with $f_{ck,cube}$.
- The minimum reinforcement amount is currently somewhat low and it is recommended that it is increased; e.g. in line with what is proposed in Eurocode 2.
- In order to improve the possibility to increase FKR it is suggested that the maximum reinforcement amount allowed is increased to $\rho_{max} \approx 1.0$ %. The current maximum amount of 0.5 %, though, can still be recommended to be used in structures whose main purpose is to withstand impulse loading from e.g. explosions. Such a change would increase the possibility to use FKR in civilian structures such as tunnels.

8.2.4 Shear

Concept for control of shear forces

- It is recommended that the method used in FKR for control of shear forces should at least partly be replaced; e.g. the concept used in Eurocode 2, UFC and Cormie *et al*. is recommended to be used as a basic design (corresponding to the plastic deformation phase).
- The basic concept for the initial elastic deformation phase does not have any equivalence in the regulations compared with in this report and can be kept. It should be considered, though, if it is possible to modify the current method and perhaps use an energy concept similar to what is currently used for barriers in FKR.
- If the method used in FKR is kept the current descriptions and equations need to be thoroughly modified; now there are several misprints and/or unclear guidelines. The

Direct shear

- No changes recommended if current concept of initial elastic deformation phase is kept.
- If the current concept of initial elastic deformation phase is removed there may perhaps be a need to also include a control for direct shear.

Shear reinforcement

- The maximum spacing of shear reinforcement is recommended to be $s_{max} = 0.5 \cdot d$ for all cases.

8.2.5 Spalling and breaching

Charge in contact with slab

- In the current expression in FKR to determine a reduction thickness for the slab thickness, see equation (6.5), f_{ck} should be replaced with $f_{ck,cube}$.

8.3 Suggestions for further research

Based on the comparisons made in this report it can be concluded that there are some distinct areas which are of special interest for further research that can be implemented in FKR:

- Plastic deformation capacity
- Control of shear capacity
- Material properties

These areas are briefly discussed below.

Plastic deformation capacity

The plastic deformation capacity is of essential importance for a structure to effectively resist impulse loading. The method used in FKR to determine this parameter was originally developed for another, more ductile, type of reinforcement than that used in Sweden today. Hence, there is need to update the current method with regard to this change.

Control of shear capacity

The method used in FKR to control the effect of shear is conceptually rather different to that used in other regulations. The comparisons made in this report indicate that the current method used in FKR need to be modified. One possible way to do this could be to accept a method described in another regulation.

However, a unique aspect of FKR is that it strive take into account the effect of initial loading in the elastic deformation phase; something that is not explicitly handled at all in any of the other regulations compared in this report. Even though this aspect is unique it is still believed that there is need to further look into the model of how to determine both the design shear force and the design shear strength.

Material properties

Based on the regulations compared in this report it can be concluded that there is a lack of information of how to handle high strength concrete and fibre reinforced concrete in structures subjected to impulse loading. Therefore, this is a field in which there is need for further research; is it possible to use the same type of models applied for normal strength concrete or is there need to change these in order to make use of concrete with different material properties.

Another part of the material strength is how to handle the effect of high strain rates. Such effects are currently not explicitly included in FKR. It can be worthwhile, though, to consider incorporating such effects to some degree. For what cases and how this should be done, though, has not been further treated in this report and is therefore a possible area for further studies.

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