

École polytechnique de Louvain

Steel reinforced concrete columns using high strength grades

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Abstract

This Master's thesis aims to adjust the EN 1994-1-1 Simplified Method to allow the use of high-strength materials for fully and partially encased sections. Four analytical design methods, using plastic stress blocks, were chosen from the literature and adapted to authorize the use of high-performance materials for concrete and steel grades up to C120 and S600, respectively. A FEM 2D-model was developed and calibrated according to experimental tests found in open literature. This numerical model served as a basis to compare the studied methods. A sensitive study was performed and analyzed the different analytical methods for fully and partially encased sections along strong axis. This investigation proves that an easy-to-use method can be adopted in the design of composite column considering high-strength materials.

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Contents

1	Introduction	11
1.1	Context	11
1.2	Objective	13
1.3	State of the art	15
1.3.1	High-strength materials	15
1.3.2	Construction using high-strength materials	17
2	Design methods for steel-concrete composite columns	19
2.1	EN 1994-1-1 - Simplified Method of design	19
2.1.1	General	19
2.1.2	Resistance of cross-section to pure compression	21
2.1.3	Buckling resistance of members to axial compression	22
2.1.4	Resistance of cross-section to combined compression and bending moment	24
2.1.5	Buckling resistance of members to combined axial compression and bending moment	28
2.1.6	Buckling resistance of members to combined axial compression and bi-axial bending moment	31
2.1.7	Method 1 - Adapted Simplified Method of EN 1994-1-1 for high-strength materials	31
2.2	BC4:2015 - Extension of EN 1994-1-1	32
2.2.1	General	32
2.2.2	Materials	32
2.2.3	Material compatibility	35
2.2.4	Method 2 - Adapted Simplified Method of BC4:2015	36
2.3	Australian/New Zealand Standard	37
2.3.1	Method 3 - Adapted method of AS/NZ Standard	38
2.4	RFCS SMARTCoCo project	38
2.4.1	Method 4 - Adapted method of the SMARTCoCo project	39
2.5	Fiber Method	39
3	Experimental tests and numerical validation	43
3.1	Experimental test	43
3.1.1	Materials	44
3.1.2	Test setup and instrumentation	45
3.1.3	Test results	46
3.2	SAFIR numerical model	46
3.2.1	Model characteristics	46
3.2.2	Materials definition	49

3.3	Comparison between numerical and experimental values	51
4	Comparisons between analytical methods with FEM model	53
4.1	Analytical methods	53
4.1.1	Method 1 - Adapted Simplified Method of EN 1994-1-1	53
4.1.2	Method 2 - Adapted method of BC4:2015 design guide	53
4.1.3	Method 3 - Adapted method of AS/NZ Standard	54
4.1.4	Method 4 - Adapted Method of SMARTCoCo project	54
4.2	Considered parameters	54
4.3	Sensitive study	57
4.3.1	Comparison of the different methods for FEC	59
4.3.2	Influence of parameters $\bar{\lambda}$, δ and material compatibility for FEC . .	66
4.3.3	Comparison of the different methods for PEC	71
4.3.4	Influence of parameters $\bar{\lambda}$ and δ for PEC	77
4.4	Interpretation of the results	79
5	Conclusion	81
5.1	Solution developed	81
5.2	Limitations and future work	82
A	Interaction curve	85
A.1	Fully concrete encased steel section	85
A.2	Partially concrete encased steel section	87
B	Sensitivity study results	88
B.1	Fully encased concrete cross-section	88
B.1.1	Box plot for high-strength materials : test 3	145
B.1.2	Influence of the material compatibility for methods 3 and 4	147
B.2	Partially encased concrete cross-section	148
B.2.1	Box plot for high-strength materials : test 3	176

List of Figures

1.1	Latitude building in Sidney	12
1.2	Reinforced concrete element	12
1.3	Steel-concrete composite element	12
1.4	Typical cross-section of composite columns	13
1.5	Stress-strain diagram for normal- and high-strength materials	14
1.6	Master's thesis overall objective	15
1.7	Lotte World Tower	17
2.1	Buckling curves	23
2.2	Evolution of the stress distribution in a cross-section subjected to an eccentric axial load	25
2.3	Interaction curve for fully encased section	25
2.4	Calculation of point C of the interaction curve	26
2.5	Calculation of point D of the interaction curve	27
2.6	Parameters of the composite cross-section	27
2.7	Reduced design steel strength due to transverse shear forces	27
2.8	Uniform bending moment distribution	28
2.9	Factors β for the determination of moments to second order theory	29
2.10	Interaction curve in the critical cross-section	30
2.11	Non-uniform bending moment distribution	30
2.12	Plastic and reduced interaction curves	32
2.13	Types of concrete filled tubular cross-sections	32
2.14	Parabola-rectangle stress-strain relations for different concrete strength classes	33
2.15	Derivation of rectangular stress block from parabolic-rectangle stress distribution	33
2.16	Axial load-bending moment interaction curve from Kim and <i>al.</i> test	36
2.17	Evolution of K_{sa} with λ	39
2.18	Evolution of K_c with λ	39
2.19	Interaction curves of the Fiber Method implemented in an Excel spreadsheet	40
2.20	Discretization of the section	40
2.21	Stress distribution in function of the position of neutral axis for FEC cross-section	41
2.22	Stress distribution in function of the position of neutral axis for PEC cross-section	41
3.1	Sections of the specimens from the experimental tests	43
3.2	Stress-strain relationship of thick steel plates	44
3.3	Stress-strain relationship of high-strength concrete	45
3.4	Test setup	45

3.5	Cross-section and mesh from GiD software for column C1	47
3.6	Cross-section and mesh from GiD software for column C11	47
3.7	Cross-section and mesh from GiD software for column C6	47
3.8	Boundary conditions of the 2D column	48
3.9	Applied load along y axis on the column	48
3.10	Result from DIAMOND software	48
3.11	Stress-strain relation for concrete in compression according to EN 1992-1-2	49
3.12	Stress-strain relation for structural steel according to EN 1993-1-2	50
3.13	Stress-strain relation for reinforcing steel according to EN 1992-1-2	51
3.14	P-d diagram for experimental and numerical values of column C1	52
3.15	P-d diagram for experimental and numerical values of column C6	52
3.16	P-d diagram for experimental and numerical values of column C11	52
4.1	Comparison between SAFIR software and analytical methods implemented with a Fiber Method	57
4.2	Points of the interaction curve related to the eccentricity e	58
4.3	Maximum applied bending moment on a composite column	58

List of Symbols

$(EI)_{eff,II}$	Effective flexural stiffness for use in second-order analysis
$(EI)_{eff}$	Effective flexural stiffness for calculation of relative slenderness
α	Factor; parameter
α_{cr}	Factor by which the design loads would have to be increased to cause elastic instability
$\alpha_{M,y}, \alpha_{M,z}$	Coefficient related to bending of a composite column about the y-y axis and the z-z axis respectively
α_M	Coefficient related to bending of a composite column
β	Factor
χ	Reduction factor for flexural buckling
δ	Factor; steel contribution ratio
η_a, η_{ao}	Factors related to the confinement of concrete
η_c, η_{co}	Factors related to the confinement of concrete
γ_c	Partial factor for concrete
μ	Nominal factor
μ_{dy}, μ_{dz}	Factor μ_d related to plane of bending
μ_d	Factor related to design for compression and uniaxial bending
$\bar{\lambda}$	Relative slenderness
ϕ	Diameter (size) of a steel reinforcing bar
ρ	Parameter related to reduced design bending resistance accounting for vertical shear
ρ_s	Parameter; reinforcement ratio
σ_s	Stress in the tension reinforcement
$\varphi(t, t_0)$	Creep coefficient, defining creep between times t and to, related to elastic deformation at 28 days
φ_t	Creep coefficient

A	Cross-sectional area of the effective composite section neglecting concrete in tension
A_a	Cross-sectional area of the structural steel section
A_c	Cross-sectional area of concrete
A_s	Cross-sectional area of reinforcement
A_v	Shear area of a structural steel section
b	Width of the flange of a steel section
b_c	Width of the concrete encasement to a steel section
b_f	Width of the flange of a steel section
c_y, c_z	Thickness of concrete cover
d	Overall diameter of circular hollow steel section
e	Eccentricity of loading
E_a	Modulus of elasticity of structural steel
$E_{c,eff}$	Effective modulus of elasticity for concrete
E_{cm}	Secant modulus of elasticity of concrete
E_s	Design value of modulus of elasticity of reinforcing steel
f_{cd}	Design value of the cylinder compressive strength of concrete
f_{ck}	Characteristic value of the cylinder compressive strength of concrete at 28 days
f_{cm}	Mean value of the measured cylinder compressive strength of concrete
f'_c	Cylinder compressive strength of concrete at 28 days
f_{sd}	Design value of the yield strength of reinforcing steel
f_{sk}	Characteristic value of the yield strength of reinforcing steel
f_u	Specified ultimate tensile strength
f_{yd}	Design value of the yield strength of structural steel
f_{ys}	Nominal value of the yield strength of structural steel
f_y	Nominal value of the yield strength of structural steel
h	Overall depth; thickness
h_a	Depth of the structural steel section
h_c	Depth of the concrete encasement to a steel section; thickness of the concrete flange
h_f	Thickness of concrete flange
h_n	Position of neutral axis

h_s	Distance between the longitudinal reinforcement in tension and the centre of compression
I_a	Second moment of area of the structural steel section
I_c	Second moment of area of the un-cracked concrete section
I_s	Second moment of area of the steel reinforcement
k	Amplification factor for second-order effects; coefficient; empirical factor for design shear resistance
L	Length
M	Bending moment
M_{Ed}	Design bending moment
$M_{max,Rd}$	Maximum design value of the resistance moment in the presence of a compressive normal force
$M_{pl,N,Rd}$	Design value of the plastic resistance moment of the composite section taking into account the compressive normal force
$M_{pl,Rd}$	Design value of the plastic resistance moment of the composite section with full shear connection
$M_{pl,y,Rd}$	Design value of the plastic resistance moment about the y-y axis of the composite section with full shear connection
$M_{pl,z,Rd}$	Design value of the plastic resistance moment about the z-z axis of the composite section with full shear connection
$M_{y,Ed}$	Design bending moment applied to the composite section about the y-y axis
$M_{z,Ed}$	Design bending moment applied to the composite section about the z-z axis
N	Compressive normal force
$N_{cr,eff}$	Elastic critical load of a composite column corresponding to an effective flexural stiffness
N_{cr}	Elastic critical normal force
N_{Ed}	Design value of the compressive normal force
$N_{G,Ed}$	Design value of the part of the compressive normal force that is permanent
$N_{pl,Rd}$	Design value of the plastic resistance of the composite section to compressive normal force
$N_{pl,Rk}$	Design value of the plastic resistance of the composite section to compressive normal force
$N_{pm,Rd}$	Characteristic value of the plastic resistance of the composite section to compressive normal force
r	Ratio of end moments

t	Thickness
t_0	Age at loading
t_f	Thickness of a flange of the structural steel section
t_w	Thickness of the web of the structural steel section
V_{Ed}	Design value of the shear force acting on the composite section
$V_{pl,Rd}$	Design value of the plastic resistance of the composite section to vertical shear
y	Cross-section axis parallel to the flanges
z	Cross-section axis perpendicular to the flanges; lever arm

Chapter 1

Introduction

1.1 Context

The construction sector, as a major producer of carbon dioxide, is facing several challenges related in particular to sustainability, climate change and environmental issues. The move towards sustainable building products and practices is an inevitable development and there is a growing demand for such products.

The current trend is to improve steel consumption for construction to provide solutions that enable energy-efficient and low-carbon buildings. Efficient steel solutions reduce the environmental impact over the structure's life cycle through steel recycling quality.

The use of steel-concrete composite structures has increased in the construction of high-rise buildings and long-span structures. The two most widely used construction materials, steel and concrete, can have a significant effect on sustainability by providing higher strength solutions. In fact, for composite columns, concrete alloyed with steel reduces the buckling sensitivity and thermally insulates the steel profile in the event of a fire. Moreover, steel increases the plastic rotation capacity of composite elements which provides good seismic behaviour.

The development of higher steel grades, especially, when these are used intelligently in combination with high-strength concrete, is a step forward in improving the durability of construction methods. The advanced high-strength steel is used in many sectors such as bridges, offshore oil rigs and skyscrapers. There are several benefits to using high-strength materials in the construction of composite structures:

- Reduces of the carbon footprint of structures as fewer materials are required;
- Increases the effective net area floors by reducing column sections;
- Reduces costs by reducing the amount of materials employed;
- Produces lighter structures which result in smaller foundations.

European codes and specifications do consider composite structural elements, however, they do not offer specific provisions on the design of composite sections using high-strength materials. Nevertheless, some buildings using them already exists. For example, for the construction of the Latitude building (Fig. 1.1) in Sydney, concrete filled high-strength steel ($f_y = 690$ MPa) box sections were used [1]; for the construction of the 300 Latrobe Street in Melbourne, encased columns with high-strength steel profile ($f_y = 690$ MPa) were used [2].



Figure 1.1: Latitude building in Sydney.

It should be noted that reinforced concrete elements (Fig. 1.2) should not be confused with steel-concrete composite elements (Fig. 1.3). Composite elements are composed of one or more steel profiles and reinforced concrete.

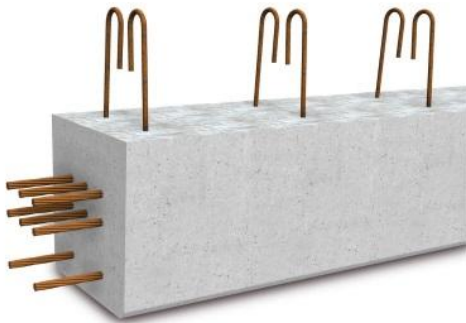


Figure 1.2: Example of a reinforced concrete element [3].

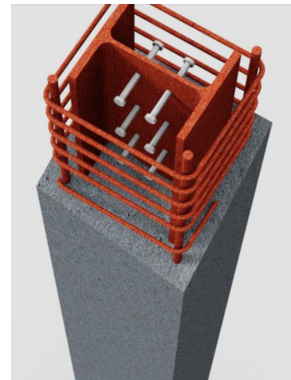


Figure 1.3: Example of a steel-concrete composite element [4].

1.2 Objective

Eurocodes are a set of harmonised technical design rules for structures and structural components, established by the European Committee for Standardisation (CEN). The main objective of harmonised technical specifications for building and civil engineering works is to promote trades across member states and to eliminate technical barriers.

EN 1994-1-1 (Eurocode 4) [5] groups European standards for the design and dimensioning of steel-concrete composite elements and structures, such as beams, slabs and columns. Composite columns can be designed using two methods: a simplified design method and a more sophisticated general design method. The simplified method works for the design of composite columns and composite compression members with concrete-encased sections, partially encased sections and concrete-filled rectangular and circular tubes (Fig. 1.4). This method is based on the full plastic resistance of the cross-section and neglecting the strain gradient.

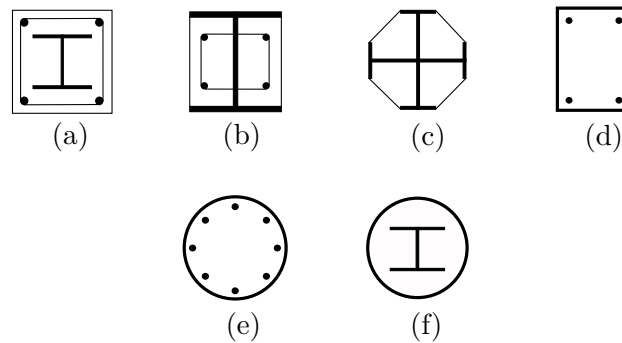


Figure 1.4: Typical cross-section of composite columns: (a) concrete-encased steel; (b) partially concrete-encased steel; (c) partially concrete-encased profiles; (d) concrete-filled rectangular tube; (e) (f) concrete-filled circular tube.

Moreover, materials strength is limited, steel grades must be lower than S460 and concrete strength class must be lower than C50/60. Whereas EN 1992-1-1 [7], *fib* Model Code 2010¹ [8] and EN 1993-1-12 [6] allow the use of higher strength materials until C90/105, C120/140 and S700 respectively. When high-strength steel is used for composite columns, much concern should be given to early concrete crushing: when the yield strain of the steel exceeds the crushing strain of concrete, crushing failure occurs before yielding of the steel (Fig. 1.5). Thus the steel section cannot develop its full plastic strength, which degrades the structural capacity and economy of composite columns. The lack of knowledge on the behaviour of steel-concrete composite columns with high grades, leads to the need for adapting/modifying the simplified method presented in EN 1994-1-1 [5].

¹The *fib* Model Code 2010, developed by the International Federation for Structural Concrete (*fib*), presents new developments and ideas with regard to concrete structures and structural materials, and serves as a basis for future codes.

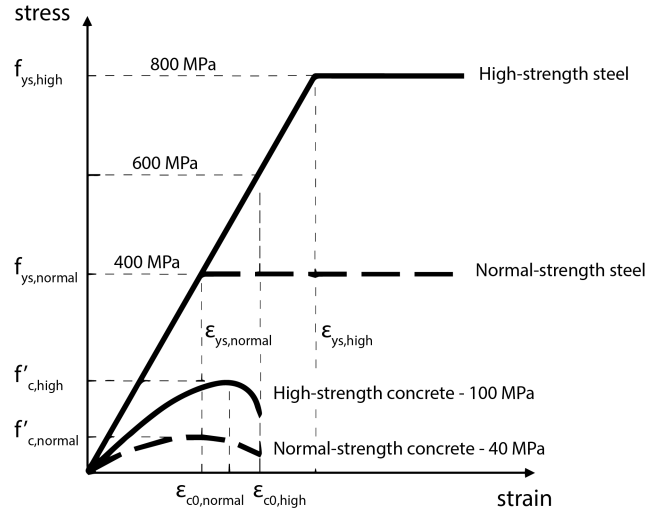


Figure 1.5: Stress-strain diagram for normal- and high-strength concrete and steel.

This Master's thesis aims to adjust the Simplified Method of EN 1994-1-1 for composite column, to allow the use of high-strength materials for the design of fully and partially concrete-encased steel sections to concrete strength class C120 and steel grade S600. Therefore, literature searches have identified various methods for the design of steel-concrete composite column:

1. Simplified Method of EN 1994-1-1 for the design of composite column based on Plastic Design Method using normal-strength materials;
2. The Building and Construction Authority of Singapore [9] developed an extension of the EN 1994-1-1 Simplified Method (BC4:2015) for concrete-filled tubular members (cross-sections (d), (e) and (f) of Fig. 1.4) using high-strength materials: concrete class up to C90 and steel grade up to S550;
3. The design method for composite column of the Australian/New Zealand Standard [10], which is almost similar to EN 1994-1-1 except that it allows the use of high-strength materials: steel grades until S690 and concrete strength class until C100. This method has the particularity of having been tested on real buildings as mentioned in the section 1.1;
4. The RFCS SMARTCoCo project [11] developed a design method based on EN 1994-1-1 Simplified Method and validated it using statistical data obtained from numerical analysis.

In order to achieve this overall goal, the thesis has been divided in a set of different objectives:

- Based on experimental tests in the open literature, create 2D-element models using GiD-SAFIR [12] (a finite element analysis software) and calibrate the models;
- Adaptation/modification of the analytical methods to allow the design of fully and partially concrete-encased steel cross-sections using high strength materials;
- Implement the 4 different methods with a Fiber Method: simplified method of EN 1994-1-1, adapted simplified method of the BC4:2015, simplified method of AS/NZ Standard and modified method based on EN 1992-1-1 from SMARTCoCo project;

- Carry out a sensitivity study by varying the cross-sections, length and materials grades for each method;
- Compared the analytical methods with the numerical analysis based on the $N - M$ interaction curves.

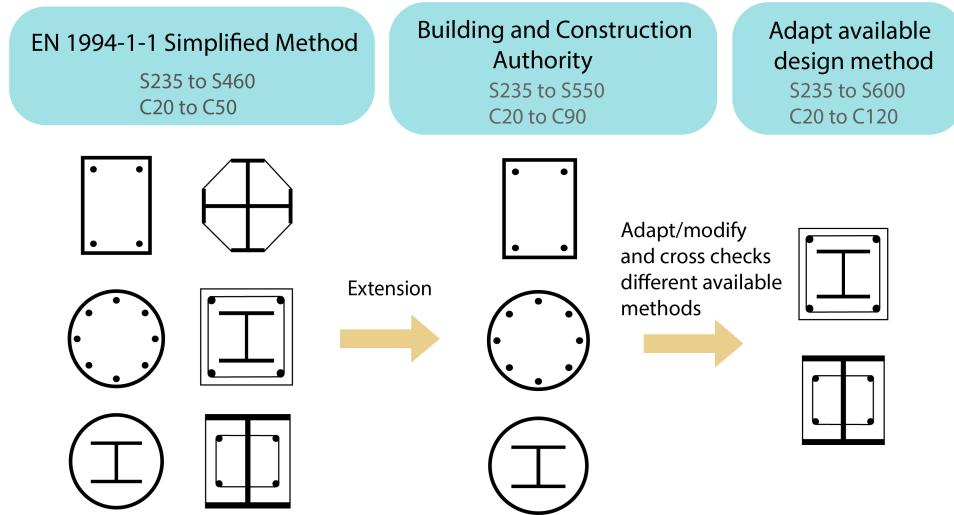


Figure 1.6: Representation of Master thesis overall objective.

1.3 State of the art

This section includes the main experimental tests and numerical simulations concerning columns with fully and partially concrete-encased steel cross-sections using high-strength materials.

1.3.1 High-strength materials

Ricles and Paboojian (1994) [13] studied the behaviour of composite columns subjected to simulated seismic loading conditions. The experimental tests considered concrete-encased steel columns using high-strength concrete of compressive strength $f'_c = 68,9$ MPa and normal-strength steel of yield strength $f_{ys} = 374,4$ MPa. The specimen flexural capacity under combined axial and flexural loading exceeded the American Concrete Institute (ACI) and American Institute of Steel Construction (AISC) provisions for composite column strength, implying that United States codes are conservative in strength prediction.

Kato (1996) [14] investigated buckling strength curves of centrally loaded composite columns based on the available test results. The experimental tests considered concrete-encased steel column with high-strength steel of $f_{ys} = 502$ MPa and normal-strength concrete of $f'_c = 38$ MPa. Kato recommended for concrete-encased steel columns that normal-strength concrete should not be used when high-strength steel is used.

Bergmann et al. (2000) [15] examined the use of high-strength steel in order to check the rules of EN 1994-1-1 based on finite element methods. Experimental static strength investigations were carried out on 32 composite columns using high-strength steel of S460 grades. The concrete strength classes (C20, C45 and C70), the arrangement of mechanical

shear connectors, the longitudinal reinforcement as well as the transverse reinforcement and the loading were varied. The parts of the section remaining elastic may decrease the bearing capacity significantly compared with the full plastic assumption, so that the simplified design methods of EN 1994-1-1 may lead to unsafe results. In order to check this problem a theoretical investigation has been carried out using finite element methods. Good agreement could be observed when the simplified design method was modified by adding additional reduction factors. In order to confirm or to modify these reduction factors, a series of tests with 30 short and 2 slender columns has been carried out.

Oh et al. (2006) [16] studied the flexural strength of a steel-concrete composite column with a non-compact² section that is under a constant axial force, and the results were compared with the evaluations of various design code provisions. The EN 1994-1-1 provisions are considered desirable for use in evaluating the capacity of the axial force-bending moment of steel-concrete composite columns with non-compact steel section. The AISC provisions evaluate the load-carrying capacity of the composite column too conservatively. The experimental tests considered 4 composite columns using high-strength steel ($f_{ys} = 451$ MPa) and normal-strength concrete ($f'_c = 35,1$ MPa).

Prickett and Driver (2006) [17] tested 11 full-scale partially encased columns with high-strength concrete under either concentric or eccentric axial loading to investigate the Canadian steel design standard limitations. In seven concentric tests, concrete type and link spacing were studied. In four eccentric tests, bending axis and various eccentricities were studied. Test results indicate that the columns attain full composite behaviour, but have a more brittle failure mode than those with normal-strength concrete. Smaller link spacing or the addition of steel fibres to the concrete improves the failure mode. The tests used normal-strength tensile steel $f_{ys} = 392$ MPa and high-strength concrete $f'_c = 60$ MPa.

Ellobody et al. (2011) [18] developed a nonlinear 3-D finite element model for eccentrically loaded concrete-encased steel composite columns. The model accounted for the inelastic behaviour of steel, concrete, longitudinal and transverse reinforcement bars as well as the effect of concrete confinement of the concrete encased steel composite columns. The finite element model has been validated against existing test results. The concrete strengths varied from normal to high strength ($f'_c = 30 - 110$ MPa). The steel section yield stresses also varied from normal to high strength ($f_{ys} = 275 - 690$ MPa). The strength of composite columns obtained from the finite element analysis were compared with the design strengths calculated using the EN 1994-1-1 for composite columns. Generally, it is shown that the 1994-1-1 accurately predicted the eccentrically loaded composite columns, while overestimated the moment.

Kim et al. (2012) [19] from the University of Seoul, investigated the eccentric axial load-carrying capacity and the deformation capacity of different concrete-encased steel columns using high-strength steel ($f_{ys} = 800$ MPa) and high-strength concrete ($f'_c = 100$ MPa). The tests parameters were: full or partial concrete-encasement, the eccentricity of the axial load and the effect of lateral reinforcement. The current study focused on the effect of early concrete crushing on the behaviour of composite columns because the yield strain of the high-strength steel is greater than ultimate compressive strain of the concrete. The test results showed that in case of inadequate confinement of the concrete by

²a non-compact section has no longitudinal rebar and has a reduced quantity of steel for economy, different from the partially encased composite column that is widely used in Europe

the lateral reinforcement, the load-carrying capacity was limited by the early crushing of concrete. However, because of the high-strength steel section, the columns showed ductile flexural behaviour after the delamination of the concrete. The results showed that EN 1994-1-1, using full plastic capacity of the steel section, overestimated the experimental tests.

Kim et al. (2014) [20] focused on maximising the contribution of the high-strength steel, preventing early crushing of the concrete. Two of the test specimens were concrete-encased steel columns, using high-strength steel ($f_{ys} = 800$ MPa) and high-strength concrete ($f'_c = 100$ MPa), with high lateral confinement of the concrete. One of those was a slender column to investigate the slenderness effect on the strength and stiffness. The test results showed ductile behaviour, maintaining their strengths even after initial crushing of the concrete due to good lateral confinement. The load-carrying capacity of the slender column was less than the shorter column due to slenderness effect. EN 1994-1-1 unsafely overestimated the test results.

1.3.2 Construction using high-strength materials

High-strength construction materials are not only used in experimental tests. As mentioned in the context, there are several high rise buildings already built using high-performance materials.

The most recent is the Lotte World Tower of 555 m high in Seoul, built in 2017. High-strength steel with a yield strength of 650 MPa is used for the belt trusses, hotel perimeter steel column and parts of the outriggers (Fig. 1.7) [21].

An other example using high-strength materials is the Taipei 101 in Taiwan. This tower of 508 m high built in 2004 is composed of concrete filled steel boxes. These columns were filled with high-performance concrete of 69 MPa. Finally, the outrigger trusses is made of high-performance steel with yield strength up to 510 MPa [22].

To conclude, high-strength materials are already used in real situation for high rise building in Asia and Australia. The European code for the design of composite column needs to be adapted to consider the economic potential offered by the opportunity of constructing lighter/climate friendly buildings with high-performance materials.

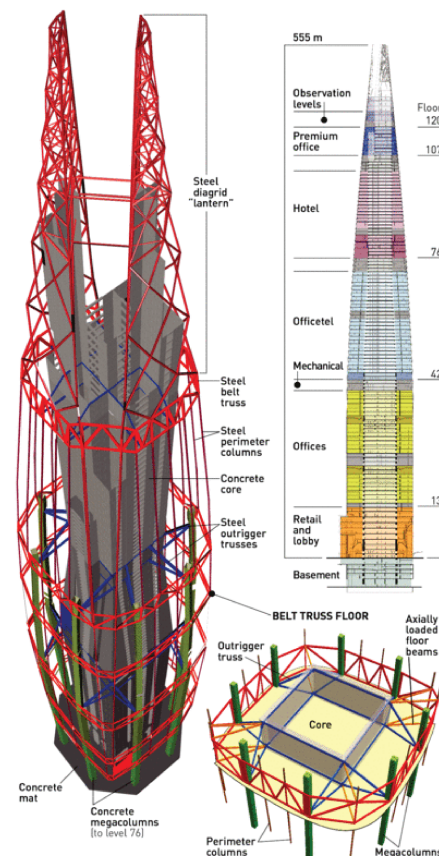


Figure 1.7: Schematic of the belt trusses of the Lotte World Tower in Seoul [23].

Chapter 2

Design methods for steel-concrete composite columns

This chapter describes the four analytical methods considered to adjust the EN 1994-1-1 Simplified Method. Each method will be adapted to allow the use of materials up to C120 for concrete and S600 for steel. Thereafter, a Fiber Method will be implemented for each one of the four methods to calculate the interaction curve for the plastic design of the section, as well as the reduced interaction curve taking into account second-order and long-term effects. An interaction curve gives the envelope of all possible axial force N -bending moment M . The four adapted methods of design will be referred to as follows:

1. Method 1: Simplified Method of EN 1994-1-1;
2. Method 2: BC4:2015 method corresponding to an extension of EN 1994-1-1 Simplified Method;
3. Method 3: AS/NZ Standard design method for composite columns;
4. Method 4: Adapted design method developed by SMARTCoCo project.

2.1 EN 1994-1-1 - Simplified Method of design

Composite columns should be verified for the cross-sectional resistance like compression, bending and shear, for the interaction of actions and for the instability effects. In the verification, the second-order effects, long-term effects and column imperfections should be considered. To perform the verification process, two methods are considered. The general one includes non-symmetrical and non-uniform cross-sections over the column length and the simplified one requires double-symmetrical and uniform cross-sections over the columns length.

This section focuses on the Simplified Method for the design of steel-concrete composite columns of EN 1994-1-1. The objective is to explain the analytical method currently used for the design of a composite column, using normal-strength materials.

2.1.1 General

Initially, composite columns were steel profile encased in low-strength concrete to protect the steel in the event of a fire. However, research studies showed that by using higher concrete strength, significant enhancements in the column strength were possible, leading

to the use of smaller steel sections. Nowadays, concrete-filled hollow sections are more often used than steel encased concrete sections because construction speed is considerably improved by using the steel tube as formwork. Moreover, protecting steel against fire is in many cases not necessary and the steel can be exposed to achieve attractive appearance. In order to continue to use universal sections in composite columns, partial encasement can have the same benefit of not using formwork, as concrete is cast between the flanges of the steel section. Hence the steel web is sufficiently thermally insulated in the event of a fire [24].

As explained in section 1.2, the Simplified Method of European design code for composite steel and concrete structures considers columns with concrete encased sections, partially encased sections and concrete filled rectangular and circular tubes. These design rules apply to columns and compression members with steel grade S235 to S460 and normal weight concrete of strength classes C20/25 to C50/60 [5].

Columns are considered as composite columns if the condition of the steel contribution ratio δ is fulfilled. This coefficient is defined by the ratio of the steel structural plastic strength to the concrete plastic strength of the section:

$$\delta = \frac{A_a f_{yd}}{N_{pl,Rd}} \quad (2.1)$$

The limit values are defined as follows: $0,2 \leq \delta \leq 0,9$. For $\delta < 0,2$, the steel contribution is low enough for columns to be considered as a reinforced concrete column. For $\delta > 0,9$, the column is considered as a steel column.

When steel is subject to compression, local buckling of a steel profile can occur, which degrades the capacity and economy of columns. To avoid local buckling of steel sections and in order for steel to reach its yield stress, steel column sections should conform to Class 3 or better. In composite columns, the steel plates should be Class 2 to prevent local buckling in the column itself because, for normal-strength materials, yield strain of steel is lower than concrete strain at peak stress. So steel will yield before concrete reaches its peak stress. For the concrete to reach its peak compression stress, the steel plate will have to undergo further strain without local buckling. Hence, requirements to control local buckling are more restrictive in a composite column than in a bare steel column. Nevertheless, concrete provides substantial restraint to prevent local buckling of steel in a composite cross-section therefore, as described in Table 2.1, the allowable ratio of steel width to thickness is higher than in a bare steel cross-section [24].

EN 1994-1-1 is only applicable if local buckling of the steel plates is prevented. The effects of local buckling may be neglected for a fully encased steel section if the minimal concrete covering is 40 mm or 1/6 of the flange width. For other types, the limit values to neglect these effects are given in Table 2.1.

Cross-section	Composite limit values	Steel limit values
Circular hollow steel sections	$\frac{d}{t} \leq 90 \sqrt{\frac{235}{f_y}}$	$\frac{d}{t} \leq 90 \sqrt{\frac{235}{f_y}}$
Rectangular hollow steel sections	$\frac{h}{t} \leq 52 \sqrt{\frac{235}{f_y}}$	$\frac{h}{t} \leq 42 \sqrt{\frac{235}{f_y}}$
Partially encased I-sections	$\frac{b}{t_f} \leq 44 \sqrt{\frac{235}{f_y}}$	$\frac{b}{t_f} \leq 30 \sqrt{\frac{235}{f_y}}$

Table 2.1: Limits of steel width to thickness ratio to avoid local buckling according to Table 6.3 of EN 1994-1-1 and Table 5.2 of EN 1993-1-1 [25].

The Simplified Method for the design of composite steel-concrete columns is limited to members of doubly symmetrical and uniform cross-section over the member length with rolled, cold-formed or welded steel sections. Other conditions must be met in order to be designed according to the Simplified Method:

- The relative slenderness $\bar{\lambda}$ should fulfil the following condition: $\bar{\lambda} \leq 2$,
where $\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}}$.
- For fully encased steel section, limits to the maximum thickness of concrete cover are: $c_z \leq 0,3 h$ and $c_y \leq 0,4 b$.
- The longitudinal reinforcement that may be used in calculation should not exceed 6% of the concrete area.
- The ratio of the depth to the width of the composite cross-section should be within the limits 0,2 and 5.

Any column which does not fulfil these conditions needs to be designed according to the general design method [5].

2.1.2 Resistance of cross-section to pure compression

The plastic resistance of a composite column cross-section to compression should be calculated by adding the plastic resistance of its components:

$$N_{pl,Rd} = A_a f_{yd} + \alpha A_c f_{cd} + A_s f_{sd} \quad (2.2)$$

$$\text{where } \alpha = \begin{cases} 1 & \text{for concrete filled sections;} \\ 0,85 & \text{for concrete encased and partially concrete encased steel sections.} \end{cases}$$

The α coefficient takes into account concrete deterioration due to environmental exposure and splitting. For concrete-filled steel sections where concrete is protected, its cylinder strength can be fully developed.

For circular concrete-filled steel tubular cross-section, the increase of concrete strength due to confinement effect may be permitted, as long as this effect is non-uniform and small in the case of rectangular sections and may be safely ignored. The confinement effect is due to the drastically increase of Poisson's coefficient when concrete approaches plastic failure. This lateral dilatation is restrained by high stiffness of the steel hollow section, which leads to an increase in the compressive strength of concrete beyond its normal peak stress [24]. When steel is restraining concrete, a tensile circumferential stress is produced in the steel tube, leading to a reduction in steel strength in the longitudinal direction; this is taken into account by the coefficient η_a in equation 2.4.

In the case of long columns or columns under large bending moments, concrete failure strain is small and its Poisson's coefficient is similar to that of steel. Consequently steel does not confine concrete because lateral expansion of steel and concrete are similar under longitudinal compression, this is taken into account by the coefficients η_a and η_c in equation 2.4.

EN 1994-1-1 Simplified Method allows this increase of concrete strength if the following conditions are fulfilled:

- The relative slenderness $\bar{\lambda}$ does not exceed 0,5;
- The eccentricity of loading e does not exceed $d/10$:

$$e = \frac{M_{Ed}}{N_{Ed}} \leq 0,1 d \quad (2.3)$$

In this case, the plastic resistance of the cross section $N_{pl,Rd}$ should be calculated with the following expression:

$$N_{pl,Rd} = \eta_a A_a f_{yd} + A_c f_{cd} \left(1 + \eta_c \frac{t}{d} \frac{f_y}{f_{ck}} \right) + A_s f_{sd} \quad (2.4)$$

For members in pure compression ($e = 0$):

$$\eta_a = \eta_{a0} = 0,25(3 + 2\bar{\lambda}) \leq 1 \quad (2.5)$$

$$\eta_c = \eta_{c0} = 4,9 - 18,5\bar{\lambda} + 17\bar{\lambda}^2 \quad (2.6)$$

For members in combined compression and bending with $0 < e/d \leq 0,1$:

$$\eta_a = \eta_{a0} + (1 - \eta_{a0})(10 e/d) \quad (2.7)$$

$$\eta_c = \eta_{c0}(1 - 10 e/d) \quad (2.8)$$

For members in combined compression and bending with $e/d > 0,1$: $\eta_a = 1$ et $\eta_c = 0$.

2.1.3 Buckling resistance of members to axial compression

For slender columns, instabilities will occur before the composite cross-section reaches its plastic resistance of the cross-section $N_{pl,Rd}$. In the case of a perfect column with ideal pin supports at the ends, the column buckling load is given by the Euler load:

$$N_{cr} = \frac{\pi^2 (EI)_{eff}}{L^2} \quad (2.9)$$

where $(EI)_{eff}$ is the effective flexural rigidity of the cross-section and is given by:

$$(EI)_{eff} = E_a I_a + 0,6 E_{c,eff} I_c + E_s I_s \quad (2.10)$$

Due to initial imperfections in realistic columns, second order bending moments are generated. These bending moments give rise to strain gradients which are amplified by the influence of creep and shrinkage. These lead to an increase in bending and earlier failure of the composite column. To take into account the effects of creep and shrinkage, $E_{c,eff}$ is defined as:

$$E_{c,eff} = \frac{E_{cm}}{1 + \frac{\varphi_t N_{G,Ed}}{N_{Ed}}} \quad (2.11)$$

As the columns slenderness depends on the plastic resistance of the cross-section $N_{pl,Rd}$, in order to avoid iterations, concrete confinement should be ignored when calculating the plastic resistance of the column. Verification of buckling resistance of steel-concrete composite columns under axial compression is similar as one in EN 1993-1-1 for steel columns. The relative slenderness $\bar{\lambda}$ is given by:

$$\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \quad (2.12)$$

where $N_{pl,Rk}$ is equal to $N_{pl,Rd}$ excepted that material partial safety factors are not included.

When the relative slenderness is known, the verification of the column buckling resistance is defined by:

$$\frac{N_{Ed}}{\chi N_{pl,Rd}} \leq 1 \quad (2.13)$$

where χ is the column strength reduction factor for the relevant buckling mode gives in EN 1993-1-1 and is a function of the column slenderness $\bar{\lambda}$ (Fig. 2.1). The relevant buckling curves for cross-sections of composite columns are given in Table 2.2, where ρ_s is the reinforcement ratio A_{sl}/A_c [26].

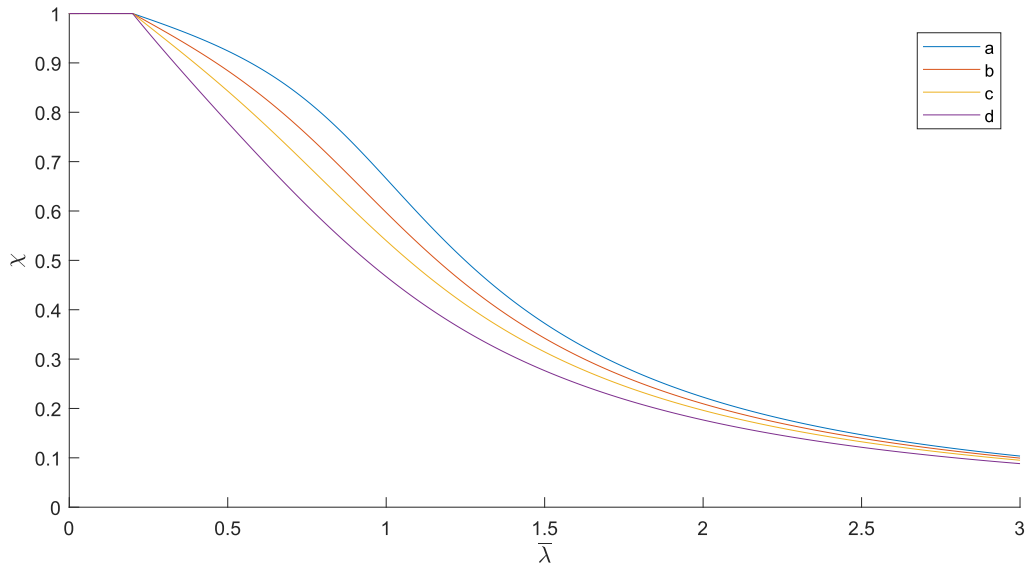


Figure 2.1: Buckling curves [25].

Cross-sections	Limits	Axis of buckling	Buckling curve	Member imperfection
		y-y	b	L/200
		z-z	c	L/150
		y-y	b	L/200
		z-z	c	L/150
	$\rho_s \leq 3\%$		a	L/300
	$3\% \leq \rho_s \leq 6\%$		b	L/200
		y-y	b	L/200
		z-z	b	L/200
			b	L/200

Table 2.2: Buckling curves and member imperfections for composite columns which corresponds to Table 6.5 of the EN 1994-1-1. y-y buckling axis corresponds to strong axis of the section and z-z buckling axis to weak axis of the section.

2.1.4 Resistance of cross-section to combined compression and bending moment

The resistance of a cross-section under combined compression and bending moment may be described by the corresponding interaction diagram. This diagram gives the failure conditions of the composite cross-section.

For steel columns, when axial compression is added to bending, the moment of resistance decreases due to second-order effect. Unlike steel-concrete composite columns, the moment of resistance could be increased by compression. This phenomenon can be noticed by the "belly" of the interaction curve (Fig. 2.3), this is due to the pre-stressing of concrete which increases the compression area, therefore decreasing the cracked concrete area [26].

The variation of the neutral axis position as well as the calculation of the internal forces and the plastic stress distribution at each point makes it possible to determine the interaction curve. This method is exclusively feasible with numerical calculations. As a simplification, the interaction curve may be replaced by a polygonal diagram (Fig. 2.3) and the representation is based on the calculation of some specific situations. When evaluating these key points, rigid-plastic material behaviour is assumed. Thereby, steel is assumed to have reached yield stress in either tension or compression and concrete is assumed to have reached its peak stress in compression.

Neutral axis

The position of the neutral axis changes according to the stress distribution, the elastic neutral axis will not be at the same position as the plastic neutral axis. This axis corresponds to the position where there are no stresses or strains. Consider a section subjected to an

eccentric axial force N_{Ed} (Fig. 2.2), if N_{Ed} increases then the stress distribution will evolve from (a) to (d):

- (a) is the elastic state;
- from (b) to (c), plasticization spreads until it reaches an elasto-plastic state;
- (d) represent the plastic state where the entire cross-section has reached its yield strength.

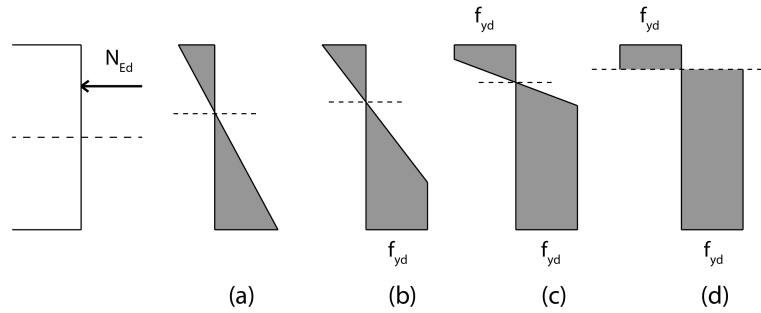


Figure 2.2: Representation of the evolution of the stress distribution in a cross-section subjected to an eccentric axial load whose value increases.

Interaction curve for plastic stress blocks

The resistance of a cross-section to combined compression and bending moment and the corresponding interaction curve may be calculated assuming rectangular stress blocks. This plastic distribution takes into account the design shear force V_{Ed} according to Fig. 2.7. The tensile strength of concrete should be neglected. The minimum approach is the *ABC* curve which is often sufficient (see Fig. 2.3). A fourth point *D* can be added to this polynomial diagram to take into account the pre-stressing effect of concrete due to axial compression.

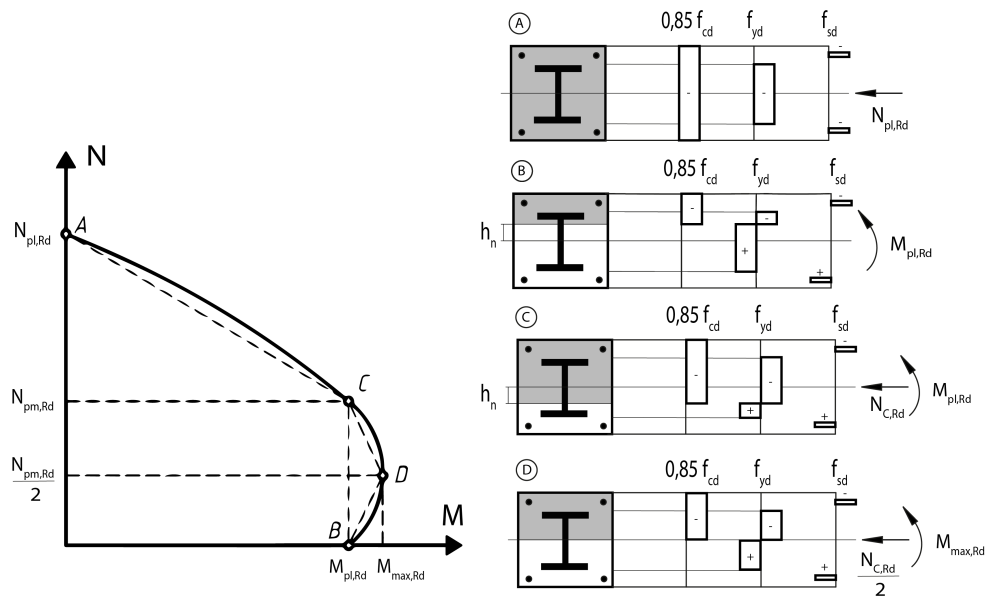


Figure 2.3: Interaction curve for fully concrete encased steel section [5] (grey area represents the compressed area and white area represents the area in tension).

The simplified interaction curve is determined as follows:

- **Point A:** Describes a cross-section under pure compression. The resistance value is $N_{pl,Rd}$.
- **Point B:** Describes a cross-section under pure bending. The resistance is the plastic resistant moment $M_{B,Rd} = M_{pl,Rd}$.
- **Point C:** The neutral axis is symmetrical to that corresponding point B, with respect to the mid-height of the section. The compression stresses did not produce any bending moment therefore $M_{C,Rd} = M_{B,Rd} = M_{pl,Rd}$. This bending moment can be mobilised in the presence of an axial compression load equal to the load-bearing capacity of the concrete section: $N_{C,Rd} = A_c 0,85 f_{cd}$. This can be explained by summing the results of the constraints relating to points B and C, by reversing the direction of the moment $M_{B,Rd}$ (Fig. 2.4).

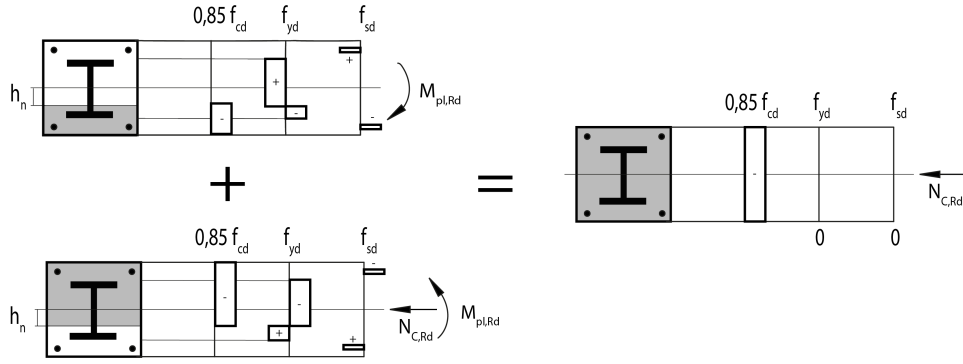


Figure 2.4: Sum of points B and C stresses distribution, by reversing the direction of the moment $M_{B,Rd}$.

- **Point D:** The neutral axis corresponds to the center of the section. Stresses in the steel profile and reinforcements do not contribute to axial force. Half of the concrete section is compressed, which leads to:

$$N_{D,Rd} = \frac{1}{2} A_c 0,85 f_{cd} \quad (2.14)$$

All stresses give a positive contribution to the bending moment:

$$M_{D,Rd} = M_{max,Rd} = 0,5 W_{pl,c} 0,85 f_{cd} + W_{pl,a} f_{yd} + W_{pl,s} f_{sd} \quad (2.15)$$

The neutral axis for points B and C depends on the distance h_n and can be determined by the difference between the stresses distribution of points B and C (see Fig. 2.5). There are different possible positions for this axis, which are detailed in Appendix A. If the axis is in the web of the steel profile, h_n should be derived by the following equations:

$$2 h_n (b - t_w) 0,85 f_{cd} + 2 h_n t_w 2 f_{yd} = A_c 0,85 f_{cd} \quad (2.16)$$

which allows to determine h_n :

$$h_n = \frac{A_c 0,85 f_{cd}}{2(b - t_w) 0,85 f_{cd} + 2 t_w 2 f_{yd}} \quad (2.17)$$

where the different parameters are shown in Fig. 2.6.

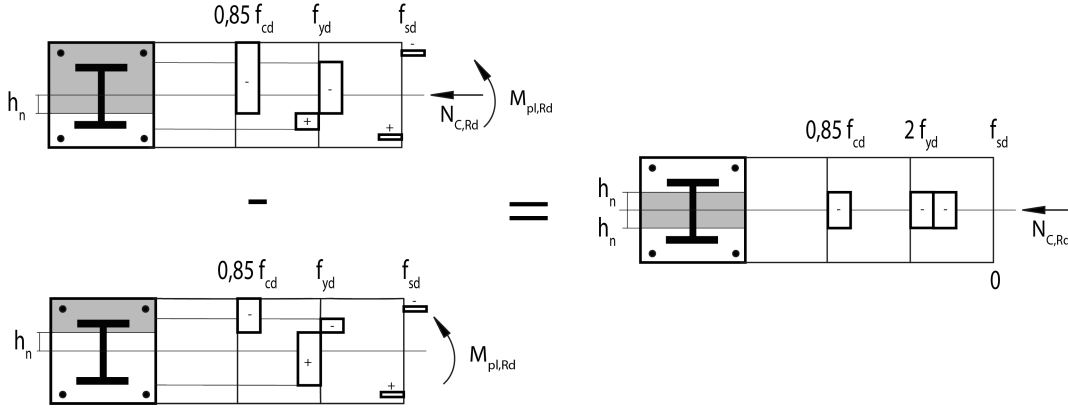


Figure 2.5: Stresses distribution difference between point C and B.

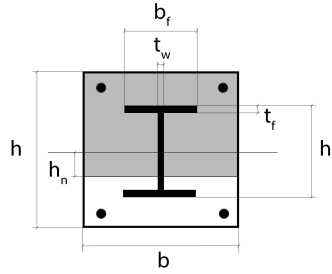


Figure 2.6: Representation of the different parameters of the section.

The influence of transverse shear forces on the resistance to combine bending and axial compression should be considered when determining the interaction curve if: $V_{a,Ed} > 0,5V_{pl,a,Rd}$. In this case, a reduced design steel strength $(1 - \rho)f_{yd}$ is considered in the shear area A_v in accordance to Fig. 2.7, ρ is defined in Eq. 2.20. Otherwise the influence of transverse shear forces could be neglected. Unless a more accurate analysis is used, V_{Ed} may be distributed into $V_{a,Ed}$ acting on the structural steel and $V_{c,Ed}$ acting on the reinforced concrete section by:

$$V_{a,Ed} = V_{Ed} \frac{M_{pl,a,Rd}}{M_{pl,Rd}} \quad (2.18)$$

$$V_{c,Ed} = V_{Ed} V_{a,Ed} \quad (2.19)$$

$$\rho = \left(2 \frac{V_{Ed}}{V_{Rd}} - 1\right)^2 \quad (2.20)$$

For simplification V_{Ed} may be assumed to act on the structural steel section alone.

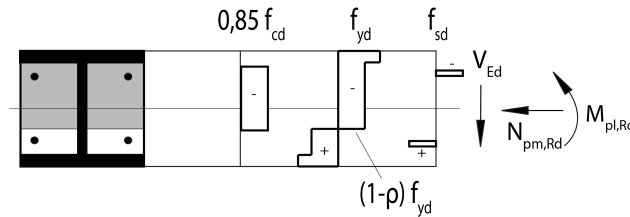


Figure 2.7: Reduced design steel strength due to transverse shear forces [5].

2.1.5 Buckling resistance of members to combined axial compression and bending moment

The analysis should be based on second-order linear elastic analysis for columns with member imperfections depicted as an initial deformation. The amplitude e_0 of the initial deformation due to the member imperfection is determined such that the analysis gives the same load-bearing capacity as the method in section 2.1.3 based on the use of the buckling curves [26].

Second-order effects need to be considered when $\alpha_{cr} = \frac{N_{cr}}{N_{Ed}} < 10$ [25] and the elastic critical load N_{cr} is determined with the flexural stiffness $(EI)_{eff,II}$ in accordance with Eq. 2.24 [5].

Uniform bending moment distribution

Consider a steel-concrete composite column with a member imperfection and loaded by an eccentric axial load N_{Ed} . The member imperfection is considered as a sinusoidal curve with an amplitude e_0 . The effort N_{Ed} is applied with an eccentricity e . The column is subject to combined compression N_{Ed} and bending moment $M_{Ed} = eN_{Ed}$ added to bending moment from imperfection $M_{imp} = N_{Ed}e_0$ (Fig. 2.8).

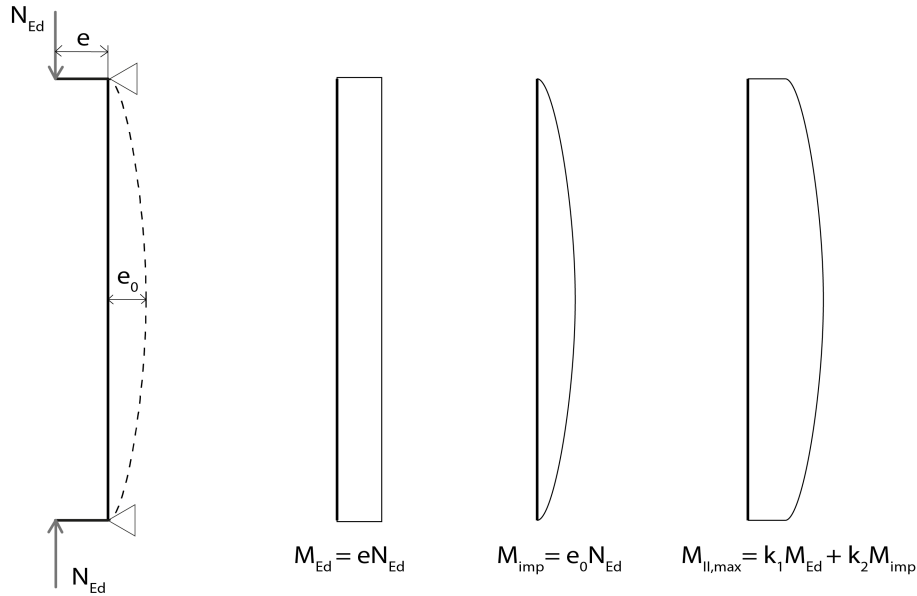


Figure 2.8: Representation of the bending moments diagram, respectively, for: primary bending moments due to the end moments, primary bending moments due to the initial geometric imperfection and superposition of the amplified bending moments due to the second-order effects.

This primary configuration of bending moments is increased by second-order effects to provide a distribution of amplified bending moments, $M_{Ed,II}$ and $M_{imp,II}$. The superposition of the 2 types of primary bending moments gives the maximum primary bending moment M_{max} which makes it possible to determine the maximum second-order bending moment $M_{II,max}$:

$$M_{II,max} = k M_{max} \quad (2.21)$$

where k is the amplification coefficient related to the primary distribution moment, characterised by a coefficient β (see Fig. 2.9).

$$M_{II,max} = k_1 M_{Ed} + k_2 M_{imp} \quad (2.22)$$

The factors k_i are given by:

$$k_i = \frac{\beta_i}{1 - \frac{N_{Ed}}{N_{cr,eff}}} \quad (2.23)$$

where $N_{cr,eff}$ is the critical normal force for the relevant axis and corresponding to the effective flexural stiffness $(EI)_{eff,II}$ given by:

$$(EI)_{eff,II} = 0,9(E_a I_a + 0,5E_{c,eff} I_c + E_s I_s) \quad (2.24)$$

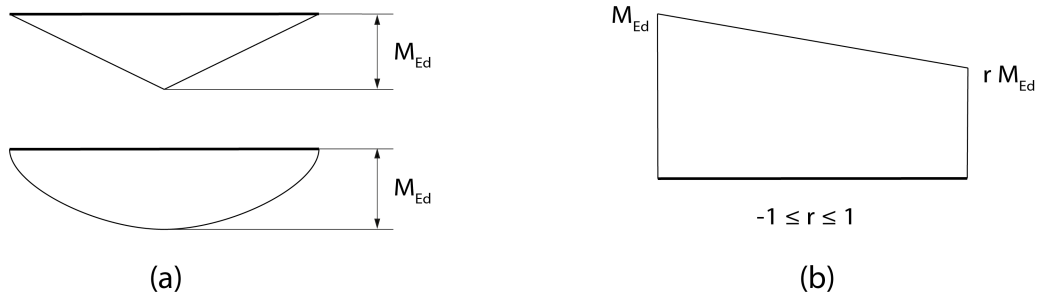


Figure 2.9: Equivalent moment factor β , which corresponds to Table 6.4 of EN 1994-1-1. (a) First-order bending moment from member imperfection or lateral load: $\beta = 1$. (b) End moments: $\beta = \max(0,44; 0,66 + 0,44 r)$.

Based on the interaction diagram of the most highly stressed cross-section (see Fig. 2.10), the resistance cross-section verification should satisfy:

$$\frac{M_{II,max}}{M_{pl,N,Rd}} = \frac{M_{II,max}}{\mu M_{pl,Rd}} \leq \alpha_M \quad (2.25)$$

with:

$$\alpha_M = \begin{cases} 0,9 & \text{for steel grade from S235 to S355} \\ 0,8 & \text{for steel grade from S420 to S460} \end{cases} \quad (2.26)$$

The α_M values were determined from a sensitive study of numerical simulations results from experimental tests values to protect against the following simplifications:

- The interaction diagram is based on a rectangular distribution of stresses, therefore it assumes yielding of materials;
- M_{Ed} is determined based on effective flexural stiffness, to which the effects of concrete cracking are taken into account on a flat-rate basis.

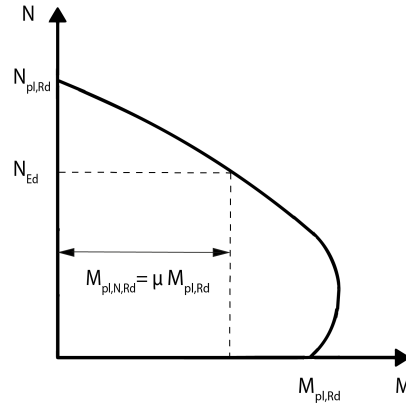


Figure 2.10: Interaction curve for combined compression and bending in the critical cross-section [5].

Non-uniform bending moment distribution

Consider a steel-concrete composite column with a member imperfection as in the previous situation. This column is subject to combined axial load N_{Ed} and unequal end bending moments, M_{Ed} and $r M_{Ed}$, where factor r is the end moments ratio.

The primary bending moment due to initial imperfection is not represented in Fig. 2.11 because it is the same as Fig. 2.8. Since the end bending moments are unequal, the element deformation is S-shaped. Therefore, the axial load applied to the deformed member provides additional S-shaped bending moments distribution due to second-order effects.

Compared to the uniform bending moment distribution, the amplified bending moment is lower in this situation. The bending moment due to the initial imperfection is maximum at mid-height of the column but the section, where the maximum second-order bending moment is applied, cannot be determined. Nevertheless, the method beginning at Eq. 2.22 is still used.

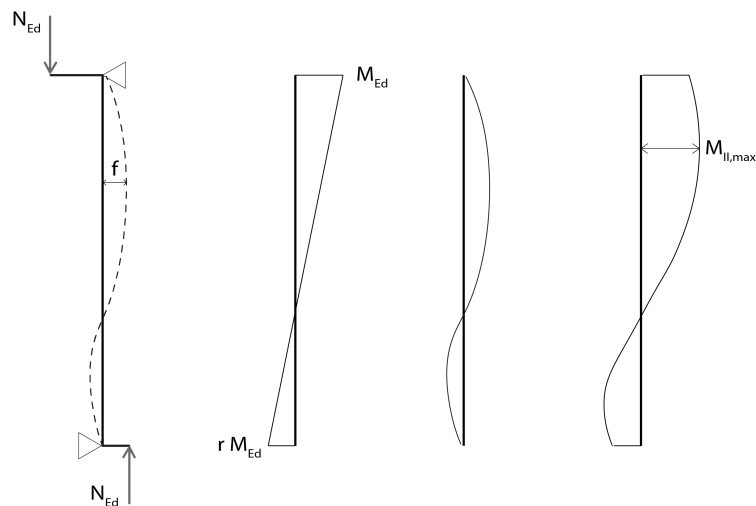


Figure 2.11: Representation of moments bending diagram, respectively, for: primary bending moments due to end bending moments, second-order effects due to end bending moments and superposition of the amplified bending moments due to end bending moments.

2.1.6 Buckling resistance of members to combined axial compression and bi-axial bending moment

The composite column capacity is determined separately for the buckling modes along y and z axis. From these two $N - M$ interaction curves, the bending moment capacities of the composite column under axial compression are separately obtained along each axis, as in section 2.1.5. It is assumed that the second-order bending moment arising from axial compression acting on imperfection is only effective in the expected buckling axis of the column. The following conditions should be satisfied:

$$\frac{M_{y,Ed}}{\mu_y M_{pl,y,Rd}} \leq \alpha_{M,y} \quad (2.27)$$

$$\frac{M_{z,Ed}}{\mu_z M_{pl,z,Rd}} \leq \alpha_{M,z} \quad (2.28)$$

The combined compression and bi-axial bending moment resistance verification is given by:

$$\frac{M_{y,Ed}}{\mu_y M_{pl,y,Rd}} + \frac{M_{z,Ed}}{\mu_z M_{pl,z,Rd}} \leq 1 \quad (2.29)$$

where μ_z and μ_y are determined in section 2.1.5 along each axis.

2.1.7 Method 1 - Adapted Simplified Method of EN 1994-1-1 for high-strength materials

The Simplified Method of design is adapted by changing the authorized material grades. The compressive strength class of concrete is extended to C120 and steel grade to S600. The plastic interaction curve will be determined by varying the neutral axis position and, for each neutral axis position, the calculation of the internal forces using plastic stress distribution will allow to determine point by point the interaction curve.

Afterwards, the reduced interaction curve can be calculated by taking into account buckling effects, which are translated by a decrease in the resistant axial force N_{Rd} and resistant bending moment M_{Rd} (see Fig. 2.12).

The reduced N_{Rd} is named $N_{b,Rd}$ and is obtained by multiplying the first-order resistant axial force with coefficient χ , which is defined as follows:

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \leq 1 \quad (2.30)$$

where coefficient $\bar{\lambda}$ is defined in section 2.1.3 and $\phi = 0,5(1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2)$. The coefficient $\bar{\lambda}$ depends, for second-order and long-term effects, on the effective flexural rigidity:

$$(EI)_{eff,II} = K_0(E_a I_a + K_{e,II} E_{c,eff} I_c + E_s I_s) \quad (2.31)$$

with $K_0 = 0,9$ and $K_{e,II} = 0,5$.

The reduced M_{Rd} is named $M_{b,Rd}$ and is calculated by multiplying the first-order bending moment by a magnification factor k , defined by:

$$k = \frac{\beta}{1 - \frac{N_{Ed}}{N_{cr,eff}}} \geq 1 \quad (2.32)$$

where $N_{cr,eff}$ depends on the effective flexural rigidity.

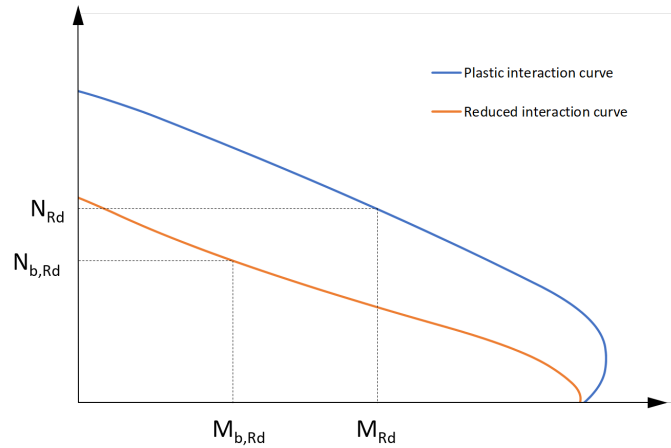


Figure 2.12: Representation of the plastic and reduced interaction curves.

2.2 BC4:2015 - Extension of EN 1994-1-1

2.2.1 General

This design guide [9], established by the Building and Construction Authority of Singapore, is an extension of EN 1994-1-1 for concrete-filled tubular members using high-strength materials. This is an adapted method of the Simplified Method for steel-concrete composite column considering concrete cylinder strength up to 90 MPa and steel of yield strength up to 550 MPa. More than 2000 test data collected from the literature on concrete-filled steel tubes with normal and high-strength materials were analyzed to formulate this design guide. The types of cross-section are given in Fig. 2.13; these are doubly symmetrical and uniform over the member length. The main differences in this extension are the materials properties, that change in the case of high-performance materials and a rule for the compatibility of materials.

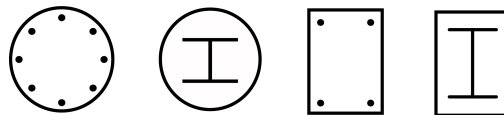


Figure 2.13: Types of concrete filled tubular cross-sections.

2.2.2 Materials

High-strength materials allow the construction of high-rise buildings due to smaller construction elements compared to normal-strength materials. High-strength and low-weight are beneficial in seismic design with seismic response being reduced by the low-weight of a structure.

Concrete

High-strength concrete is defined as concrete with cylinder compressive strength f_{ck} greater than 50 MPa. High-strength concrete is more brittle than normal-strength concrete and may not exhibit load displacement behaviour after the compression peak when it is not reinforced, as shown in Fig. 2.14, C80 class concrete does not exhibit any yielding plateau compared to C30 class concrete.

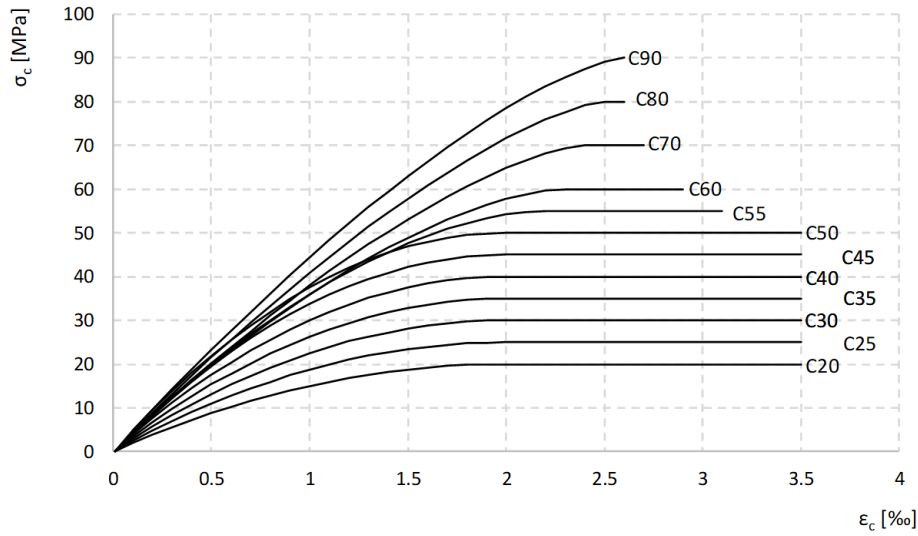


Figure 2.14: Parabola-rectangle stress-strain relations for different concrete strength classes according to EN 1992-1-1.

In this case, to derive the rectangular stress block, used in the Simplified Method, from parabolic-rectangle stress distribution, the characteristic cylinder strength should be multiplied by a reduction factor η , according to Fig. 2.15 defined by:

$$\eta = \begin{cases} 1 - (f_{ck} - 50)/200 & 50 \text{ MPa} < f_{ck} \leq 90 \text{ MPa} \\ 0,8 & f_{ck} > 90 \text{ MPa} \end{cases} \quad (2.33)$$

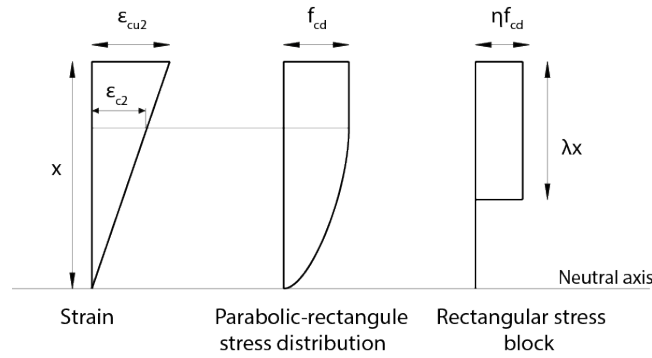


Figure 2.15: Derivation of rectangular stress block from parabolic-rectangle stress distribution relative to the neutral axis for high-strength concrete. Coefficients ϵ_{c2} and ϵ_{cu2} refer to the parabolic-rectangle relation defined in EN 1992-1-1 and correspond respectively to the maximum strain in pure compression and in combined bending and compression.

The factor λ in Fig. 2.15, defines the height reduction of concrete strength and is equal to 1 in this design guide. Whereas EN 1992-1-1 defined λ as follows:

$$\lambda = \begin{cases} 0,8 & f_{ck} \leq 50 \text{ MPa} \\ 0,8 - (f_{ck} - 50)/400 & 50 \text{ MPa} < f_{ck} \leq 90 \text{ MPa} \end{cases} \quad (2.34)$$

This extension of EN 1994-1-1 considers a reduction of the cylinder compressive strength f_{ck} and modulus of elasticity E_{cm} for cylinder compressive strength f_{ck} higher than 50 MPa with a coefficient η according to:

$$f'_{ck} = \eta f_{ck} \quad (2.35)$$

$$E'_{cm} = 22((\eta f_{ck} + 8)/10)^{0,3} \quad (2.36)$$

Table 2.3 presents the cylinder compressive strength f_{ck} and modulus of elasticity E_{cm} reduced and unreduced. It can be observed that the maximum reduction for the cylinder compressive strength is 20% and for the modulus of elasticity is 5,7%.

Strength classes	C55	C60	C70	C80	C90
f_{ck} [MPa]	55	60	70	80	90
f'_{ck} [MPa]	53,6	57	63	68	72
E_{cm} [GPa]	38,2	39,1	40,7	42,2	43,6
E'_{cm} [GPa]	38	38,6	39,6	40,4	41,1

Table 2.3: Reduced cylinder compressive strength and modulus of elasticity for higher strength concrete according to Table 2.2 of BC4:2015.

Structural steel

High-strength steel is defined as steel with yield strength f_y higher than 460 MPa. The limiting values of ratio f_u/f_y , elongation at failure, and the ultimate strain ϵ_u are recommended in Table 2.4.

Steel	Ratio f_u/f_y	Elongation at failure	Ultimate strain ϵ_u
≤ 460 MPa	$\geq 1,1$	15%	$\geq 15 \epsilon_y$
> 460 MPa	$\geq 1,05$	10%	$\geq 15 \epsilon_y$

Table 2.4: Limitation on ductility, elongation at failure and ultimate strain of steel according to Table 2.4 of BC4:2015.

According to EN 1993-1-1, the elastic modulus and the Poisson's ratio are taken as 210 GPa and 0,3 respectively. From Hooke's law, the elastic strain ϵ_y should be determined as follows:

$$\epsilon_y = f_y/E \quad (2.37)$$

Reinforcing steel

The yield strength of reinforcing steel should be limited from 400 to 600 MPa and the Young modulus should be taken as 210 GPa, according to EN 1992-1-1. Different classes of ductility are allowed as shown in Table 2.5.

Class	f_{yk} [MPa]	Ratio f_{uk}/f_{yk}	Ultimate elongation
B500A	500	1,05	2,5%
B500B	500	1,08	5%
B500C	500	$\leq 1,05$ and $< 1,35$	7,5%

Table 2.5: Ductility classes of reinforcing steel according to Table C.1 of EN 1992-1-1.

2.2.3 Material compatibility

The material compatibility is a main criterion to prevent early crushing of concrete before yielding of the steel. Otherwise the fully plastic resistance of the cross-section cannot be guaranteed due to brittle failure of the high-strength concrete. For structural steel and concrete used in the concrete-filled tubular members, the material compatibility recommended by this design guide should be conformed to Table 2.6. The maximum steel strength should be determined in accordance with the concrete characteristic cylinder strength with class up to C90/105 using the following expression:

$$f_y \leq 0,7 E_a (f_{ck} + 8)^{0,31} \quad (2.38)$$

The objective of this rule is to extend the material strength limits while keeping a sustainable and economical design. This compatibility between steel and concrete limits the use of certain steel grades with certain concrete strength classes. For example, according to Table 2.6, this design guide does not permit the use of steel grade S460 with concrete strength class C30/37 whereas EN 1994-1-1 does not have any restrictions in this regard.

	S235	S355	S420	S460	S500	S550	S600	S690
C12/15	✓	✓	✗	✗	✗	✗	✗	✗
C16/20	✓	✓	✗	✗	✗	✗	✗	✗
C20/25	✓	✓	✗	✗	✗	✗	✗	✗
C25/30	✓	✓	✓	✗	✗	✗	✗	✗
C30/37	✓	✓	✓	✗	✗	✗	✗	✗
C35/45	✓	✓	✓	✓	✗	✗	✗	✗
C40/50	✓	✓	✓	✓	✗	✗	✗	✗
C45/55	✓	✓	✓	✓	✓	✗	✗	✗
C50/60	✓	✓	✓	✓	✓	✗	✗	✗
C55/67	✓	✓	✓	✓	✓	✗	✗	✗
C60/75	✓	✓	✓	✓	✓	✗	✗	✗
C70/85	✓	✓	✓	✓	✓	✓	✗	✗
C80/95	✓	✓	✓	✓	✓	✓	✗	✗
C90/105	✓	✓	✓	✓	✓	✓	✓	✗
C100/115	✓	✓	✓	✓	✓	✓	✓	✗
C110/130	✓	✓	✓	✓	✓	✓	✓	✗
C120/140	✓	✓	✓	✓	✓	✓	✓	✗

Table 2.6: Compatibility between steel and concrete, based on Eq. 2.38, for a composite column. Rows refer to the strength concrete classes and columns refer to the steel grades. The arrows indicate the extension of the table based on the Eq. 2.38 for materials of higher grades.

2.2.4 Method 2 - Adapted Simplified Method of BC4:2015

Based on this design guide, which extends the Simplified Method of EN 1994-1-1 [9], for concrete-filled tubular members using high strength materials and defines a compatibility rule between steel and concrete to prevent early concrete crushing occurs before yielding of the steel. It would be possible to modify this method for fully and partially concrete-encased steel sections.

According to Kim *et al.* [20], for concrete filled steel tube columns using high-strength materials ($f'_c = 100$ MPa and $f_{ys} = 800$ MPa), the axial load-flexural moment interaction relationship of EN 1994-1-1 using plastic stress distribution conform with the tests. However, for concrete-encased steel columns, EN 1994-1-1 unsafely overestimated the test results, as observed in Fig. 2.16. This suggests that adapting the BC4:2015 [9] design guide to concrete-encased steel columns should not be sufficient and should require further changes to the EN 1994-1-1 Simplified Method.

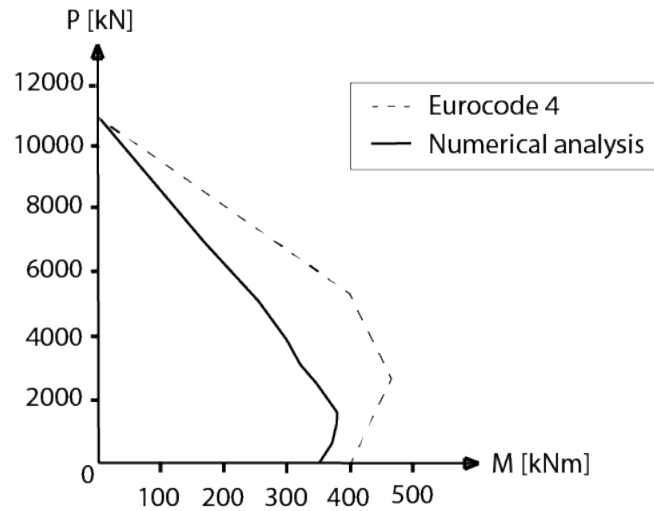


Figure 2.16: Axial load-bending moment interaction curve at mid height section for concrete-encased steel columns (column C10 from Kim *et al.* experimental tests [20]).

Nevertheless this Master's thesis aims to adjust the Simplified Method of EN 1994-1-1 by adapting/modifying the BC4:2015 design guide to concrete encased column. Therefore, the adapted method is the same as EN 1994-1-1 Simplified Method but taking into account:

- Material grades until concrete class C120 and steel grade S600;
- Materials compatibility rules;
- Reduction in concrete compressive strength f_{ck} with the factor η and λ for high-strength concrete.

This adaptation of BC4:2015 design guide proposes a factor $\lambda = 0,7$ for concrete strength class from C90 to C120 and keeps values of λ from EN 1992-1-1 for concrete class from C20 to C90.

In order to vary only one parameter (f_{ck}), the modulus of elasticity will not be reduced. Moreover, the maximum reduction of E_{cm} in BC4:2015 is 5,7%, which could be neglected.

Table 2.7 summarised the cylinder compressive strength, reduced according to this adapted simplified method, which is defined by:

$$f'_{ck} = \eta \lambda f_{ck} \quad (2.39)$$

where η and λ are defined by:

$$\eta = \begin{cases} 1 - (f_{ck} - 50)/200 & 50 \text{ MPa} < f_{ck} \leq 90 \text{ MPa} \\ 0,8 & f_{ck} > 90 \text{ MPa} \end{cases} \quad (2.40)$$

$$\lambda = \begin{cases} 0,8 & f_{ck} \leq 50 \text{ MPa} \\ 0,8 - (f_{ck} - 50)/400 & 50 \text{ MPa} < f_{ck} \leq 90 \text{ MPa} \\ 0,7 & f_{ck} > 90 \text{ MPa} \end{cases} \quad (2.41)$$

Strength classes	C55	C60	C70	C80	C90	C100	C110	C120
f_{ck}	55	60	70	80	90	100	110	120
f'_{ck}	42	44	47	49	50	56	62	67

Table 2.7: Reduced cylinder compressive strength for higher strength concrete according to the adapted simplified method.

Finally, the procedure to calculate the plastic and reduced interaction curves for method 2 is the same as method 1, except that the characteristic compressive cylinder strength f_{ck} is reduced with factor η and λ and that a material compatibility rule limits the possible combination between concrete and steel grades.

2.3 Australian/New Zealand Standard

The Australian and New Zealand (AS/NZ) Standard [10] for the design of composite steel-concrete structures and elements allow the use of steel strength up to 690 MPa and concrete strength from 20 to 100 MPa. These standards provide a method for the design of concrete-filled circular and rectangular hollow steel compression members and concrete encased steel sections, the steel section shall be symmetrical.

This design method is essentially the same as the Simplified Method of EN 1994-1-1, the purpose being to determine the resistance of the cross-section then the resistance of members by including second-order and long-term effects. The main difference concerns the materials strength, which are more limited in the EN 1994-1-1, and the effective flexural stiffness as observed in Table 2.8.

	EN 1994-1-1	Australian/New Zealand Standard
$(EI)_{eff}$	$E_a I_a + 0,6 E_{c,eff} I_c E_s I_s$	$E_a I_a + E_{c,eff} I_c + E_s I_s$
$(EI)_{eff,II}$	$0,9 (E_a I_a + 0,5 E_{c,eff} I_c E_s I_s)$	$0,9 (E_a I_a + 0,2 E_{c,eff} I_c + E_s I_s)$

Table 2.8: Differences in effective elastic flexural stiffness between EN 1994-1-1 and Australian/New Zealand Standard.

The AS/NZ Standard considers an effective elastic flexural stiffness $(EI)_{eff}$ higher than EN 1994-1-1 for first-order analysis. However, for second-order effects, $(EI)_{eff,II}$

defined in AS/NZ Standard is lower than EN 1994-1-1. This difference is mainly due to the fact that second-order effects are characterized by greater deformations; the use of high-strength materials can lead, under these conditions, to early crushing of concrete occurring before yielding of steel. Therefore the reduction of the elastic flexural rigidity of concrete prevents a non-conservative behaviour.

2.3.1 Method 3 - Adapted method of AS/NZ Standard

The adaptation of the AS/NZ Standard extends the authorized concrete strength class until C120. It should be noted that, unlike the BC4:2015 method, the AS/NZ standard does not impose any compatibility on materials or reduction of the characteristic strength of the materials.

The plastic interaction curve will be the same as for the EN 1994-1-1 Simplified Method. However, this reduction of $(EI)_{eff,II}$ will change the reduced $N - M$ interaction curve: N_{Rd} and M_{Rd} reduced both depend on the elastic flexural rigidity, via coefficients χ and k as follows:

$$N_{b,Rd} = \chi N_{Rd} \quad (2.42)$$

$$M_{b,Rd} = \frac{M_{Rd}}{k} \quad (2.43)$$

2.4 RFCS SMARTCoCo project

The SMARTCoCo project [11] aims at studying the behaviour of composite steel concrete components in which steel profiles are used to reinforce concrete structures. During this project, a method to design the hybrid columns, submitted to a compression force and bending moments, has been proposed. In order to insert it easily in actual European standards, it was chosen to try to adapt the method proposed in EN 1992-1-1. It has been decided to develop this method as an adaptation of the nominal stiffness EI presented in EN 1992-1-1.

In EN 1994-1-1, second-order effects are taken into account by multiplying the first-order bending moment and the first-order axial resistance with coefficient k and χ respectively, as explained in the EN 1994-1-1 Simplified Method. These coefficients depend on the effective flexural rigidity $(EI)_{eff,II}$ defined as follows:

$$(EI)_{eff,II} = K_c E_{c,eff} I_c + K_s E_s I_s + K_a E_a I_a \quad (2.44)$$

where according to EN 1994-1-1: $K_c = 0,45$ and $K_s = K_a = 0,9$.

This project proposes to modify the coefficient K_c according to EN 1992-1-1 and the coefficients K_a and K_s by a unique coefficient K_{sa} , using statistical data, obtained from numerical studies. This leads to a coefficient K_c defined by:

$$K_c = \frac{k_1 k_2}{1 + \varphi_{ef}} \quad (2.45)$$

with $k_1 = \sqrt{\frac{f_{ck}}{20}}$; $k_2 = n \frac{\lambda}{170} \leq 0,2$; $\lambda = L \sqrt{\frac{I_g}{A_g}}$ and $n = \frac{N_{Ed}}{N_{pl,Rd}}$. And a coefficient K_{sa} defined by:

$$K_{sa} = \frac{0,45 \lambda^{0,147}}{1 + 1,433 \varphi_{ef} \exp(-0,027 \lambda)} \quad (2.46)$$

Fig. 2.17 and 2.18 show that the slender the column, the higher the coefficient K_{sa} and K_c . Nonetheless, compared to the AS/NZ Standard, K_{sa} and K_c are genuinely lower than $K_0 = 0,9$ and $K_0 K_{e,II} = 0,18$ for stockier column.

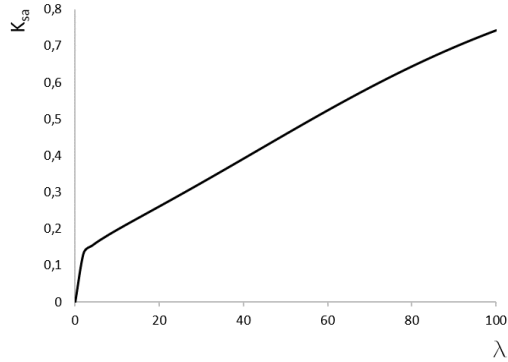


Figure 2.17: Evolution of K_{sa} with λ for $\varphi_{eff} = 2$.

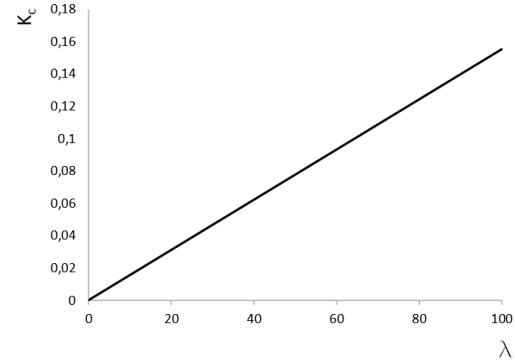


Figure 2.18: Evolution of K_c with λ for $\varphi_{eff} = 2$, $n = 0,5$ and $f_{ck} = 50$ MPa.

To conclude, this adapted method proposes to reduce the coefficient K_c compared to EN 1994-1-1, as in the AS/NZ Standard [10], but also to replace K_a and K_s coefficients by a K_{sa} coefficient lower than EN 1994-1-1 and AS/NZ Standard. However, the report of the SMARTCoCo project indicates that if the method is applied to a composite column, the quantity of steel becomes very large, and the collapse may be produced by yielding of steel. In such a case, it is possible that the value of K_{sa} proposed in this method is too conservative [11].

2.4.1 Method 4 - Adapted method of the SMARTCoCo project

As for the other methods, the adapted method extends the authorized material grades until concrete C120 and steel S600.

The plastic interaction curve will be the same as method 1 but the reduced interaction curve will change due to a decrease of the effective flexural rigidity with coefficient K_c and K_{sa} :

$$(EI)_{eff,II} = K_c E_{c,eff} I_c + K_{sa} E_s I_s + K_{sa} E_a I_a \quad (2.47)$$

which leads to a lower $(EI)_{eff,II}$ than method 1 and 3.

2.5 Fiber Method

The adapted methods, developed in the previous sections, will be implemented in an Excel spreadsheet using a Fiber Method. This method is commonly used in the design of reinforced concrete structures and steel-concrete composite structures when bending moment occurs. It will be used to compare the analytical methods with the numerical model to validate and compare the four methods. In particular, the Fiber Method provides all the points of the interaction curve compared to the 4-points $ABCD$ used as a simplified interaction curve in the Simplified Method of EN 1994-1-1. This number of points, thanks to the Fiber Method, allows to compare analytical and numerical values due to the lack of couple $N - M$ given by the numerical analysis.

In the Excel spreadsheet, a composite section is divided into layers and the force-equilibrium, using plastic distribution, is considered. This force-equilibrium for each layer allows to determine the number of points required to represent the plastic interaction curve. Then it was necessary to modify the spreadsheet to generate the reduced interaction curve, which takes into account the influence of buckling and second-order effects. Finally, this Fiber Method gives a graph with the plastic and the reduced interaction curves as represented in Fig. 2.19.

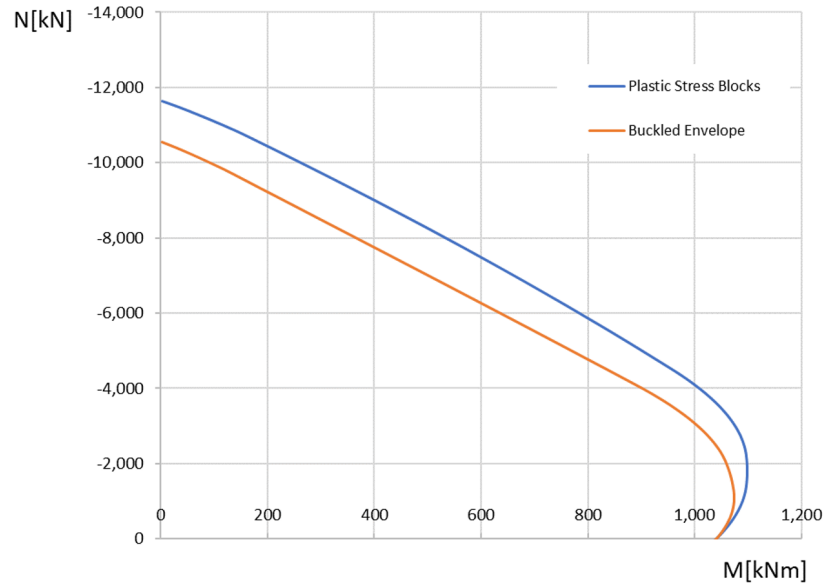


Figure 2.19: Interaction curves of the Fiber Method implemented in an Excel spreadsheet for a steel-concrete composite column. Negative values correspond to compression.

The procedure to evaluate the plastic interaction curve begins with the division of the section in different zones referring to the main elements composing the section (rebars, web of the profile, flange of the profile...). Then the section is divided in 1000 layers of thickness $t = h_c/1000$, where h_c is the height of the concrete for a fully encased section, and each layer has a vertical position Y_i as shown on Fig. 2.20.

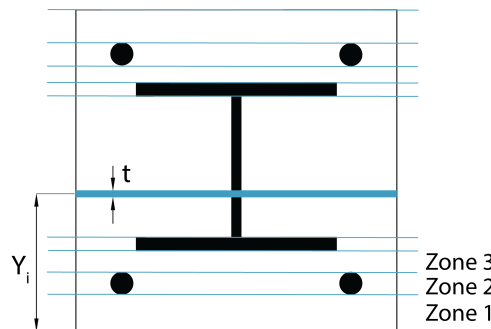


Figure 2.20: Discretization of the section.

This gives a table, which allows, in function of the zone, to determine the area of the corresponding material for each layer, see Table 2.9. Afterwards, it is possible to calculate for each layer and for each material, the second moment area I and plastic section

modulus W_{pl} . Knowing these parameters, by varying the position of the neutral axis Z_p and considering full plastic distribution of stresses, the resistant axial force and resistant bending moment can be calculated with the force-equilibrium. Therefore, this position of Z_p gives one $N - M$ point of the plastic interaction curve. The reduced interaction curve will be calculated based on the plastic interaction curve by multiplying, for each point of the curve, N_{Rd} by coefficient χ and M_{Rd} by coefficient k , as explained in the previous sections.

	Zone 1	Zone 2		Zone 3	Zone 4		...
Y_i [mm]	Concrete	Concrete	Rebars	Concrete	Concrete	Profile	...
0							
0.5							
1							
...							

Table 2.9: Representation of the tableau concerning the discretization of the section.

Fig. 2.21 shows the plastic stress blocks in function of the neutral axis position for fully encased section and Fig. 2.22 for partially encased section considering high-strength materials, hence the concrete has no plastic plateau and the parabola-rectangle stress-strain relation is adapted to a rectangular relation.

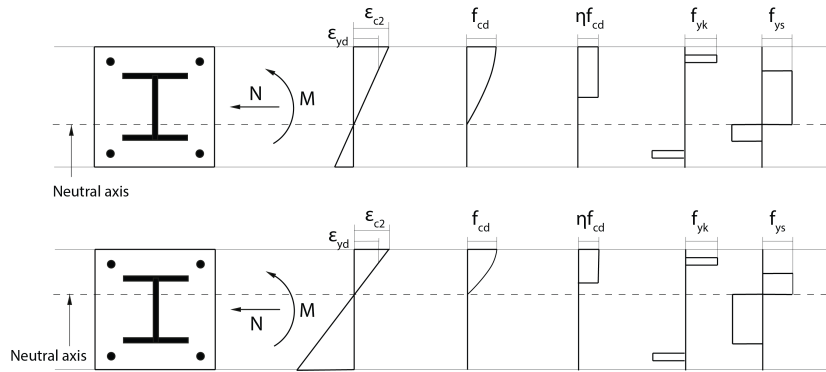


Figure 2.21: Representation of the stress distribution for the different materials in function of the position of neutral axis for FEC cross-section, considering Plastic Design Method.

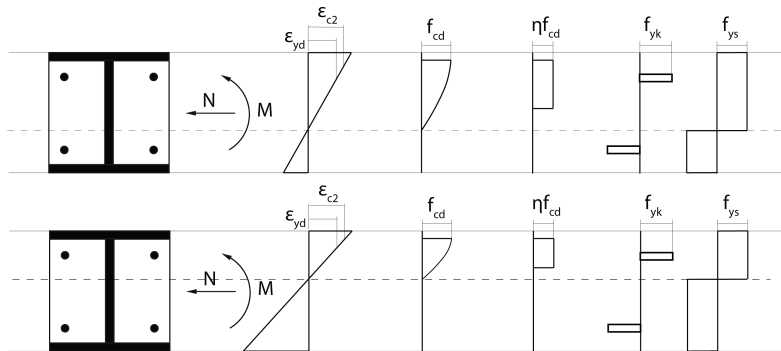


Figure 2.22: Representation of the stress distribution for the different materials in function of the position of neutral axis for PEC cross-section, considering Plastic Design Method.

Chapter 3

Experimental tests and numerical validation

3.1 Experimental test

In order to calibrate the numerical model, experimental tests must be selected from open literature. This thesis aims to adapt the EN 1994-1-1 Simplified Method for fully and partially concrete-encased column using high-strength materials. However, there is a lack of experimental testing on steel-concrete composite columns using high-strength materials. The majority of tests in the literature concerns high-strength concrete with normal-strength steel or *vice versa*, but rarely both sets. Therefore, the numerical model will be calibrated with a limited amount of data.

The only tests considering fully and partially concrete encased steel sections using high-strength materials were performed by Kim *et al.* [19] [20]. The following columns were chosen from these experimental tests: 2 fully concrete-encased steel columns (C1 and C11) and 1 partially concrete-encased steel column (C6) as shown in Fig. 3.1 and Table 3.1. The C11 column is slender to obtain a larger influence of second-order effects.

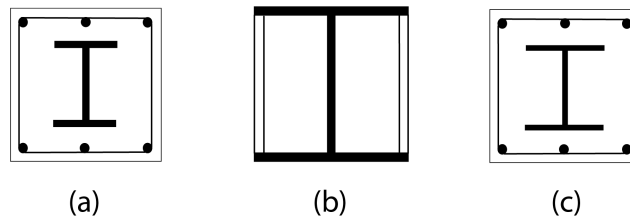


Figure 3.1: Sections of the specimens: (a) C1; (b) C6; (c) C11.

	C1 (shorter)	C6 (shorter)	C11 (slender)
Specimens length [mm]	900	900	1800
Section [mm ²]	260 x 260	260 x 260	260 x 260
Concrete f'_c [MPa]	91,3	113	104
Steel profile $h \times b \times t_w \times t_f$ [mm]	150 x 100 x 17,6 x 17,6	260 x 260 x 15 x 15	150 x 150 x 15 x 15
Steel profile f_{ys} [MPa]	913	806	812
Steel profile E_s [GPa]	205	205	205
Longitudinal rebar	6 – $\phi 13$	/	6 – $\phi 13$
Steel rebar f_{yt} [MPa]	525	/	512
Steel rebar E_t [GPa]	193	/	193

Table 3.1: Properties of test specimens.

3.1.1 Materials

Steel profiles were fabricated using 17,6 mm and 15 mm thick steel plates (HSA800 from Korean Industrial Standards) and groove welding. The steel plates specimen were tested under direct tension and Fig. 3.2 shows the stress-strain relationship of the 17,6 mm thick steel plate. The high-strength steel does not show apparent yield plateau, therefore, the 0,2% yield strength was measured.

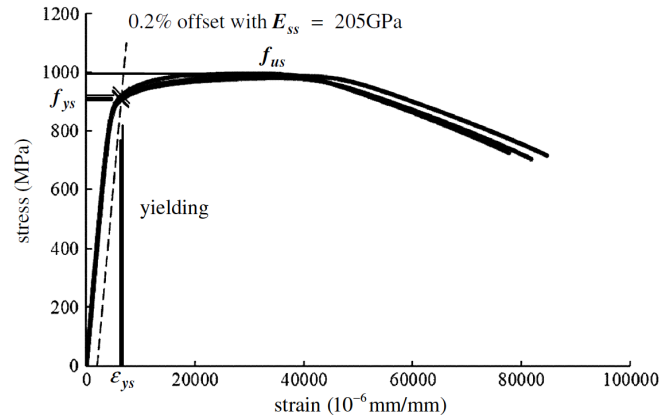


Figure 3.2: Stress-strain relationship of 17,6 mm thick steel plates [19].

The concrete was designed for 28-days cylinder compressive strength $f'_c = 100$ MPa. The strain corresponding to the maximum compressive strength was $\epsilon = 0,0026$ on average, which is lower than the yield stress of the steel $\epsilon_{ys} = 0,0045$. Consequently the steel will not be able to yield in the compressed area. Fig. 3.3 shows the brittle behaviour of the high-performance concrete under axial compression.

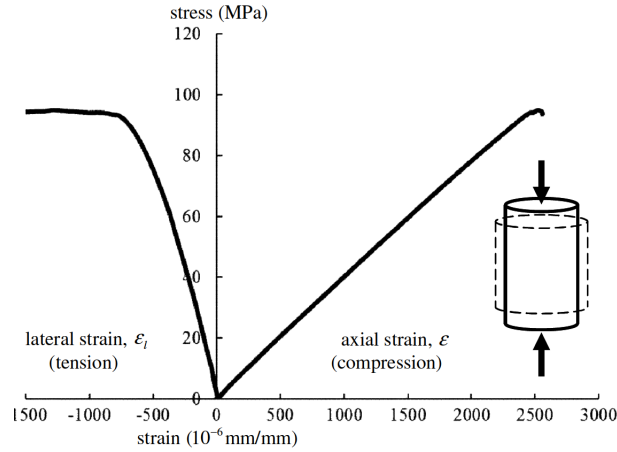


Figure 3.3: Stress-strain relationship of concrete cylinder with $f'_c = 94$ MPa [19].

3.1.2 Test setup and instrumentation

Fig. 3.4 describes the test setup and instrumentation for the specimens. Both ends of the columns were strengthened by concrete-filled steel tubes to secure the structural integrity between steel and concrete. An eccentric axial load of 120 mm was applied to the specimen through the knife edges in the direction of the strong axis. The buckling lengths were respectively, for the shorter and the slender columns, 2,62 m and 3,52 m due to all the equipment.

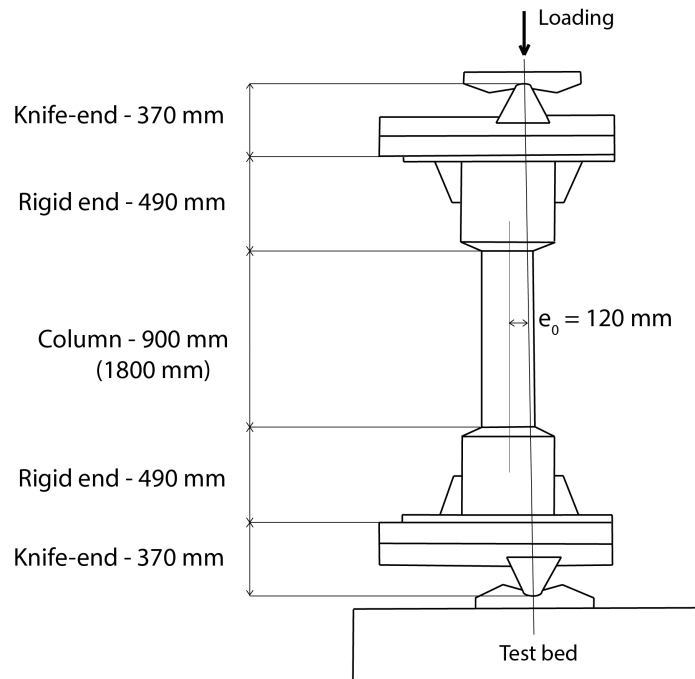


Figure 3.4: Description of the test setup [19].

3.1.3 Test results

Failure mode

The specimens failed because of concrete crushing in the compression side. For the fully encased specimens, after concrete cover spalling, crushing of the confined concrete occurred in the compression side. Moreover, buckling of the longitudinal rebars and yielding of the ties occurred [19] [20]. For the partially encased specimen, crushing of the confined concrete, welding failure of the link bars and local buckling of the steel flanges were observed [19]. The measurements by the strain gauges showed that yielding of the ties occurred in column C1 and C11. For column C6, the link bars between the flanges failed at the welded connection without yielding, and at the same moment, flange local buckling occurred.

Axial load-displacement relationship

Considering axial load-displacement relationship, for C1 and C11, even after complete cover spalling ($\epsilon_{cu} = 0.003$ designated as the initial concrete-crushing load), the integrity and strength were maintained, due to good lateral confinement. The ties yielded immediately after the concrete cover spalling. Before the peak load, yielding of the compression and tension steel flanges occurred. In the partially encased column C6, yielding of the compression steel flange appeared before the first peak, because of steel flange was located at the extreme compression side of the cross-section.

Summary

The test results showed that in the case of inadequate lateral confinement, the load-carrying capacity was limited by the early crushing of concrete. However, because of the high-strength steel section, all specimens shown ductile flexural behaviour, maintaining the moment-carrying capacity after the delamination of concrete, which demonstrated a good structural performance. The study also compared the current design codes with a numerical analysis, which was validated with the experimental test and showed that EN 1994-1-1, using full plastic capacity of steel section, overestimated the test results.

3.2 SAFIR numerical model

SAFIR is a special purpose computer program, developed at the University of Liège, for the analysis of structures under ambient and elevated temperature conditions. The program, which is based on the Finite Element Method, can be used to study the behaviour of one, two and three-dimensional structures [12].

3.2.1 Model characteristics

The model was created from GiD-SAFIR software. GiD is the pre-processor and allows sections and structures definition. It is possible to define geometries, assign materials, provide information on thermal and mechanical characteristics and generate a mesh. SAFIR is the processor, it takes the GiD values as input, solves the problem by finite element analysis and outputs the results. This output file is interpreted by the post-processor, DIAMOND, and allows to visualise the deformed structure and the results [12].

First step begins with the description of the section, for columns C1 and C11, there are 3 materials: concrete, steel profile and longitudinal rebars (Fig. 3.5 and 3.6); and for column

C6, there are 2 materials: concrete and steel profile (Fig. 3.7). SAFIR software considers perfect bond between structural components by default. The transverse reinforcement could not be modelled in the case of a beam element analysis. Afterwards, it is necessary to define the mesh, GiD allows to assign the size of the mesh, which is an important step, because a fine mesh allows to obtain a model closer to reality, however, it increases the calculation time.

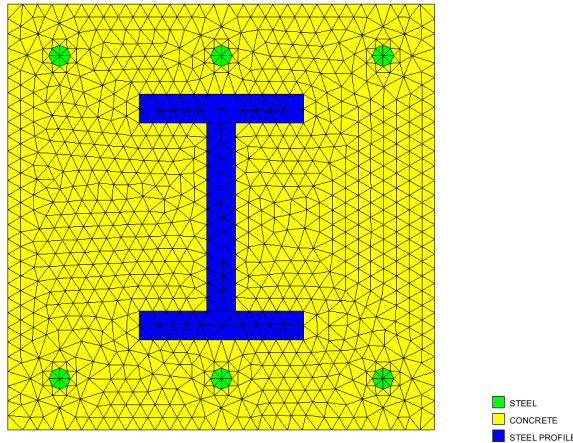


Figure 3.5: Representation of the cross-section and the mesh from GiD software for column C1.

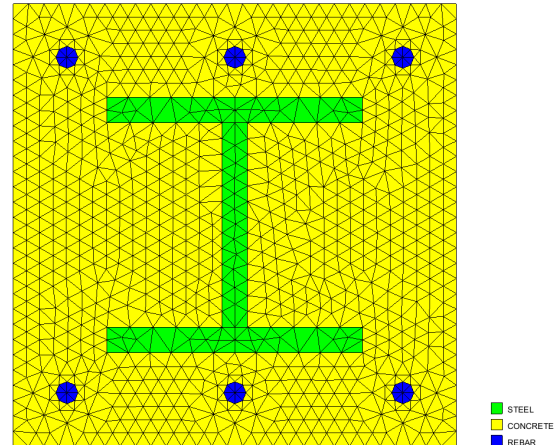


Figure 3.6: Representation of the cross-section and the mesh from GiD software for column C11.

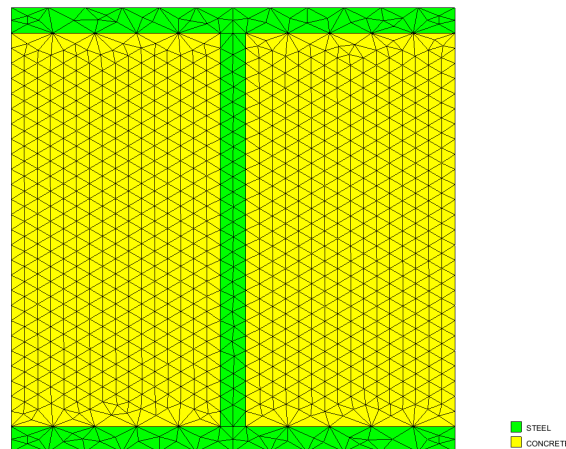


Figure 3.7: Representation of the cross-section and the mesh from GiD software for column C6.

Second step consists to define the 2D structure, in this case, a column. A real column is never perfectly straight, EN 1994-1-1 imposes an initial imperfection depending on the type of section and buckling axis, see Table 2.1. For a modelling of these experimental tests, since the eccentricity is along the strong axis and given the sections, the initial imperfection is $L/200$, with L the length of the column. The total length of the column will not be that of the specimen but the length between the knife edges, *i.e.* 2,62 m and 3,52 m for, respectively, shorter and longer columns. This is done to take into account the total length of the column and consequently all second-order effects.

The boundary conditions of the model are pinned at both ends, *i.e.* that displacement along x and y axis are blocked (except along y axis at the top of the column to allow the application of the load). The load is applied at the top of the column with an eccentricity along the strong axis of 120 mm (see Fig. 3.9). The loading function F1000PS is equivalent to a load of 1 [kN] applied each second, *e.g.* after 200 seconds the load applied on the top of the column is 200 kN. This loading function allows to neglect the additional constraints of dynamic actions due to the speed of the loading.

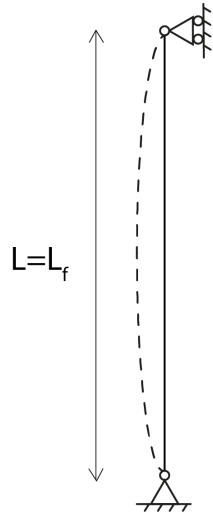


Figure 3.8: Representation of the boundary conditions of the 2D column. These give a buckling length equal to the column length.



Figure 3.9: The load F is applied along y axis and the amplitude is 1 [kN/sec] (compression) given by loading function F1PS.

Third step consists to assign the section defined in step 1 to the structure in step 2. Unlike the experimental test, the concrete-encased steel section is applied over the entire length of the column. The mechanical characteristics of the materials are defined as values in Table 3.1. Materials definition are explained in section 3.2.2.

The final step is to create the mesh of the column, as previously explained, the finer the mesh, the closer the model results will be to reality, but the slower the speed calculation will be. Then it is necessary to run the calculation and to analyse the output file with DIAMOND.



Figure 3.10: Result from DIAMOND software showing deflection of the composite column C11 (amplified 10 times).

3.2.2 Materials definition

SAFIR is a software that models the behaviour of structures in buildings under fire conditions. Therefore, the law of materials are defined according to the part for structural fire design of the European codes. Nevertheless, cold structural analyses, such as in this study, are possible. This section describes the materials laws implemented in SAFIR according to EN 1992-1-2 and EN 1993-1-2.

Concrete

SAFIR software considers the stress-strain relation according to EN 1992-1-2 [27] for concrete in compression, defined by:

$$\sigma_c = \frac{3\epsilon f_{c,\theta}}{\epsilon_{c1,\theta} \left(2 + \left(\frac{\epsilon}{\epsilon_{c1,\theta}} \right)^3 \right)} \quad (3.1)$$

in the case of this study, the concrete is considered as siliceous aggregates at a temperature of 20°C, which leads to concrete parameters: $f_{c,\theta}/f_{ck} = 1$, $\epsilon_{c1,\theta} = 0,0025$ and $\epsilon_{cu1,\theta} = 0,02$.

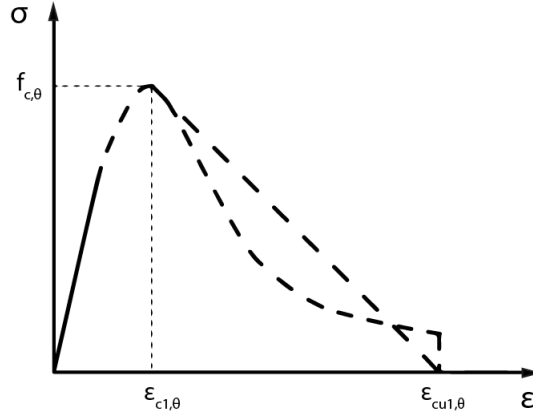


Figure 3.11: Stress-strain relation for concrete in compression according to EN 1992-1-2 [27].

Structural steel

Structural steel is defined according to EN 1993-1-2 [28], and considers a stress-strain relationship described as follows:

$$\sigma_a = \begin{cases} E_{a,\theta} & \text{for } 0 \leq \epsilon \leq \epsilon_{sp,\theta} \\ f_{p,\theta} - c + (b/a)(a^2 - (\epsilon_{y,\theta} - \epsilon)^2)^{0,5} & \text{for } \epsilon_{p,\theta} < \epsilon < \epsilon_{y,\theta} \\ f_{y,\theta} & \text{for } \epsilon_{y,\theta} \leq \epsilon \leq \epsilon_{t,\theta} \\ f_{y,\theta}(1 - (\epsilon - \epsilon_{t,\theta})/(\epsilon_{u,\theta} - \epsilon_{t,\theta})) & \text{for } \epsilon_{t,\theta} < \epsilon < \epsilon_{u,\theta} \\ 0 & \text{for } \epsilon = \epsilon_{u,\theta} \end{cases} \quad (3.2)$$

with:

$$a^2 = (\epsilon_{y,\theta} - \epsilon_{p,\theta})(\epsilon_{y,\theta} - \epsilon_{p,\theta} + c/E_{a,\theta}) \quad (3.3)$$

$$b^2 = c(\epsilon_{y,\theta} - \epsilon_{p,\theta})E_{a,\theta} + c^2 \quad (3.4)$$

$$c = \frac{(f_{y,\theta} - f_{p,\theta})^2}{(\epsilon_{y,\theta} - \epsilon_{p,\theta})E_{a,\theta} - 2(f_{y,\theta} - f_{p,\theta})} \quad (3.5)$$

for structural steel at ambient temperature (20°C), the parameters are given in Table 3.2.

$\epsilon_{y,\theta}$	$\epsilon_{p,\theta}$	$\epsilon_{t,\theta}$	$\epsilon_{u,\theta}$	$f_{y,\theta}/f_y$	$f_{p,\theta}/f_y$	$E_{a,\theta}/E_a$
0,02	$f_{sp,\theta}/E_{s,\theta}$	0,15	0,2	1	1	1

Table 3.2: Parameters for the stress-strain relation according to Table 3.1 and Fig. 3.1 of EN 1993-1-2 [28].

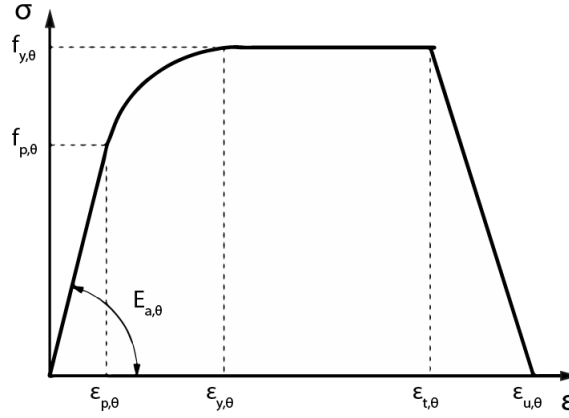


Figure 3.12: Stress-strain relation for structural steel according to EN 1993-1-2.

Reinforcement steel

The characteristics of the reinforcing steel are in accordance with EN 1992-1-2 [27], and the stress-strain relationship is defined by:

$$\sigma_s = \begin{cases} E_{s,\theta} & \text{for } 0 \leq \epsilon \leq \epsilon_{sp,\theta} \\ f_{sp,\theta} - c + (b/a)(a^2 - (\epsilon_{sy,\theta} - \epsilon)^2)^{0,5} & \text{for } \epsilon_{sp,\theta} \leq \epsilon \leq \epsilon_{sy,\theta} \\ f_{sy,\theta} & \text{for } \epsilon_{sy,\theta} \leq \epsilon \leq \epsilon_{st,\theta} \\ f_{sy,\theta}(1 - (\epsilon - \epsilon_{st,\theta})/(\epsilon_{su,\theta} - \epsilon_{st,\theta})) & \text{for } \epsilon_{st,\theta} \leq \epsilon \leq \epsilon_{su,\theta} \\ 0 & \text{for } \epsilon = \epsilon_{su,\theta} \end{cases} \quad (3.6)$$

with:

$$a^2 = (\epsilon_{sy,\theta} - \epsilon_{sp,\theta})(\epsilon_{sy,\theta} - \epsilon_{sp,\theta} + c/E_{s,\theta}) \quad (3.7)$$

$$b^2 = c(\epsilon_{sy,\theta} - \epsilon_{sp,\theta})E_{s,\theta} + c^2 \quad (3.8)$$

$$c = \frac{(f_{sy,\theta} - f_{sp,\theta})^2}{(\epsilon_{sy,\theta} - \epsilon_{sp,\theta})E_{s,\theta} - 2(f_{sy,\theta} - f_{sp,\theta})} \quad (3.9)$$

for a hot rolled reinforcing steel at 20°C, the parameters are given in Table 3.3.

$\epsilon_{sy,\theta}$	$\epsilon_{sp,\theta}$	$\epsilon_{st,\theta}$	$\epsilon_{su,\theta}$	$f_{sy,\theta}/f_{yk}$	$f_{sp,\theta}/f_{yk}$	$E_{s,\theta}/E_s$
0,02	$f_{sp,\theta}/E_{s,\theta}$	0,15	0,2	1	1	1

Table 3.3: Parameters for the stress-strain relation according to Table 3.2a and Fig. 3.3 of EN 1992-1-2 [27].

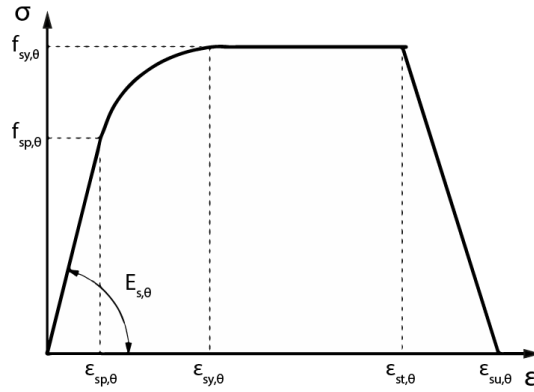


Figure 3.13: Stress-strain relation for reinforcing steel according to EN 1992-1-2.

3.3 Comparison between numerical and experimental values

Before proceeding to the comparison of the adapted analytical methods with a sensitive study, the experimental values must be compared with the numerical model values to prove its reliability. As explained in section 3.2.1, 3 types of columns have been chosen and modelled using SAFIR software. The results of the experimental tests and the SAFIR simulations are represented in Fig. 3.14 to 3.16. These figures compared the P-d diagram (vertical applied force-vertical displacement) for each column types.

The purpose of these models is to obtain a good flexural rigidity, compared to experimental tests, in order to achieve a faithful representation of the real interaction diagrams compared to those calculated by the adapted methods. Fig. 3.14 to 3.16 show a match between numerical and experimental tests during the elastic behaviour then the model overestimated the plastic behaviour. However, concerning the partially encased section (column C6), it could be observed that the agreement with the test is not as satisfying as for the fully encased sections. Due to the lack of experimental test for such material grades, this difference between numerical and experimental values is quite suitable. Table 3.4 presents the difference of the ratio between loads obtained from the finite element analyses (P_{FE}) and experimental test (P_{Test}) for different displacement.

Column		C1	C6	C11
$\frac{P_{FE} - P_{Test}}{P_{FE}}$	6 [mm]	7%	25%	7%
	8 [mm]	3%	15%	2%
	10 [mm]	1%	13%	5%
	12 [mm]	4%	5%	8%

 Table 3.4: Difference of the ratios between finite element analyses and experimental test for different displacement d for each tested column.

To conclude, the suitable structural performance of experimental tests and the good agreement between these tests and numerical analysis allows to validate the numerical model, which can be used as a reference for the future sensitive study of the different analytical methods.

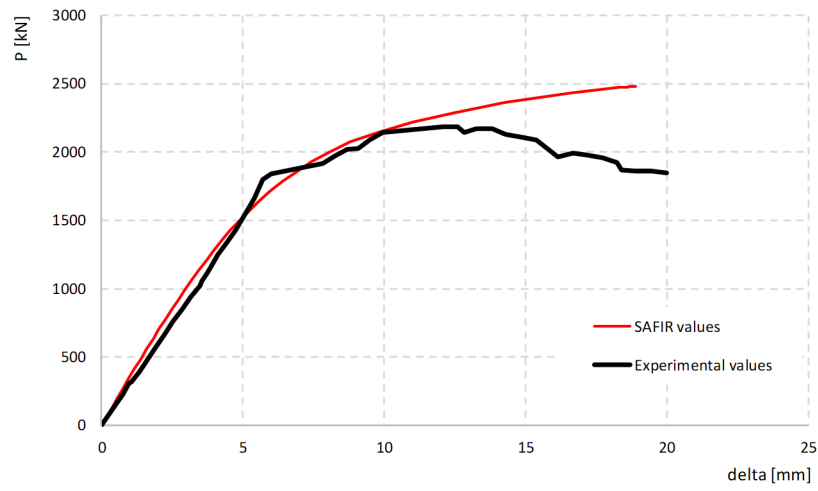


Figure 3.14: P-d diagram for experimental and numerical values of column C1.

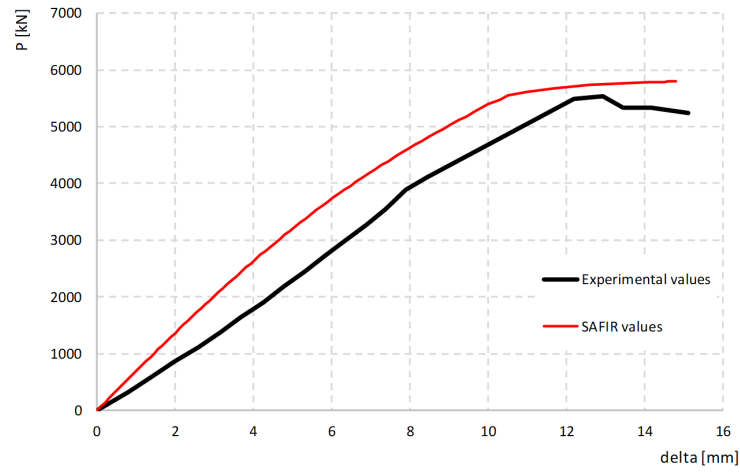


Figure 3.15: P-d diagram for experimental and numerical values of column C6.

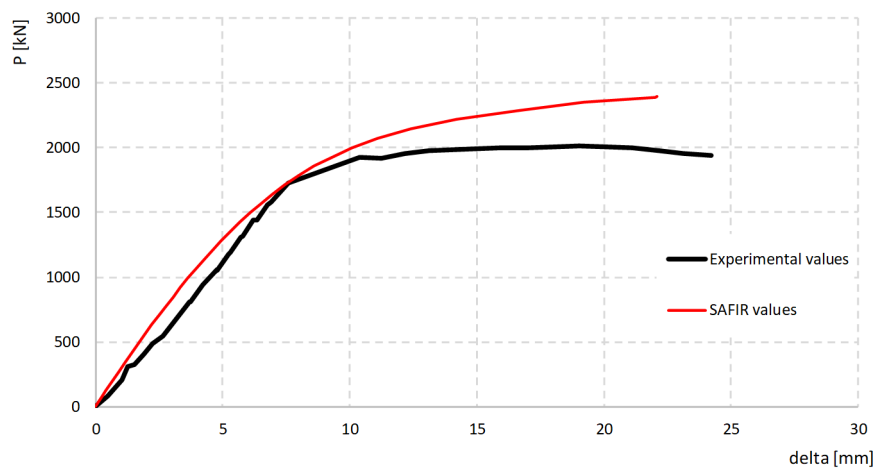


Figure 3.16: P-d diagram for experimental and numerical values of column C11.

Chapter 4

Comparisons between analytical methods with FEM model

4.1 Analytical methods

As explained in Chapter 2, four current analytical methods for the design of composite columns were chosen. These methods were adjusted to allow the use of material grades up to C120 for concrete and S600 for steel. All the methods are based on the calculation of the plastic interaction curve by varying the neutral axis position and, for each neutral axis position, the calculation of the internal forces, using plastic stress blocks. The second-order and long-term effects are taken into account by reducing the plastic interaction diagram, as explained in section 2.1.5.

4.1.1 Method 1 - Adapted Simplified Method of EN 1994-1-1

This first method, considered as the reference method, is not influenced by the reduction of the materials. The effective flexural rigidity $(EI)_{eff,II}$ takes into account second-order and long-term effects and is defined as:

$$(EI)_{eff,II} = K_0(I_a E_a + I_s E_s + K_{e,II} I_c E_{c,eff}) \quad (4.1)$$

where $K_0 = 0,9$ and $K_{e,II} = 0,5$. This effective flexural rigidity is used to defined coefficient χ and k to reduce the resistant axial force N_{Rd} and resistant bending moment M_{Rd} of the reduced interaction curve.

4.1.2 Method 2 - Adapted method of BC4:2015 design guide

The second method considers material compatibility according to Table 2.6 and reduction of the characteristic compressive cylindric strength of concrete f_{ck} as follows:

$$f'_{ck} = \eta \lambda f_{ck} \quad (4.2)$$

with coefficient η and λ defined by:

$$\eta = \begin{cases} 1 - (f_{ck} - 50)/200 & 50 \text{ MPa} < f_{ck} \leq 90 \text{ MPa} \\ 0,8 & f_{ck} > 90 \text{ MPa} \end{cases} \quad (4.3)$$

$$\lambda = \begin{cases} 0,8 & 50 f_{ck} \leq 50 \text{ MPa} \\ 0,8 - (f_{ck} - 50)/400 & 50 \text{ MPa} < f_{ck} \leq 90 \text{ MPa} \\ 0,7 & f_{ck} > 90 \text{ MPa} \end{cases} \quad (4.4)$$

This method, as an extension of method 1, considers the same effective flexural rigidity for the reduced interaction curve:

$$(EI)_{eff,II} = K_0(I_a E_a + I_s E_s + K_{e,II} I_c E_{c,eff}) \quad (4.5)$$

where $K_0 = 0,9$ and $K_{e,II} = 0,5$.

4.1.3 Method 3 - Adapted method of AS/NZ Standard

The third method is mainly the same as the adapted EN 1994-1-1 Simplified Method (method 1) excepted that coefficient $K_{e,II}$ introduced in the effective flexural rigidity is equal to 0,2, which leads to:

$$(EI)_{eff,II} = K_0(I_a E_a + I_s E_s + K_{e,II} I_c E_{c,eff}) \quad (4.6)$$

where $K_0 = 0,9$ and $K_{e,II} = 0,2$. Therefore the part of the concrete flexural rigidity is even more reduced than for method 1 and 2 and the reduced interaction curve should be more reduced as well.

4.1.4 Method 4 - Adapted Method of SMARTCoCo project

The final method is also based on EN 1994-1-1 and proposes a reduction of the effective flexural rigidity, according to EN 1992-1-1 for K_c and it relies on a numerical study for K_{sa} , as follows:

$$(EI)_{eff,II} = K_{sa} I_a E_a + K_{sa} I_s E_s + K_c I_c E_{c,eff} \quad (4.7)$$

where K_{sa} and K_c are defined in section 2.4 and are generally lower, respectively, than 0,9 and 0,18 as defined in the AS/NZ Standard. Consequently, the reduced interaction diagram should be more reduced than for method 3.

4.2 Considered parameters

The parameters take into account for this sensitive study are the section, the length and the materials strength.

The variation of the section allows to test different steel profiles such as HEA, HEB, HEM or HD. The HEA profile will be used to design the partially concrete encased steel sections because the width of the flange over the height of the web is higher than the other profile types. This leads to a larger area of concrete relative to the area of steel. The HEB, HEM and HD profiles will be used to design the fully concrete encased steel sections to observe the variation of proportion of the different profiles on the behaviour of the composite column. Moreover, not only the type of the profile but also the dimensions will be tested, *e.g.* HEB100 and HEB1000.

In the case of the fully encased sections, the area of concrete will vary compared to the dimension of the steel profile to investigate the influence of the steel contribution coefficient δ , defined by:

$$\delta = \frac{A_a f_{yd}}{N_{pl,Rd}} \quad (4.8)$$

EN 1994-1-1 Simplified Method limits this coefficient as follows: $0,2 \leq \delta \leq 0,9$. The larger the δ value, the closer the column will be to a steel column and *vice versa*, the smaller the

δ value, the closer the column will be to a reinforced concrete column. Consequently, *a priori*, the sections with a low δ value should expect to underestimate the resistance of the column for analytical methods compared to the numerical analysis.

When defining the section of the studied specimens, it will also be necessary to keep in mind the rules concerning the concrete cover c_y and c_z defined in clause 6.7.3.1 of EN 1994-1-1. For fully encased sections, this rule makes it difficult to obtain delta values greater than 0,7.

Another key parameter is the length of the column. In fact, the second-order effects are not negligible with the increases of the length of a column and could become the major part influencing the resistance of the column. The length also plays a dominant role in the relative slenderness $\bar{\lambda}$ of the column and EN 1994-1-1 limits the $\bar{\lambda}$ value to 2, beyond this value the second-order effects are too important. In order to observe the behaviour of the length, 3 tests for each kind of column will be tested: the resistance of the cross-section corresponding to a column of 0 m long, the resistance of a 3,5 m long column and the resistance of a 7 m long column.

Finally, the last parameters tested are the materials strength. As a reminder, EN 1994-1-1 allows the use of concrete strength class up to C50 and steel grade up to S460. In order to verify the EN 1994-1-1 Simplified Method, a set of materials respecting the EN 1994-1-1 rules are chosen for each kind of tested column. Table 4.1 presents the different sets of materials, named "test" for this analysis. This first set of materials, test 1, concerns normal-strength materials to observe the agreement of the methods, in particular the EN 1994-1-1 Simplified Method, with the numerical values. Furthermore, this Master's thesis aims to extend the Simplified Method to the use of higher strength materials. Therefore, two sets of high-strength materials will be used: test 2 and test 3. All of these sets of materials respect the material compatibility defined in Table 2.6. In order to go further, incompatible materials were also chosen: test 4, 5 and 6. Test 6 is, in fact, composed of compatible materials but at the limit according to Table 2.6. It should be noted that the material strength of the reinforcing steel is constant and corresponds to S500.

	Test 1	Test 2	Test 3	Test 4	Test 5	Test 6
Concrete	C30	C90	C120	C30	C30	C90
Structural steel	S355	S500	S600	S500	S600	S600
Reinforcing steel	S500					

Table 4.1: Description of the different material sets or the different tests.

Table 4.2 to 4.5 summarise the different specimens tested in this parametric study. Table 4.2 refers to the fully encased concrete sections (FEC) along the strong flexural axis, Table 4.3 is the same but for the weak flexural axis and Table 4.4 and 4.5 correspond to partially encased concrete sections (PEC), respectively for strong and weak flexural axis.

N°	Profile	Concrete [mm^2]	Reinforcement	Concrete cover c_v [mm]	Test performed
FEC 1	HEB260	400x400	4 ϕ 16	35	1,2
FEC 2	HEB180	300x300	4 ϕ 14	38	1,2,3
FEC 3	HEM320	500x500	4 ϕ 16	40	1,2,3
FEC 4	HD400x237	600x600	4 ϕ 20	46	1,2,3
FEC 5	HD260x68,2	360x360	4 ϕ 16	40	1,2,3
FEC 6	HD260x68,2	450x450	4 ϕ 16	40	1,2,3
FEC 7	HD260x68,2	375x375	4 ϕ 20	46	1,2,3
FEC 8	HD260x68,2	450x450	4 ϕ 20	46	1,2,3,4,5,6
FEC 9	HD260x68,2	400x400	4 ϕ 25	55,5	1,2,3
FEC 10	HD260x68,2	450x450	4 ϕ 25	55,5	1,2,3
FEC 11	HEB100	180x180	4 ϕ 10	31	1,2,3
FEC 12	HEB1000	1200x600	10 ϕ 28	60	1,2,3
FEC 13	HEM260	410x380	4 ϕ 14	37	1,2,3

Table 4.2: Specimens dimensions for fully concrete encased steel sections along strong axis.

N°	Profile	Concrete [mm^2]	Reinforcement	Concrete cover c_v [mm]	Test performed
FEC 1	HEB260	400x400	4 ϕ 16	35	1,2
FEC 2	HEB180	300x300	4 ϕ 14	38	1,2,3
FEC 3	HEM320	500x500	4 ϕ 16	40	1,2,3
FEC 4	HD400x237	600x600	4 ϕ 20	46	1,2,3
FEC 11	HEB100	180x180	4 ϕ 10	31	1,2,3
FEC 12	HEB1000	1200x600	10 ϕ 28	60	1,2,3

Table 4.3: Specimens dimensions for fully concrete encased steel sections along weak axis.

N°	Profile	Reinforcement	c_y [mm]	c_z [mm]	Test performed
PEC 1	HEA180	4 ϕ 14	37	27.5	1,2
PEC 2	HEA400	4 ϕ 20	46	55	1,2,3
PEC 3	HEA500	4 ϕ 20	46	50	1,2,3
PEC 4	HEA100	4 ϕ 8	30	20	1,2,3
PEC 11	HEA1000	4 ϕ 28	60	65	1,2,3

Table 4.4: Specimens dimensions for partially concrete encased steel sections along strong axis.

N°	Profile	Reinforcement	c_y [mm]	c_z [mm]	Test performed
PEC 1	HEA180	4 ϕ 14	37	27.5	1,2
PEC 2	HEA400	4 ϕ 20	46	55	1,2,3
PEC 3	HEA500	4 ϕ 20	46	50	1,2,3
PEC 4	HEA100	4 ϕ 8	30	20	1,2,3
PEC 11	HEA1000	4 ϕ 28	60	65	1,2,3

Table 4.5: Specimens dimensions for partially concrete encased steel sections along weak axis.

4.3 Sensitive study

The four adapted simplified methods will be implemented using a Fiber Method working on an Excel spreadsheet. The use of this method is driven by the fact that the software SAFIR gives only one point $N - M$ per simulation. The solution is to calculate 5 $N - M$ points for each interaction diagram with SAFIR and to use a Fiber Method, which generates easily a lot of $N - M$ points to create the interaction curve. Then it is possible to compare the numerical analysis with the adapted methods implemented with this Fiber Method, as shown in Fig. 4.1. To compare the values, a ratio R is introduced and is defined as the ratio between the distance OB over the distance OA , where O corresponds to the coordinate $(0;0)$, *i.e.* $R = \overline{OB}/\overline{OA}$. If $R > 1$ than the analytical method unsafely overestimates the numerical analysis and *vice versa*.

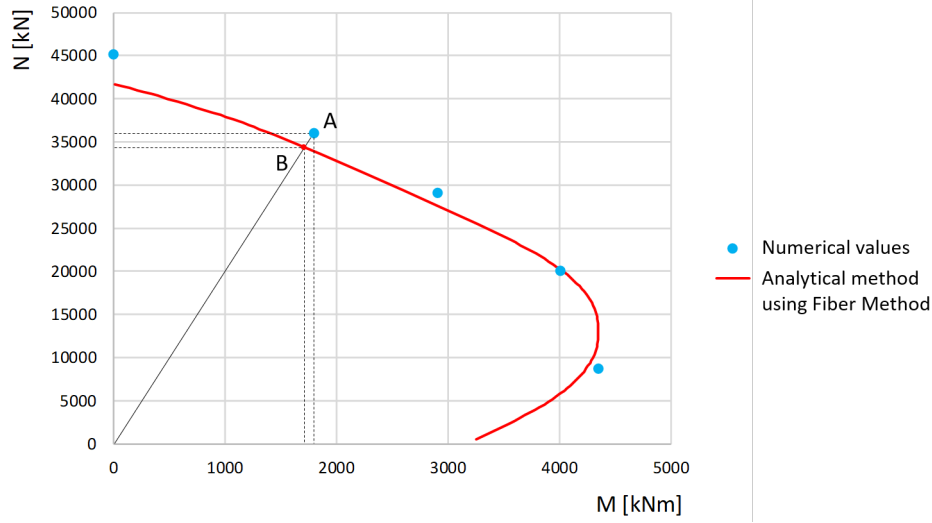


Figure 4.1: Representation of the comparison between SAFIR software and EN 1994-1-1 implemented with a Fiber Method. The point A of the numerical analysis could be compared with the corresponding point B of the interaction curve determined with the EN 1994-1-1.

As previously explained, 5 points for each $N - M$ interaction curves have been determined. Each simulation with SAFIR software gives the resistant axial force of the column N_{Rd} , the related bending moment M_{Rd} , produced by this axial force, is determined as follows:

$$M_{Rd} = N_{Rd} e \quad (4.9)$$

with e the eccentricity imposed during the simulation. Different values of the eccentricity were chosen to describe the complete interaction curve: $e = [0; 0,05; 0,1; 0,2; 0,5]$ m which corresponds respectively to points=[A; B; C; D; E] in Fig. 4.2.

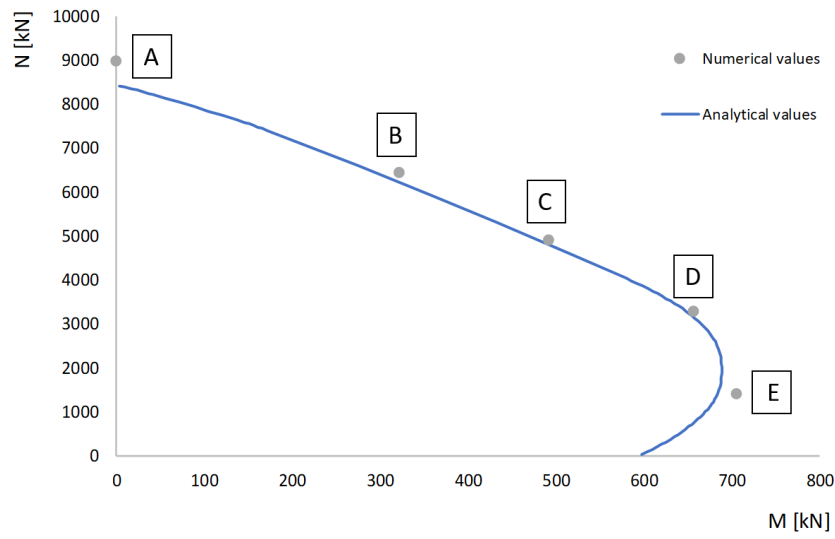


Figure 4.2: Representation of the points related to the eccentricity e .

SAFIR software gives the bearing capacity of the column for different eccentricities. Since columns are subjected mainly to axial force with low eccentricity, generally up to 20%, there is no model to determine the bending moment when e tends to ∞ and N is close to 0. Consequently, the maximum bending moment is often defined by $M_{Ed} \leq 0,2 h N_{Ed}$ (Fig. 4.3).

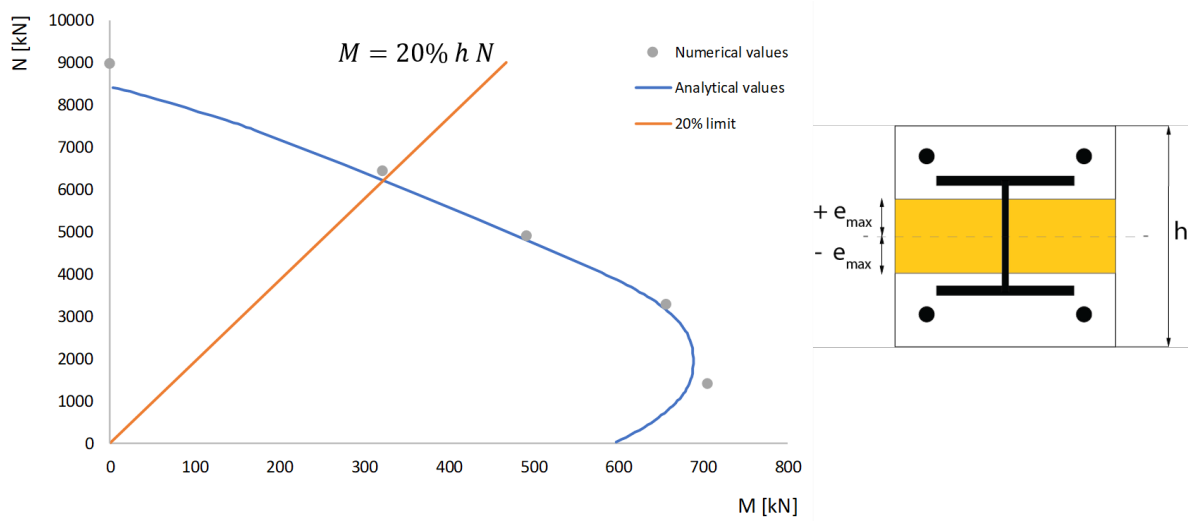


Figure 4.3: Representation of the generally maximum applied bending moment on a composite column (left) and orange zone represents the maximal eccentricity applied on a composite column (right).

This analysis will be divided as follows:

- Fully concrete encased steel (FEC) sections along strong axis;
 - Global analysis to compare the different methods;
 - Local analysis to determine the influence of the different parameters;
- Partially concrete encased steel (PEC) sections along strong axis;
 - Global analysis to compare the different methods;
 - Local analysis to determine the influence of the different parameters.

It should be noted that this study only concern fully and partially encased sections along strong axis, the specimens along weak axis were not analysed but the results of numerical simulations and analytical methods are presented in Appendix B.

4.3.1 Comparison of the different methods for FEC

To analyse the performance of the analytical methods compared to the numerical model, a ratio R was introduced in the previous section. The value of R defines the behaviour of the analytical method compared to the reference, *i.e.* the numerical model. Generally a tolerance interval of 10% is considered around $R = 1$ and the ratios could be interpreted as:

$$R \begin{cases} < 1 & \text{Conservative} \\ = 1 & \text{Perfect agreement} \\ > 1 & \text{Unsafe} \end{cases}$$

To compare each method, it is necessary to compare the methods for the 3 different materials grades and the 3 different considered length (0m long corresponding to the resistance of cross-section, 3,5m long and 7m long). The letters A, B, C and D of the 4 different figures refer to eccentricities of charge $e = [0; 0,05; 0,1; 0,2]$ m of Fig. 4.2. To compare the methods between each other, box plots were used but attention must be paid to the small number of specimens (13 specimens per boxplot).

Normal-strength materials: test 1

Firstly, let's consider the plastic interaction curve, the AS/NZ Standard (method 3) and SMARTCoCo project (method 4) do not consider reduction of the plastic interaction curve, therefore the ratios are the same than method 1. Fig. 4.4 to 4.7 show clearly that the methods without reduction of materials, *i.e.* 1, 3 and 4, give the best agreement with the numerical results. For some types of sections, the ratios R are over 100%, which could give an unsafe overestimation of the resistance of the cross-section but stay in the interval of 10%. For the adapted method of the BC4:2015 design guide (method 2), results are always safe but may be considered as over conservative compared to other methods.

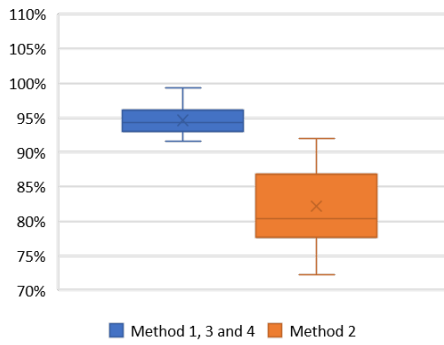


Figure 4.4: Point A: Distribution of the ratios for the plastic interaction curve for test 1.

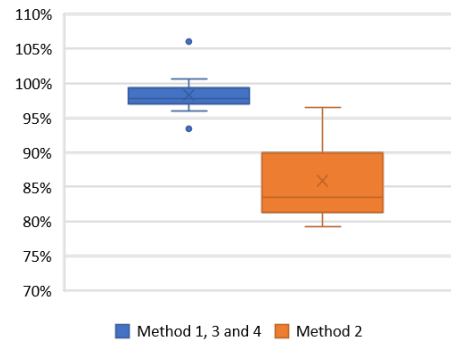


Figure 4.5: Point B: Distribution of the ratios for the plastic interaction curve for test 1.

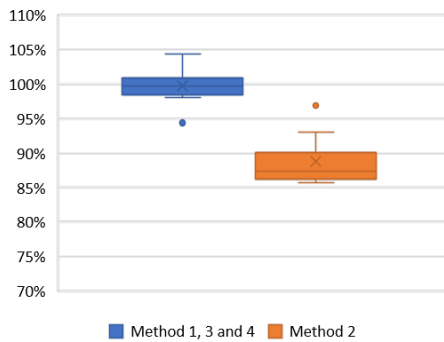


Figure 4.6: Point C: Distribution of the ratios for the plastic interaction curve for test 1.

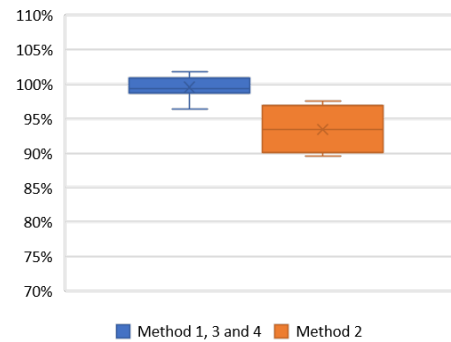


Figure 4.7: Point D: Distribution of the ratios for the plastic interaction curve for test 1.

Secondly, for the reduced interaction curve of 3,5 m long columns, it could be observed in Fig. 4.8 to 4.11 that, as for plastic interaction diagram, the method 2 is too conservative compared to method 1 and 3. The method of SMARTCoCo project (method 4) is also too conservative compared to the others. The method with the better conformity with numerical simulations is the AS/NZ Standard (method 3), which overestimate the resistance for point B, C and D of the interaction curve but stay under 110%. Moreover, methods 1 and 3 provide concentrate results compared to other methods.

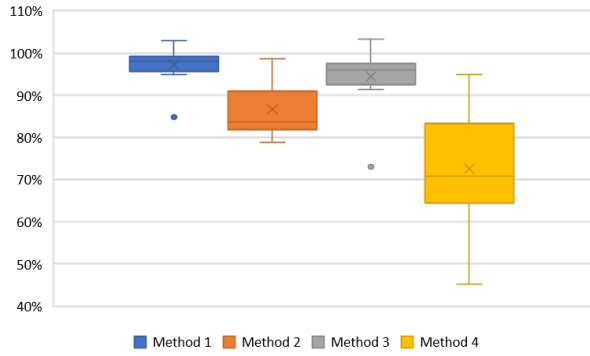


Figure 4.8: Point A: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 1.

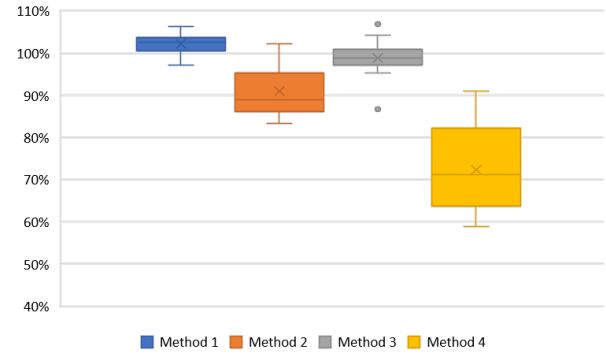


Figure 4.9: Point B: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 1.

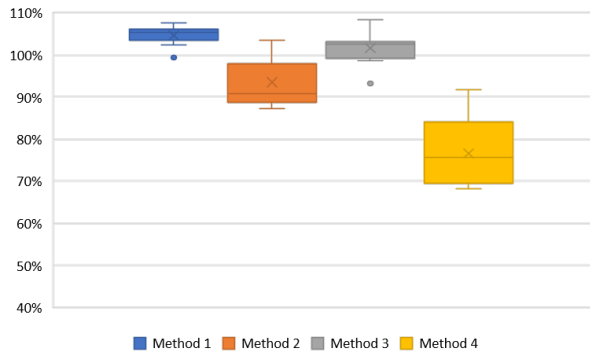


Figure 4.10: Point C: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 1.

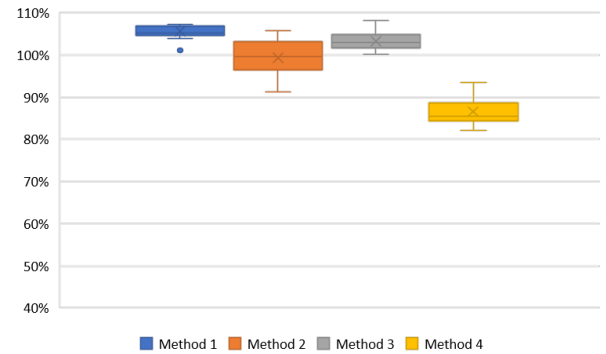


Figure 4.11: Point D: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 1.

Thirdly, for the reduced interaction curve of 7 m long columns, Fig. 4.12 to 4.15 show the same conclusion as for columns of 3,5 m long, the methods with the best agreement to the numerical values are the method 1 and 3. However in this case, the Simplified Method of EN 1994-1-1 (method 1) does not overestimate the resistance of the column and present a quite better precision than the AS/NZ Standard (method 3) compared to the numerical analysis. Results given by method 1 are also more concentrated than method 3.

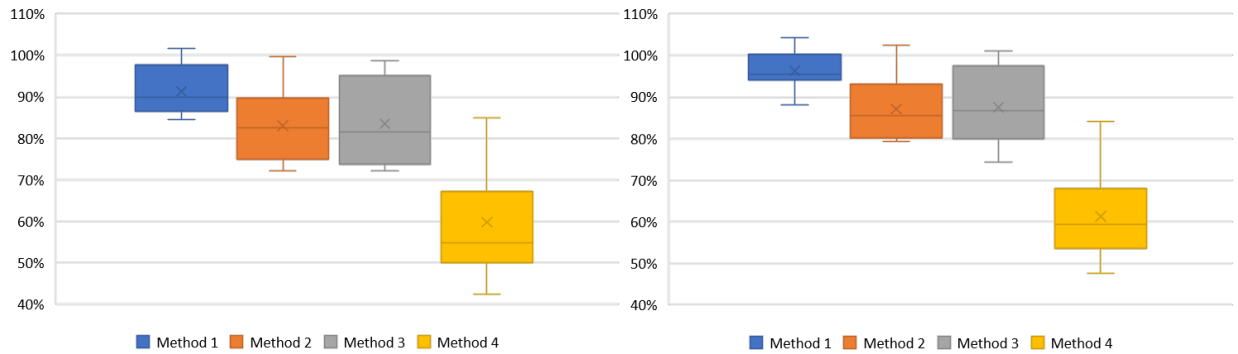


Figure 4.12: Point A: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 1.

Figure 4.13: Point B: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 1.

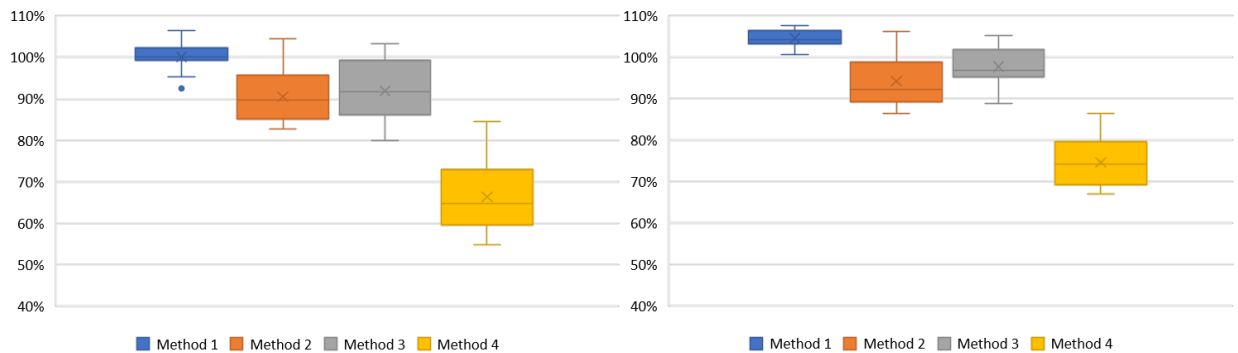


Figure 4.14: Point C: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 1.

Figure 4.15: Point D: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 1.

High-strength materials: test 2

Firstly, let's consider the plastic interaction curve, due to the lack of data, only point A and B of the interaction curve will be analysed. Fig. 4.16 and 4.17 show that with the increase of resistance of the materials, the adapted simplified method of BC4:2015 (method 2) is really too conservative. This was expected because coefficients η and λ decrease with the increase of the cylindrical compressive strength of concrete f_{ck} which decreases more and more the reduced f_{ck} . However, for the methods 1 and 3, the agreement with the numerical values is quite good and safe.

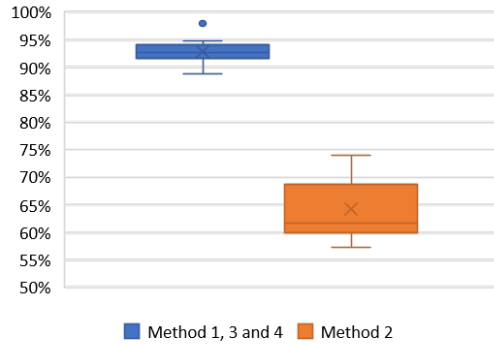


Figure 4.16: Point A: Distribution of the ratios for the plastic interaction curve for test 2.

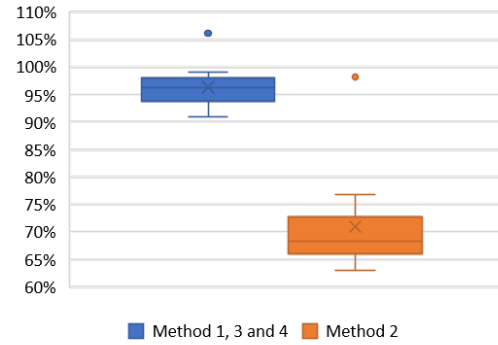


Figure 4.17: Point B: Distribution of the ratios for the plastic interaction curve for test 2.

Secondly, for the reduced interaction curve of 3,5 m long columns, due to the lack of data, the point D of the interaction curve is not presented. Fig 4.18 to Fig. 4.20 show the same conclusion as previously, methods 2 and 4 are over-conservative and only methods 1 and 3 give quite good agreement but the Simplified Method of EN 1994-1-1 (method 1) gives the best precision compared to the numerical values.

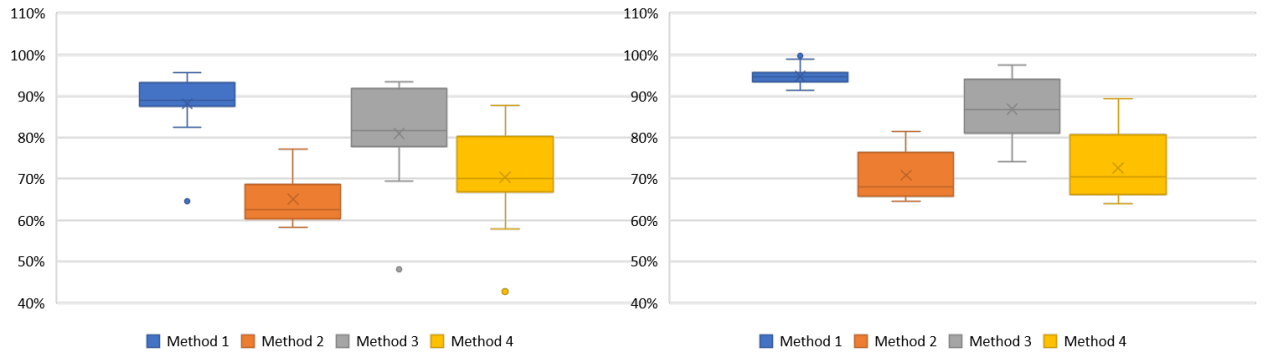


Figure 4.18: Point A: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 2.

Figure 4.19: Point B: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 2.

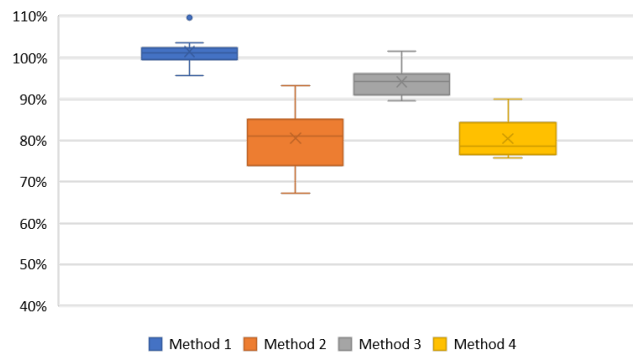


Figure 4.20: Point C: Distribution of the reduced interaction curve of 3,5 m long columns for test 2.

Thirdly, for the reduced interaction curve of 7 m long columns, Fig. 4.21 to 4.24 show a good accordance of method 1 unlike other methods. However, for point D of the interaction curve, the first method unsafely overestimate the resistance of the column. On the other hand, this point is not in the 20% limit eccentricity of load, therefore this eccentricity of charge will not occur in the vast majority of cases.

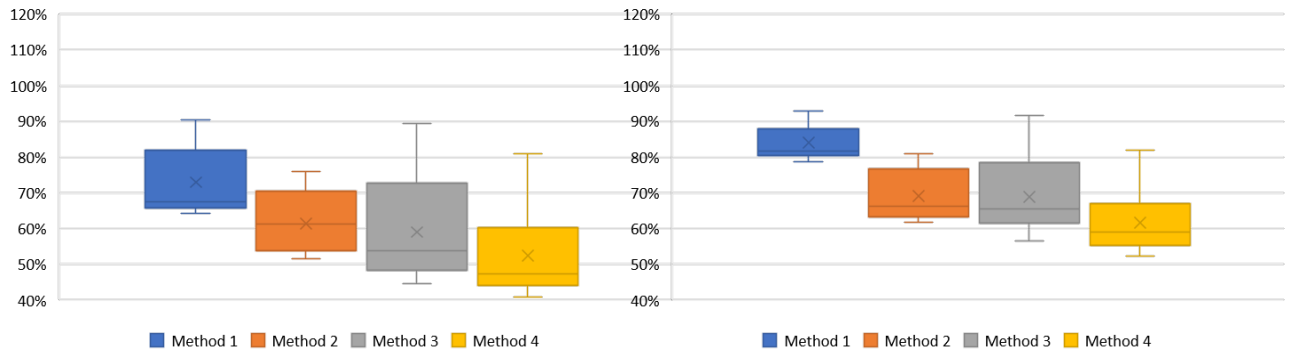


Figure 4.21: Point A: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 2.

Figure 4.22: Point B: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 2.

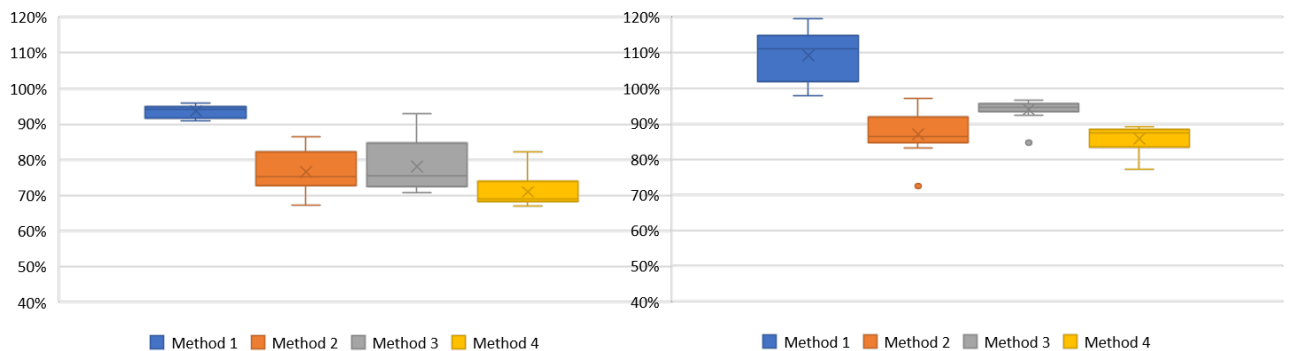


Figure 4.23: Point C: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 2.

Figure 4.24: Point D: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 2.

High-strength materials: test 3

For high-strength materials of test 3, the conclusions are the same as for test 2 and the different graphs are summarised in Appendix B.1.1.

4.3.2 Influence of parameters $\bar{\lambda}$, δ and material compatibility for FEC

Influence of $\bar{\lambda}$

In this analysis, specimens FEC2, FEC5 and FEC4 will be compared according to length, material grades and analytical methods. Specimens FEC2 and FEC5 are slender and FEC4 is stockier. The increase of the length induces a greater relative slenderness as well as the increase of the material grades. Compared to the Simplified Method of EN 1994-1-1 (method 1), the method 2 decreases the relative slenderness, due to the reduction of the concrete strength f_{ck} , in contrast to the methods 3 and 4 which increase parameter $\bar{\lambda}$.

Firstly, consider the same material grades and the same length of the column, in this case, test 1 and 7 m long, respectively. Table 4.6, Fig. 4.25 and 4.26 show that, for the same analytical method, the increase of the relative slenderness decreases the agreement of the analytical methods compared to numerical values. Another observation concerns the method 2, although the relative slenderness is higher than the method 1, the agreement with the numerical values is lower.

		Method 1	Method 2	Method 3	Method 4
FEC2	$\bar{\lambda}$	1,39	1,32	1,53	1,96
	Point A	84%	83%	72%	50%
	Point B	91%	88%	80%	58%
	Point C	95%	91%	85%	63%
	Point D	104%	99%	94%	74%
FEC4	$\bar{\lambda}$	0,66	0,63	0,72	1,18
	Point A	99%	92%	94%	64%
	Point B	102%	95%	97%	65%
	Point C	104%	97%	100%	68%
	Point D	106%	99%	102%	74%

Table 4.6: Ratios R for columns FEC2 and FEC4 of 7 m long and for normal-strength materials (test 1).

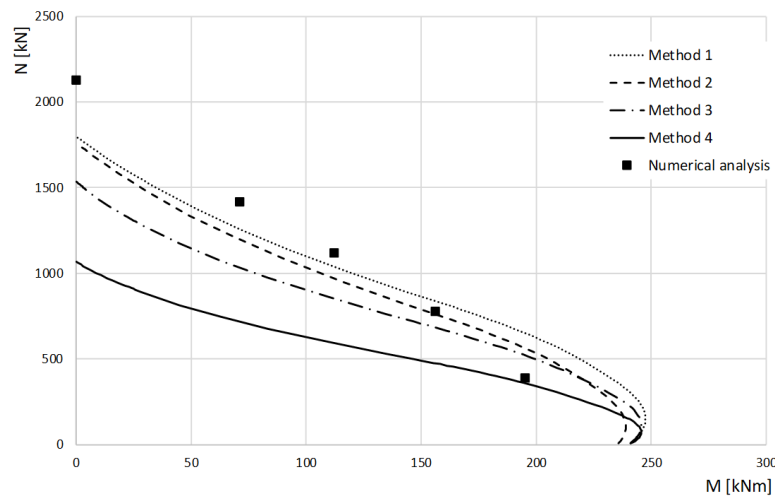


Figure 4.25: Comparison of the analytical methods for specimen FEC2 (slender column) considering test 1 and 7m long column.

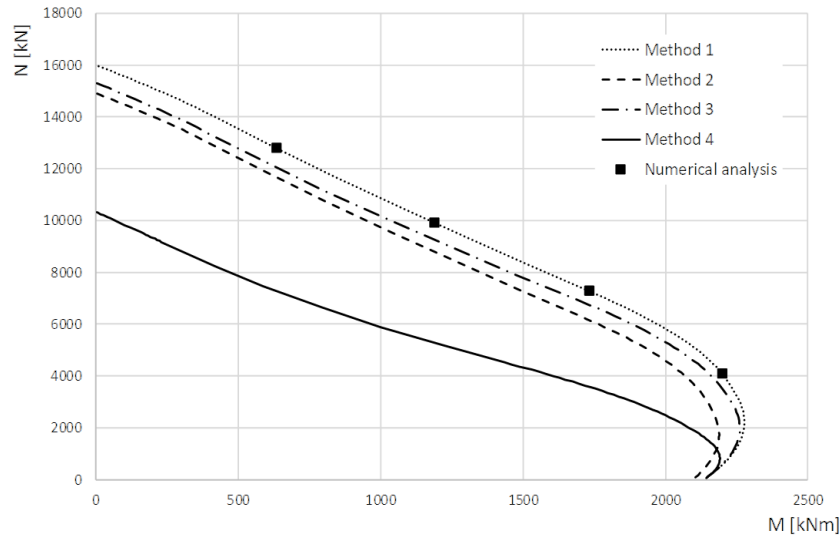


Figure 4.26: Comparison of the analytical methods for specimen FEC4 (stockier column) considering test 1 and 7m long column.

Secondly, consider the same analytical methods and the same length of the column: the Simplified Method of EN 1994-1-1 (method 1) and the AS/NZ Standard method (method 3) with a column length of 7 m. In this case, FEC5 is considered instead of FEC2 because $\bar{\lambda} > 2$, which is the maximum limit imposed by EN 1994-1-1 Simplified Method. Tables 4.7 and 4.8 show that the relative slenderness increases with the material grades and the ratios R decrease with the increase of relative slenderness and, obviously, the material grades as well. As a reminder, test 1 is concrete C30 and steel S355, test 2 is C90 and S500 and test 3 is C120 and S600.

		Test 1	Test 2	Test 3
FEC5	$\bar{\lambda}$	1,05	1,56	1,73
	Point A	90%	67%	62%
	Point B	95%	81%	79%
	Point C	99%	91%	91%
	Point D	104%	111%	112%
FEC4	$\bar{\lambda}$	0,66	0,84	0,91
	Point A	99%	84%	79%
	Point B	102%	88%	83%
	Point C	104%	94%	91%
	Point D	106%	102%	101%

Table 4.7: Ratios R of method 1 for columns FEC5 and FEC4 of 7 m long and for different material grades.

		Test 1	Test 2	Test 3
FEC5	$\bar{\lambda}$	1,14	1,35	1,47
	Point A	82%	54%	47%
	Point B	87%	66%	62%
	Point C	92%	76%	73%
	Point D	98%	96%	94%
FEC4	$\bar{\lambda}$	0,72	0,97	1,08
	Point A	94%	73%	66%
	Point B	97%	77%	70%
	Point C	100%	83%	77%
	Point D	102%	92%	89%

Table 4.8: Ratios R of method 3 for columns FEC5 and FEC4 of 7 m long and for different material grades.

Influence of δ

For this analysis, the specimens are chosen to have a large difference of the steel contribution coefficient δ but with a similar relative slenderness $\bar{\lambda}$. Therefore, specimens FEC10 and FEC13 were chosen. In the same way than for the relative slenderness, the influence of the analytical methods and the material grades will be shown. The steel contribution factor is defined as $\delta = \frac{A_a f_{yd}}{N_{pl,Rd}}$ therefore methods 3 and 4 do not influence δ compared to the method 1.

The adapted method of the BC4:2015 design guide (method 2), which reduced the compressive cylindric strength of the concrete f_{ck} , is really impacted by the steel contribution ratio because the higher the coefficient δ , the lower the contribution of concrete. This is shown in Table 4.9, the influence of δ is significant between FEC13 (high δ) and FEC10 (low δ) for a same test. No correlation could be demonstrated for the other methods on the influence of the coefficient δ to the ratios R .

		FEC13			FEC10		
		Test 1	Test 2	Test 3	Test 1	Test 2	Test 3
	δ	0,71	0,64	0,62	0,37	0,31	0,28
Resistance of cross-section	Point A	92%	74%	72%	78%	59%	58%
	Point B	95%	77%	75%	83%	65%	67%
	Point C	97%	79%	78%	86%	/	/
	Point D	97%	/	/	/	/	/
L=3,5 m	$\bar{\lambda}$	0,43	0,5	0,54	0,45	0,48	0,52
	Point A	99%	77%	75%	81%	60%	59%
	Point B	102%	81%	78%	86%	65%	64%
	Point C	103%	84%	82%	89%	/	/
	Point D	103%	89%	89%	/	/	/
L=7 m	$\bar{\lambda}$	0,86	0,99	1,08	0,9	0,97	1,04
	Point A	100%	73%	69%	75%	54%	50%
	Point B	102%	79%	75%	82%	63%	60%
	Point C	105%	83%	80%	86%	75%	75%
	Point D	106%	87%	86%	90%	/	90%

Table 4.9: Representation of the influence of coefficient δ for different material grades and different column length for method 2.

Influence of materials compatibility

To observe the influence of the materials compatibility, specimen FEC8 was tested with compatible and incompatible materials according to Table 2.6. As explained in previous section, tests 1,2,3,4,5 and 6 were performed for column FEC8 and only tests 4 and 5 are composed of incompatible materials. The material grades influence coefficient δ and $\bar{\lambda}$ for a same section, analytical method and length. For methods 1, 3 and 4, it could be observed that if the tests are ranked according to the relative slenderness, the ratios R decrease with the increase of $\bar{\lambda}$, as it is shown in Table 4.10 for method 1 (Tables for methods 3 and 4 are present in Appendix B.1.2). Moreover the incompatible material tests (4 and 5) has no effects on the accordance of the analytical method compared to numerical values.

		Test 1	Test 4	Test 5	Test 2	Test 6	Test 3
	δ	0,34	0,42	0,47	0,21	0,24	0,19
Resistance of cross-section	Point A	93%	93%	93%	91%	91%	91%
	Point B	97%	97%	98%	94%	93%	93%
	Point C	99%	99%	99%	98%	98%	98%
	Point D	/	101%	102%	/	/	/
L=3,5 m	$\bar{\lambda}$	0,48	0,51	0,53	0,6	0,61	0,64
	Point A	95%	95%	96%	88%	88%	85%
	Point B	101%	100%	100%	92%	92%	95%
	Point C	104%	104%	104%	100%	100%	98%
	Point D	104%	106%	108%	110%	110%	114%
L=7 m	$\bar{\lambda}$	0,96	1,03	1,07	1,2	1,22	1,29
	Point A	86%	84%	85%	65%	66%	60%
	Point B	94%	93%	94%	79%	80%	74%
	Point C	100%	99%	100%	95%	96%	93%
	Point D	106%	105%	106%	118%	116%	120%

Table 4.10: Ranking of the tests according to the increase of the relative slenderness for specimen FEC8 calculated with method 1 and for different column lengths.

However, for method 2, a ranking according to the relative slenderness has no effects on the trend in the evolution of the ratios R . A ranking according to the steel contribution coefficient, as shown in Table 4.11, gives a decrease in the ratios values with the decrease of δ . Once again, the materials compatibility have no effects on the agreement of the analytical method compared to numerical values.

		Test 5	Test 4	Test 1	Test 6	Test 2	Test 3
	δ	0,52	0,47	0,39	0,35	0,31	0,3
Resistance of cross-section	Point A	78%	77%	77%	59%	58%	57%
	Point B	81%	81%	80%	65%	66%	70%
	Point C	85%	85%	86%	/	/	/
	Point D	/	/	/	/	/	/
L=3,5 m	$\bar{\lambda}$	0,51	0,49	0,45	0,5	0,49	0,52
	Point A	96%	80%	80%	60%	59%	57%
	Point B	84%	84%	85%	65%	64%	64%
	Point C	87%	88%	88%	/	/	/
	Point D	/	/	108%	/	/	/
L=7 m	$\bar{\lambda}$	0,96	1,03	1,07	1,2	1,22	1,29
	Point A	73%	72%	73%	53%	52%	49%
	Point B	81%	80%	81%	63%	62%	59%
	Point C	85%	84%	86%	76%	75%	75%
	Point D	89%	89%	90%	88%	/	90%

Table 4.11: Ranking of the tests according to the decrease of the steel contribution ratio for specimen FEC8 calculated with method 2 and for different column lengths.

4.3.3 Comparison of the different methods for PEC

As well as for the fully encased sections (FEC), the analysis of the partially encased sections for the comparison of the different analytical methods will consider the different material grades and lengths. To compare the methods between each other, unlike for FEC sections, only 5 specimens were studied, so boxplots were not conceivable.

Normal-strength materials: test 1

Firstly, let's consider the plastic interaction curve, in the same way as for FEC, the methods 3 and 4 do not consider reduction of the plastic interaction curve, therefore the ratios R are the same as method 1. Fig. 4.27 to 4.30 show that method 2 is over conservative compared to methods 1,3 and 4. The latter methods present a good conformity with the numerical analysis for all points of the interaction curve.

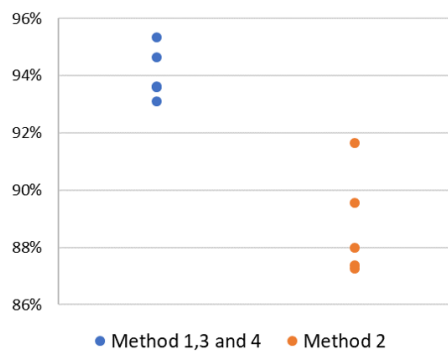


Figure 4.27: Point A: Distribution of the ratios for the plastic interaction curve for test 1.

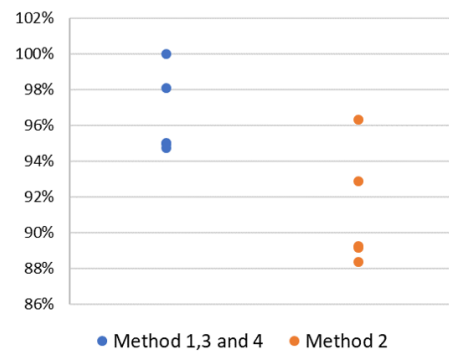


Figure 4.28: Point B: Distribution of the ratios for the plastic interaction curve for test 1.

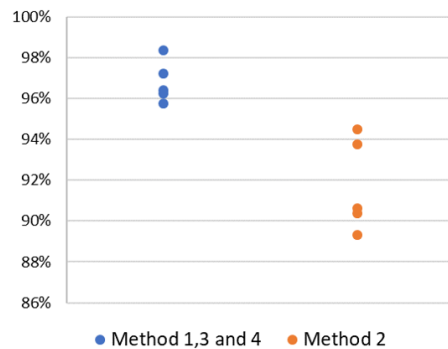


Figure 4.29: Point C: Distribution of the ratios for the plastic interaction curve for test 1.

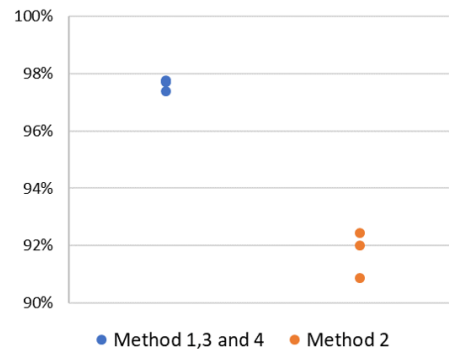


Figure 4.30: Point D: Distribution of the ratios for the plastic interaction curve for test 1.

Secondly, for the reduced interaction curve of 3,5 m long columns, it could be observed on Fig. 4.31 that method 1, 3 and 4 present the same agreement with the numerical model and stay in the interval of 10%. However, on Fig. 4.32 to 4.34, method 4 is close from $R = 1$ and on the safe side, unlike methods 1 and 3, which are also close to $R = 1$ but on the unsafe side and stay in the interval of 10%.

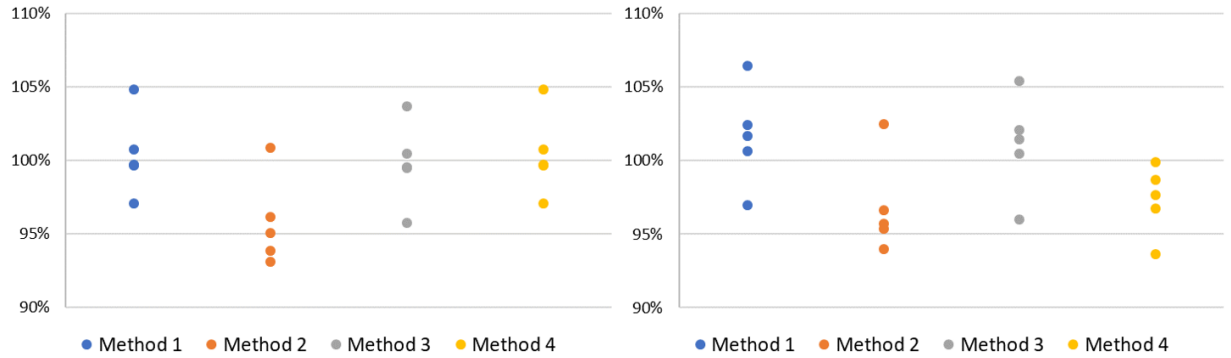


Figure 4.31: Point A: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 1.

Figure 4.32: Point B: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 1.

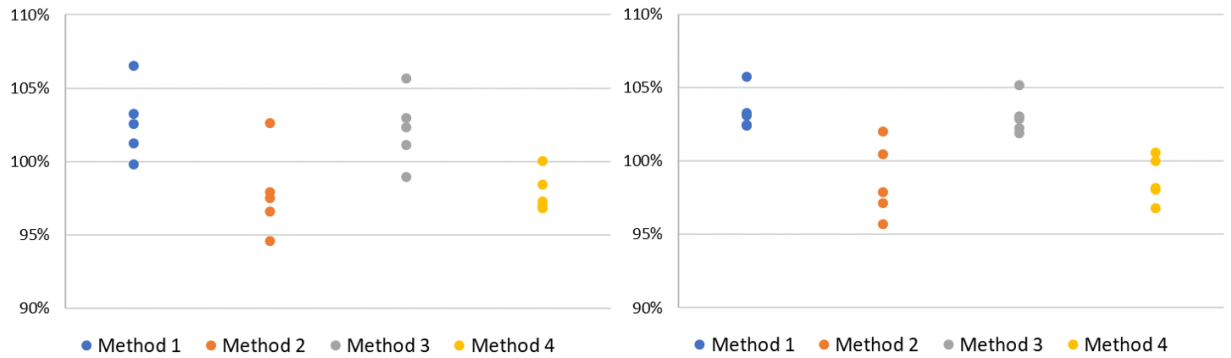


Figure 4.33: Point C: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 1.

Figure 4.34: Point D: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 1.

Thirdly, for the reduced interaction curve of 7 m long columns, Fig. 4.35 shows, as for 3,5 m long column, that methods 1, 3 and 4 give similar affinity with the numerical analysis. On Fig. 4.36 to 4.38, method 3 and 4 are close to $R = 1$ and on the safe side but method 4 gives more concentrated results.

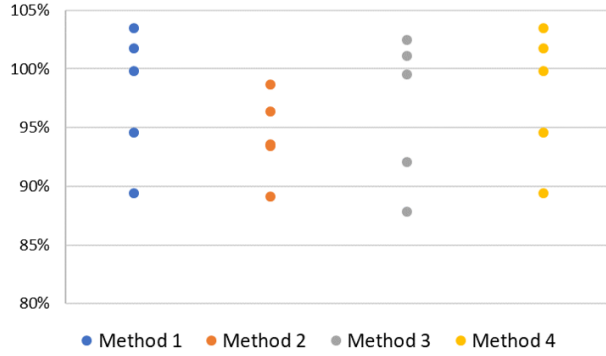


Figure 4.35: Point A: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 1.

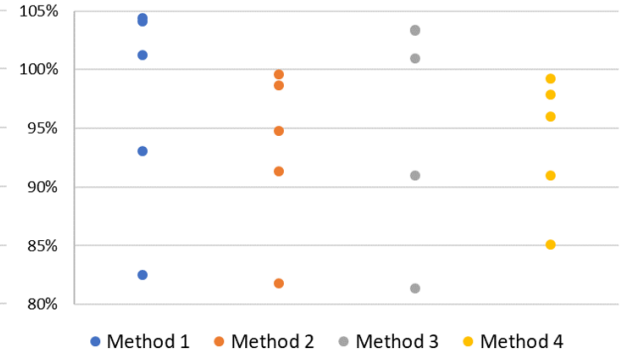


Figure 4.36: Point B: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 1.

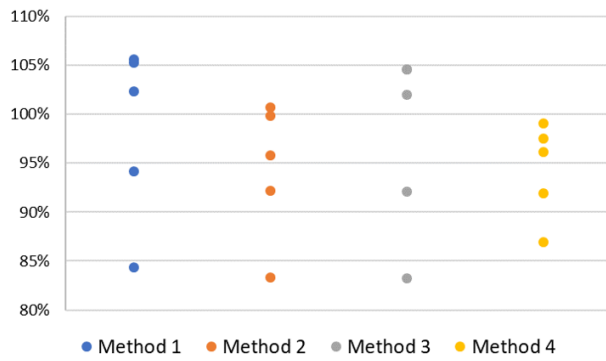


Figure 4.37: Point C: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 1.

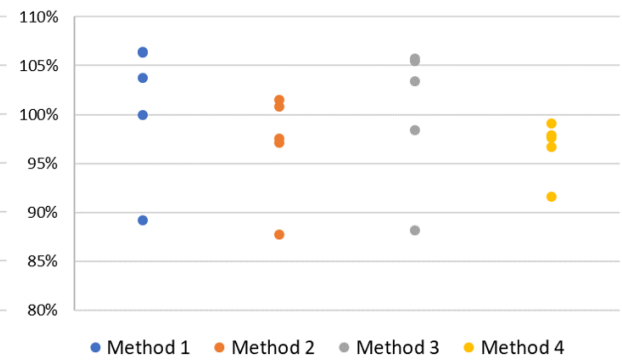


Figure 4.38: Point D: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 1.

High-strength materials: test 2

Firstly, for plastic interaction curve, due to the lack of data, only points A, B and C of the interaction curve are analysed. Fig. 4.42 to 4.45 show that with the increase of the material strength, the method 2 gives results that are really too conservative compared to other methods. Methods 1, 3 and 4 give results with a good agreement to the numerical analysis.

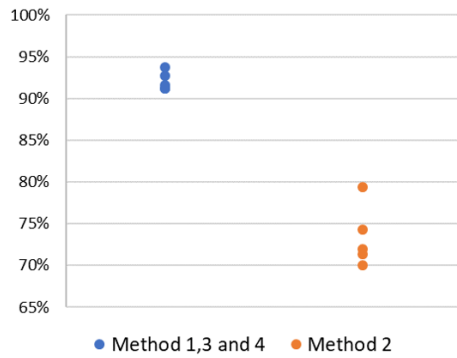


Figure 4.39: Point A: Distribution of the ratios for the plastic interaction curve for test 2.

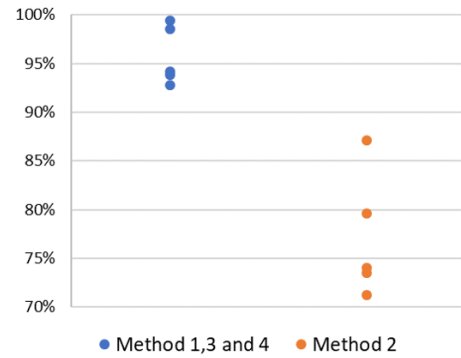


Figure 4.40: Point B: Distribution of the ratios for the plastic interaction curve for test 2.

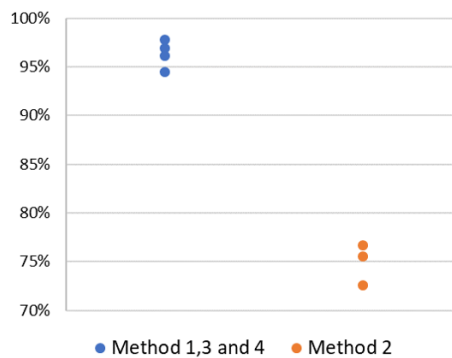


Figure 4.41: Point C: Distribution of the ratios for the plastic interaction curve for test 2.

Secondly, for the reduced interaction curve of 3,5 m long columns, Fig. 4.31 to 4.32 show that, as for plastic interaction curve, the method 2 is too conservative. Method 1, 3 and 4 are close from each other but the method 1 and 3 give result closer to $R = 1$ than method 4, with an overestimation of the resistance under 10%.

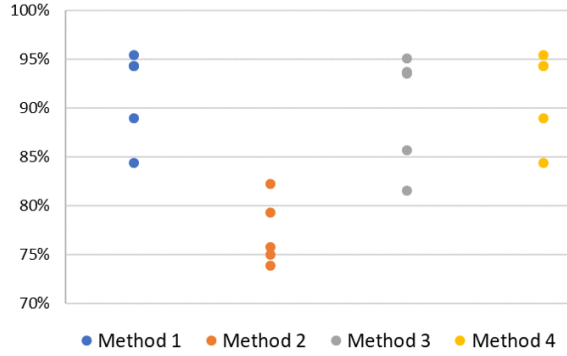


Figure 4.42: Point A: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 2.

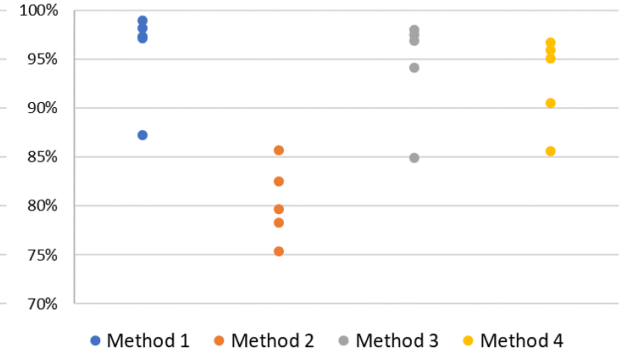


Figure 4.43: Point B: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 2.

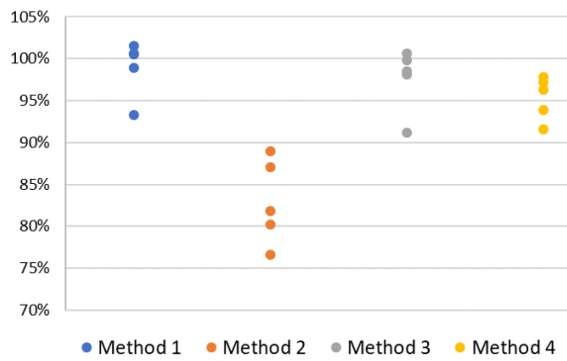


Figure 4.44: Point C: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 2.

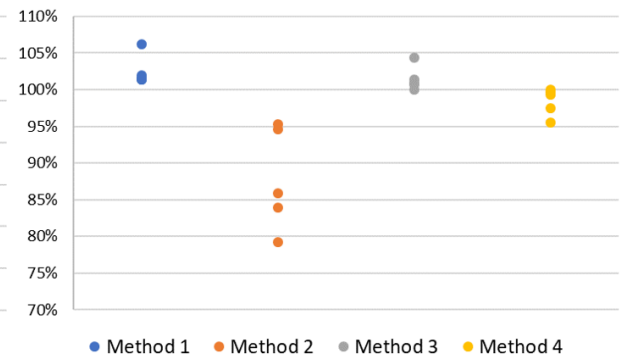


Figure 4.45: Point D: Distribution of the ratios for the reduced interaction curve of 3,5 m long columns for test 2.

Thirdly, for the reduced interaction curve of 7 m long columns, Fig. 4.46 shows that method 1, 3 and 4 are close from each other. Nonetheless, Fig. 4.48 and 4.49 show a better agreement to numerical values for methods 1 and 3. As well as for the other cases, method 2 is too conservative.

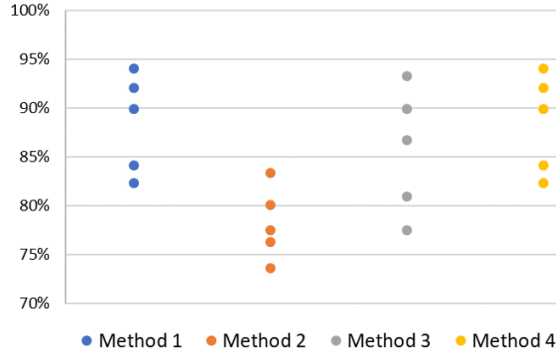


Figure 4.46: Point A: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 2.

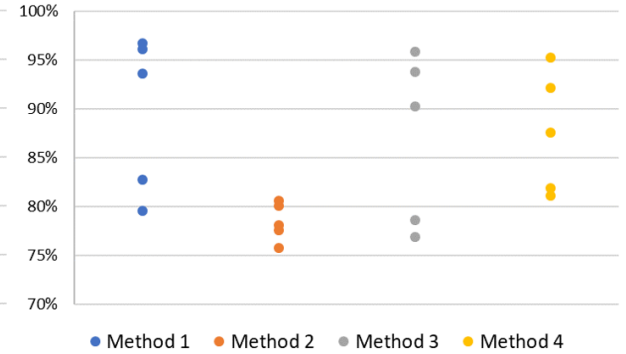


Figure 4.47: Point B: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 2.

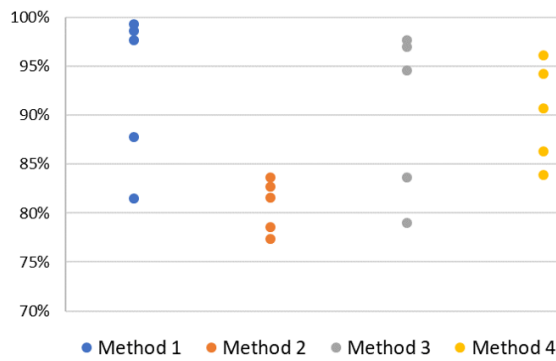


Figure 4.48: Point C: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 2.

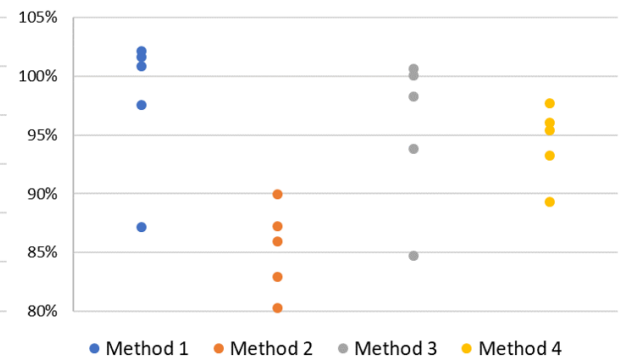


Figure 4.49: Point D: Distribution of the ratios for the reduced interaction curve of 7 m long columns for test 2.

High-strength materials: test 3

For high-strength materials of test 3, the conclusions are the same as for test 2 and the results are summarised in Appendix B.2.1.

4.3.4 Influence of parameters $\bar{\lambda}$ and δ for PEC

Influence of $\bar{\lambda}$

In the same way as for FEC, 2 specimens will be compared according to length, material grades and analytical methods. Specimens PEC1 and PEC5 are chosen because the first is slender then the last and the coefficients δ of the specimens are close.

Firstly, to continue in the same way than for FEC, consider the same material grades and the same length of the column, test 1 and 7 m long, respectively. Table 4.12 shows the same conclusion than for FEC, the increase of the relative slenderness, induces a slightly decrease of the agreement of the analytical methods compared to numerical values. It could also be observed that even if method 2 gives a lower slenderness for a same specimen, the ratios R are slightly lower than the other methods.

		Method 1	Method 2	Method 3	Method 4
PEC1	$\bar{\lambda}$	1,55	1,51	1,57	1,62
	Point A	95%	94%	92%	95%
	Point B	93%	91%	91%	91%
	Point C	94%	92%	92%	92%
	Point D	100%	98%	98%	98%
PEC5	$\bar{\lambda}$	0,29	0,28	0,29	0,55
	Point A	100%	93%	100%	100%
	Point B	101%	95%	101%	99%
	Point C	102%	96%	103%	99%
	Point D	104%	97%	103%	99%

Table 4.12: Ratios R for columns PEC1 and PEC5 of 7 m long and for normal-strength materials (test 1).

Secondly, consider the same analytical methods and the same length of the column to compare the material grades. With the increase of the material grades, the relative slenderness of specimen FEC1 becomes higher than 2. Therefore a column length of 3,5 m will be used and the results of methods 1 and 3 will be presented. Tables 4.13 and 4.14 show that the $\bar{\lambda}$ increase with the increase of the material grades and for the two methods, the increase of $\bar{\lambda}$ induces a decrease of the agreement of analytical methods with the numerical values. However, the influence of $\bar{\lambda}$ could not be insured.

		Test 1	Test 2	Test 3
FEC1	$\bar{\lambda}$	0,77	1	1,11
	Point A	105%	89%	83%
	Point B	106%	97%	93%
	Point C	107%	101%	98%
	Point D	106%	106%	105%
FEC5	$\bar{\lambda}$	0,14	0,19	0,21
	Point A	100%	95%	93%
	Point B	101%	97%	95%
	Point C	101%	99%	97%
	Point D	102%	101%	101%

Table 4.13: Ratios R of method 1 for columns FEC1 and FEC5 of 3,5 m long and for different materials.

		Test 1	Test 2	Test 3
FEC1	$\bar{\lambda}$	0,79	1,04	1,15
	Point A	104%	86%	79%
	Point B	105%	94%	89%
	Point C	106%	98%	94%
	Point D	105%	104%	103%
FEC5	$\bar{\lambda}$	0,15	0,2	0,22
	Point A	100%	95%	94%
	Point B	100%	97%	96%
	Point C	101%	99%	98%
	Point D	102%	101%	101%

Table 4.14: Ratios R of method 3 for columns FEC1 and FEC5 of 3,5 m long and for different materials.

Influence of δ

For partially encased sections, the contribution of steel is much greater than for fully encased sections and the tested specimens have large values of δ but none of them have a relatively small δ to allow a comparison based on this parameter. Therefore the analysis of the influence of δ could not be performed with the available data.

4.4 Interpretation of the results

This parametric study compares four analytical methods with numerical model:

- Method 1 consists of the adapted EN 1994-1-1 Simplified Method for high-strength materials;
- Method 2 is the modified BC4:2015 design guide for concrete encased sections considering reduction of the concrete compressive strength f_{ck} ;
- Method 3 adjusts the AS/NZ Standard and considers a reduced effective flexural rigidity $(EI)_{eff,II}$;
- Method 4 adapts the SMARTCoCo project design method to consider high-performance materials and reduces the effective flexural rigidity $(EI)_{eff,II}$ compared to Method 3.

In case of fully encased sections, methods 1 and 3 have the best agreement with SAFIR model. However method 1 overestimates the resistance of the column but R stay in the 10% tolerance. Method 2 and 4 generate a design for composite columns too conservative.

For partially encased sections, method 4 produces a good and safe design compared to the numerical analysis. Methods 1 and 3 give satisfying accordance with numerical values as well, and stay in the interval of 10%. Method 2 is too conservative compared to other analytical methods. The study also exposed, that partially encased sections have a better general agreement with numerical analysis than fully encased section for similar relative slenderness and material grades.

The relative slenderness has an important influence for fully encased sections. A high $\bar{\lambda}$ drives to a drastically reduction of the ratio R . However, $\bar{\lambda}$ has a slight effect on the agreement between analytical methods and numerical values for partially encased sections.

Method 2 is quite sensitive to the value of δ . A section with high δ gives better accordance to numerical values. On the other hand, no link could be found between δ and the values of ratios R for the other methods.

The sections composed of incompatible materials, according to Table 2.6, has a good agreement between the analytical methods and the numerical analysis. Therefore, no link between compatible materials and ratio R was found.

Chapter 5

Conclusion

The constant increase in the height of buildings and spans, coupled with the climate change issues, are driving the construction sector to reduce the weight of buildings, which leads to a lower quantity of materials and carbon footprint. Therefore, the adoption of high-performance materials and composite structures will be more and more recurrent in the future. Hence the need to adapt/modify the Simplified Method of EN 1994-1-1 to authorize the use of high-strength materials.

5.1 Solution developed

Based on design methods for steel-concrete composite column in the literature, four methods were chosen to adjust the EN 1994-1-1 Simplified Method established on Plastic Design Method. These four approaches of design were adapted to allow the use of high-strength materials (concrete and steel up to C120 and S600 respectively) and to approve the design of fully and partially encased sections.

1. Method 1: Simplified Method of EN 1994-1-1;
2. Method 2: Method of BC4:2015 corresponding to an extension of EN 1994-1-1;
3. Method 3: AS/NZ Standard design method;
4. Method 4: Adapted design method developed by the SMARTCoCo project.

In order to compare these analytical design methods, a sensitive study was carried out. Therefore, a FEM 2D-model, developed with SAFIR software, was created to serve as a base for the comparison. However, the model was calibrated with a limited number of tests due to a lack of experimental testing, in the open literature. The chosen experimental investigations were performed by Kim *et al.* on concrete encased sections using steel up to 800 MPa and concrete up to 100 MPa. The tests results demonstrates a reliable structural performance of the specimens. The comparison between the models and the tests reveals a strong conformity and validates the numerical analysis and its use for the parametric study.

This study considered 13 specimens for fully encased sections along strong axis and 6 along weak axis; 5 specimens for partially encased sections along strong axis and 5 along weak axis. Each specimen was tested with three different lengths and three different material grades. It should be noted that the analysis was only executed for eccentricities along the strong axis.

The sensitive study shows that the method 2 is too conservative compared to the others for all specimens. For fully encased sections, methods 1 and 3 present the best agreement according to numerical simulations. As well as method 2, method 4 is too conservative. In the case of partially encased sections, method 4 generates good and safe accordance to numerical values. Methods 1 and 3 produce as well satisfying compliance. The study also highlights that partially encased columns produce a larger agreement of the analytical methods according to numerical results than fully encased sections.

The analysis of the parameters has demonstrated that fully encased specimens are widely dependant on the relative slenderness $\bar{\lambda}$, unlike partially encased sections. The influence of steel contribution ratio is important for specimens designed according to method 2. The other methods do not present differences of the ratio R with a variation of δ . Finally, no correlation between the material compatibility, according to Table 2.6, and the value of ratio R was found.

To conclude, the adapted method of the AS/NZ Standard (method 3) presents the best compliance with the SAFIR simulations in comparison with the other analytical methods. For fully encased sections, the current limit of the relative slenderness $\bar{\lambda}$ should be modify to preserve the economy of composite columns.

5.2 Limitations and future work

The mainly limitations concern the FEM model, a better behaviour of the numerical model could be obtained using non-linear stress-strain relation for the materials and 3D-elements instead of 2D-elements. Moreover, the study does not examine the behaviour of the column along to combined bi-axial bending and axial compression.

As previously explained, the calibration of the model was based on a restraint number of experimental tests. Thus, a test campaign should be performed on a larger number of specimens and taking into account hot rolled profiles using high-strength steel instead of groove welded steel plates such as in tests executed by Kim *et al.*

Finally, a more in-depth numerical study should be achieved for fully and partially encased sections to establish the necessity of a compatibility rule of the materials, as the Building and Construction Authority provides in the BC4:2015 report. Moreover, a larger set of data could contribute to a better interpretation of the parameters $\bar{\lambda}$ and δ and thus limit them to improve the economy of composite columns.

Bibliography

- [1] Brian Uy, Stephen J. Hicks, Won-Hee Kang, Huu-Tai Thai and Farhad Aslani. *Australasian advances in steel-concrete composite bridge and building structures*, presented at the 8th International Conference on Steel and Aluminium Structures, Hong Kong, China, 2016.
- [2] Brian Uy. *High Strength Steel-Concrete Composite Columns: Applications and Design*, presented at the ASCE Structures Congress, New-York, United States, 2005.
- [3] Batirama's website. <https://www.batirama.com/produit>. Accessed : 23-05-2019.
- [4] Batiproduits' website. <https://www.batiproduits.com/fiche/produits>. Accessed : 23-05-2019.
- [5] CEN (European Committee for Standardisation). 2004. *Eurocode 4: Design of composite steel and concrete structures. Part 1-1: General rules and rules for buildings*. Brussels, Belgium.
- [6] CEN (European Committee for Standardisation). 2007. *Eurocode 3: Design of steel structures. Part 1-12: Additional rules for the extension of EN 1993 up to steel grades S 700*. Brussels, Belgium, p. 5.
- [7] CEN (European Committee for Standardisation). 2005. *Eurocode 2: Design of concrete structures. Part 1-1 : General rules and rules for buildings*. Brussels, Belgium.
- [8] Fédération internationale du béton. 2010. *fib Model Code*. Lausanne, Switzerland, vol. 1 and 2.
- [9] Prof. Richard Liew and al. 2015. *Design Guide for Concrete Filled tubular Members with High Strength Materials - An Extension of Eurocode 4 Method to C90/105 Concrete and S550 Steel*. Building and Construction Authority, Singapore.
- [10] Australian/New Zealand Standard. 2017. *Composite structures—Composite steel-concrete construction in buildings*. Sydney, Australia.
- [11] Pisey Keo and al. 2015. *Simplified design method for slender hybrid columns - Instability analysis of hybrid columns*. Smartcoco project, Rennes, France.
- [12] Uliege website. https://www.uee.uliege.be/cms/c_2383458/fr/safir. Accessed : 30-05-2019.
- [13] James M. Ricles and Shannon D. Paboojian. *Seismic performance of steel-encased composite columns*, Journal of Structural Engineering, Volume 120, Issue 8 (1994), pp. 2474-2494.

- [14] Ben Kato. *Column Curves of Steel-Concrete Composite Members*, Journal of Construction Steel Research, Volume 39, Issue 2 (1996), pp. 121-135.
- [15] Reinhard Bergmann, Ram Puthli, Olivier Fleischer. *Behavior of composite columns using high strength steel sections*, presented at the Composite Construction in Steel and Concrete IV, Banff, Canada, 2000.
- [16] Myoung-Ho Oh, young-Kyu ju, Myeong-Han Kim and Sang-Dae Kim. *Structural performance of Steel-Concrete Composite Column Subjected to Axial and Flexural Loading*, Journal of Asian Architecture and Building Engineering, Volume 5, Issue 1 (2006), pp. 153-160.
- [17] Brent Prickett, Robert Driver. *Behaviour of partially encased composite columns made with high performance concrete*, Structural Engineering Report, Issue 262 (2006).
- [18] Ehab Ellobody and Ben Young. *Numerical simulation of concrete encased steel composite columns*, Journal of Construction Steel Research, Volume 67, Issue 2 (2011), pp.211-222.
- [19] Chang-Soo Kim, Kyung-Soo Chung and In-Rak Choi. *Eccentric Axial Load Testing for Concrete-Encased Steel Columns Using 800 MPa Steel and 100 MPa Concrete*, Journal of Construction Steel Research, Volume 138, Issue 8 (2012), pp. 1019-1031.
- [20] Chang-Soo Kim, Kyung-Soo Chung and In-Rak Choi. *Eccentric Axial Load Capacity of High-Strength Steel-Concrete Composite Columns of Various Sectional Shape*, Journal of Construction Steel Research, Volume 140, Issue 4 (2014).
- [21] Jong-Ho et al. *Challenges and opportunities for the structural design of the 123-story Jamsil Lotte World Tower*, Conference, Global Interchanges: Resurgence of Skyscraper city (2015).
- [22] Dennis Poon and al. *Structural design of Taipei 101, the World's tallest building*, Seoul Conference (2004).
- [23] Engineering News-Record's website. <https://www.enr.com/articles/153-lotte-korea-s-first-supertower-is-an-all-in-the-family-affair?v=preview>. Accessed : 26-05-2019.
- [24] David A. Nethercot. 2004. *Composite construction*. Taylor Francis, Spon Press, London, United Kingdom.
- [25] CEN (European Committee for Standardisation). 2005. *Eurocode 3: Design of steel structures. Part 1-1: General rules and rules for buildings*. Brussels, Belgium.
- [26] René Maquoi, Rik Debruyckere, Jean-François Demonceau and Lincy Pyl. 2011. *Construction mixte acier-béton dans les bâtiments: Conception et dimensionnement à température ordinaire et en situation d'incendie selon l'Eurocode 4*. Poteaux mixtes acier-béton. Infosteel, Bruxelles, Belgique.
- [27] CEN (European Committee for Standardisation). 2004. *Eurocode 2: Design of concrete structures. Part 1-2 : General rules - Structural fire design*. Brussels, Belgium.
- [28] CEN (European Committee for Standardisation). 2005. *Eurocode 3: Design of steel structures. Part 1-2 : General rules - Structural fire design*. Brussels, Belgium.

Appendix A

Interaction curve

This Appendix contains the various calculations used to determine the simplified interaction curve in the case of fully and partially concrete encased steel sections.

A.1 Fully concrete encased steel section

The interaction curve is created by taking into account 4 different positions of the neutral axis:

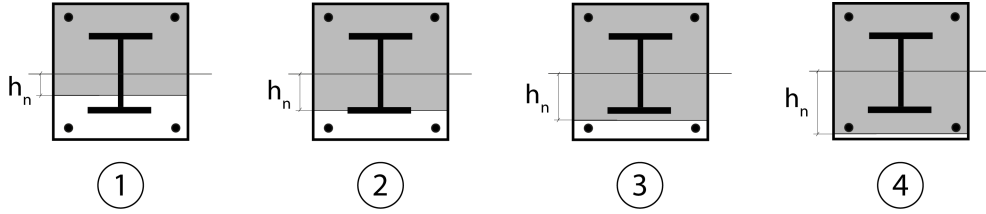


Figure A.1: Representation of the different positions of the neutral axis.

1. Neutral axis in the web:

$$N_{C,Rd} = A_c f_{cd} 0,85 = 2 h_n (b - t_w) 0,85 f_{cd} + 2 h_n t_w 2 f_{yd} \quad (\text{A.1})$$

$$h_n = \frac{A_c 0,85 f_{cd}}{2(b - t_w) 0,85 f_{cd} + 2 t_w 2 f_{yd}} \quad (\text{A.2})$$

2. Neutral axis in the flange:

$$N_{C,Rd} = A_c f_{cd} 0,85 = 0,85 f_{cd} (b - t_w) (h_f - 2t_f) + 0,85 f_{cd} (b - b_f) 2 \left(t_f - \frac{h_f}{2} + h_n\right) + 2 f_{yd} 2 \left(t_f - \frac{h_f}{2} + h_n\right) b_f + 2 f_{yd} t_w (h_f - 2t_f) \quad (\text{A.3})$$

$$h_n = \frac{0,85 f_{cd} (A_c - (h_f - 2t_f)(b - t_w)) - 2(h_f - 2t_f)t_w f_{yd}}{2(b - b_f) 0,85 f_{cd} + 4b_f f_{yd}} + \frac{h_f}{2} - t_f \quad (\text{A.4})$$

3. Neutral axis out of the profile:

$$N_{C,Rd} = A_c f_{cd} 0,85 = (2 h_n b - A_s) 0,85 f_{cd} + 2 A_s f_{yd} \quad (\text{A.5})$$

$$h_n = \left(\frac{A_c 0,85 f_{cd} - 2 A_s f_{yd}}{0,85 f_{cd}} + A_s \right) \frac{1}{2b} \quad (\text{A.6})$$

4. Neutral axis out of the profile but taking into account the reinforcement (the reinforcement section is considered as an infinitesimal point to simplify the equations):

$$N_{C,Rd} = A_c f_{cd} 0,85 = (2 h_n b - A_s) 0,85 f_{cd} + 2 A_s f_{yd} + 2 A_{sl} f_{sd} \quad (\text{A.7})$$

$$h_n = \left(\frac{A_c 0,85 f_{cd} - 2 A_s f_{yd} - 2 A_{sl} f_{sd}}{0,85 f_{cd}} + A_s \right) \frac{1}{2b} \quad (\text{A.8})$$

Once the position of the neutral axis is determined, the resistant plastic moment $M_{pl,Rd}$ must be calculated.

First of all, the centre of gravity, in relation to the average fibre, of the concrete section in compression Y_c , the steel section in compression Y_a and the steel section in tension Y'_a must be obtained, see Fig. A.2. In this example, the reinforcements do not participate in the resistance of the plastic moment due to their symmetry and the position of the neutral axis in the web of the profile. Then the forces F_c , F_a and F'_a can be calculated knowing the stress distributions and the different areas.

This allows to calculate the resistant moment plastic:

$$M_{pl,Rd} = (Y_c + Y'_a) F_c + (Y_a + Y'_a) F_a \quad (\text{A.9})$$

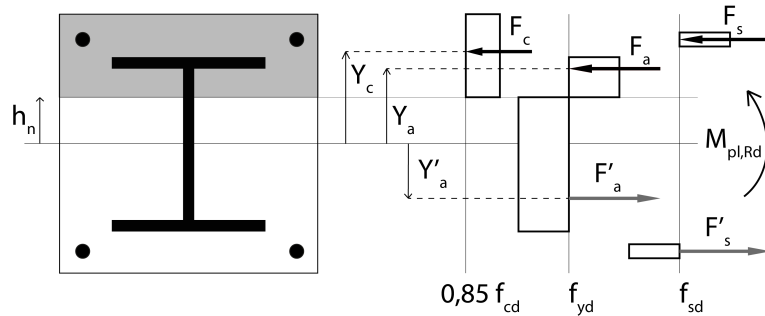


Figure A.2: Representation of the forces involved in the calculation of the resistant plastic moment.

A.2 Partially concrete encased steel section

The interaction curve is created taking into account 3 different positions of the neutral axis:

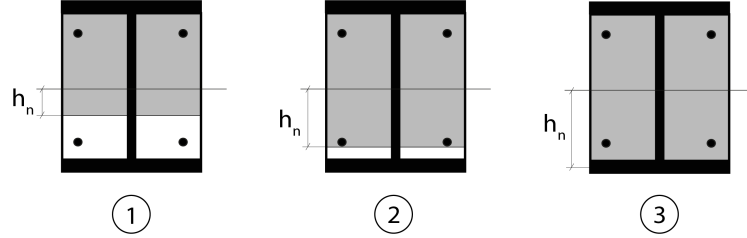


Figure A.3: Representation of the different positions of the neutral axis.

1. Neutral axis in the web:

$$N_{C,Rd} = A_c f_{cd} 0,85 = (b_f - t_w) 2 h_n 0,85 f_{cd} + 2 h_n t_w 2 f_{yd} \quad (\text{A.10})$$

$$h_n = \frac{A_c 0,85 f_{cd}}{2 f_{cd} 0,85 (b_f - t_w) + 4 t_w f_{yd}} \quad (\text{A.11})$$

2. Neutral axis in the web but taking into account the reinforcement:

$$N_{C,Rd} = A_c f_{cd} 0,85 = (b_f - t_w) 2 h_n 0,85 f_{cd} + 2 h_n t_w 2 f_{yd} + 4 A_{sl} 2 f_{sd} \quad (\text{A.12})$$

$$h_n = \frac{A_c 0,85 f_{cd} - 8 A_{sl} f_{sd}}{2 f_{cd} 0,85 (b_f - t_w) + 4 t_w f_{yd}} \quad (\text{A.13})$$

3. Neutral axis in the flange:

$$N_{C,Rd} = A_c f_{cd} 0,85 = h_c (b_f - t_w) 0,85 f_{cd} + h_c t_w 2 f_{yd} + 4 A_{sl} 2 f_{sd} + (h_n - h_c/2) b_f 4 f_{yd} \quad (\text{A.14})$$

$$h_n = \frac{A_c 0,85 f_{cd} - h_c (b_f - t_w) 0,85 f_{cd} - h_c t_w 2 f_{yd} - 4 A_{sl} 2 f_{sd}}{4 b_f f_{yd}} + \frac{h_c}{2} \quad (\text{A.15})$$

By proceeding in the same way as in the case of fully concrete encased steel section, the resistant plastic moment can be determined but changing parameters according to Fig. A.4

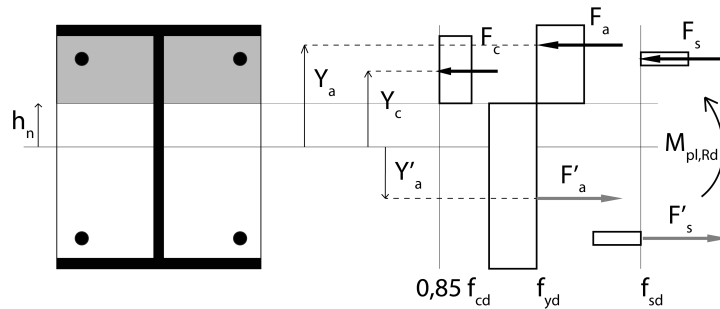


Figure A.4: Representation of the forces involved in the calculation of the resistant plastic moment.

Appendix B

Sensitivity study results

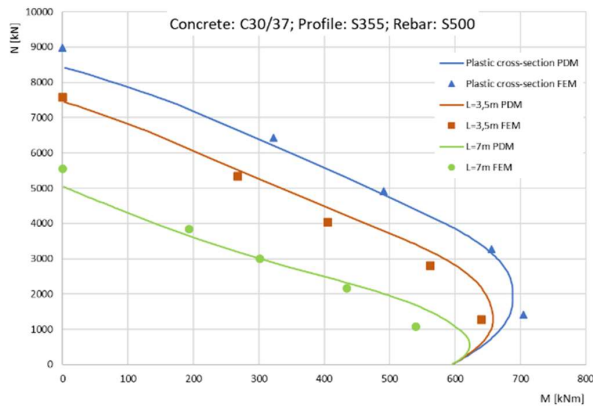
B.1 Fully encased concrete cross-section

This section contains all the results obtained for fully encased section along strong and weak axis. The Tables summarize the ratios R for the different analytical methods (A = method 1, B = method 2, C = method 3 and D = method 4) and the different parameters $\bar{\lambda}$ and δ . Each figure presents the plastic and reduced interaction curves for the 3 considered length and for one set of material grades.

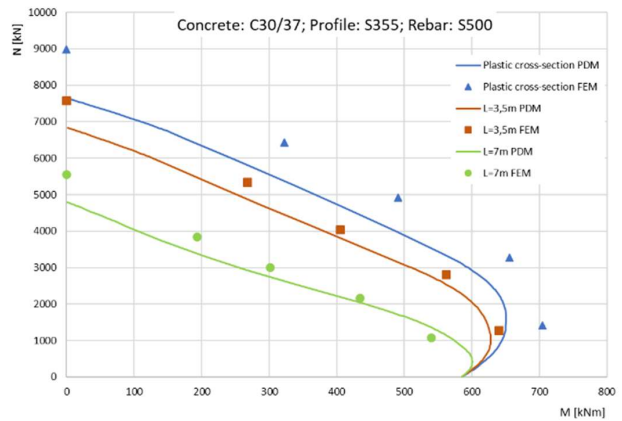
FEC 1 – Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEBE260	355	400x400	30	4 ϕ 16	500	35	0.54%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	8980	0	94%	85%	94%	94%
	6440	322	98%	88%	98%	98%
	4910	491	99%	90%	99%	99%
	3280	656	99%		99%	99%
L=3,5 m	7580	0	99%	90%	96%	71%
	5340	267	103%	94%	100%	72%
	4050	405	106%	97%	103%	76%
	2810	562	105%	97%	103%	84%
L=7 m	5556	0	91%	86%	82%	53%
	3850	192.5	96%	91%	88%	59%
	3010	301	100%	94%	92%	65%
	2170	434	103%	97%	97%	73%
δ			0.49	0.49	0.54	0.49
λ	3.5 [m]		0.49	0.53	0.47	0.98
	7 [m]		0.98	1.07	0.93	1.51

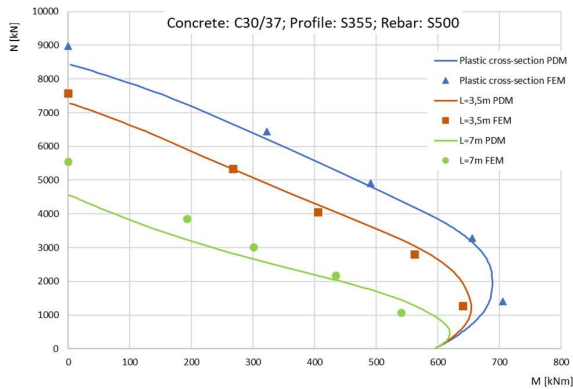
a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard

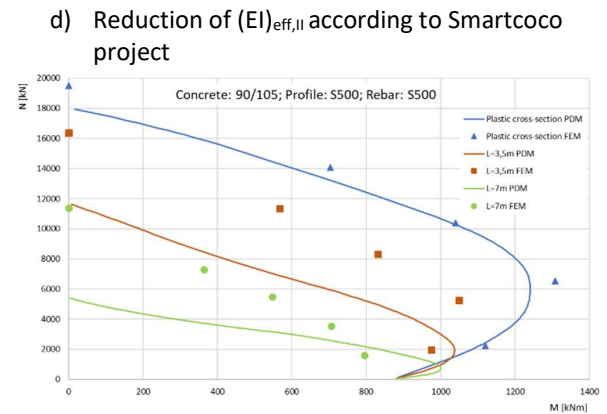
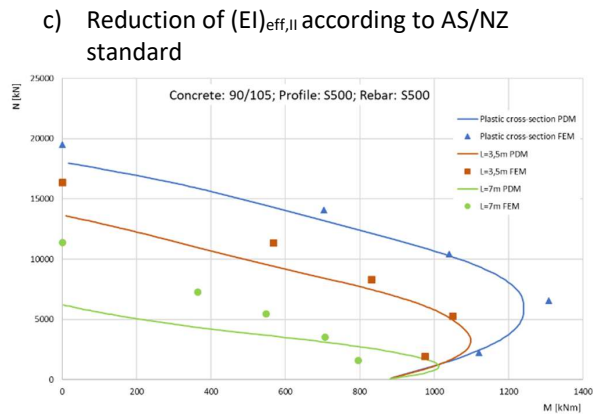
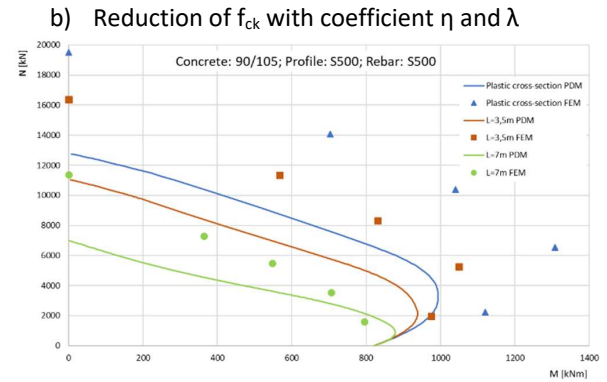
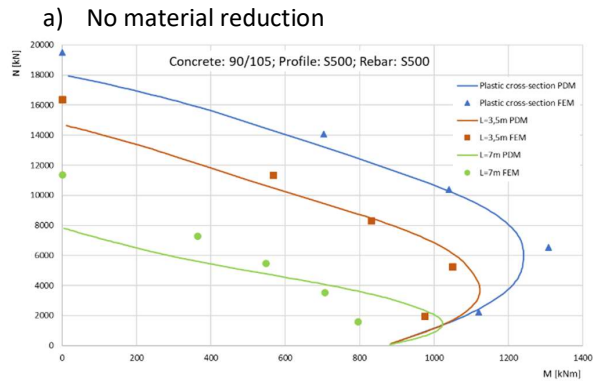


d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



FEC 1 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB260	500	400x400	90	4 ϕ 16	500	35	0.54%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	19510	0	93%	65%	93%	93%
	14070	703.5	96%	68%	96%	96%
	10400	1040	99%		99%	99%
	6540	1308				
L=3,5 m	16370	0	90%	67%	83%	71%
	11350	567.5	95%	71%	87%	71%
	8310	831	101%	79%	94%	78%
	5250	1050	104%		99%	
L=7 m	11390	0	69%	61%	54%	47%
	7280	364	82%	70%	66%	58%
	5480	548	91%	77%	75%	67%
	3530	706	108%	91%	93%	85%
	δ		0.33	0.46	0.33	0.33
λ	3.5 [m]		0.62	0.52	0.73	1.05
	7 [m]		1.24	1.05	1.45	1.71



FEC 2 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB180	355	300x300	30	4 ϕ 14	500	38	0.74%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	5050	0	96%	87%	96%	96%
	3230	161.5	101%	91%	101%	101%
	2290	229	102%	93%	102%	102%
	1420	284				
L=3,5 m	3860	0	98%	91%	92%	57%
	2380	119	104%	97%	98%	63%
	1740	174	107%	99%	102%	70%
	1130	226	107%	100%	103%	82%
L=7 m	2130	0	84%	83%	72%	50%
	1420	71	91%	88%	80%	58%
	1120	112	95%	91%	85%	63%
	780	156	104%	99%	94%	74%
δ			0.48	0.53	0.48	0.48
λ	3.5 [m]		0.69	0.66	0.77	1.29
	7 [m]		1.39	1.32	1.53	1.96

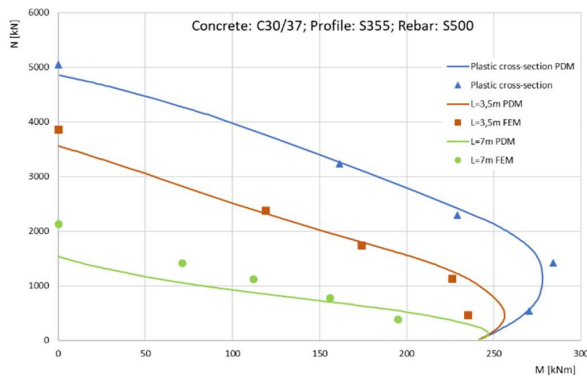
a) No material reduction



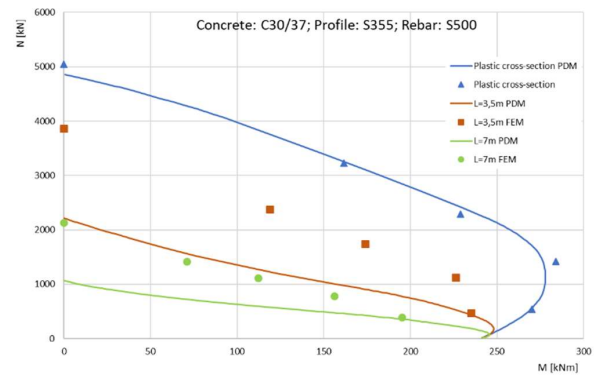
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



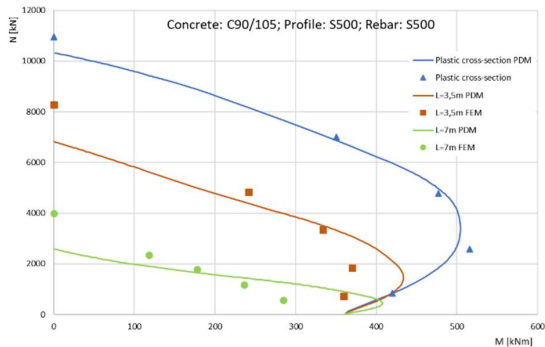
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



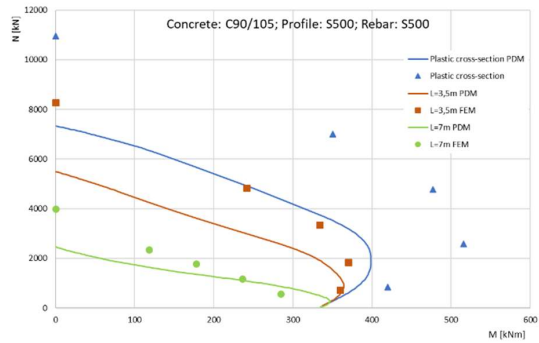
FEC 2 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB180	500	300x300	90	4 ϕ 14	500	38	0.74%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	10950	0	94%	67%	94%	94%
	7010	350.5	94%	98%	94%	94%
	4770	477	101%		101%	101%
	2580	516				
L=3,5 m	8270	0	82%	66%	70%	58%
	4830	241.5	94%	74%	81%	66%
	3340	334	102%	85%	91%	76%
	1850	370	118%		108%	93%
L=7 m	4000	0	65%	61%	48%	45%
	2360	118	84%	76%	65%	61%
	1780	178	95%	84%	75%	71%
	1180	236	113%	97%	93%	89%
δ			0.32	0.45	0.32	0.32
λ	3.5 [m]		0.87	0.73	1.03	1.38
	7 [m]		1.73	1.46	2.05	2.23

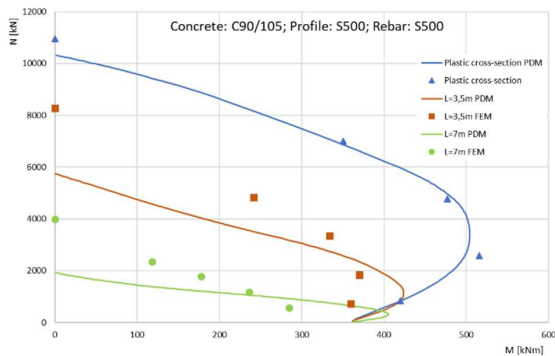
a) No material reduction



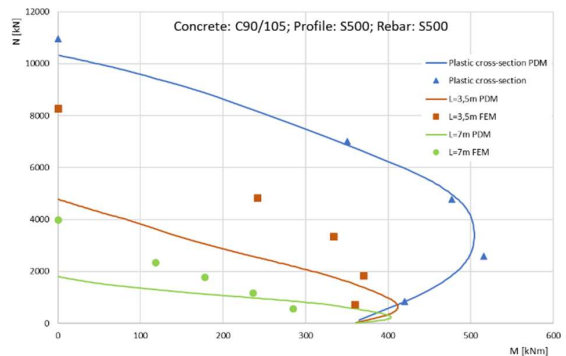
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



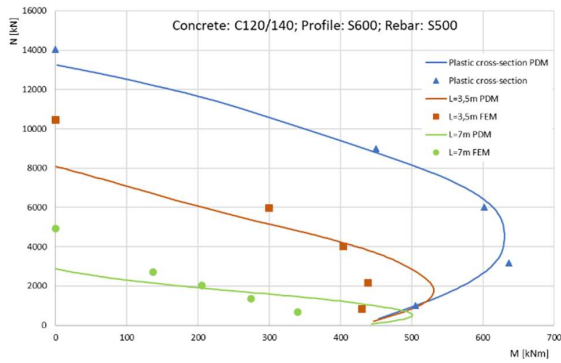
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



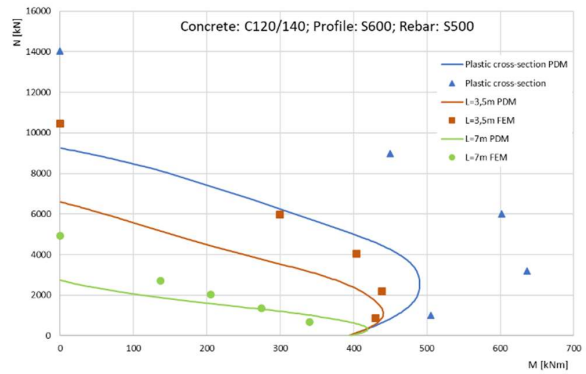
FEC 2 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB180	600	300x300	120	4 ϕ 14	500	38	0.74%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	14040	0	94%	66%	94%	94%
	8990	449.5	91%	99%	91%	91%
	6020	602	102%		102%	102%
	3180	636				
L=3,5 m	10460	0	77%	63%	62%	58%
	5980	299	90%	72%	75%	67%
	4040	404	102%	85%	88%	78%
	2190	438	122%		109%	96%
L=7 m	4940	0	58%	56%	42%	44%
	2730	136.5	82%	75%	61%	64%
	2050	205	94%	83%	72%	75%
	1370	274	114%	98%	91%	94%
δ			0.30	0.43	0.30	0.30
λ	3.5 [m]		0.94	0.78	1.13	1.43
	7 [m]		1.87	1.57	2.26	2.30

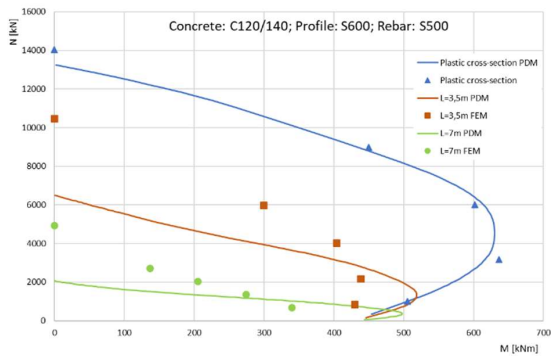
a) No material reduction



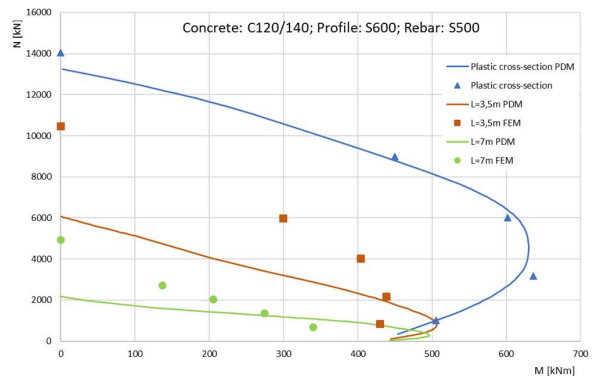
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



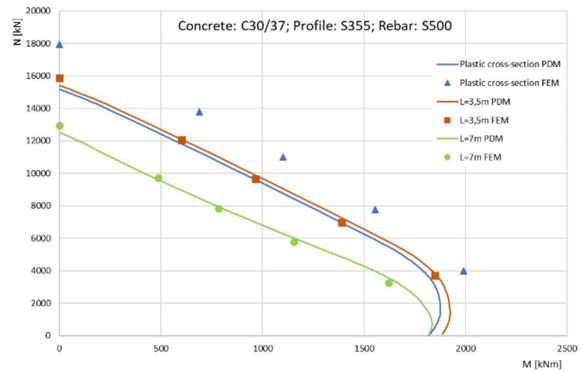
FEC 3 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEM320	355	500x500	30	4 ϕ 16	500	40	0.37%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	17940	0	96%	85%	96%	96%
	13780	689	99%	86%	99%	99%
	11000	1100	100%	88%	100%	100%
	7770	1554	101%	90%	101%	101%
L=3,5 m	15849	0	102%	97%	101%	85%
	12050	602.5	105%	100%	104%	85%
	9660	966	106%	102%	106%	87%
	6950	1390	107%	102%	106%	90%
L=7 m	12960	0	102%	97%	99%	69%
	9720	486	104%	99%	101%	71%
	7830	783	106%	101%	103%	75%
	5770	1154	108%	102%	105%	80%
δ			0.64	0.69	0.64	0.64
λ	3.5 [m]		0.36	0.35	0.38	0.73
	7 [m]		0.72	0.70	0.76	1.15

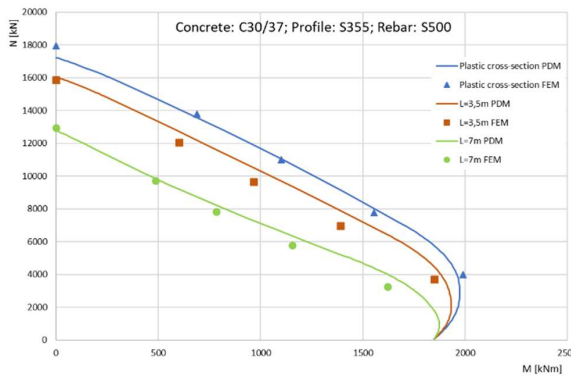
a) No material reduction



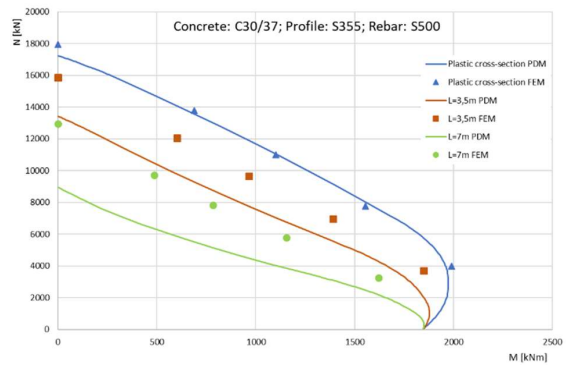
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



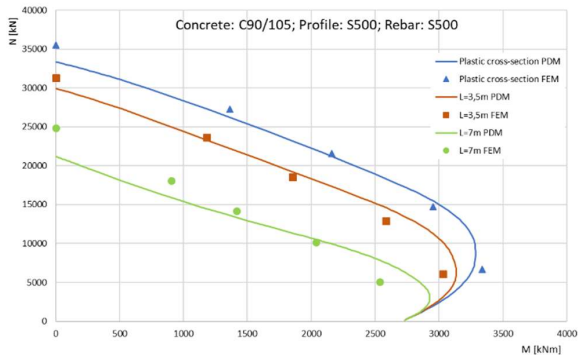
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



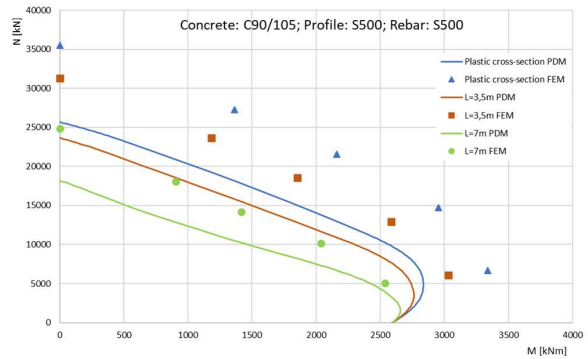
FEC 3 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEM320	500	500x500	90	4 ϕ 16	500	40	0.37%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	35510	0	94%	72%	94%	94%
	27250	1362.5	97%	74%	97%	97%
	21599	2159.9	99%	76%	99%	99%
	14760	2952	102%		102%	102%
L=3,5 m	31300	0	96%	76%	93%	83%
	23650	1182.5	99%	78%	96%	83%
	18530	1853	102%	81%	100%	86%
	12930	2586	105%	87%	103%	91%
L=7 m	24860	0	85%	73%	77%	64%
	18100	905	90%	77%	82%	68%
	14180	1418	96%	81%	88%	74%
	10180	2036	102%	86%	95%	81%
	δ		0.47	0.61	0.47	0.47
λ	3.5 [m]		0.46	0.40	0.51	0.77
	7 [m]		0.92	0.81	1.02	1.29

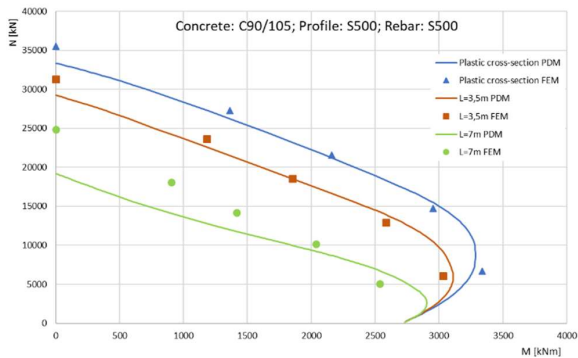
a) No material reduction



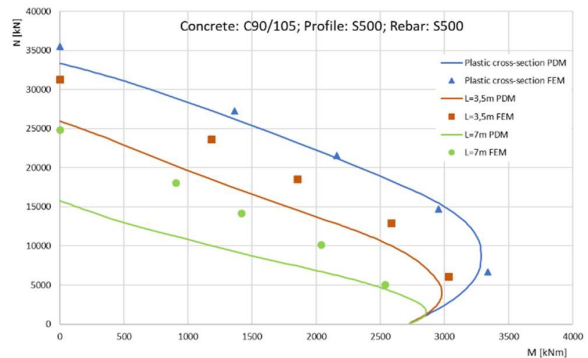
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



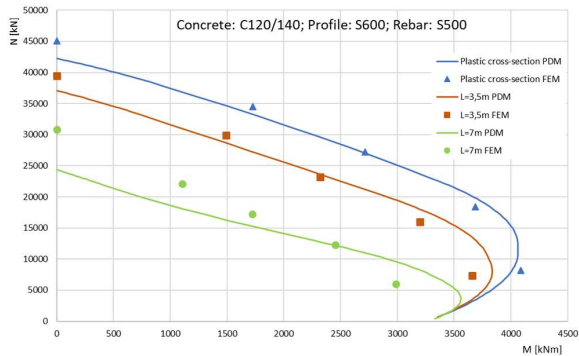
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



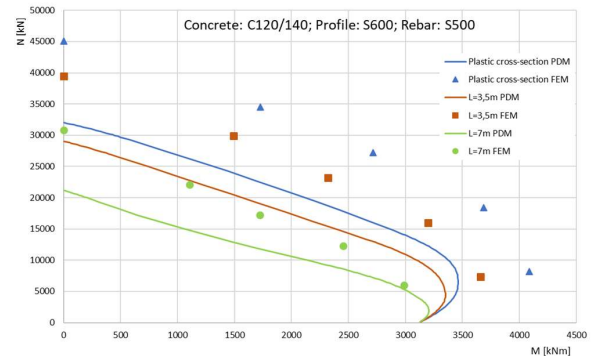
FEC 3 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEM320	600	500x500	120	4 ϕ 16	500	40	0.37%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	45090	0	94%	71%	94%	94%
	34550	1727.5	97%	73%	97%	97%
	27180	2718	100%	76%	100%	100%
	18430	3686	103%		103%	103%
L=3,5 m	39440	0	94%	74%	91%	82%
	29860	1493	97%	76%	94%	82%
	23210	2321	101%	79%	98%	85%
	16000	3200	105%	88%	102%	91%
L=7 m	30790	0	79%	69%	70%	61%
	22140	1107	85%	73%	75%	67%
	17210	1721	92%	78%	82%	74%
	12260	2452	100%	85%	91%	81%
	δ		0.45	0.59	0.45	0.45
λ	3.5 [m]		0.50	0.44	0.56	0.82
	7 [m]		1.01	0.88	1.13	1.38

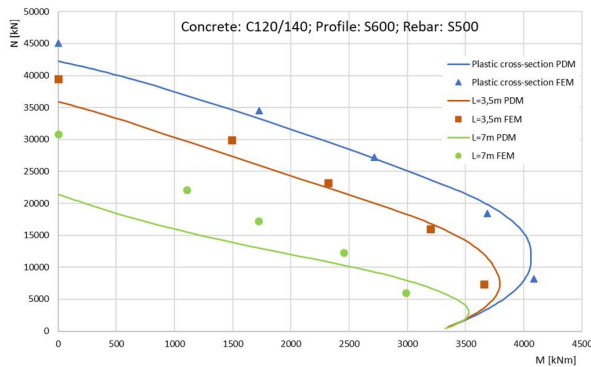
a) No material reduction



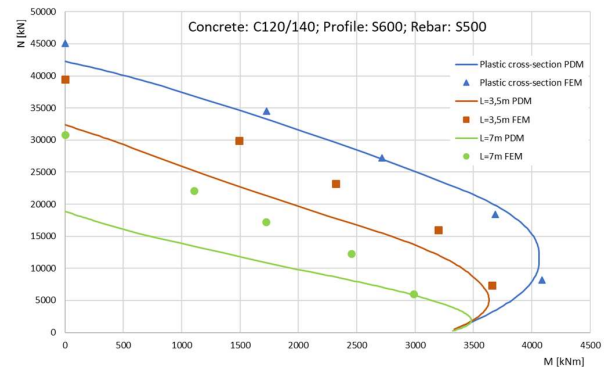
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



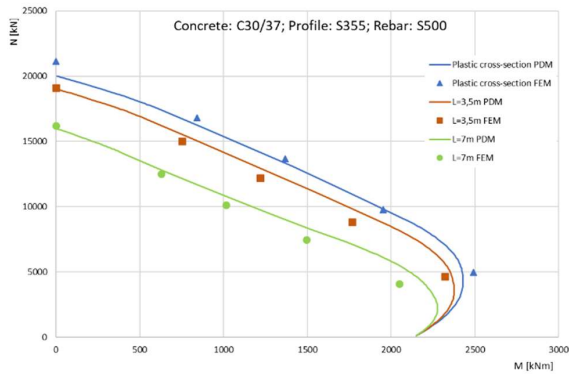
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



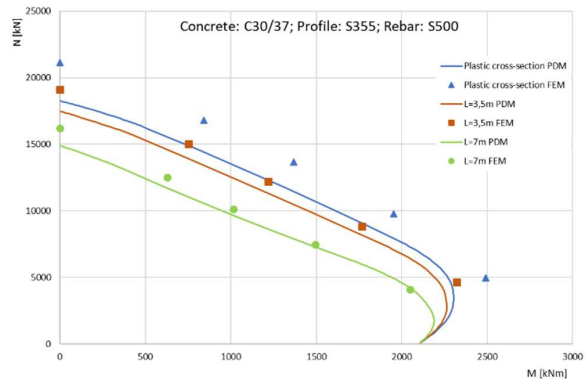
FEC 4 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD400x237	355	600x600	30	4 ϕ 20	500	46	0.38%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	21140	0	95%	86%	95%	95%
	16820	841	97%	89%	97%	97%
	13670	1367	98%	90%	98%	98%
	9760	1952	100%	92%	100%	100%
L=3,5 m	19100	0	100%	91%	98%	82%
	15030	751.5	103%	94%	101%	82%
	12200	1220	104%	95%	103%	84%
	8840	1768	105%	97%	104%	87%
L=7 m	16210	0	99%	92%	94%	64%
	12530	626.5	102%	95%	97%	65%
	10140	1014	104%	97%	100%	68%
	7470	1494	106%	99%	102%	74%
δ			0.54	0.59	0.54	0.54
λ	3.5 [m]		0.33	0.32	0.36	0.73
	7 [m]		0.66	0.63	0.72	1.18

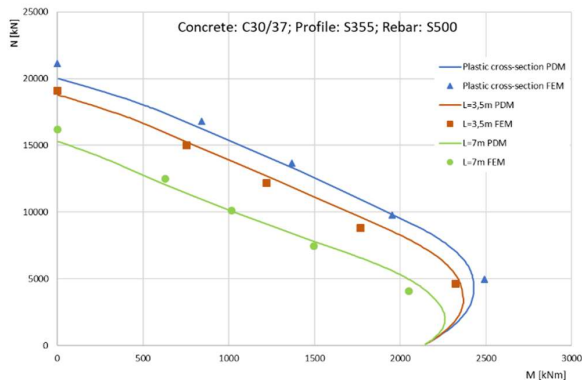
a) No material reduction



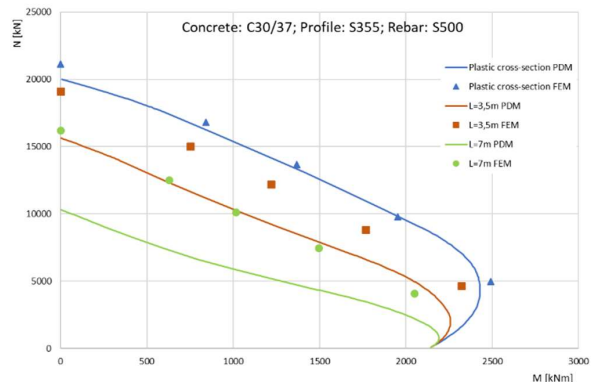
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



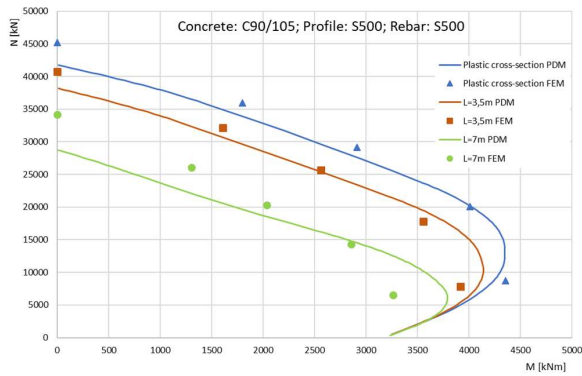
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



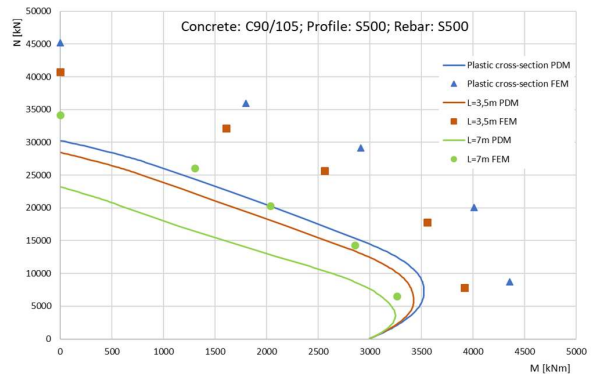
FEC 4 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD400x237	500	600x600	90	4 ϕ 20	500	46	0.38%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	45180	0	92%	67%	92%	92%
	35970	1798.5	95%	69%	95%	95%
	29120	2912	97%	70%	97%	97%
	20050	4010	100%		100%	100%
L=3,5 m	40730	0	94%	70%	91%	81%
	32150	1607.5	97%	72%	93%	80%
	25640	2564	99%	74%	96%	82%
	17780	3556	104%		101%	89%
L=7 m	34190	0	84%	68%	73%	62%
	26080	1304	88%	71%	77%	64%
	20340	2034	94%	75%	83%	68%
	14270	2854	102%	83%	92%	77%
δ			0.37	0.50	0.37	0.37
λ	3.5 [m]		0.42	0.36	0.49	0.78
	7 [m]		0.84	0.71	0.97	1.30

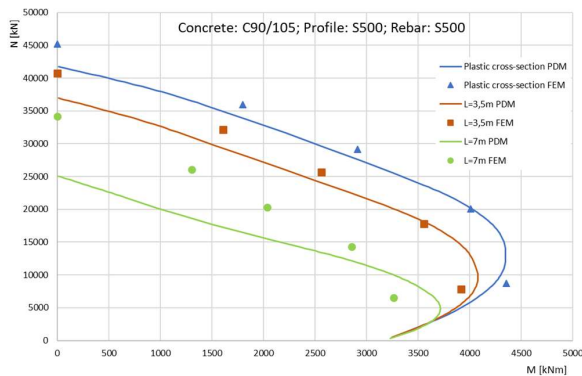
a) No material reduction



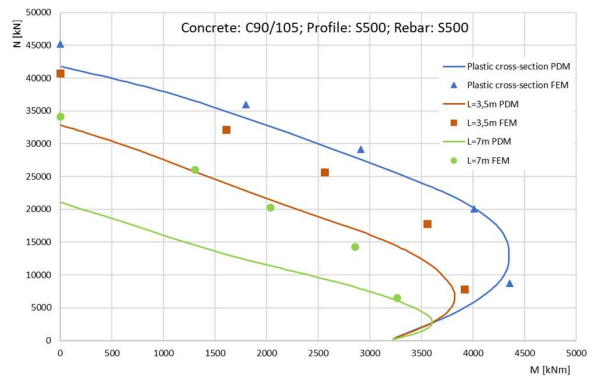
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



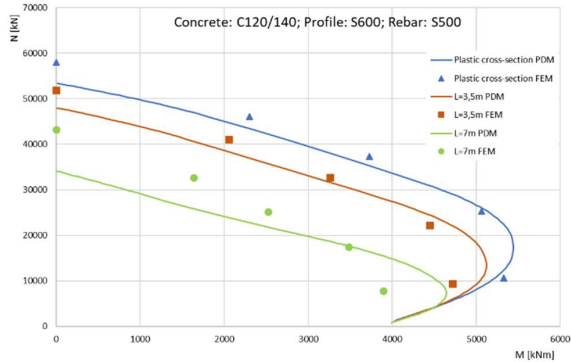
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



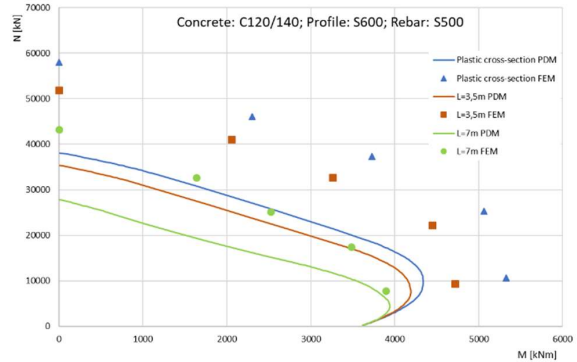
FEC 4 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD400x237	600	600x600	120	4 ϕ 20	500	46	0.38%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	57950	0	92%	66%	92%	92%
	46050	2302.5	96%	68%	96%	96%
	37290	3729	97%	70%	97%	97%
	25320	5064	101%		101%	101%
L=3,5 m	51900	0	92%	68%	89%	80%
	41110	2055.5	95%	70%	90%	79%
	32630	3263	98%	72%	94%	81%
	22230	4446	104%		100%	90%
L=7 m	43190	0	79%	64%	66%	61%
	32720	1636	83%	67%	70%	64%
	25220	2522	91%	73%	77%	69%
	17400	3480	101%	83%	89%	78%
	δ		0.34	0.48	0.34	0.34
λ	3.5 [m]		0.45	0.38	0.54	0.82
	7 [m]		0.91	0.77	1.08	1.36

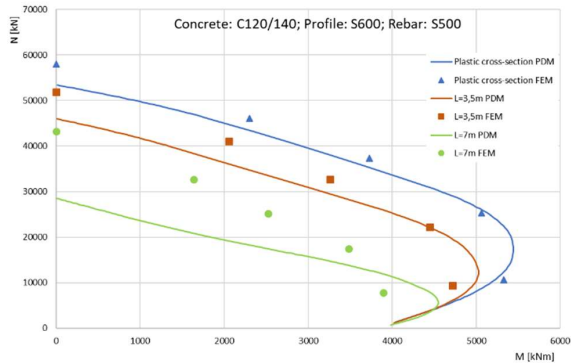
a) No material reduction



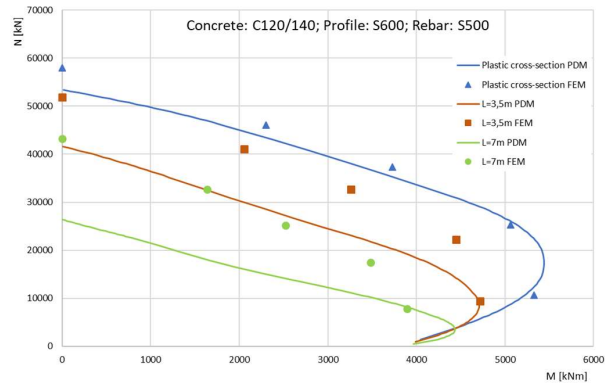
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



FEC 5 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	355	360x360	30	4 ϕ 16	500	40	0.66%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	7030	0	94%	79%	94%	94%
	4950	247.5	98%	82%	98%	98%
	3736	373.6	100%	87%	100%	100%
	2480	496	99%		99%	
L=3,5 m	5820	0	99%	83%	97%	71%
	4020	201	103%	86%	100%	72%
	3060	306	105%	89%	103%	77%
	2090	418	105%		103%	86%
L=7 m	4080	0	90%	75%	82%	55%
	2800	140	95%	79%	87%	61%
	2200	220	99%	83%	92%	67%
	1570	314	104%	87%	98%	76%
δ			0.45	0.50	0.45	0.45
λ	3.5 [m]		0.52	0.50	0.57	1.01
	7 [m]		1.05	0.99	1.14	1.54

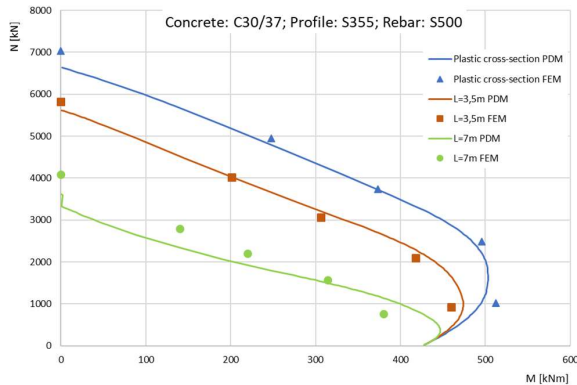
a) No material reduction



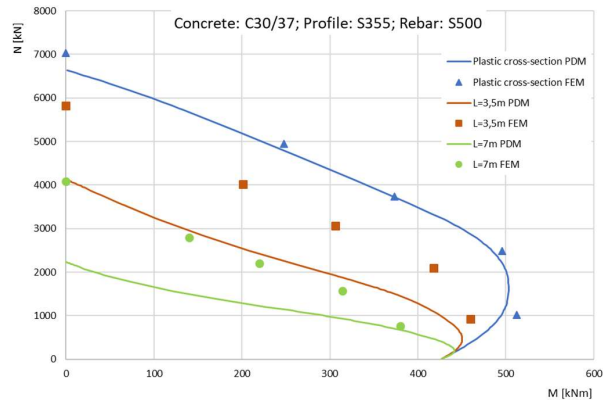
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



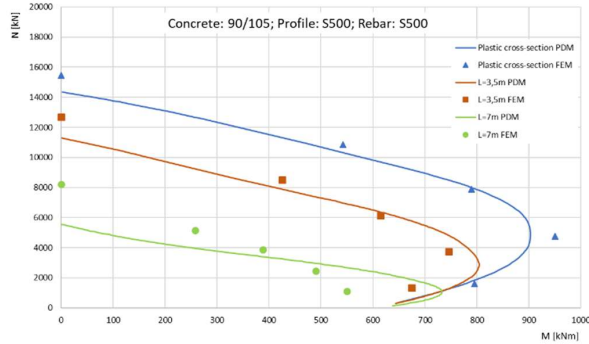
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



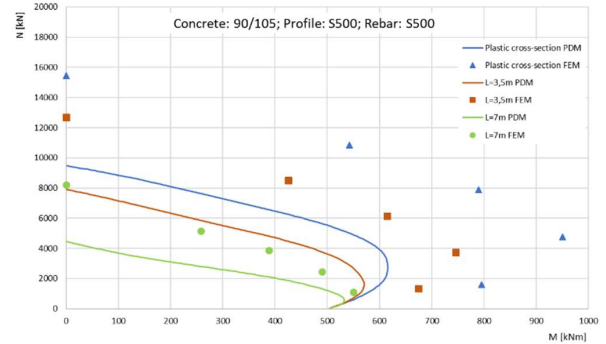
FEC 5 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	500	360x360	90	4 ϕ 16	500	40	0.66%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	15470	0	93%	61%	93%	93%
	10850	542.5	97%	68%	97%	97%
	7900	790	101%		101%	101%
	4750	950				
L=3,5 m	12700	0	89%	62%	82%	70%
	8510	425.5	95%	68%	87%	71%
	6150	615	102%		95%	79%
	3730	746	107%		101%	
L=7 m	8230	0	67%	54%	54%	47%
	5160	258	81%	64%	66%	59%
	3880	388	91%	72%	76%	69%
	2450	490	111%	85%	96%	89%
δ			0.30	0.42	0.30	0.30
λ	3.5 [m]		0.67	0.56	0.78	1.10
	7 [m]		1.35	1.13	1.56	1.78

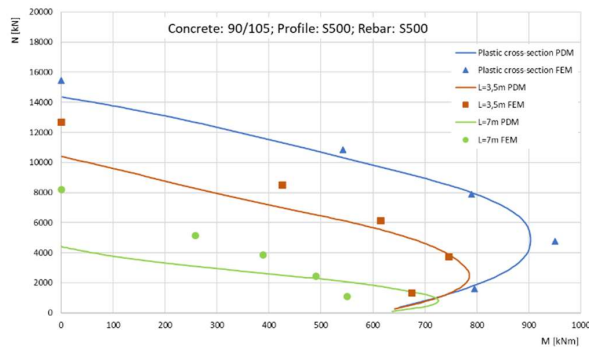
a) No material reduction



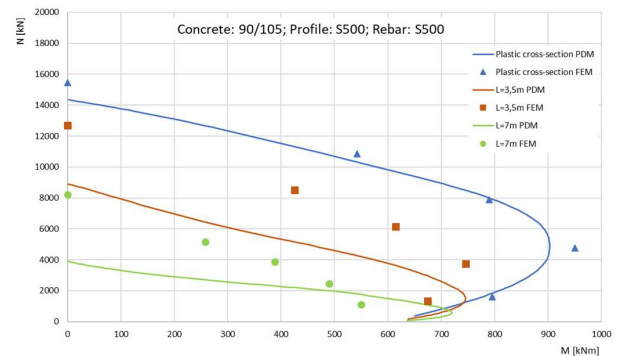
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



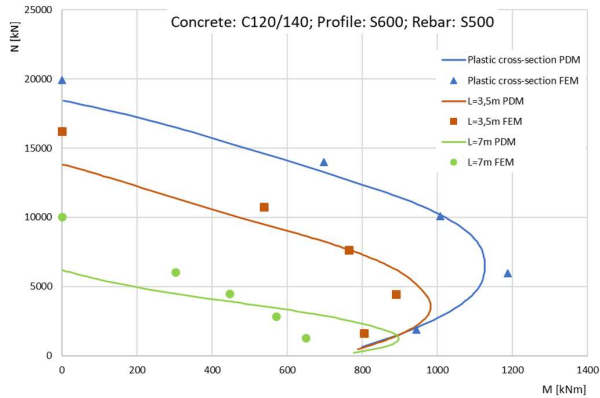
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



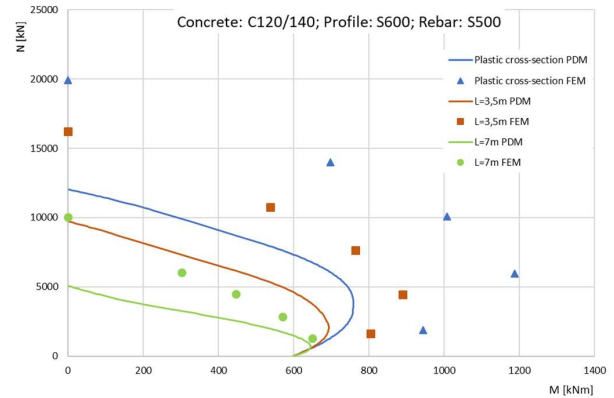
FEC 5 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	600	360x360	120	4 ϕ 16	500	40	0.66%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	19900	0	93%	60%	93%	93%
	13970	698.5	45%	70%	45%	45%
	10080	1008	100%		100%	100%
	5940	1188	#N/A			
L=3,5 m	16200	0	85%	60%	76%	70%
	10770	538.5	91%	67%	82%	70%
	7650	765	100%		91%	79%
	4450	890	110%		102%	91%
L=7 m	10040	0	62%	51%	47%	47%
	6050	302.5	79%	63%	62%	61%
	4470	447	91%	72%	73%	73%
	2850	570	112%	85%	94%	93%
δ			0.28	0.40	0.28	0.28
λ	3.5 [m]		0.73	0.61	0.86	1.15
	7 [m]		1.47	1.22	1.73	1.86

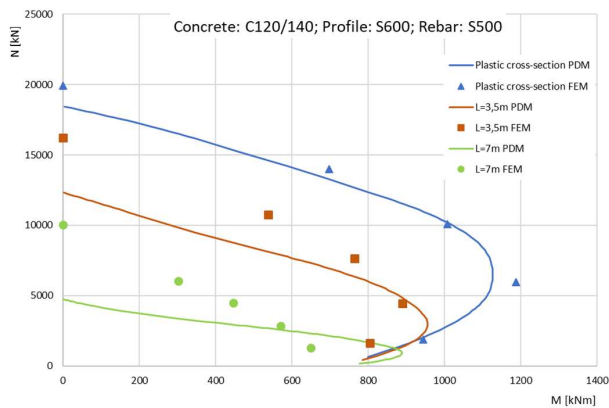
a) No material reduction



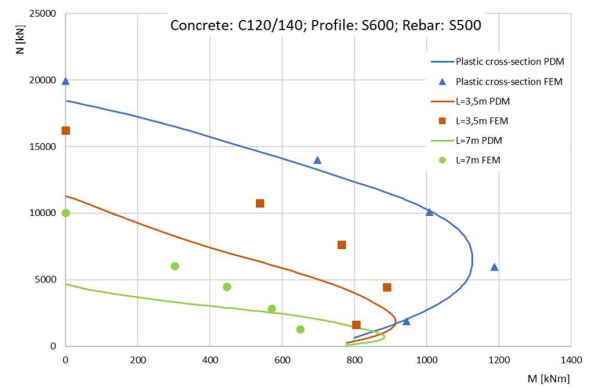
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



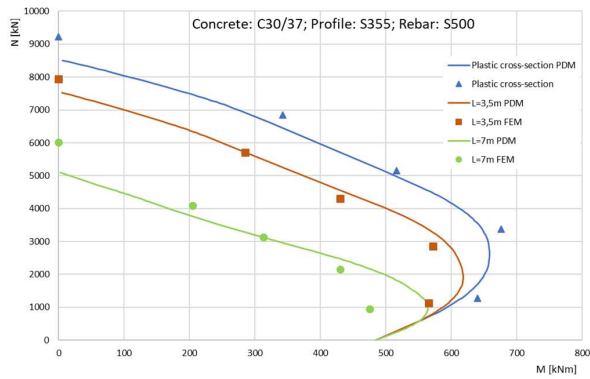
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



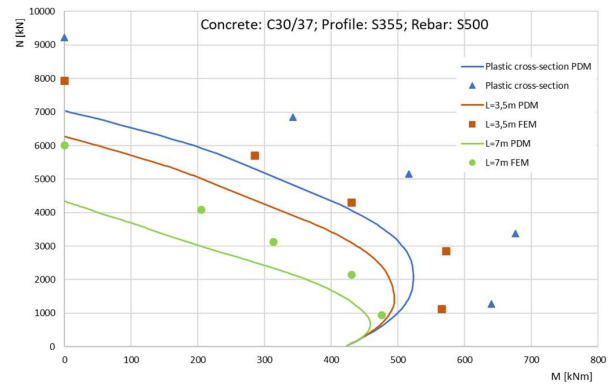
FEC 6 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	355	450x450	30	4 ϕ 16	500	40	0.41%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	9220	0	92%	76%	92%	92%
	6840	342	96%	79%	96%	96%
	5160	516	98%	90%	98%	98%
	3380	676				
L=3,5 m	7940	0	95%	79%	95%	95%
	5700	285	100%	83%	95%	62%
	4300	430	103%	87%	99%	68%
	2860	572	104%		100%	
L=7 m	6010	0	85%	72%	85%	85%
	4100	205	94%	80%	79%	48%
	3130	313	100%	85%	86%	55%
	2150	430	107%	90%	95%	67%
δ			0.35	0.40	0.35	0.35
λ	3.5 [m]		0.49	0.46	0.57	1.12
	7 [m]		0.97	0.91	1.14	1.74

a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard

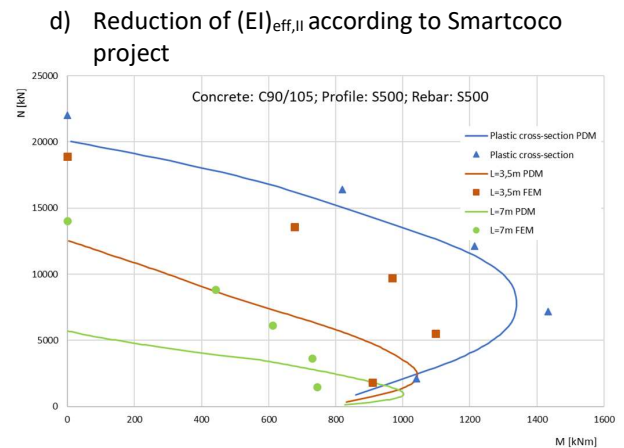
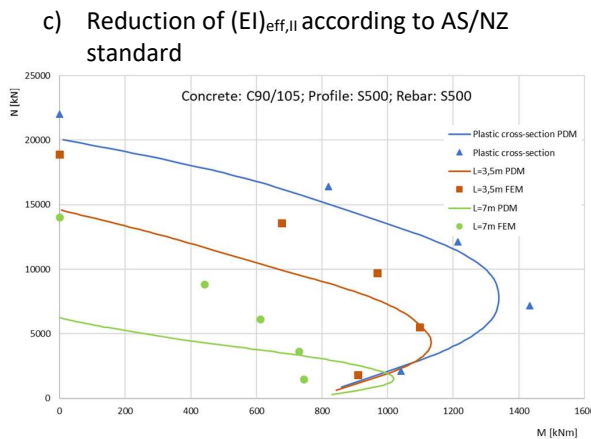
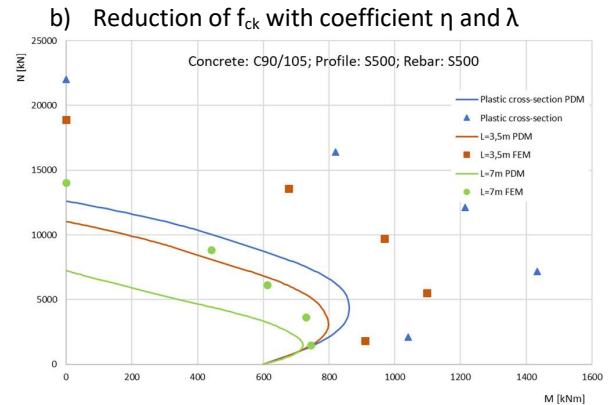
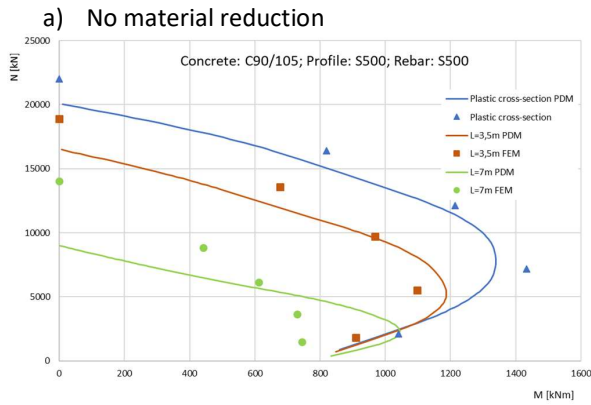


d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



FEC 6 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	500	450x450	90	4 ϕ 16	500	40	0.41%

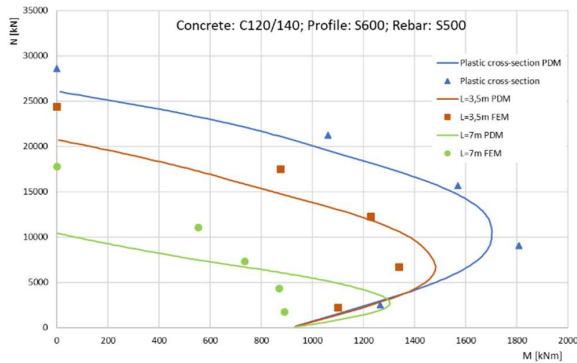
	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	22030	0	91%	57%	91%	91%
	16390	819.5	91%	69%	91%	91%
	12140	1214	97%		97%	97%
	7170	1434				
L=3,5 m	18910	0	87%	58%	77%	66%
	13560	678	91%	65%	80%	64%
	9690	969	99%		90%	76%
	5490	1098	111%		101%	
L=7 m	14040	0	64%	51%	44%	41%
	8820	441	79%	62%	56%	52%
	6120	612	95%	75%	71%	69%
	3650	730	120%		95%	88%
δ			0.21	0.32	0.21	0.21
λ	3.5 [m]		0.61	0.49	0.77	1.16
	7 [m]		1.21	0.98	1.54	1.83



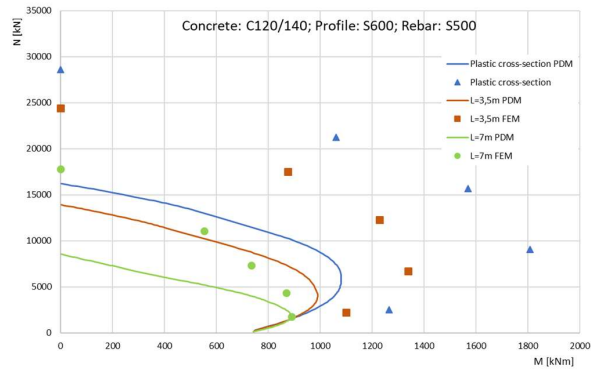
FEC 6 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	600	450x450	120	4 ϕ 16	500	40	0.41%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	28610	0	91%	57%	91%	91%
	21230	1061.5	91%	76%	91%	91%
	15690	1569	97%		97%	97%
	9040	1808				
L=3,5 m	24430	0	85%	57%	72%	68%
	17500	875	92%	65%	75%	65%
	12280	1228	98%		87%	77%
	6700	1340	116%		102%	
L=7 m	17800	0	59%	48%	39%	42%
	11070	553.5	74%	59%	50%	54%
	7350	735	94%	75%	67%	74%
	4350	870	121%		93%	95%
δ			0.20	0.30	0.20	0.20
λ	3.5 [m]		0.65	0.53	0.84	1.18
	7 [m]		1.30	1.05	1.69	1.85

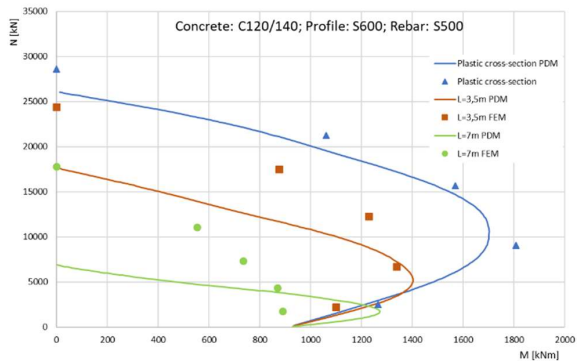
a) No material reduction



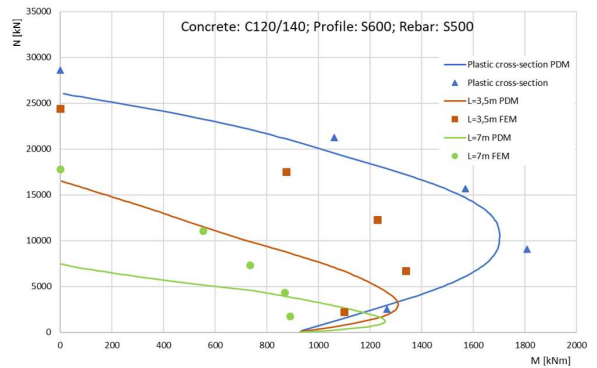
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



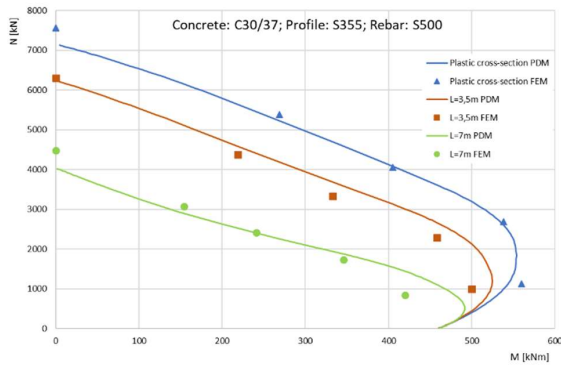
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



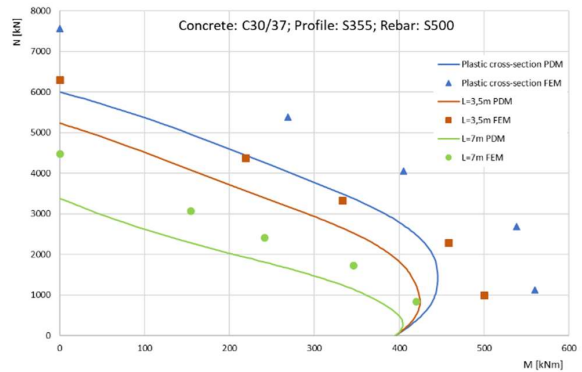
FEC 7 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	355	375x375	30	4 ϕ 20	500	46	0.95%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	7560	0	94%	79%	94%	94%
	5380	269	98%	82%	98%	98%
	4050	405	100%	87%	100%	100%
	2690	538	99%		99%	99%
L=3,5 m	6300	0	99%	83%	96%	70%
	4380	219	103%	87%	100%	71%
	3330	333	106%	90%	103%	76%
	2290	458	105%		103%	85%
L=7 m	4480	0	90%	75%	81%	53%
	3080	154	96%	80%	87%	59%
	2410	241	100%	84%	92%	65%
	1730	346	104%	87%	97%	74%
δ			0.42	0.47	0.42	0.42
λ	3.5 [m]		0.52	0.49	0.56	1.02
	7 [m]		1.03	0.98	1.13	1.56

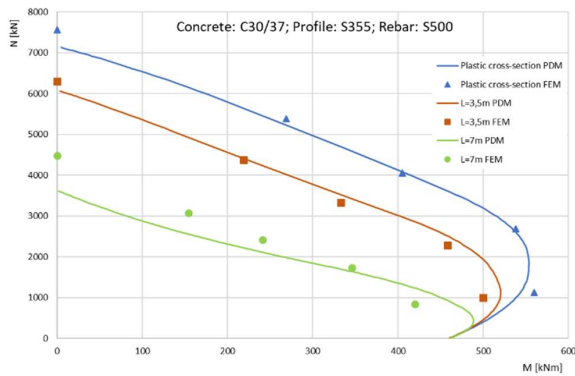
a) No material reduction



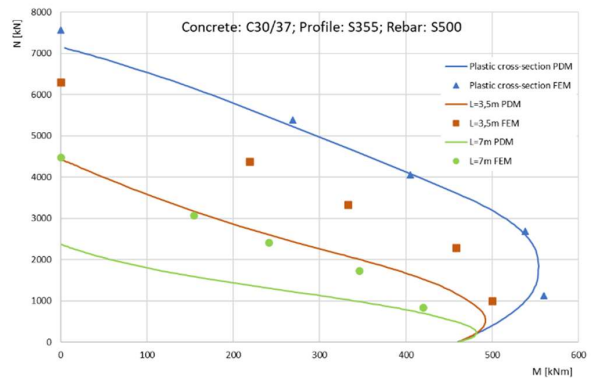
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



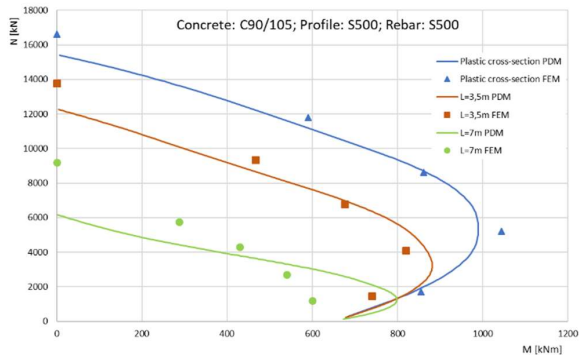
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



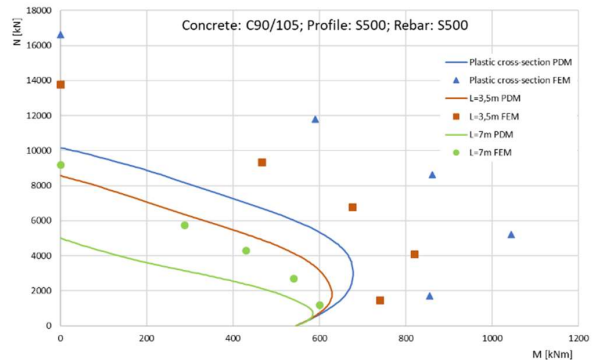
FEC 7 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	500	375x375	90	4 ϕ 20	500	46	0.95%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	16630	0	93%	61%	93%	93%
	11800	590	99%	67%	99%	99%
	8620	862	100%		100%	100%
	5220	1044				
L=3,5 m	13780	0	89%	62%	82%	70%
	9340	467	95%	67%	87%	70%
	6770	677	102%		94%	79%
	4100	820	107%		101%	
L=7 m	9180	0	67%	54%	53%	46%
	5750	287.5	81%	64%	65%	58%
	4300	430	92%	72%	76%	68%
	2700	540	112%	86%	96%	89%
δ			0.28	0.39	0.28	0.28
λ	3.5 [m]		0.66	0.55	0.77	1.10
	7 [m]		1.32	1.10	1.54	1.78

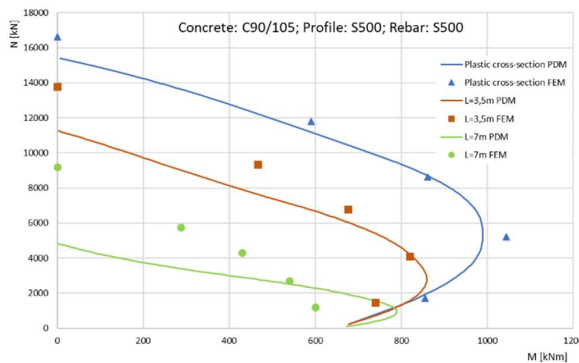
a) No material reduction



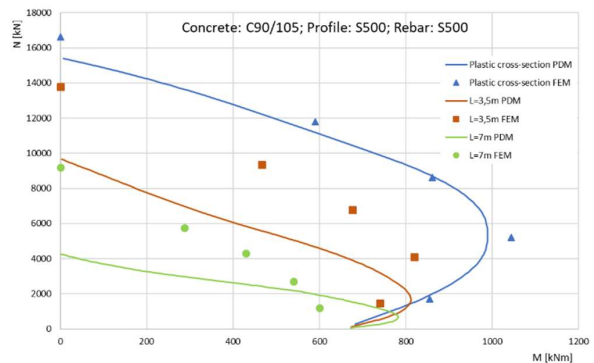
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



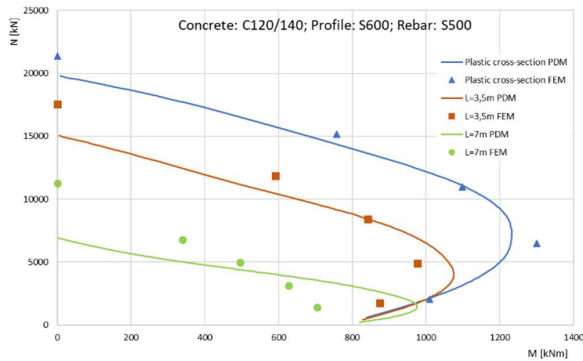
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



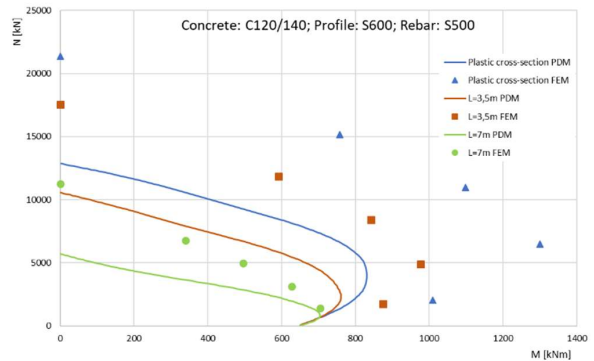
FEC 7 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	600	375x375	120	4 ϕ 20	500	46	0.95%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	21370	0	93%	60%	93%	93%
	15150	757.5	96%	69%	96%	96%
	10980	1098	100%		100%	100%
	6500	1300				
L=3,5 m	17560	0	86%	60%	76%	70%
	11840	592	91%	66%	81%	70%
	8420	842	100%		91%	79%
	4880	976	110%		102%	93%
L=7 m	11280	0	61%	51%	46%	46%
	6790	339.5	78%	63%	61%	60%
	4960	496	92%	73%	73%	73%
	3140	628	113%	86%	94%	94%
δ			0.26	0.37	0.26	0.26
λ	3.5 [m]		0.71	0.59	0.85	1.14
	7 [m]		1.43	1.19	1.70	1.85

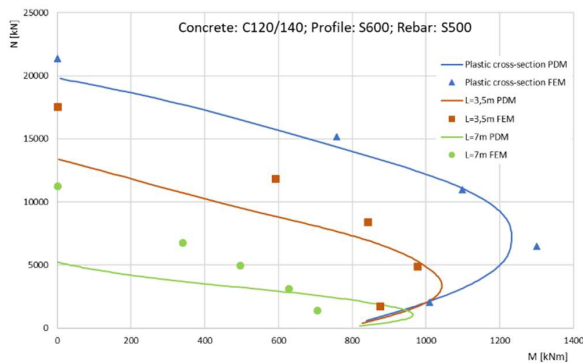
a) No material reduction



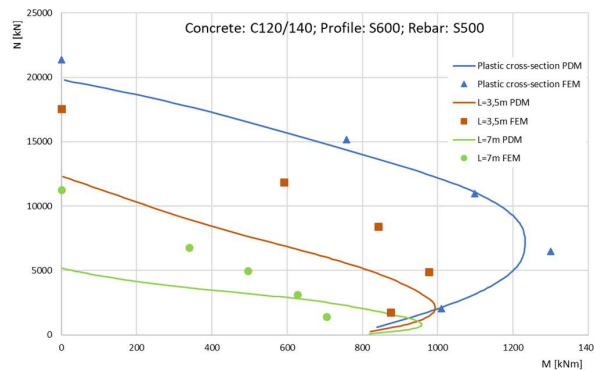
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



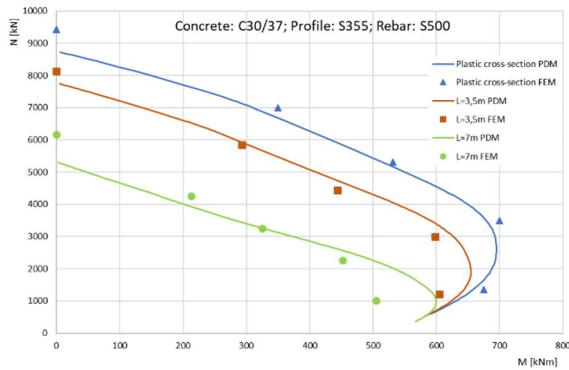
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



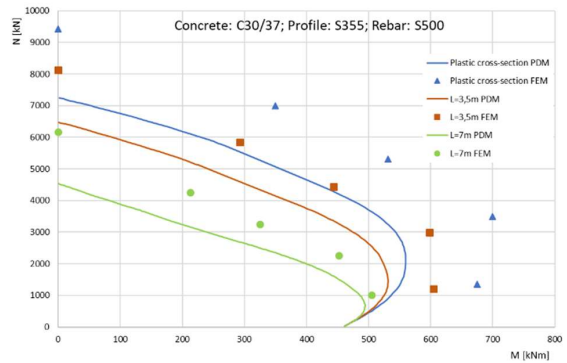
FEC 8 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	355	450x450	30	4 ϕ 20	500	46	0.65%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	9420	0	93%	77%	93%	93%
	7000	350	97%	80%	97%	97%
	5310	531	99%	86%	99%	99%
	3500	700				
L=3,5 m	8130	0	95%	80%	91%	63%
	5850	292.5	101%	85%	96%	64%
	4440	444	104%	88%	99%	69%
	2990	598	104%		101%	
L=7 m	6170	0	86%	73%	72%	42%
	4250	212.5	94%	81%	81%	49%
	3250	325	100%	86%	87%	56%
	2260	452	106%	90%	95%	67%
δ			0.34	0.39	0.34	0.34
λ	3.5 [m]		0.48	0.45	0.56	1.09
	7 [m]		0.96	0.90	1.11	1.69

a) No material reduction



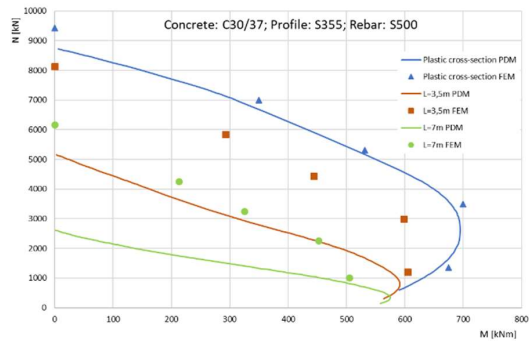
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



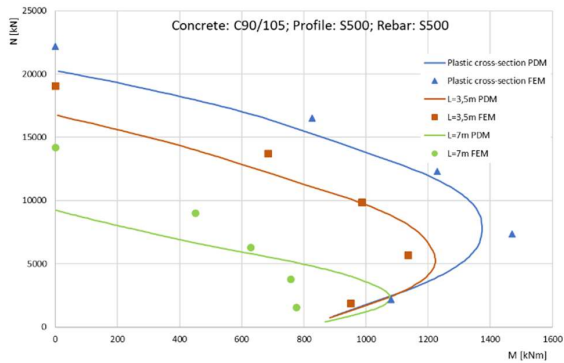
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



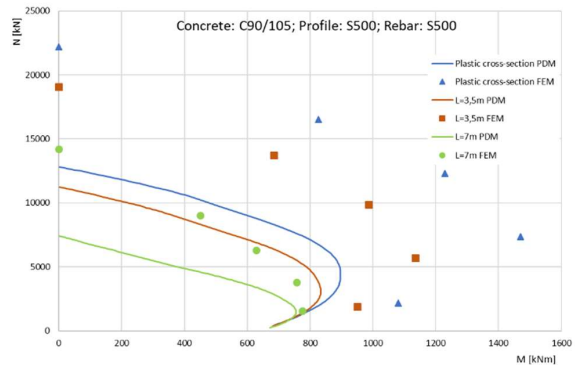
FEC 8 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	500	450x450	90	4 ϕ 20	500	46	0.65%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	22200	0	91%	58%	91%	91%
	16530	826.5	94%	66%	94%	94%
	12290	1229	98%		98%	98%
	7350	1470				
L=3,5 m	19070	0	88%	59%	78%	67%
	13710	685.5	92%	64%	82%	65%
	9870	987	100%		90%	76%
	5680	1136	110%		100%	
L=7 m	14190	0	65%	52%	46%	42%
	9000	450	79%	62%	58%	53%
	6290	629	95%	75%	72%	69%
	3790	758	118%		95%	88%
δ			0.21	0.31	0.21	0.21
λ	3.5 [m]		0.60	0.49	0.75	1.13
	7 [m]		1.20	0.98	1.51	1.80

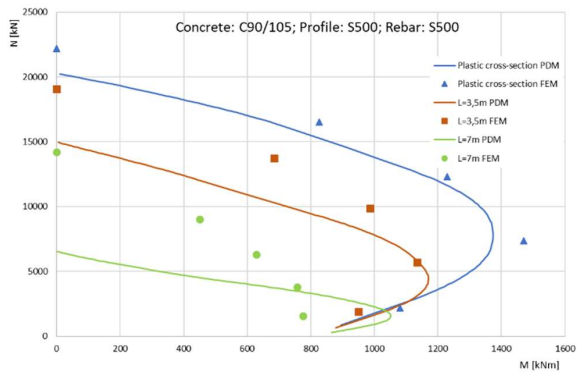
a) No material reduction



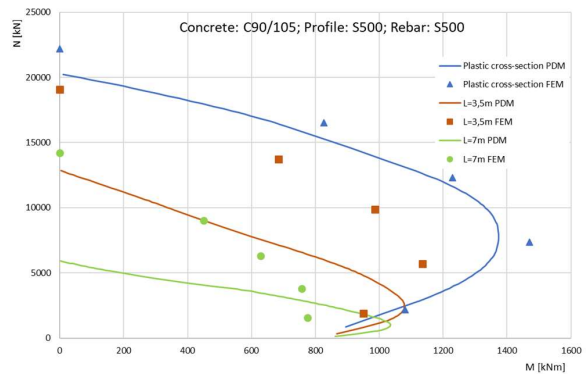
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



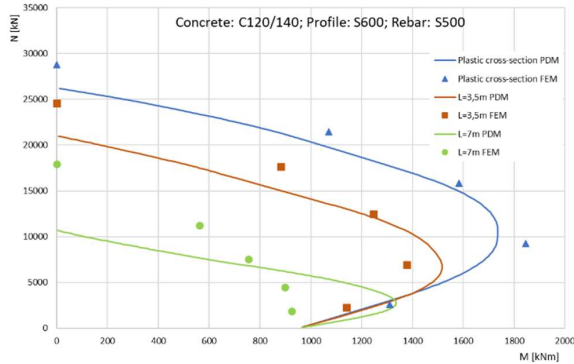
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



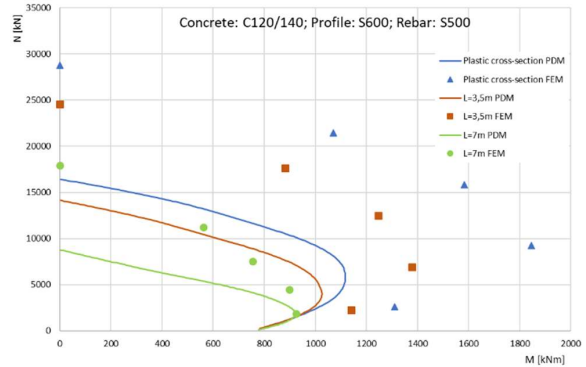
FEC 8 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	600	450x450	120	4 ϕ 20	500	46	0.65%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	28760	0	91%	57%	91%	91%
	21430	1071.5	93%	70%	94%	94%
	15840	1584	98%		98%	98%
	9230	1846				
L=3,5 m	24580	0	85%	57%	74%	68%
	17650	882.5	95%	64%	76%	66%
	12470	1247	98%		87%	77%
	6890	1378	114%		101%	
L=7 m	17940	0	60%	49%	40%	43%
	11230	561.5	74%	59%	52%	55%
	7540	754	93%	75%	68%	72%
	4490	898	120%	90%	93%	95%
δ			0.19	0.30	0.19	0.19
λ	3.5 [m]		0.64	0.52	0.83	1.15
	7 [m]		1.29	1.04	1.65	1.82

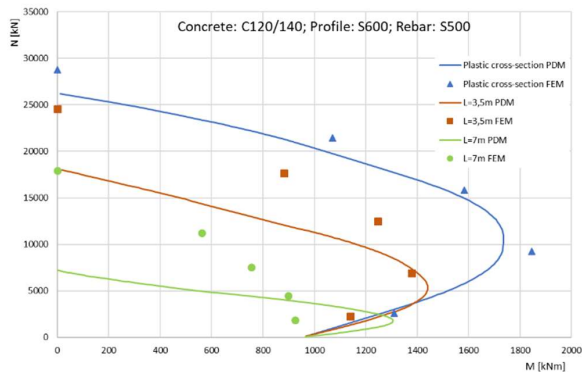
a) No material reduction



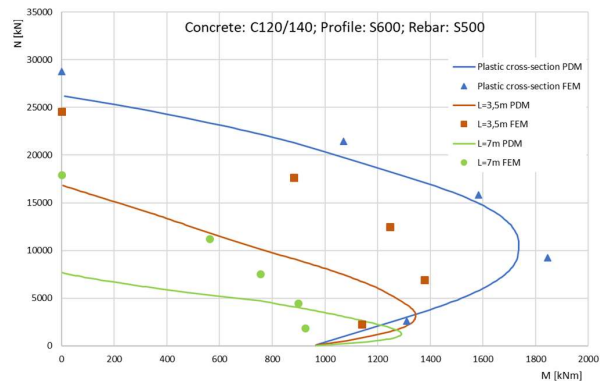
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



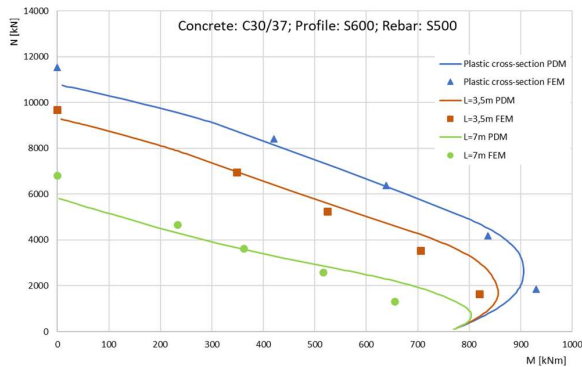
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



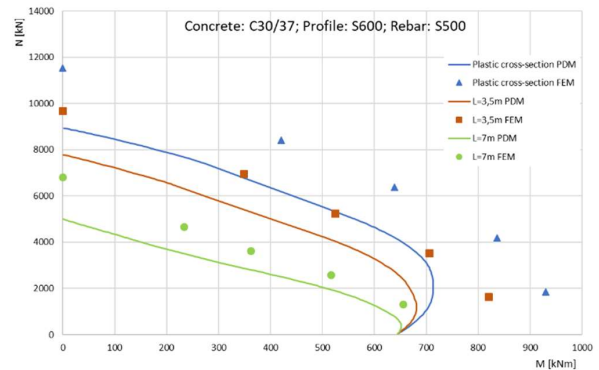
FEC 8 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	500	450x450	30	4 ϕ 20	500	46	0.65%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	10680	0	93%	77%	93%	93%
	7850	392.5	97%	81%	97%	97%
	5960	596	99%	85%	99%	99%
	3910	782	101%		101%	101%
L=3,5 m	9120	0	95%	80%	91%	60%
	6540	327	100%	84%	95%	60%
	4930	493	104%	88%	99%	65%
	3320	664	106%		102%	79%
L=7 m	6700	0	84%	72%	70%	40%
	4550	227.5	93%	80%	79%	48%
	3510	351	99%	84%	85%	53%
	2480	496	105%	89%	93%	63%
δ			0.42	0.47	0.42	0.42
λ	3.5 [m]		0.51	0.49	0.59	1.17
	7 [m]		1.03	0.97	1.19	1.82

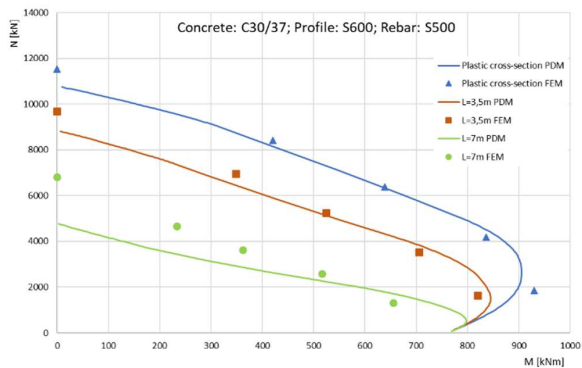
a) No material reduction



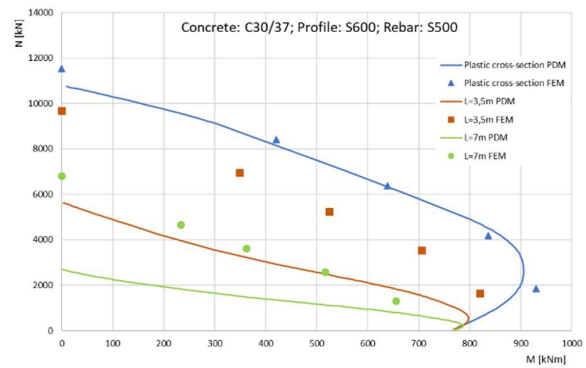
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



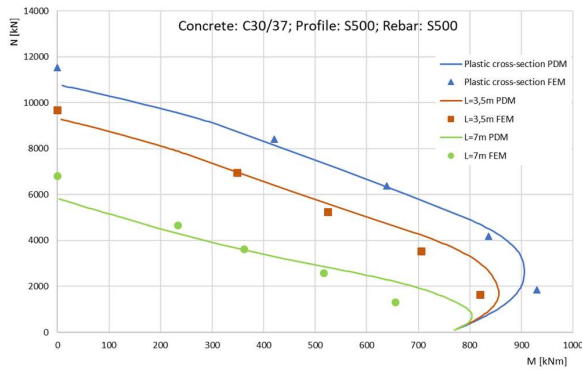
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



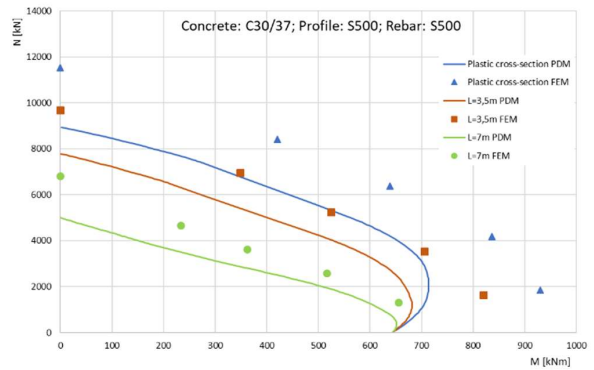
FEC 8 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	600	450x450	30	4 ϕ 20	500	46	0.65%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	11530	0	93%	78%	93%	93%
	8420	421	98%	81%	98%	98%
	6390	639	99%	85%	99%	99%
	4180	836	102%		102%	102%
L=3,5 m	9690	0	96%	96%	91%	58%
	6960	348	100%	84%	95%	58%
	5250	525	104%	87%	99%	63%
	3530	706	108%		103%	76%
L=7 m	6820	0	85%	73%	70%	40%
	4660	233	94%	81%	79%	47%
	3620	362	100%	85%	85%	53%
	2580	516	106%	89%	93%	61%
δ			0.47	0.52	0.47	0.47
λ	3.5 [m]		0.53	0.51	0.62	1.22
	7 [m]		1.07	1.02	1.24	1.90

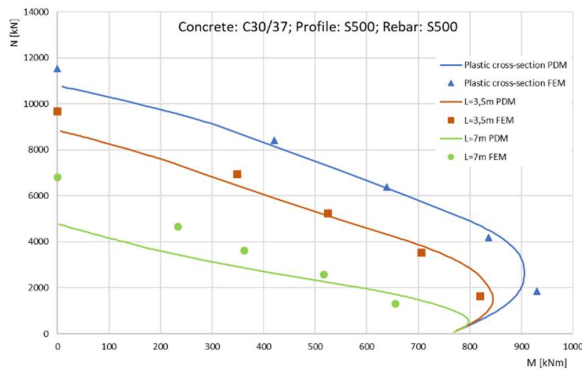
a) No material reduction



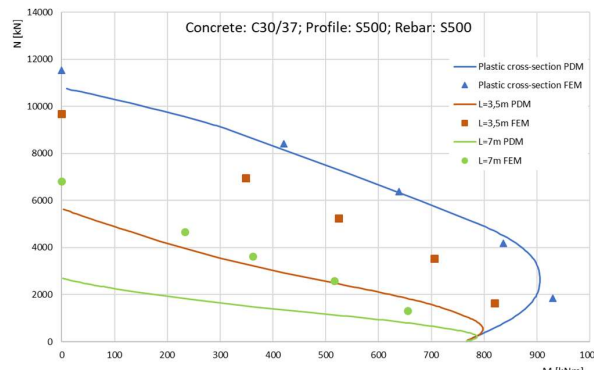
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



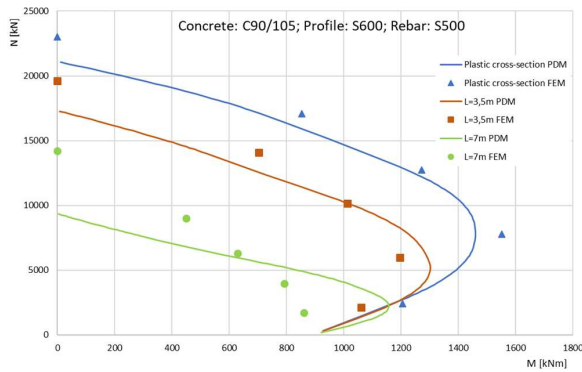
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



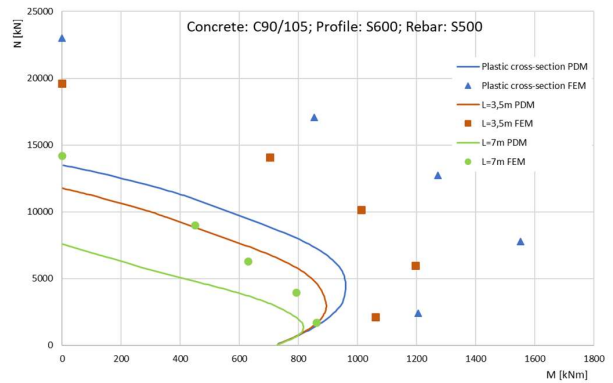
FEC 8 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	600	450x450	90	4 ϕ 20	500	46	0.65%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	23020	0	91%	59%	91%	91%
	17080	854	93%	65%	93%	93%
	12720	1272	98%		98%	98%
	7760	1552				
L=3,5 m	19600	0	88%	60%	78%	67%
	14080	704	92%	65%	81%	64%
	10130	1013	100%		90%	74%
	5980	1196	110%		100%	
L=7 m	14200	0	66%	53%	46%	42%
	9000	450	80%	63%	58%	53%
	6290	629	96%	76%	72%	69%
	3960	792	116%	88%	93%	85%
δ			0.24	0.35	0.24	0.24
λ	3.5 [m]		0.61	0.50	0.77	1.16
	7 [m]		1.22	1.01	1.54	1.85

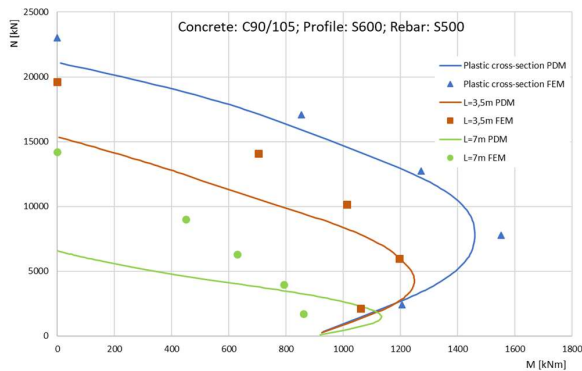
a) No material reduction



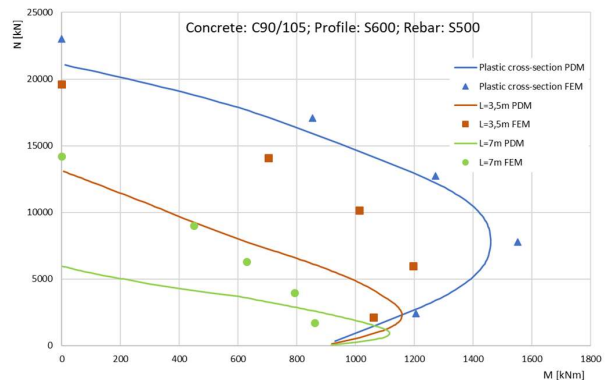
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



FEC 9 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	355	400x400	30	4 ϕ 25	500	55.5	1.29%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	8490	0	94%	80%	94%	94%
	6130	306.5	98%	83%	98%	98%
	4610	461	100%	87%	100%	100%
	3090	618	99%		99%	99%
L=3,5 m	7170	0	98%	84%	95%	69%
	5060	253	102%	87%	99%	69%
	3820	382	106%	91%	103%	74%
	2640	528	105%		103%	84%
L=7 m	5220	0	89%	76%	78%	50%
	3600	180	95%	81%	85%	56%
	2810	281	99%	85%	90%	62%
	2020	404	103%	89%	95%	71%
δ			0.37	0.42	0.37	0.37
λ	3.5 [m]		0.50	0.48	0.56	1.03
	7 [m]		1.01	0.95	1.11	1.58

a) No material reduction



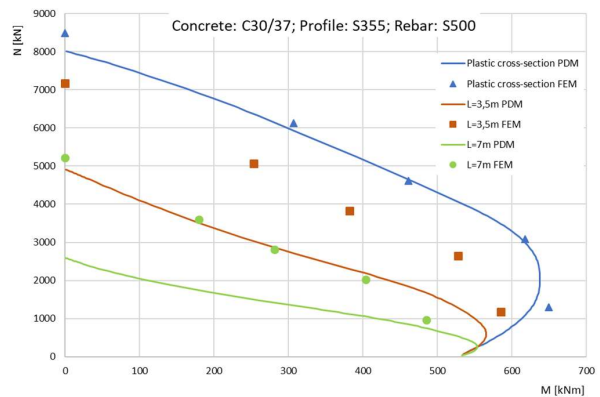
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



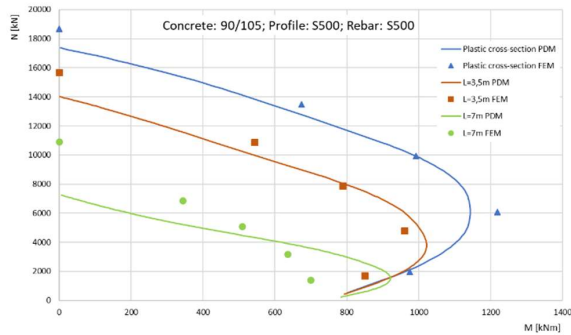
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



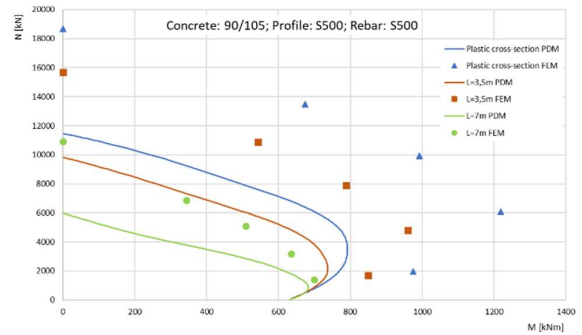
FEC 9 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	500	400x400	90	4 ϕ 25	500	55.5	1.29%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	18680	0	93%	61%	93%	93%
	13490	674.5	96%	66%	96%	96%
	9920	992	100%		100%	100%
	6090	1218				
L=3,5 m	15690	0	89%	63%	82%	70%
	10870	543.5	94%	67%	86%	69%
	7890	789	101%		93%	78%
	4800	960	105%		99%	
L=7 m	10930	0	67%	55%	51%	45%
	6870	343.5	81%	64%	64%	57%
	5090	509	92%	73%	74%	67%
	3180	636	112%	87%	95%	88%
	δ		0.25	0.35	0.25	0.25
λ	3.5 [m]		0.63	0.53	0.75	1.09
	7 [m]		1.27	1.06	1.51	1.77

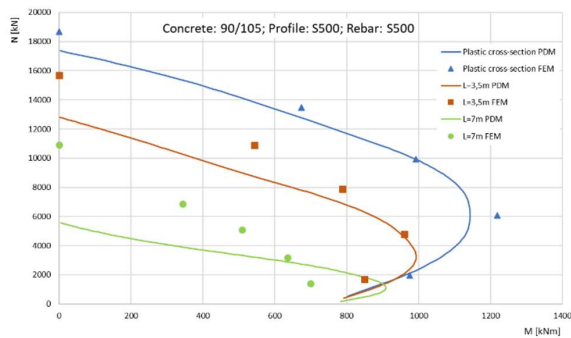
a) No material reduction



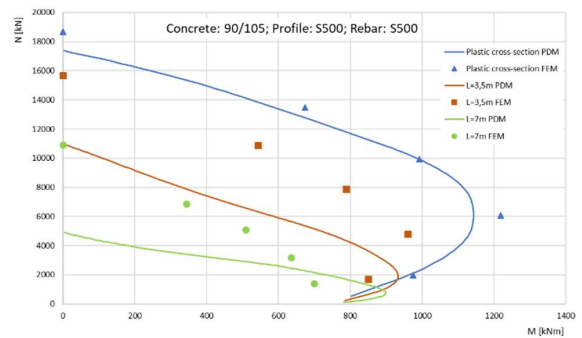
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



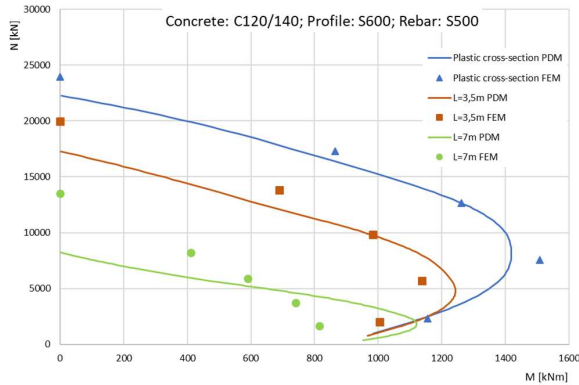
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



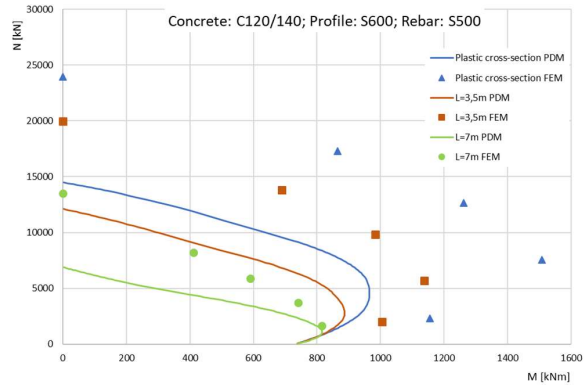
FEC 9 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	600	400x400	120	4 ϕ 25	500	55.5	1.29%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	23970	0	93%	60%	93%	93%
	17300	865	98%	67%	98%	98%
	12630	1263	100%		100%	100%
	7540	1508				
L=3,5 m	19990	0	86%	61%	76%	70%
	13790	689.5	91%	66%	81%	69%
	9830	983	100%		90%	78%
	5690	1138	110%		101%	
L=7 m	13520	0	61%	51%	45%	45%
	8220	411	77%	62%	59%	59%
	5900	590	92%	74%	71%	72%
	3700	740	113%	87%	93%	94%
	δ		0.23	0.34	0.23	0.23
λ	3.5 [m]		0.69	0.57	0.83	1.13
	7 [m]		1.37	1.13	1.66	1.82

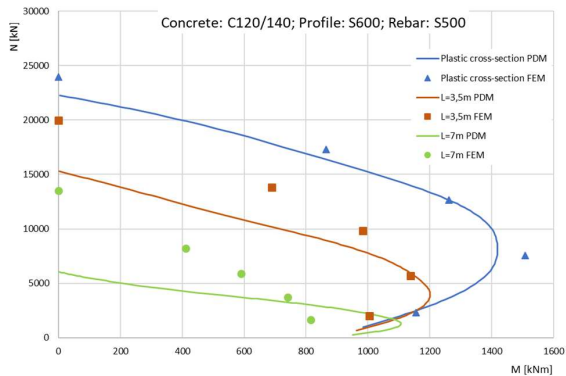
a) No material reduction



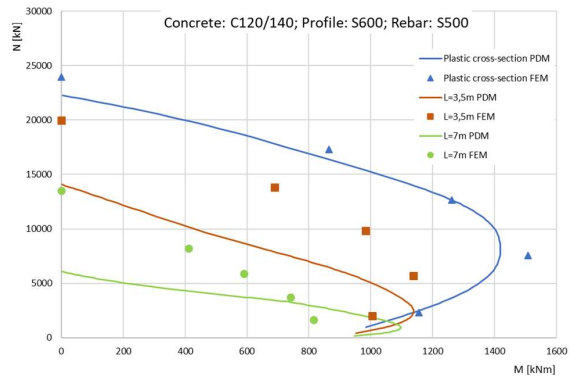
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



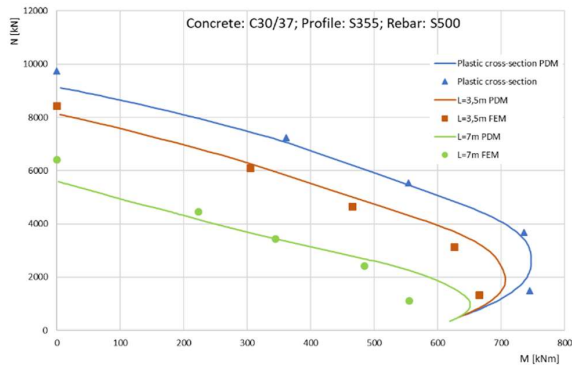
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



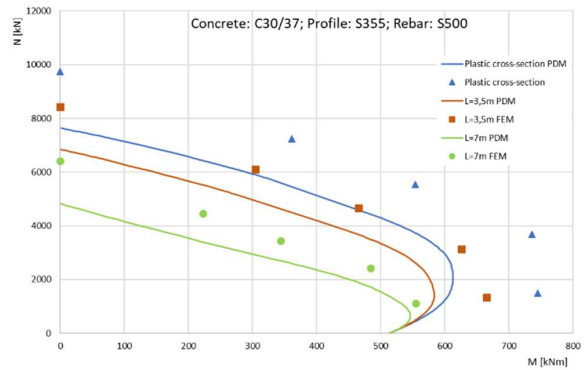
FEC 10 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	355	450x450	30	4 ϕ 25	500	55.5	1.01%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	9750	0	93%	78%	93%	93%
	7230	361.5	98%	83%	98%	98%
	5540	554	99%	86%	99%	99%
	3680	736	99%		99%	99%
L=3,5 m	8430	0	96%	81%	93%	65%
	6100	305	102%	86%	98%	66%
	4650	465	104%	89%	100%	70%
	3130	626	106%		103%	
L=7 m	6420	0	87%	75%	75%	44%
	4460	223	95%	82%	82%	51%
	3440	344	100%	86%	88%	57%
	2420	484	105%	90%	95%	68%
δ			0.33	0.37	0.33	0.33
λ	3.5 [m]		0.48	0.45	0.54	1.06
	7 [m]		0.95	0.90	1.09	1.65

a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



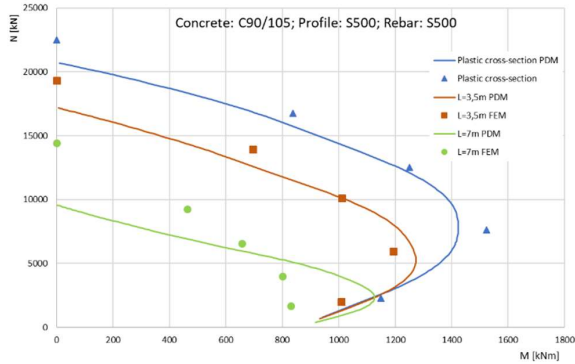
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



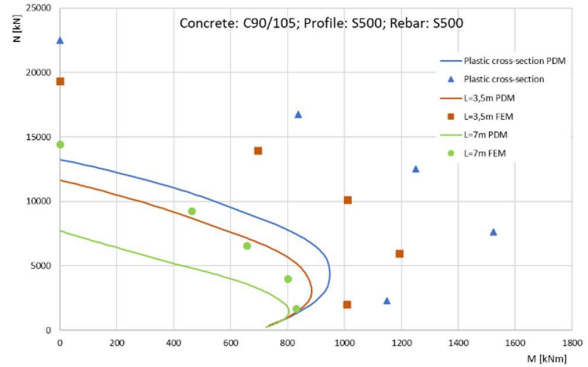
FEC 10 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	500	450x450	90	4 ϕ 25	500	55.5	1.01%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	22500	0	92%	59%	92%	92%
	16760	838	98%	65%	98%	98%
	12510	1251	98%		98%	98%
	7620	1524				
L=3,5 m	19330	0	89%	60%	80%	69%
	13930	696.5	93%	65%	83%	67%
	10110	1011	100%		91%	77%
	5970	1194	108%		99%	
L=7 m	14420	0	66%	54%	48%	43%
	9250	462.5	80%	63%	59%	54%
	6560	656	94%	75%	73%	69%
	4000	800	116%		95%	87%
δ			0.21	0.31	0.21	0.21
λ	3.5 [m]		0.59	0.48	0.73	1.10
	7 [m]		1.18	0.97	1.47	1.76

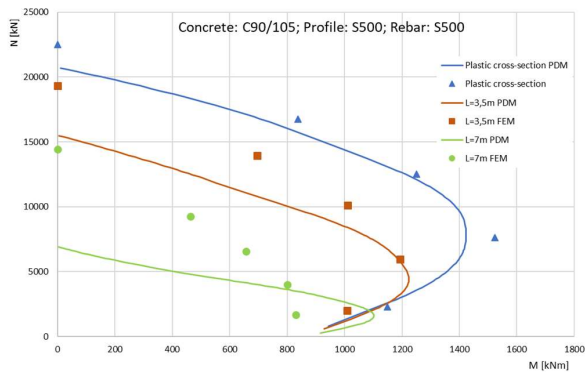
a) No material reduction



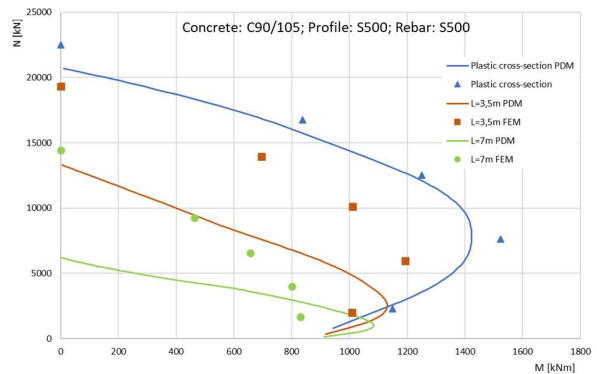
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



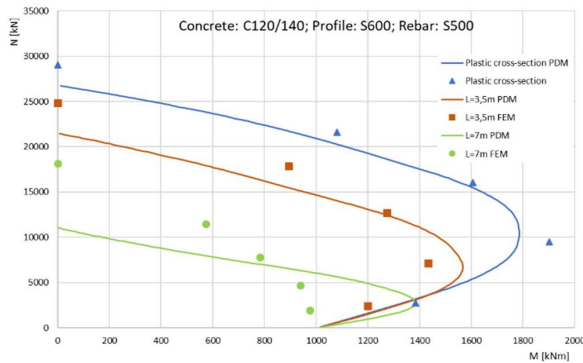
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



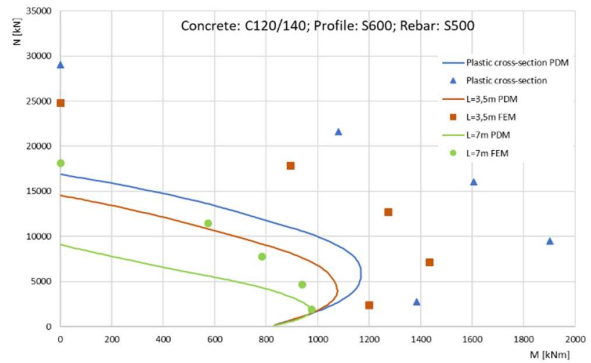
FEC 10 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD260x68,2	600	450x450	120	4 ϕ 25	500	55.5	1.01%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	29030	0	92%	58%	92%	92%
	21630	1081.5	96%	67%	96%	96%
	16060	1606	98%		98%	98%
	9510	1902				
L=3,5 m	24820	0	86%	59%	75%	70%
	17860	893	98%	64%	78%	67%
	12730	1273	99%		88%	78%
	7170	1434	112%		101%	
L=7 m	18150	0	61%	50%	42%	44%
	11470	573.5	75%	60%	53%	56%
	7820	782	93%	75%	69%	73%
	4700	940	118%	90%	93%	94%
	δ		0.19	0.29	0.19	0.19
λ	3.5 [m]		0.64	0.52	0.81	1.13
	7 [m]		1.27	1.04	1.61	1.79

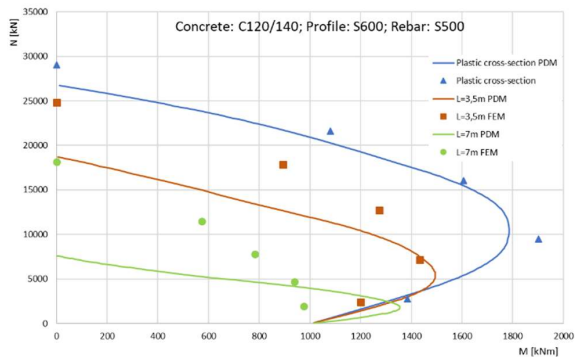
a) No material reduction



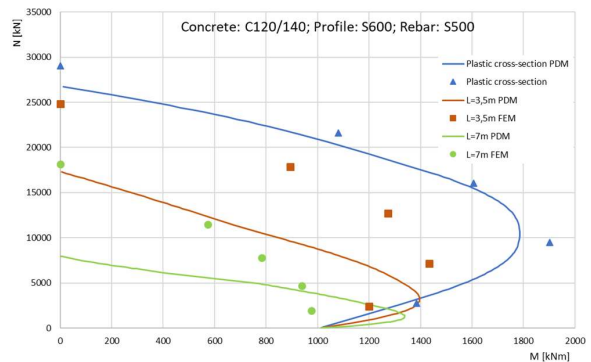
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



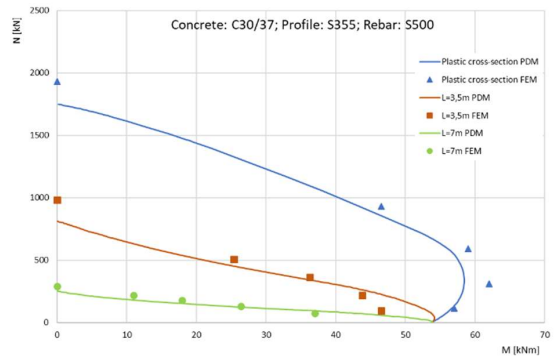
FEC 11 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB100	355	180x180	30	4 ϕ 10	500	31	1.05%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	1930	0	99%	91%	99%	99%
	930	46.5	106%	96%	106%	106%
	590	59	104%		104%	104%
	310	62				
L=3,5 m	982	0	85%	83%	73%	45%
	508	25.4	97%	93%	87%	59%
	363	36.3	102%	97%	93%	68%
	219	43.8	112%	106%	105%	85%
L=7 m	290	0	88%	87%	73%	64%
	220	11	88%	85%	74%	66%
	179	17.9	93%	90%	80%	72%
	132	26.4	101%	97%	89%	81%
δ			0.49	0.53	0.49	0.49
λ	3.5 [m]		1.24	1.18	1.36	1.92
	7 [m]		2.47	2.36	2.73	2.98

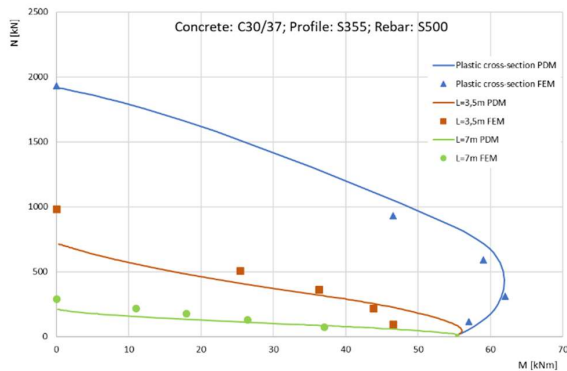
a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



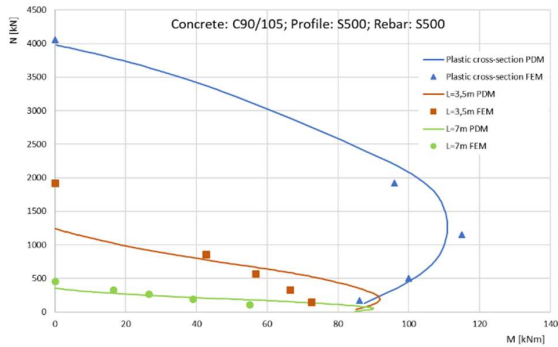
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



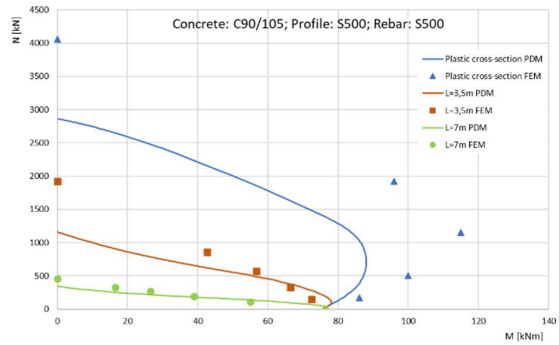
FEC 11 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB100	500	180x180	90	4 ϕ 10	500	31	1.05%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	4060	0	98%	71%	98%	98%
	1920	96	106%	71%	106%	106%
	1150	115				
	500	100				
L=3,5 m	1921	0	65%	60%	48%	43%
	854	42.7	94%	81%	74%	67%
	568	56.8	110%	93%	91%	84%
	332	66.4	127%	103%	111%	104%
L=7 m	458	0	78%	76%	56%	58%
	329	16.45	87%	81%	65%	67%
	266	26.6	95%	86%	72%	75%
	195	39	108%	96%	85%	88%
δ			0.34	0.46	0.34	0.34
λ	3.5 [m]		1.50	1.28	1.79	2.09
	7 [m]		3.01	2.56	3.59	3.41

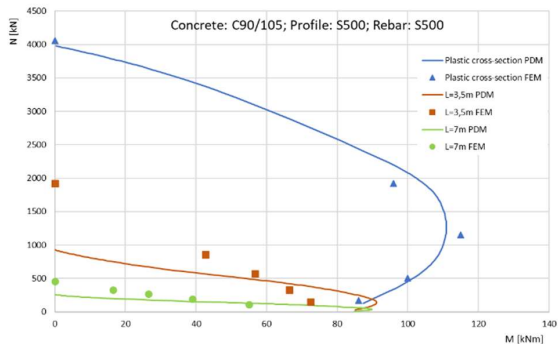
a) No material reduction



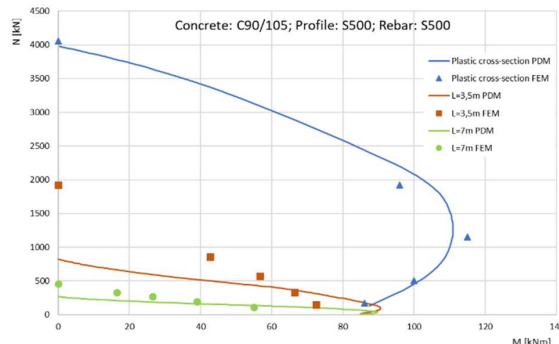
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



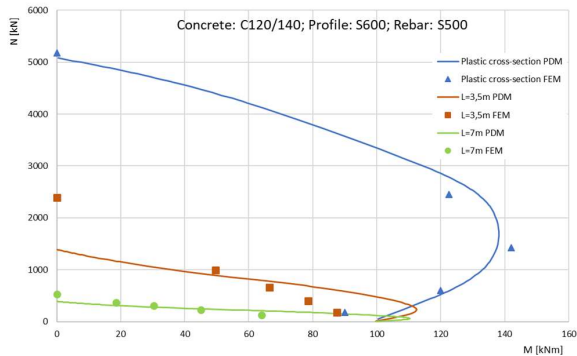
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



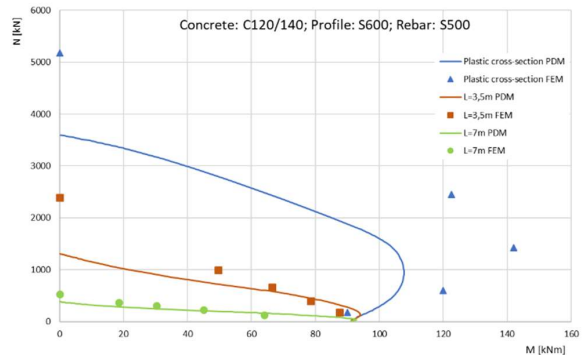
FEC 11 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB100	600	180x180	120	4 ϕ 10	500	31	1.05%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	5180	0	98%	69%	98%	98%
	2450	122.5	106%	71%	106%	106%
	1420	142				
	600	120				
L=3,5 m	2388	0	58%	55%	42%	42%
	991	49.55	93%	81%	71%	72%
	664	66.4	110%	93%	88%	89%
	393	78.6	129%	104%	110%	109%
L=7 m	532	0	74%	72%	51%	59%
	371	18.55	86%	81%	62%	71%
	304	30.4	94%	86%	69%	78%
	225	45	107%	95%	81%	91%
δ			0.32	0.44	0.32	0.32
λ	3.5 [m]		1.62	1.37	1.97	2.15
	7 [m]		3.24	2.74	3.94	3.46

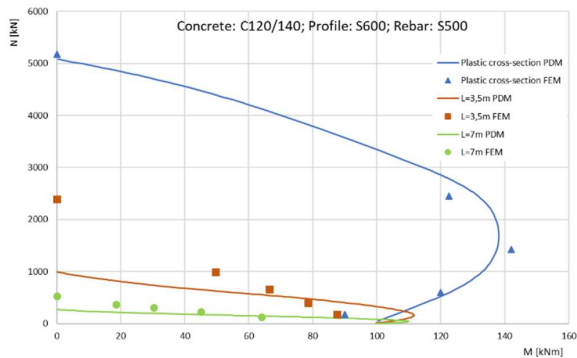
a) No material reduction



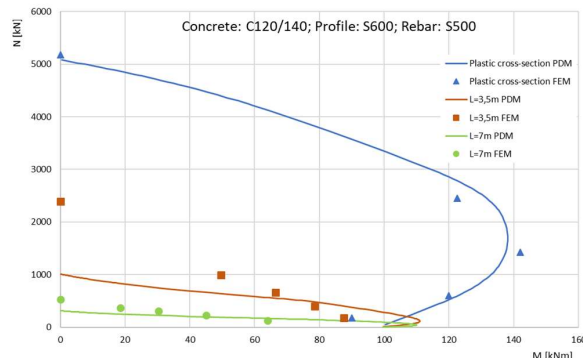
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



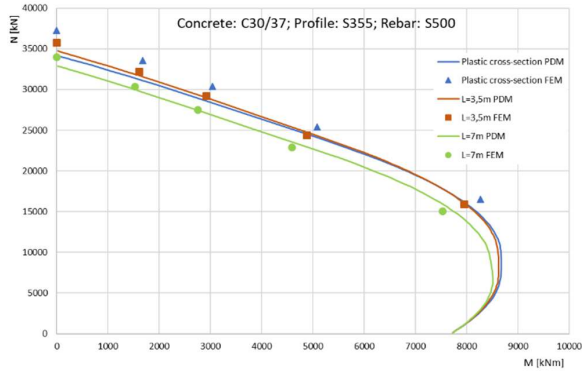
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



FEC 12 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB1000	355	1200x600	30	10 ϕ 28	500	60	0.90%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	37270	0	92%	72%	92%	92%
	33540	1677	93%	81%	93%	93%
	30410	3041	94%	86%	94%	94%
	25390	5078	96%	95%	96%	96%
L=3,5 m	35780	0	97%	88%	97%	90%
	32190	1609.5	99%	89%	98%	91%
	29200	2920	99%	90%	99%	92%
	24390	4878	101%	91%	101%	94%
L=7 m	34000	0	97%	87%	96%	83%
	30400	1520	99%	89%	98%	84%
	27540	2754	100%	90%	99%	85%
	22940	4588	102%	92%	101%	86%
δ			0.41	0.45	0.41	0.41
λ	3.5 [m]		0.15	0.14	0.16	0.36
	7 [m]		0.29	0.28	0.31	0.62

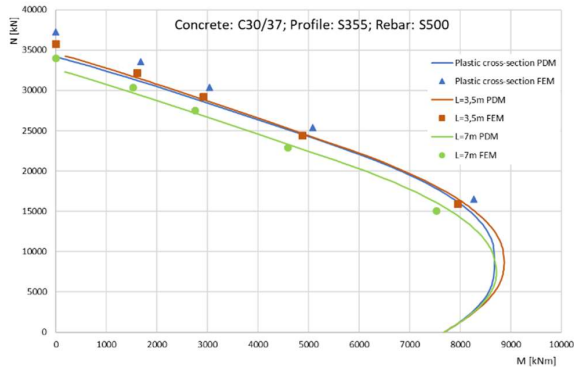
a) No material reduction



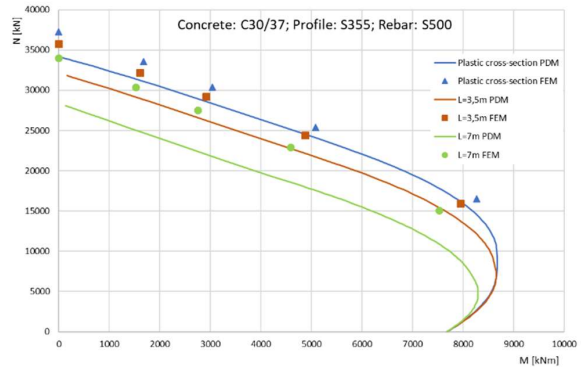
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



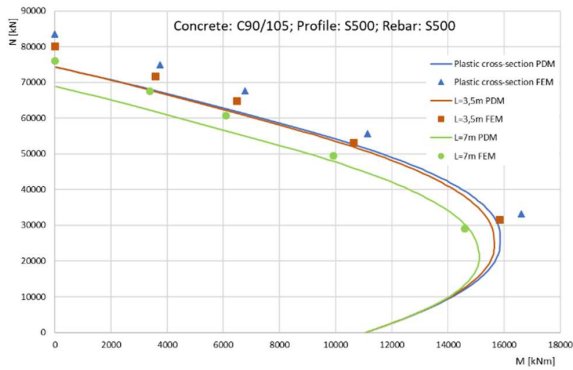
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



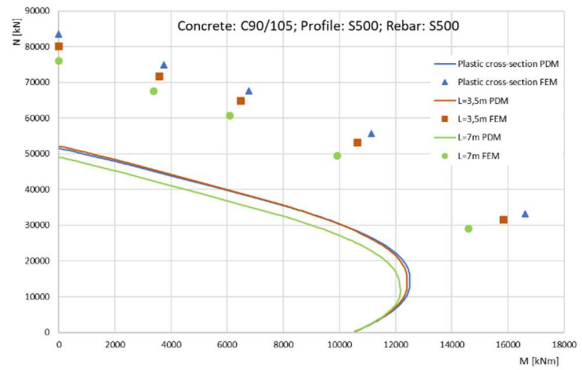
FEC 12 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB1000	500	1200x600	90	10 ϕ 28	500	60	0.90%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	83500	0	89%	62%	89%	89%
	74880	3744	91%	63%	91%	91%
	67630	6763	92%	64%	92%	92%
	55680	11136	95%	71%	95%	95%
L=3,5 m	80210	0	93%	65%	93%	88%
	71730	3586.5	95%	66%	95%	89%
	64800	6480	96%	67%	96%	90%
	53180	10636	98%	73%	99%	93%
L=7 m	76030	0	91%	65%	89%	81%
	67590	3379.5	93%	66%	92%	82%
	60830	6083	94%	67%	93%	82%
	49520	9904	98%	73%	97%	85%
δ			0.26	0.38	0.26	0.26
λ	3.5 [m]		0.19	0.16	0.22	0.40
	7 [m]		0.38	0.32	0.44	0.69

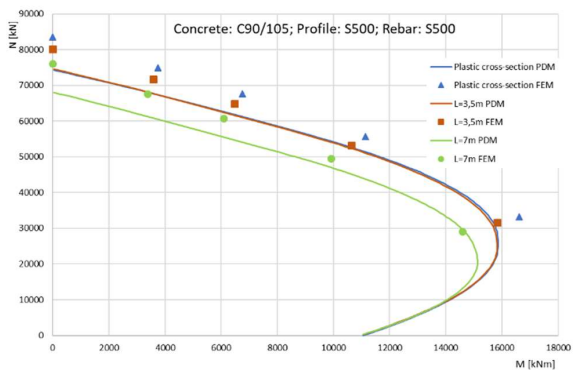
a) No material reduction



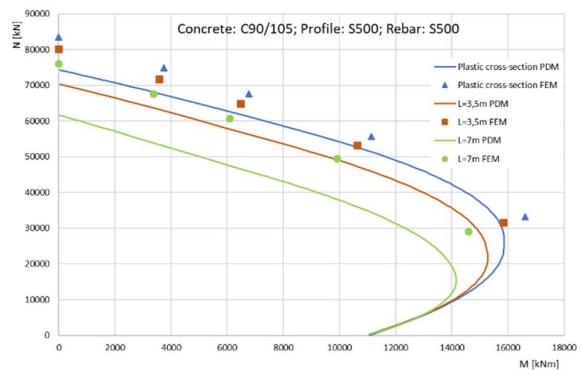
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



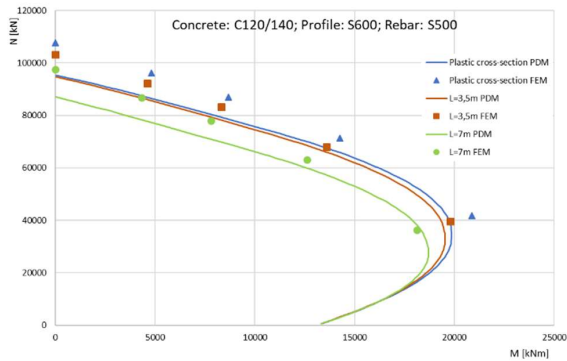
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



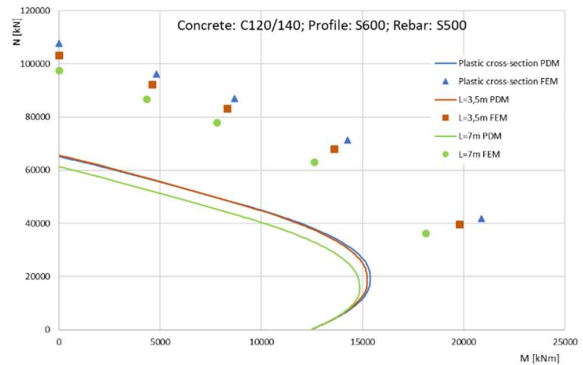
FEC 12 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB1000	600	1200x600	120	10 ϕ 28	500	60	0.90%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	107600	0	89%	60%	89%	89%
	96200	4810	91%	62%	91%	91%
	86940	8694	92%	63%	92%	92%
	71360	14272	95%	72%	95%	95%
L=3,5 m	103109	0	92%	64%	92%	87%
	92190	4609.5	94%	65%	94%	89%
	83260	8326	95%	66%	95%	90%
	68040	13608	98%	73%	98%	92%
L=7 m	97520	0	89%	63%	88%	80%
	86740	4337	91%	64%	89%	81%
	78020	7802	93%	65%	91%	81%
	63109	12621.8	97%	72%	95%	84%
δ			0.25	0.36	0.25	0.25
λ	3.5 [m]		0.21	0.17	0.24	0.42
	7 [m]		0.42	0.35	0.49	0.72

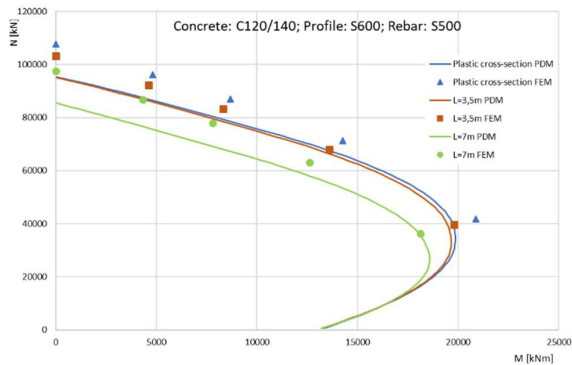
a) No material reduction



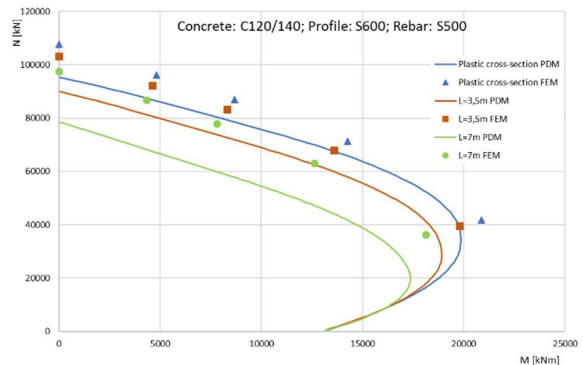
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



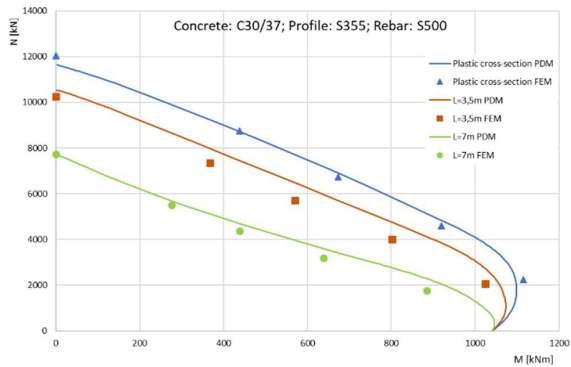
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



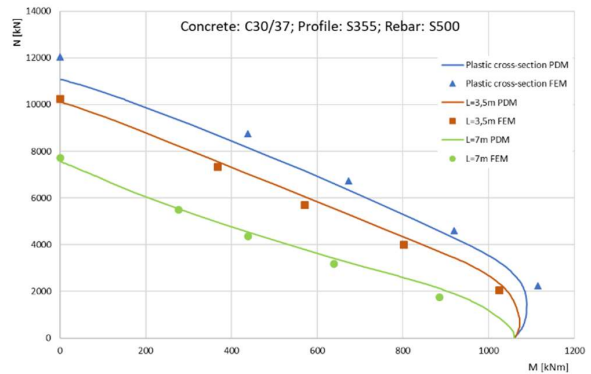
FEC 13 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEM260	355	410x380	30	4 ϕ 14	500	37	0.46%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	12040	0	97%	92%	97%	97%
	8750	437.5	100%	95%	100%	100%
	6730	673	101%	97%	101%	101%
	4600	920	102%	97%	102%	102%
L=3,5 m	10240	0	103%	99%	103%	81%
	7340	367	106%	102%	107%	82%
	5710	571	108%	103%	108%	85%
	4010	802	107%	103%	108%	88%
L=7 m	7730	0	100%	100%	97%	65%
	5510	275.5	102%	102%	100%	70%
	4380	438	105%	105%	103%	74%
	3190	638	106%	106%	105%	79%
δ			0.67	0.71	0.67	0.67
λ	3.5 [m]		0.44	0.43	0.46	0.84
	7 [m]		0.88	0.86	0.93	1.30

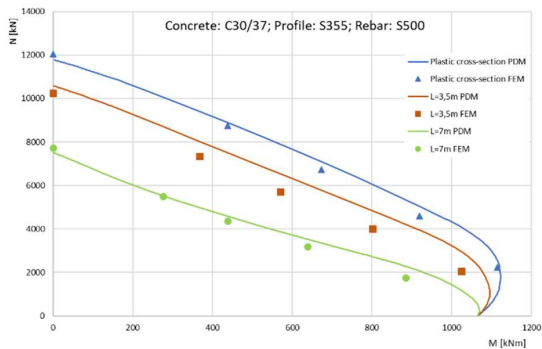
a) No material reduction



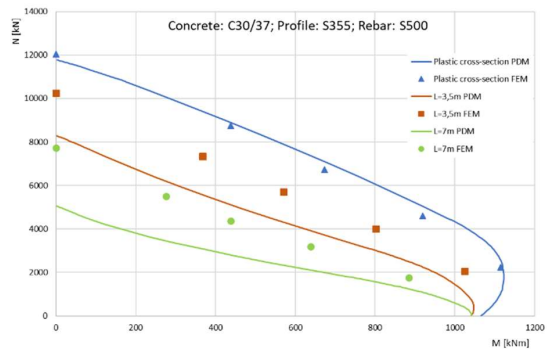
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



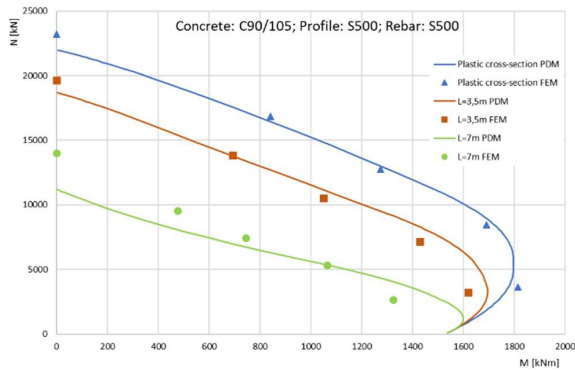
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



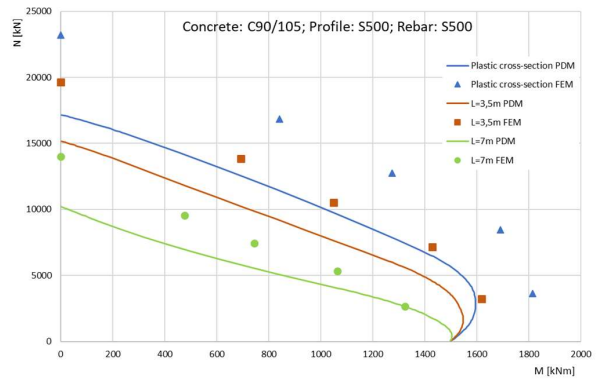
FEC 13 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEM260	500	410x380	90	4 ϕ 14	500	37	0.46%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	23190	0	95%	74%	95%	95%
	16840	842	98%	77%	98%	98%
	12750	1275	101%	79%	101%	101%
	8460	1692	101%		101%	101%
L=3,5 m	19620	0	95%	77%	93%	80%
	13850	692.5	100%	81%	97%	81%
	10500	1050	104%	84%	102%	85%
	7150	1430	106%	89%	104%	91%
L=7 m	14000	0	80%	73%	72%	59%
	9540	477	88%	79%	80%	67%
	7440	744	94%	83%	86%	74%
	5320	1064	100%	87%	94%	81%
δ			0.50	0.64	0.50	0.50
λ	3.5 [m]		0.56	0.50	0.61	0.89
	7 [m]		1.12	0.99	1.22	1.48

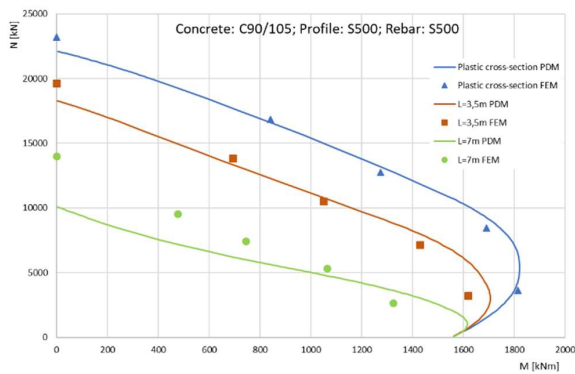
a) No material reduction



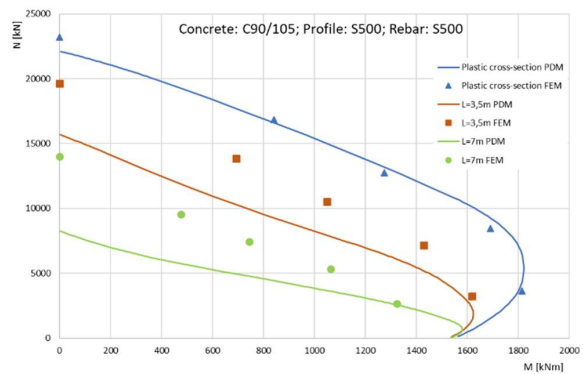
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



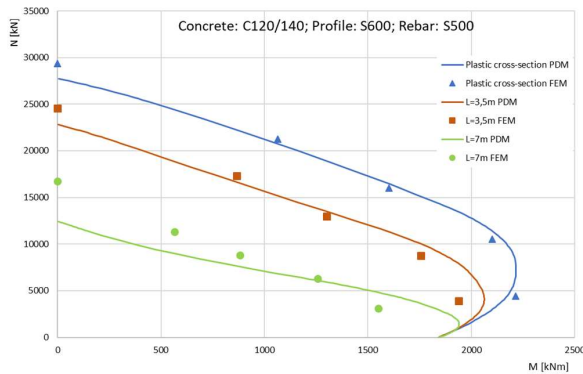
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



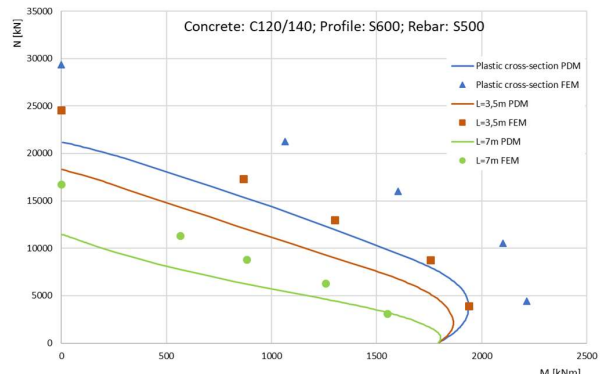
FEC 13 - Strong axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEM260	600	410x380	120	4 ϕ 14	500	37	0.46%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	29330	0	95%	72%	95%	95%
	21270	1063.5	98%	75%	98%	98%
	16010	1601	102%	78%	102%	102%
	10500	2100	102%		102%	102%
L=3,5 m	24550	0	93%	75%	90%	79%
	17310	865.5	97%	78%	94%	79%
	13000	1300	102%	82%	99%	84%
	8780	1756	105%	89%	103%	90%
L=7 m	16740	0	74%	69%	90%	57%
	11310	565.5	84%	75%	94%	66%
	8810	881	90%	80%	99%	73%
	6280	1256	98%	86%	103%	82%
	δ		0.48	0.62	0.48	0.48
λ	3.5 [m]		0.61	0.54	0.68	0.94
	7 [m]		1.22	1.08	1.36	1.58

a) No material reduction



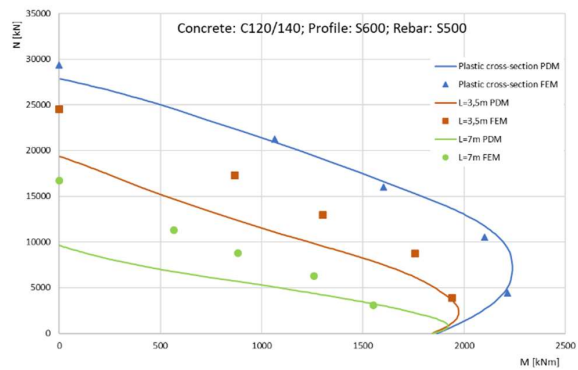
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard

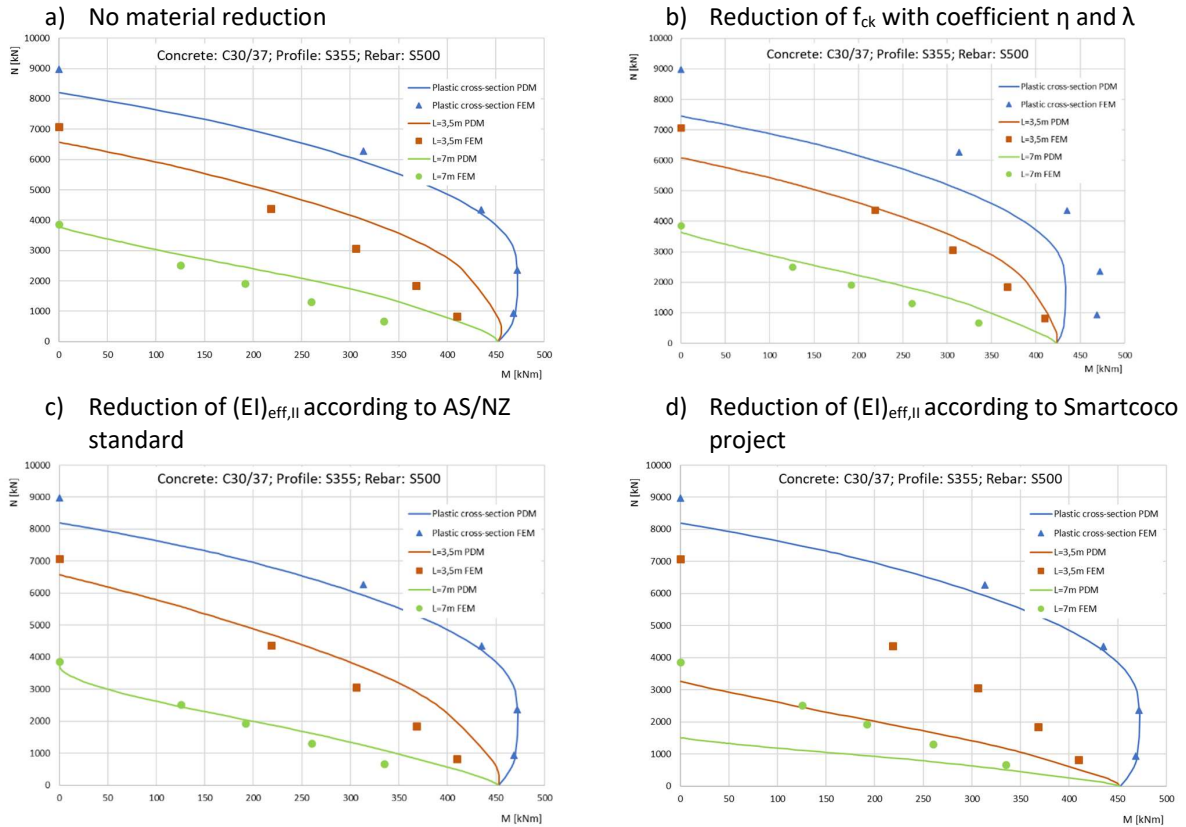


d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



FEC 1 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB260	355	400x400	30	4 ϕ 16	500	35	0.54%

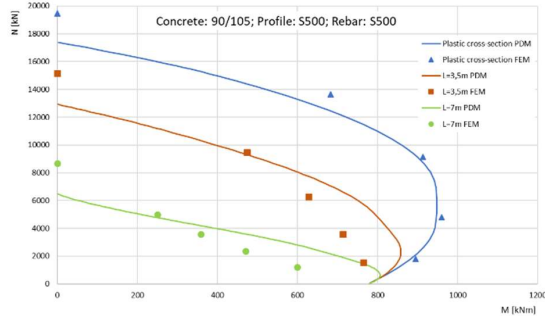
		A	B	C	D
δ		0.49	0.54	0.49	0.49
λ	3.5 [m]	0.63	0.60	0.74	1.44
	7 [m]	1.26	1.20	1.48	2.21



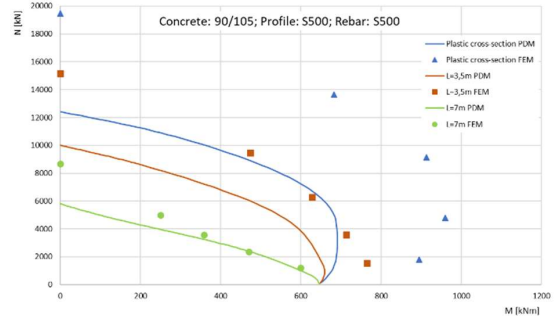
FEC 1 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB260	500	400x400	90	4 ϕ 16	500	35	0.54%

		A	B	C	D
δ		0.33	0.46	0.33	0.33
λ	3.5 [m]	0.74	0.62	0.95	1.42
	7 [m]	1.48	1.25	1.89	2.24

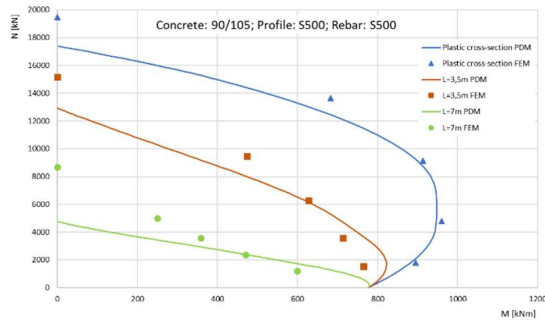
a) No material reduction



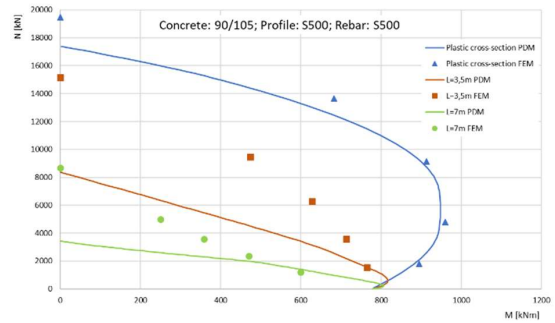
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



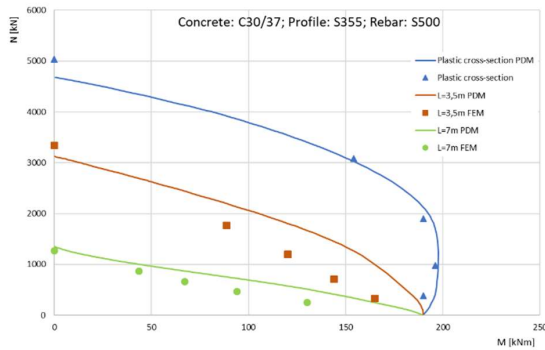
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



FEC 2 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB180	355	300x300	30	4 ϕ 14	500	38	0.74%

		A	B	C	D
δ		0.48	0.53	0.48	0.48
λ	3.5 [m]	0.87	0.83	1.03	1.83
	7 [m]	1.73	1.65	2.05	2.76

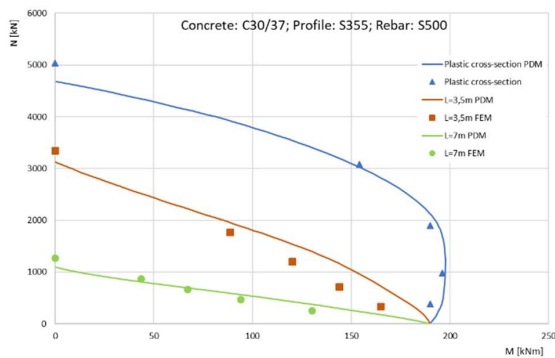
a) No material reduction



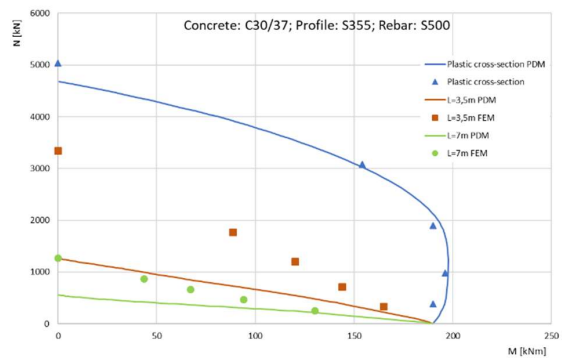
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



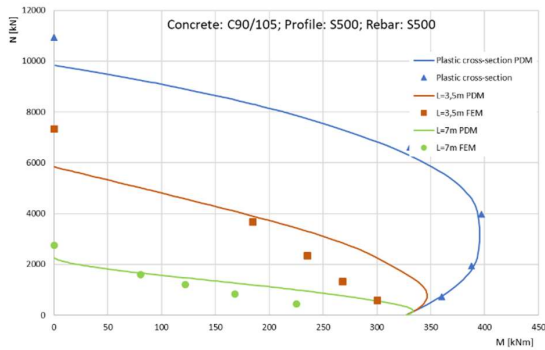
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



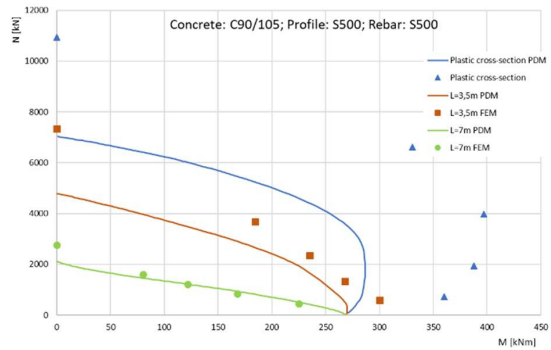
FEC 2 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB180	500	300x300	90	4 ϕ 14	500	38	0.74%

		A	B	C	D
δ		0.32	0.45	0.32	0.32
λ	3.5 [m]	1.00	0.85	1.30	1.79
	7 [m]	2.01	1.70	2.59	2.79

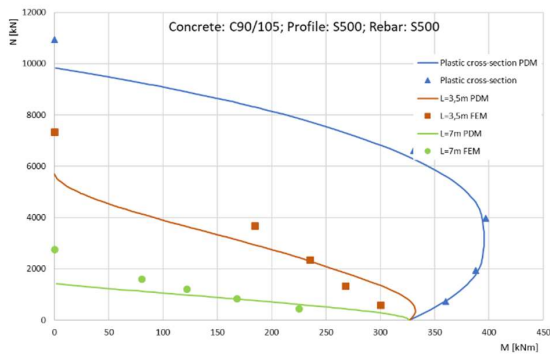
a) No material reduction



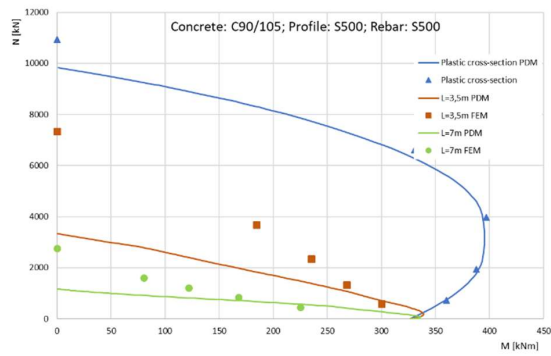
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



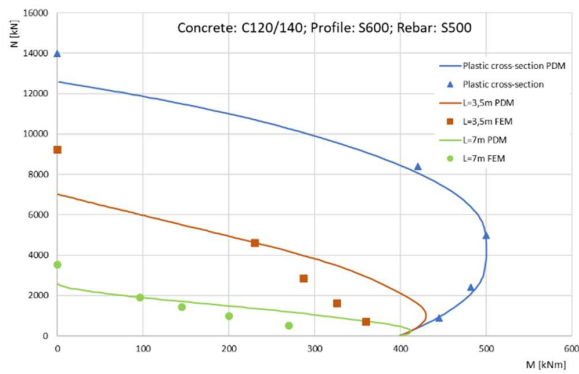
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



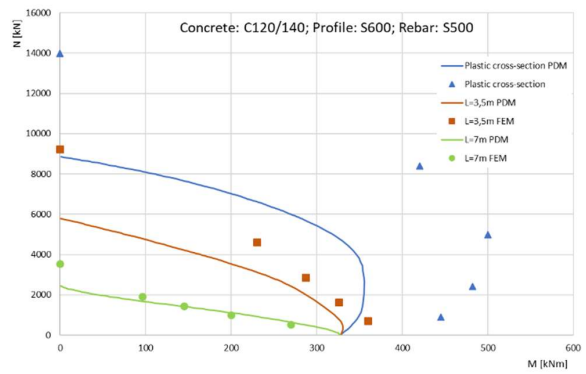
FEC 2 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB180	600	300x300	120	4 ϕ 14	500	38	0.74%

		A	B	C	D
δ		0.30	0.43	0.30	0.30
λ	3.5 [m]	1.07	0.89	1.41	1.79
	7 [m]	2.13	1.79	2.81	2.77

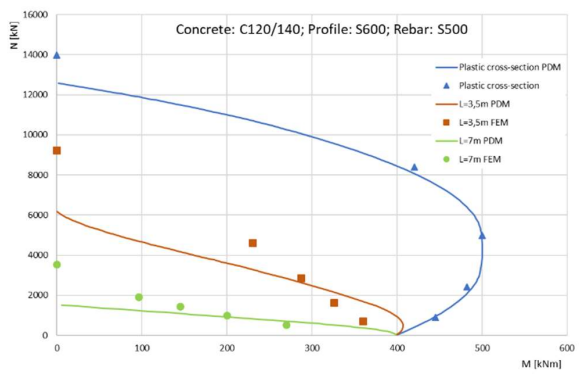
a) No material reduction



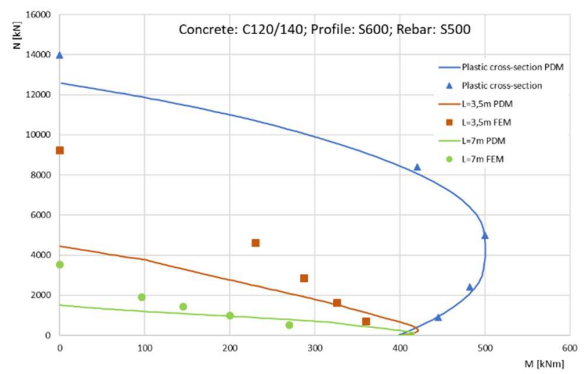
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



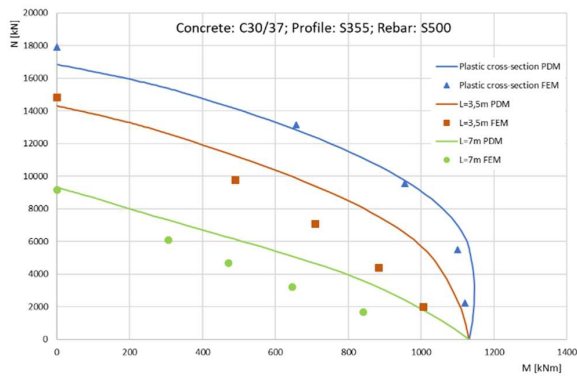
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



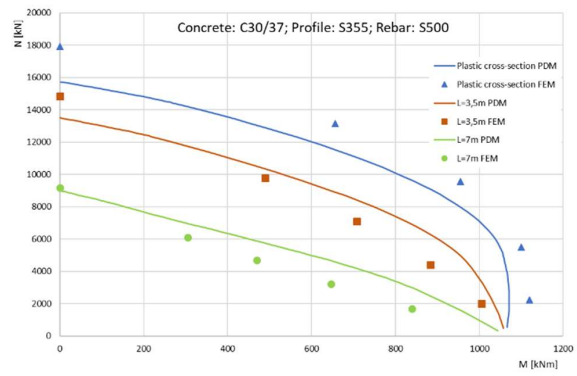
FEC 3 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEM320	355	500x500	30	4 ϕ 16	500	40	0.37%

		A	B	C	D
δ		0.64	0.69	0.64	0.64
λ	3.5 [m]	0.53	0.52	0.61	1.25
	7 [m]	1.07	1.03	1.23	1.97

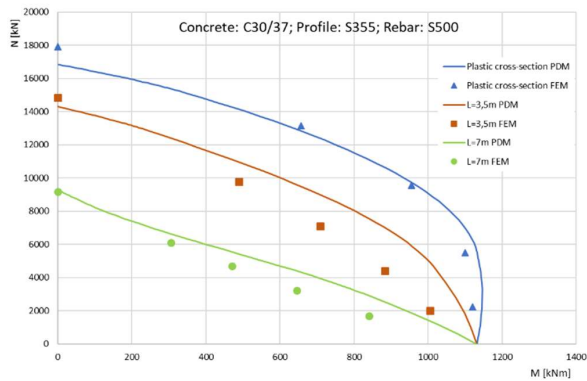
a) No material reduction



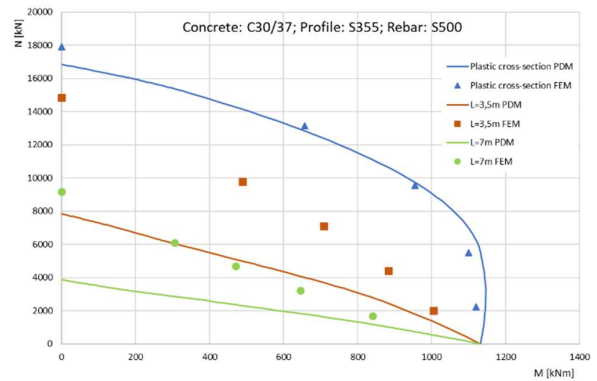
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



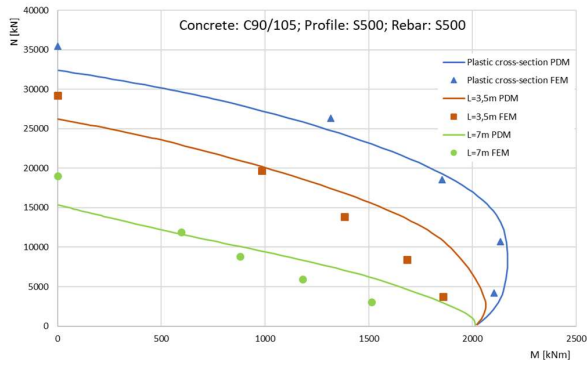
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



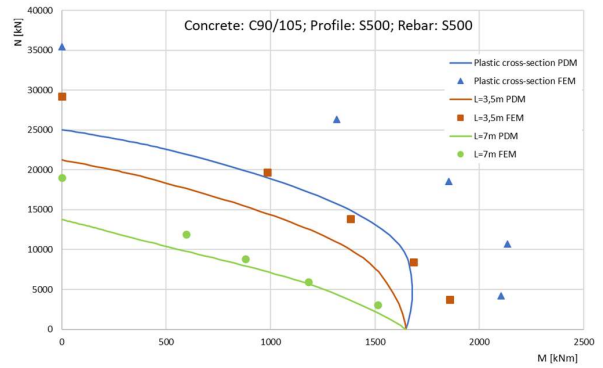
FEC 3 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEM320	500	500x500	90	4 ϕ 16	500	40	0.37%

		A	B	C	D
δ		0.47	0.61	0.47	0.47
λ	3.5 [m]	0.61	0.54	0.76	1.24
	7 [m]	1.23	1.08	1.53	2.02

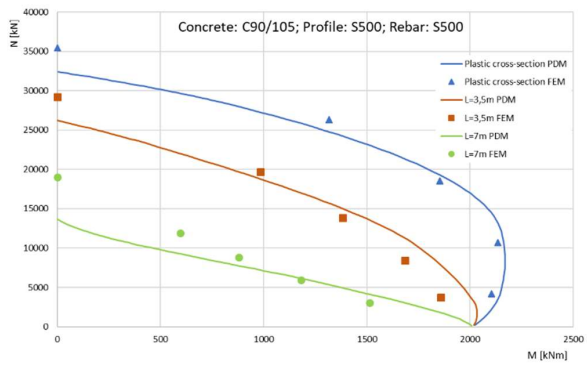
a) No material reduction



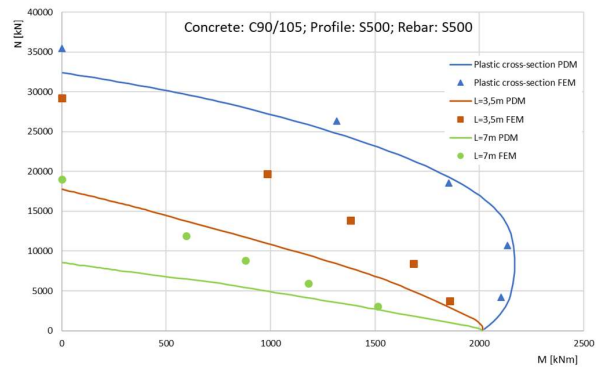
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



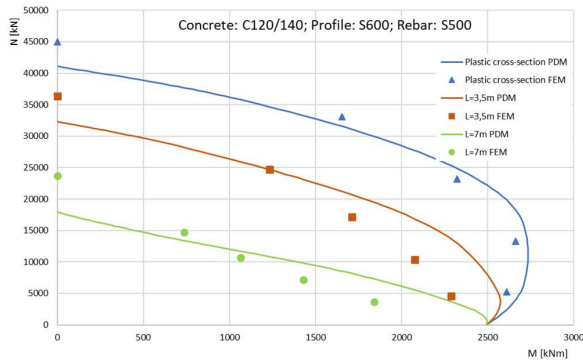
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



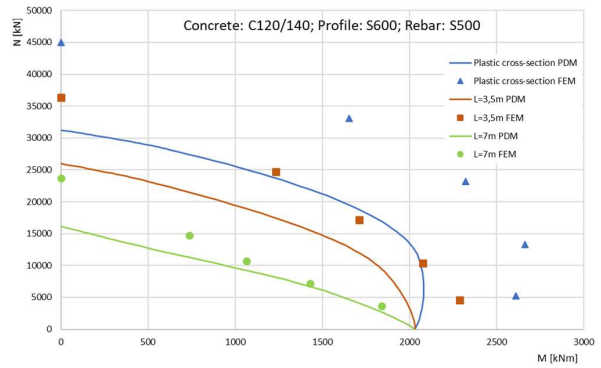
FEC 3 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEM320	600	500x500	120	4 ϕ 16	500	40	0.37%

		A	B	C	D
δ		0.45	0.59	0.45	0.45
λ	3.5 [m]	0.65	0.57	0.83	1.27
	7 [m]	1.31	1.14	1.66	2.06

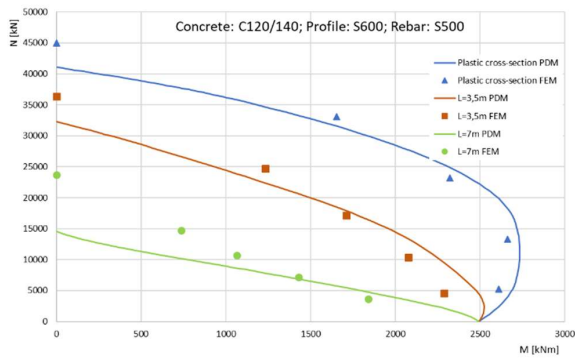
a) No material reduction



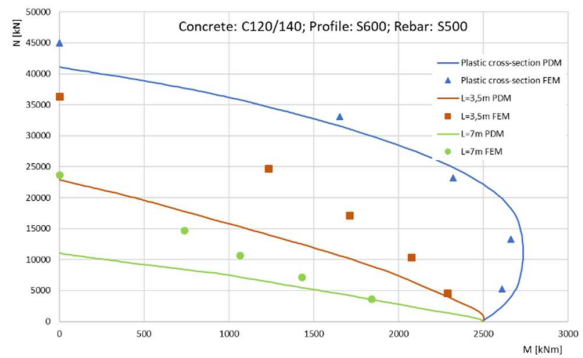
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



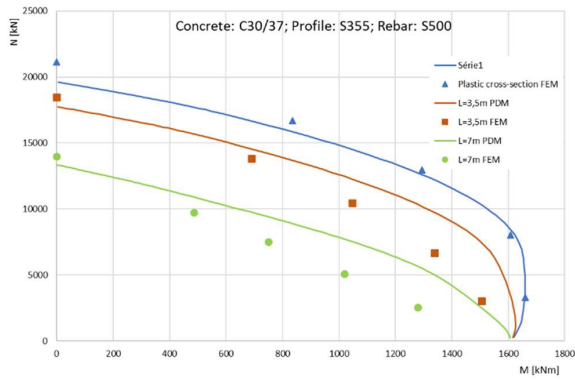
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



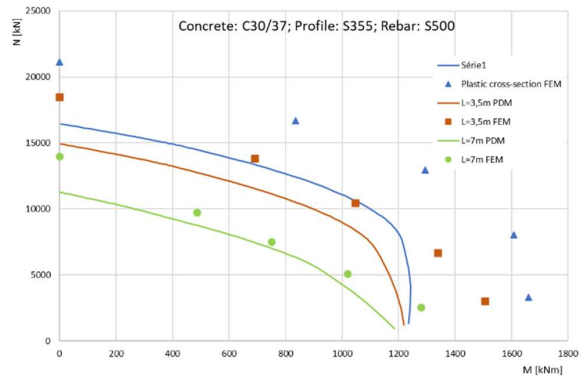
FEC 4 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD400x237	355	600x600	30	4 ϕ 20	500	46	0.38%

		A	B	C	D
δ		0.54	0.59	0.54	0.54
λ	3.5 [m]	0.42	0.40	0.50	1.07
	7 [m]	0.84	0.81	0.99	1.71

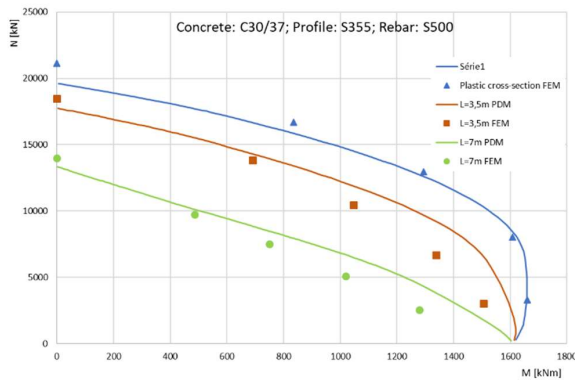
a) No material reduction



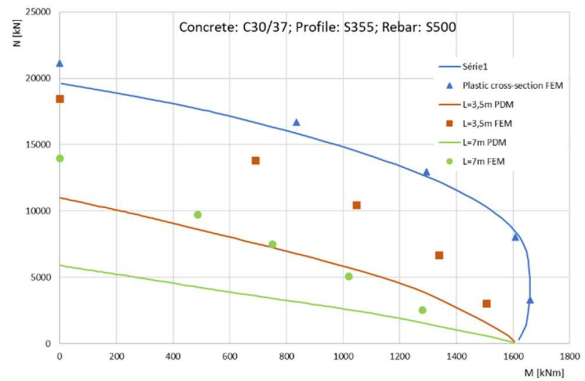
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



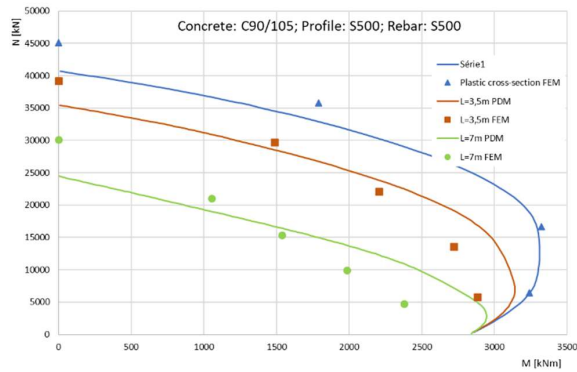
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



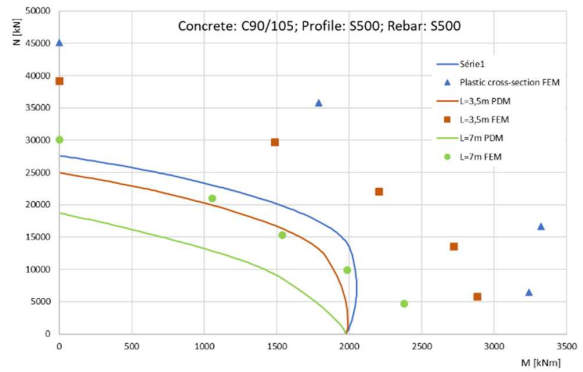
FEC 4 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD400x237	500	600x600	90	4 ϕ 20	500	46	0.38%

		A	B	C	D
δ		0.37	0.50	0.37	0.37
λ	3.5 [m]	0.49	0.42	0.63	1.07
	7 [m]	0.99	0.84	1.26	1.73

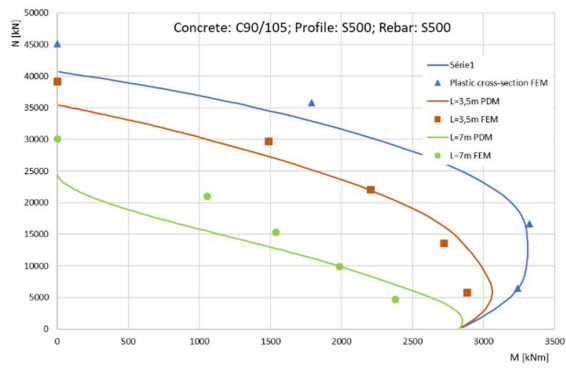
a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



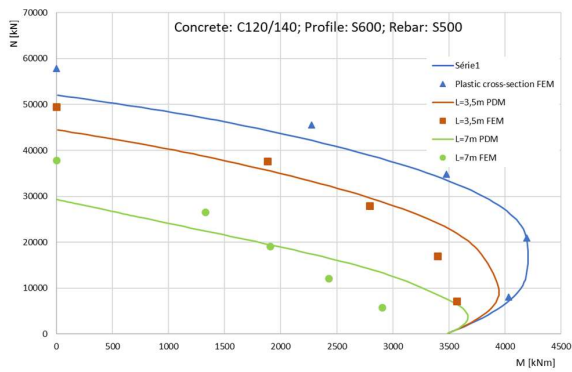
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



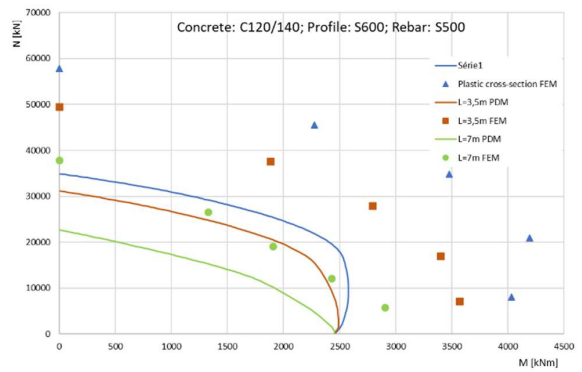
FEC 4 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HD400x237	600	600x600	120	4 ϕ 20	500	46	0.38%

		A	B	C	D
δ		0.34	0.48	0.34	0.34
λ	3.5 [m]	0.53	0.45	0.69	1.09
	7 [m]	1.06	0.89	1.37	1.74

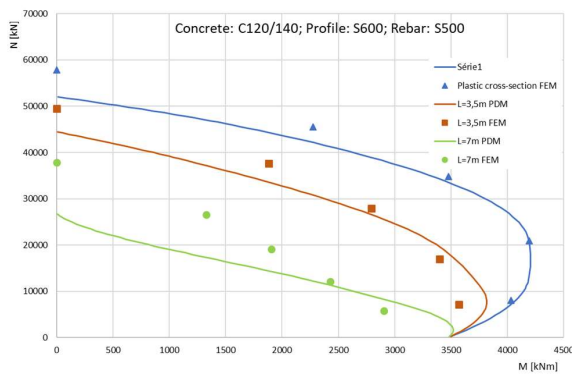
a) No material reduction



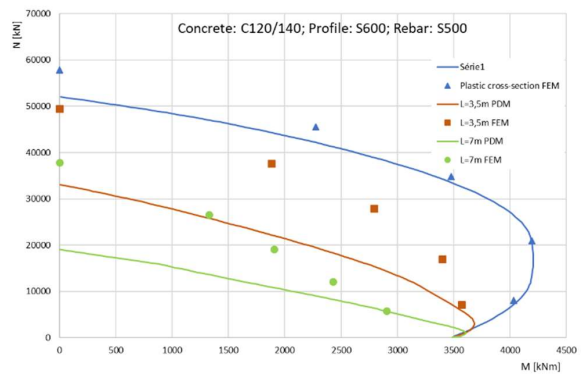
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



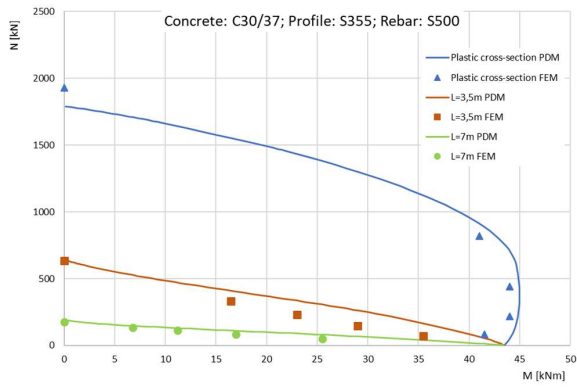
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



FEC 11 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB100	355	180x180	30	4 ϕ 10	500	31	1.05%

		A	B	C	D
δ		0.49	0.53	0.49	0.49
λ	3.5 [m]	1.51	1.45	1.78	2.62
	7 [m]	3.03	2.89	3.56	4.06

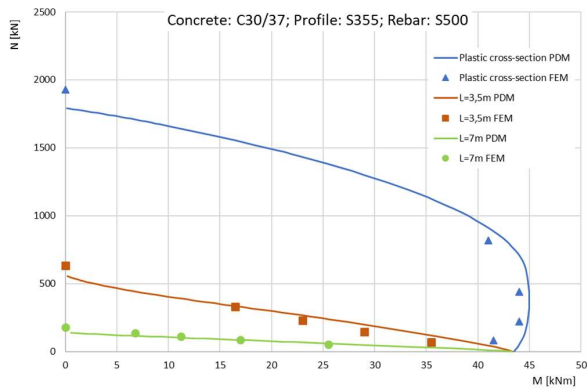
a) No material reduction



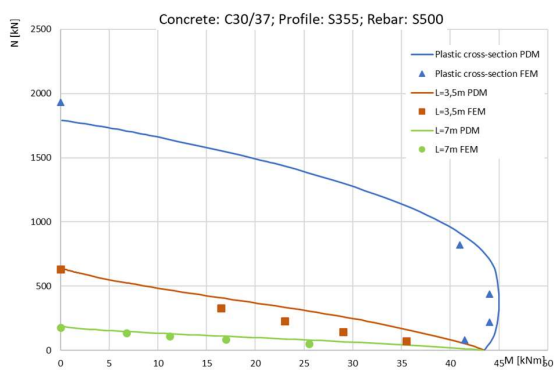
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



FEC 12 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB1000	355	1200x600	30	10 ϕ 28	500	60	0.90%

		A	B	C	D
δ		0.41	0.45	0.41	0.41
λ	3.5 [m]	0.43	0.41	0.53	1.18
	7 [m]	0.86	0.82	1.07	1.89

a) No material reduction



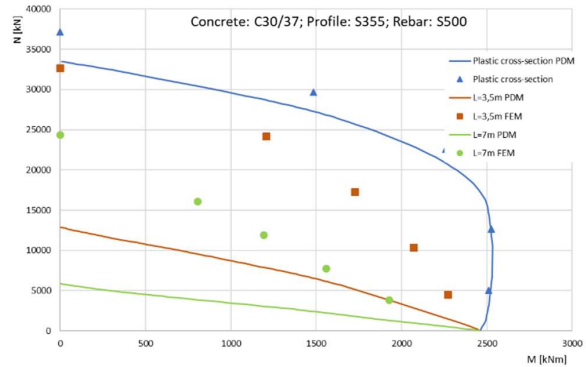
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



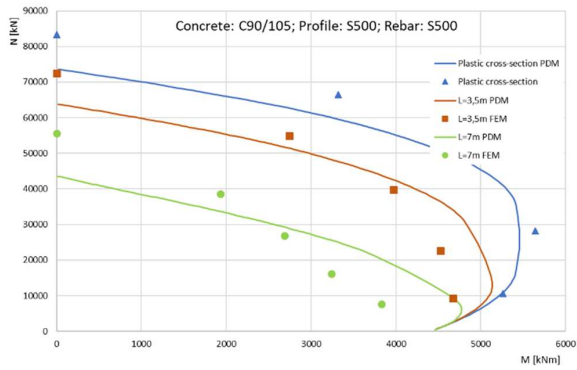
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



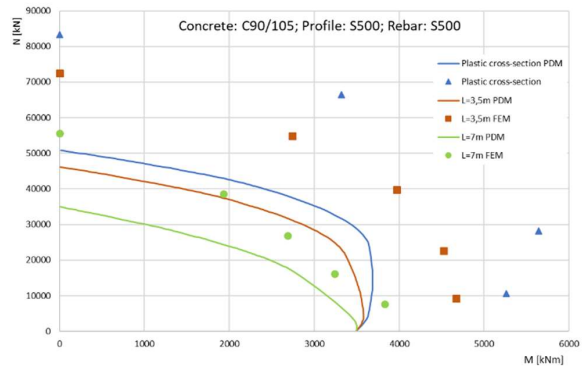
FEC 12 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB1000	500	1200x600	90	10 ϕ 28	500	60	0.90%

		A	B	C	D
δ		0.26	0.38	0.26	0.26
λ	3.5 [m]	0.50	0.42	0.67	1.14
	7 [m]	1.00	0.83	1.34	1.80

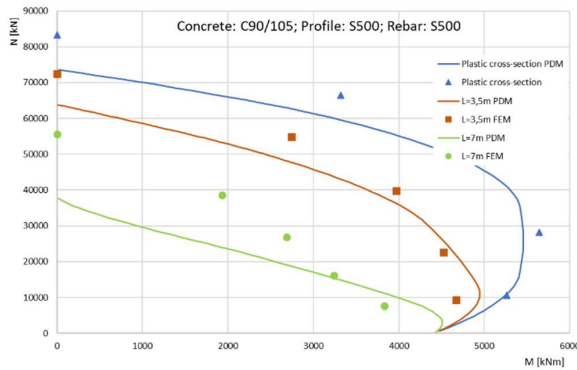
a) No material reduction



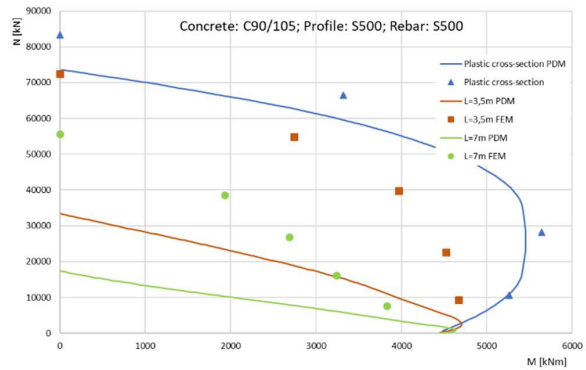
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



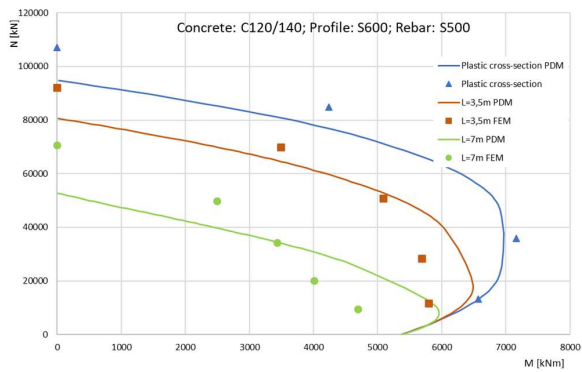
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



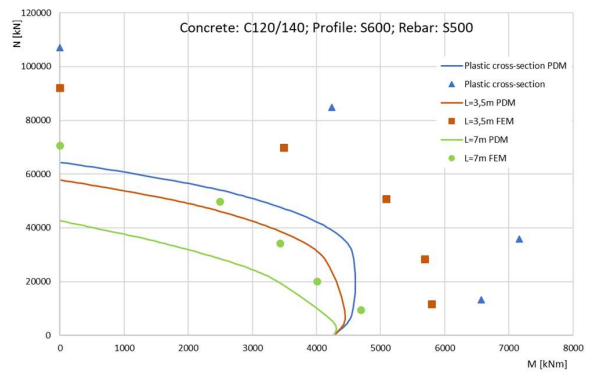
FEC 12 - Weak axis							
Profile		Concrete		Reinforcement			
Type	f_{yk} [MPa]	Section [mm ²]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_v [mm]	ρ [%]
HEB1000	600	1200x600	120	10 ϕ 28	500	60	0.90%

		A	B	C	D
δ		0.25	0.36	0.25	0.25
λ	3.5 [m]	0.53	0.44	0.72	1.14
	7 [m]	1.06	0.88	1.45	1.78

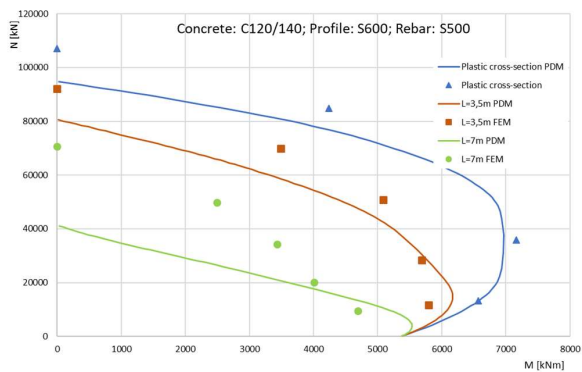
a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



B.1.1 Box plot for high-strength materials : test 3

Resistance of cross-section

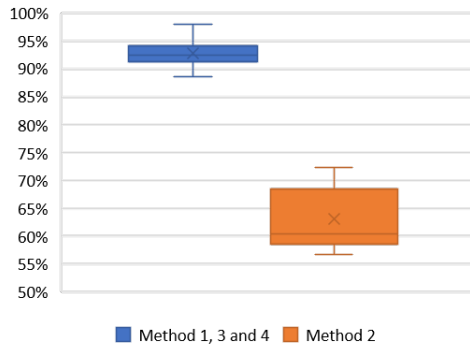


Figure B.1: Point A: Distribution of the ratios for the plastic cross-section resistance for test 3.

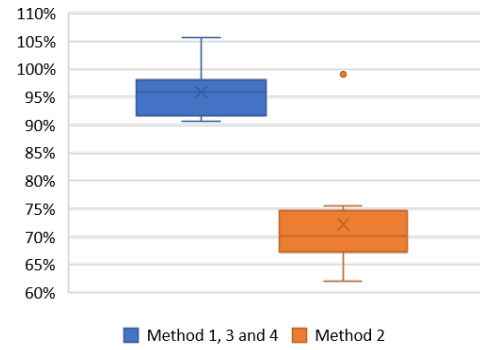


Figure B.2: Point B: Distribution of the ratios for the plastic cross-section resistance for test 3.

Columns of 3,5 m long

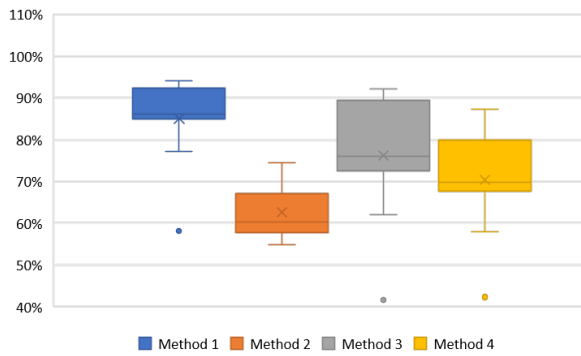


Figure B.3: Point A: Distribution of the ratios for the column of 3,5 m long for test 3.

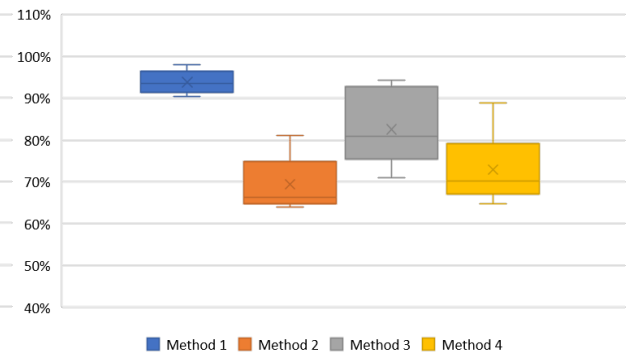


Figure B.4: Point B: Distribution of the ratios for the column of 3,5 m long for test 3.

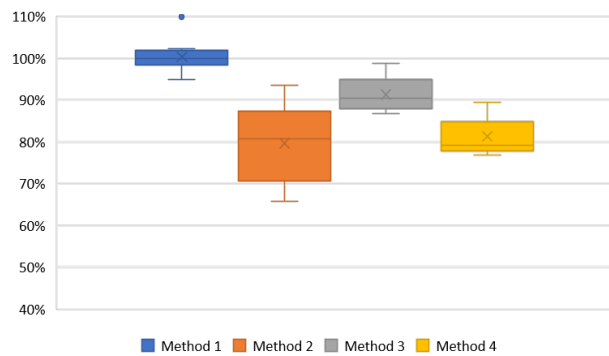


Figure B.5: Point C: Distribution of the ratios for the column of 3,5 m long for test 3.

Column of 7 m long

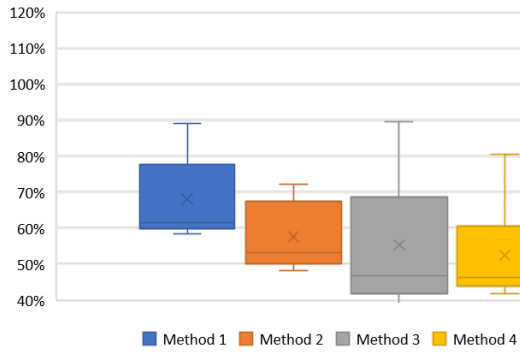


Figure B.6: Point A: Distribution of the ratios for the column of 7 m long for test 3.

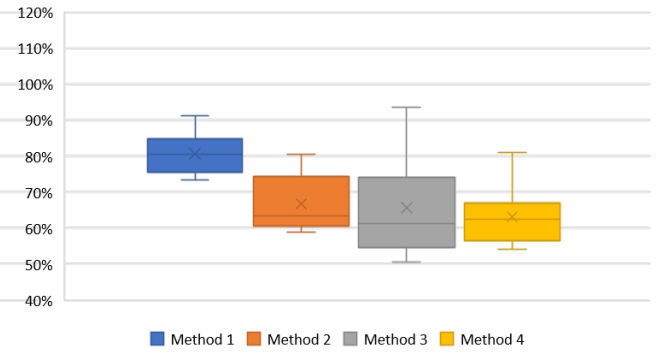


Figure B.7: Point B: Distribution of the ratios for the column of 7 m long for test 3.

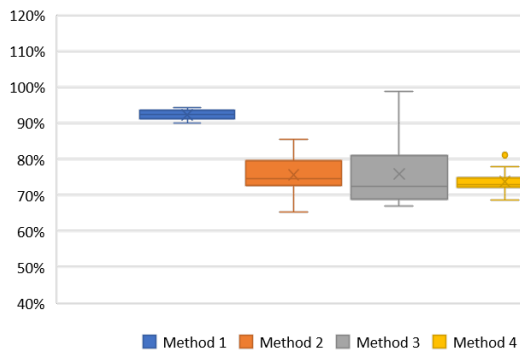


Figure B.8: Point C: Distribution of the ratios for the column of 7 m long for test 3.

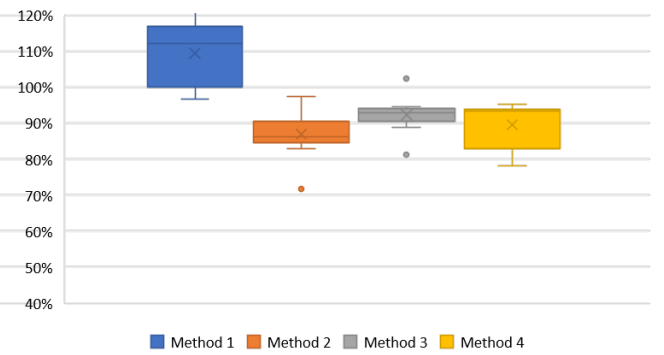


Figure B.9: Point D: Distribution of the ratios for the column of 7 m long for test 3.

B.1.2 Influence of the material compatibility for methods 3 and 4

		Test 1	Test 4	Test 5	Test 2	Test 6	Test 3
Plastic cross-section	δ	0.34	0.42	0.47	0.21	0.24	0.19
	A	93%	93%	93%	91%	91%	91%
	B	97%	97%	98%	94%	93%	94%
	C	99%	99%	99%	98%	98%	98%
	D		101%	102%			
L=3,5 m	λ	0.56	0.59	0.62	0.75	0.77	0.83
	A	91%	91%	91%	78%	78%	74%
	B	96%	95%	95%	82%	81%	76%
	C	99%	99%	99%	90%	90%	87%
	D	101%	102%	103%	100%	100%	101%
L=7 m	λ	1.11	1.19	1.24	1.51	1.54	1.65
	A	72%	70%	70%	46%	46%	40%
	B	81%	79%	79%	58%	58%	52%
	C	87%	85%	85%	72%	72%	68%
	D	95%	93%	93%	95%	93%	93%

Figure B.10: Ranking of the tests according to the increase of the relative slenderness for specimen FEC8 calculated with the AS/NZ Standard (method 3) and for different column lengths.

		Test 1	Test 2	Test 3	Test 6	Test 4	Test 5
Plastic cross-section	δ	0.34	0.21	0.19	0.24	0.42	0.47
	A	93%	91%	91%	91%	93%	93%
	B	97%	94%	94%	93%	97%	98%
	C	99%	98%	98%	98%	99%	99%
	D					101%	102%
L=3,5 m	λ	1.09	1.13	1.15	1.16	1.17	1.22
	A	63%	67%	68%	67%	60%	58%
	B	64%	65%	66%	64%	60%	58%
	C	69%	76%	77%	74%	65%	63%
	D					79%	76%
L=7 m	λ	1.69	1.80	1.82	1.85	1.82	1.90
	A	42%	42%	43%	42%	40%	40%
	B	49%	53%	55%	53%	48%	47%
	C	56%	69%	72%	69%	53%	53%
	D	67%	88%	95%	85%	63%	61%

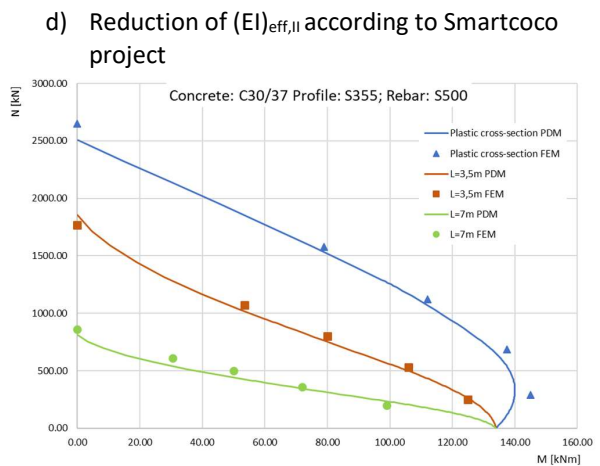
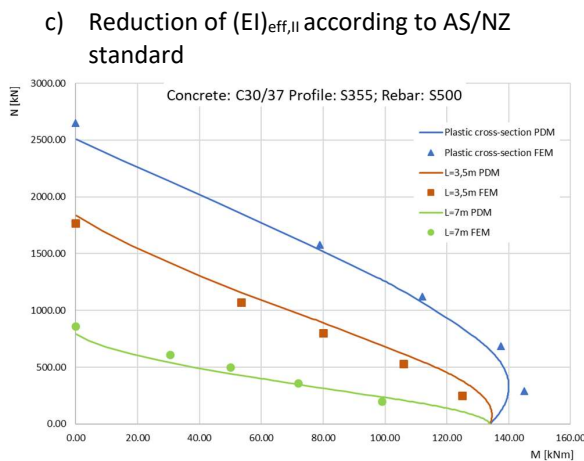
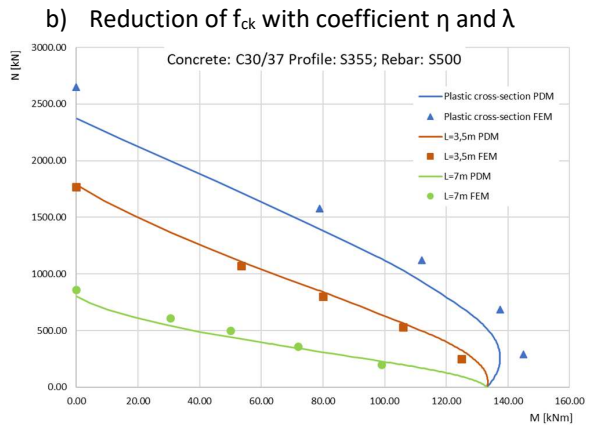
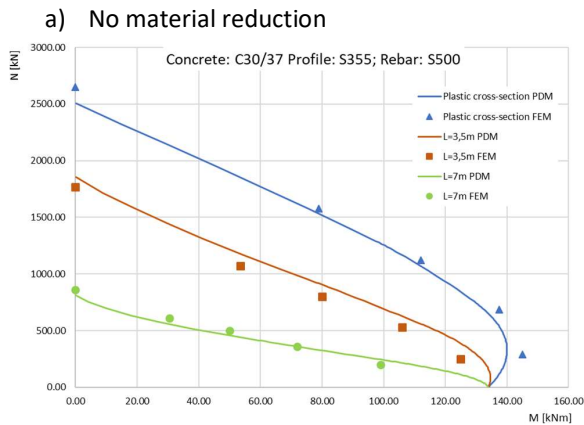
Figure B.11: Ranking of the tests according to the increase of the relative slenderness for specimen FEC8 calculated with the SMARTCoCo project method (method 4) and for different column lengths.

B.2 Partially encased concrete cross-section

This section contains all the results obtained for partially encased section along strong and weak axis. The Tables summarize the ratios R for the different analytical methods (A = method 1, B = method 2, C = method 3 and D = method 4) and the different parameters $\bar{\lambda}$ and δ . Each figure presents the plastic and reduced interaction curves for the 3 considered length and for one set of material grades.

PEC 1 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA180	355	30	4 ϕ 14	500	37	27.5	2.36%

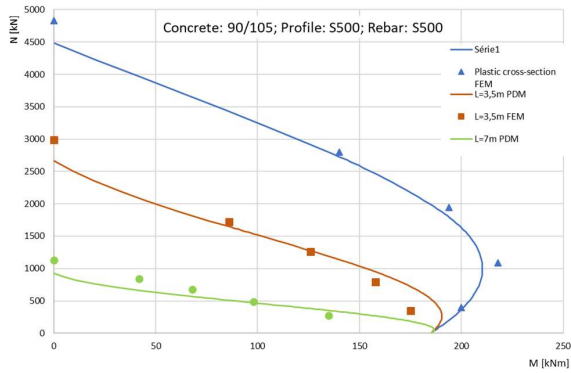
		SAFIR values		Ratio PDM/FEM			
		N [kN]	M [kNm]	A	B	C	D
Plastic cross-section		2650	0	95%	90%	95%	95%
		1580	79	98%	93%	98%	98%
		1120	112	98%	94%	98%	98%
		687	137.4	98%		98%	98%
L=3,5 m		1770	0	105%	101%	104%	105%
		1070	53.5	106%	102%	105%	97%
		800	80	107%	103%	106%	97%
		530	106	106%	102%	105%	98%
L=7 m		860	0	95%	94%	92%	95%
		610	30.5	93%	91%	91%	91%
		500	50	94%	92%	92%	92%
		360	72	100%	98%	98%	98%
		δ		0.61	0.64	0.61	0.61
	λ	3.5 [m]		0.77	0.75	0.79	1.02
		7 [m]		1.55	1.51	1.57	1.62



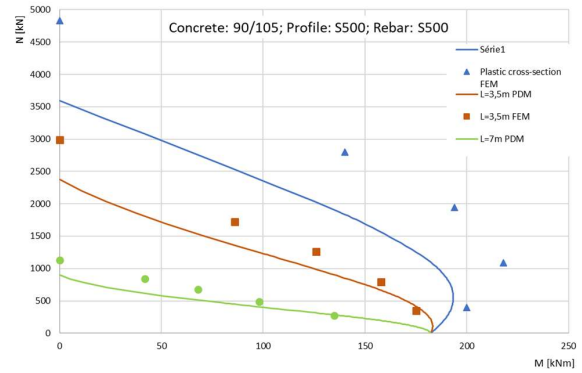
PEC 1 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA180	500	90	4 ϕ 14	500	37	27.5	2.36%

		SAFIR values		Ratio PDM/FEM			
		N [kN]	M [kNm]	A	B	C	D
Plastic cross-section		4830	0	93%	74%	93%	93%
		2800	140	99%	80%	99%	99%
		1940	194	98%		98%	98%
		1090	218				
L=3,5 m		2990	0	89%	79%	86%	89%
		1720	86	97%	86%	94%	91%
		1260	126	101%	89%	98%	94%
		790	158	106%	95%	104%	99%
L=7 m		1130	0	82%	80%	77%	82%
		840	42	83%	78%	79%	81%
		680	68	88%	82%	84%	86%
		490	98	98%	90%	94%	96%
		δ		0.48	0.60	0.48	0.48
λ		3.5 [m]		1.00	0.90	1.04	1.19
		7 [m]		2.01	1.80	2.07	2.07

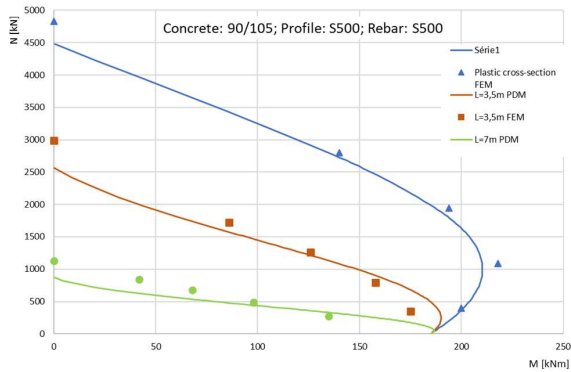
a) No material reduction



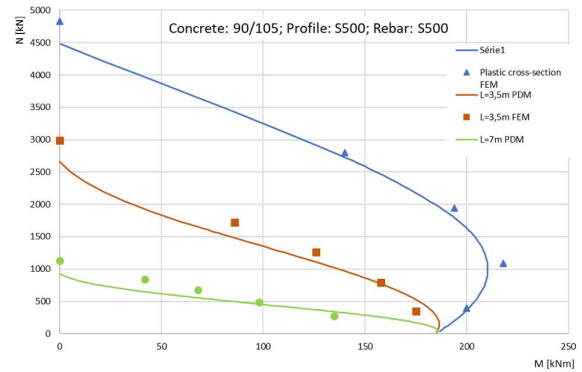
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



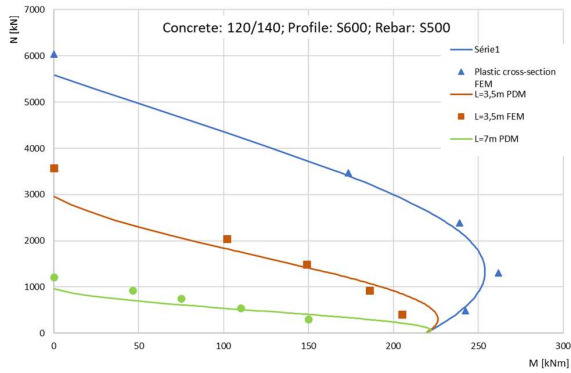
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



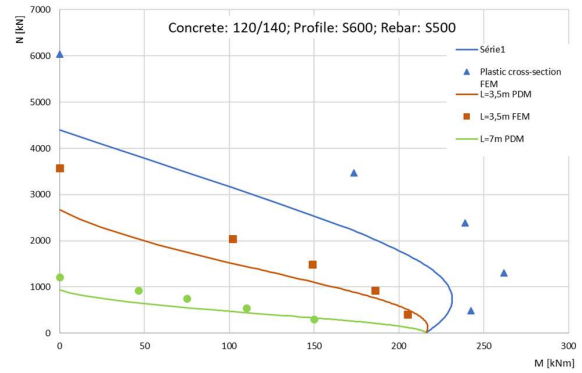
PEC 1 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA180	600	120	4 ϕ 14	500	37	27.5	2.36%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	6040	0	92%	73%	92%	92%
	3470	173.5	99%	79%	99%	99%
	2390	239	98%		98%	98%
	1310	262				
L=3,5 m	3570	0	83%	75%	79%	83%
	2040	102	93%	82%	89%	87%
	1490	149	98%	86%	94%	92%
	930	186	105%	94%	103%	99%
L=7 m	1210	0	80%	78%	74%	80%
	930	46.5	80%	75%	75%	78%
	750	75	86%	80%	81%	85%
	550	110	96%	88%	91%	95%
δ			0.46	0.59	0.46	0.46
λ	3.5 [m]		1.11	0.98	1.15	1.29
	7 [m]		2.22	1.97	2.31	2.26

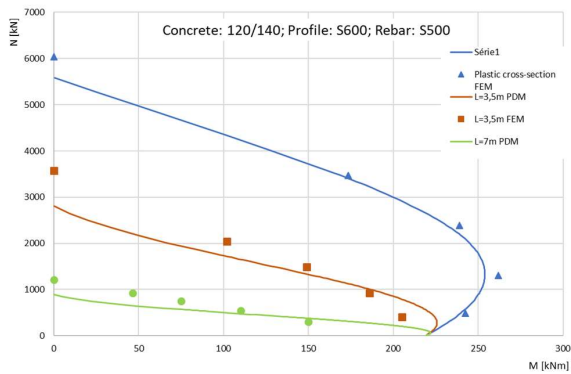
a) No material reduction



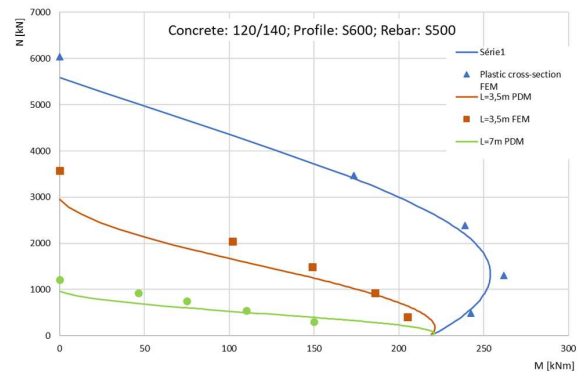
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



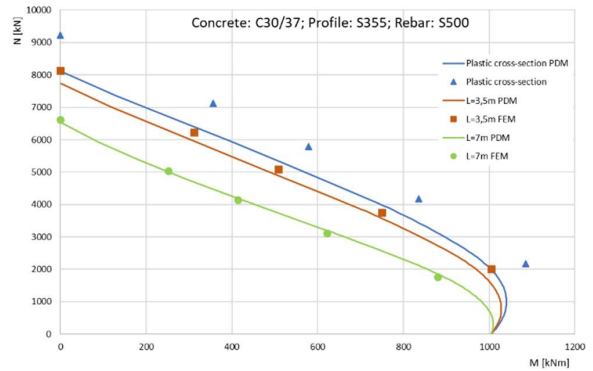
PEC 2 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA400	355	30	4 ϕ 20	500	46	55	1.24%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	9220	0	94%	88%	94%	94%
	7130	356.5	95%	89%	95%	95%
	5790	579	96%	91%	96%	96%
	4180	836	98%	92%	98%	98%
L=3,5 m	8130	0	101%	95%	100%	101%
	6230	311.5	102%	97%	102%	98%
	5090	509	103%	97%	103%	97%
	3750	750	103%	98%	103%	97%
L=7 m	6620	0	103%	99%	102%	103%
	5040	252	104%	100%	103%	96%
	4140	414	106%	101%	105%	96%
	3110	622	106%	102%	105%	97%
δ			0.63	0.67	0.63	0.63
λ	3.5 [m]		0.34	0.33	0.35	0.60
	7 [m]		0.68	0.66	0.69	0.94

a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



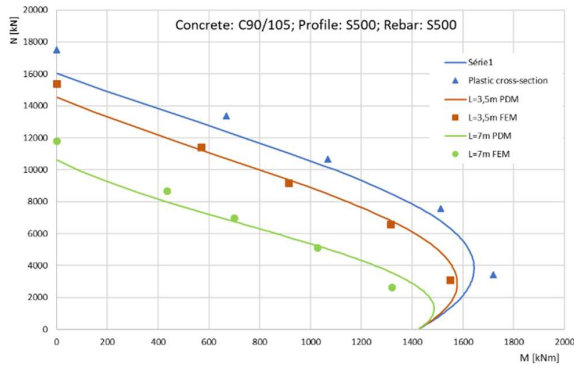
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



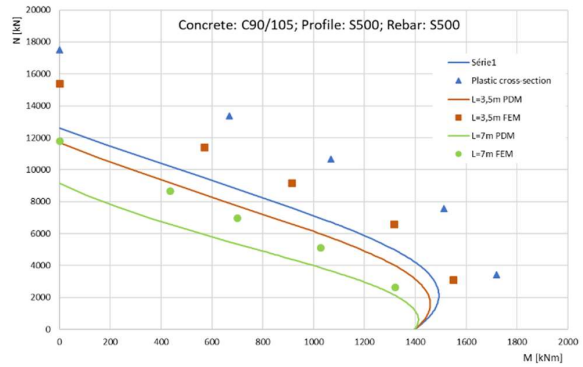
PEC 2 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA400	500	90	4 ϕ 20	500	46	55	1.24%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	17500	0	92%	72%	92%	92%
	13370	668.5	94%	74%	94%	94%
	10680	1068	97%	77%	97%	97%
	7560	1512	97%		97%	97%
L=3,5 m	15400	0	94%	76%	94%	94%
	11400	570	99%	80%	98%	95%
	9150	915	101%	82%	101%	96%
	6580	1316	101%	86%	101%	96%
L=7 m	11810	0	90%	78%	87%	90%
	8690	434.5	94%	81%	90%	88%
	6990	699	98%	84%	95%	91%
	5140	1028	101%	87%	98%	93%
	δ		0.48	0.60	0.48	0.48
λ	3.5 [m]		0.45	0.40	0.47	0.65
	7 [m]		0.89	0.79	0.93	1.10

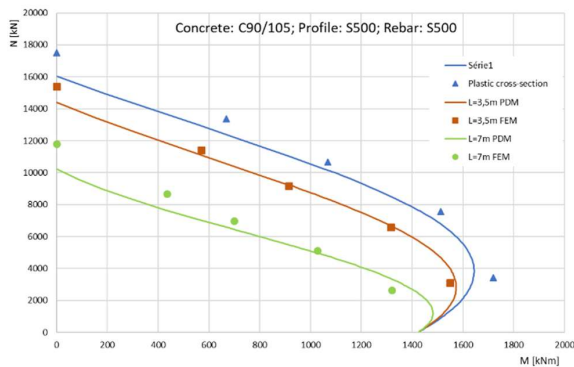
a) No material reduction



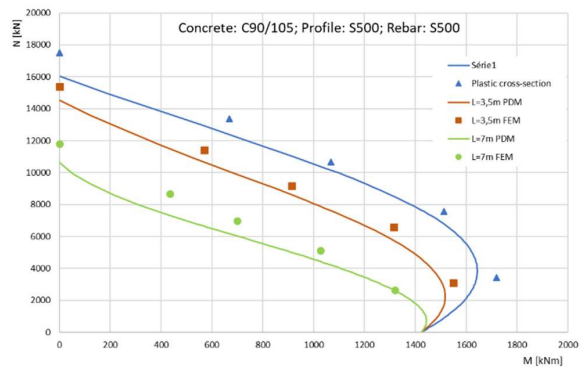
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



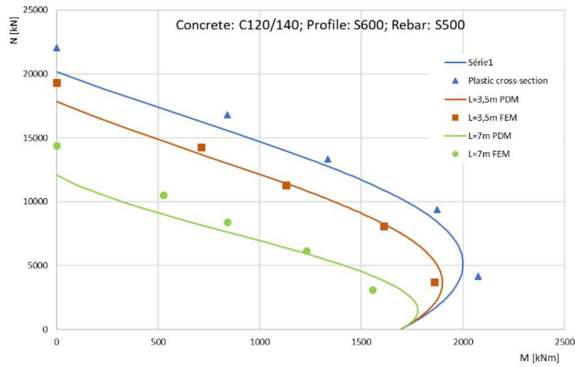
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



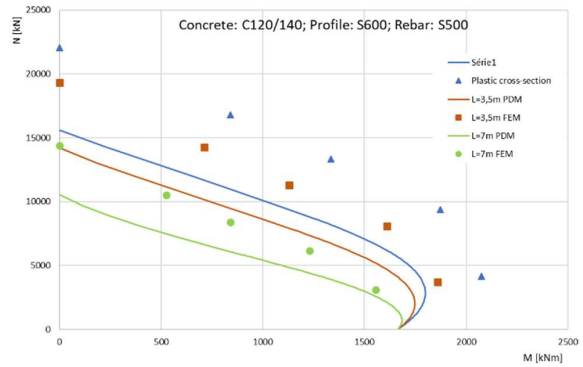
PEC 2 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA400	600	120	4 ϕ 20	500	46	55	1.24%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	22060	0	91%	71%	91%	91%
	16790	839.5	94%	73%	94%	94%
	13350	1335	97%	76%	97%	97%
	9370	1874	100%		100%	100%
L=3,5 m	19330	0	92%	73%	91%	92%
	14270	713.5	97%	77%	96%	93%
	11300	1130	101%	80%	99%	95%
	8060	1612	101%	86%	100%	95%
L=7 m	14390	0	84%	73%	80%	84%
	10530	526.5	88%	77%	84%	83%
	8400	840	94%	81%	90%	87%
	6150	1230	98%	85%	95%	91%
δ			0.45	0.59	0.45	0.45
λ	3.5 [m]		0.50	0.44	0.52	0.70
	7 [m]		0.99	0.87	1.04	1.19

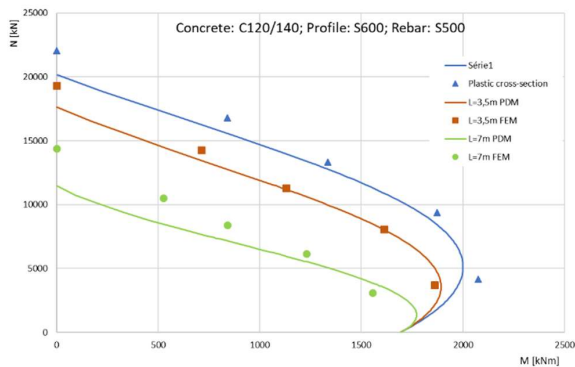
a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



PEC 3 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA500	355	30	4 ϕ 20	500	46	50	0.98%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	11380	0	93%	87%	93%	93%
	9240	462	95%	89%	95%	95%
	7750	775	96%	90%	96%	96%
	5840	1168	98%	92%	98%	98%
L=3,5 m	10350	0	100%	94%	100%	100%
	8360	418	102%	96%	101%	99%
	7030	703	103%	97%	102%	98%
	5360	1072	103%	97%	103%	98%
L=7 m	8990	0	102%	96%	101%	102%
	7170	358.5	104%	99%	103%	98%
	6030	603	105%	100%	105%	97%
	4630	926	106%	101%	106%	98%
	δ		0.64	0.68	0.64	0.64
	λ	3.5 [m]	0.27	0.26	0.28	0.52
		7 [m]	0.54	0.53	0.55	0.81

a) No material reduction



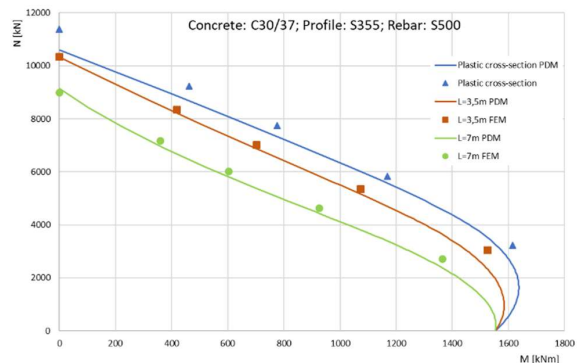
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



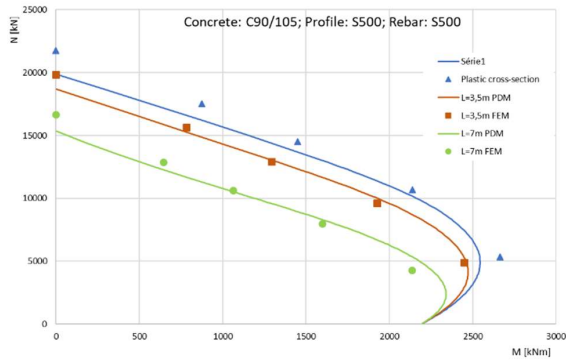
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



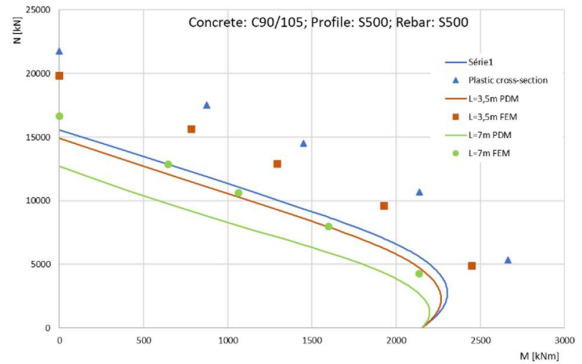
PEC 3 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA500	500	90	4 ϕ 20	500	46	50	0.98%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	21780	0	91%	71%	91%	91%
	17510	875.5	94%	74%	94%	94%
	14500	1450	96%	76%	96%	96%
	10690	2138	98%	83%	98%	98%
L=3,5 m	19830	0	94%	75%	94%	94%
	15640	782	98%	78%	97%	96%
	12940	1294	101%	80%	100%	97%
	9630	1926	102%	84%	101%	97%
L=7 m	16660	0	92%	76%	90%	92%
	12890	644.5	96%	80%	94%	92%
	10620	1062	99%	83%	97%	94%
	7990	1598	102%	86%	100%	95%
	δ		0.48	0.61	0.48	0.48
λ	3.5 [m]		0.36	0.32	0.37	0.55
	7 [m]		0.71	0.63	0.75	0.93

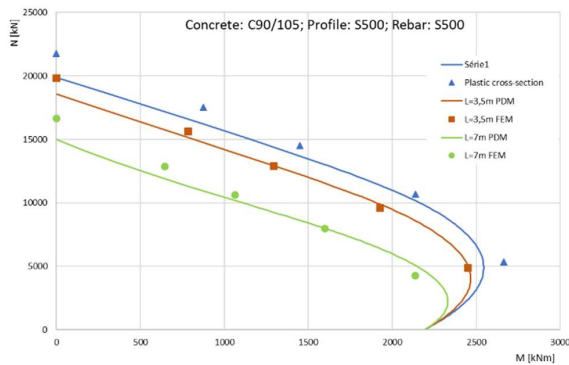
a) No material reduction



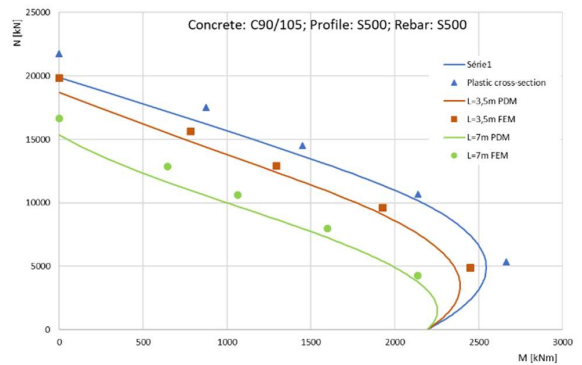
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



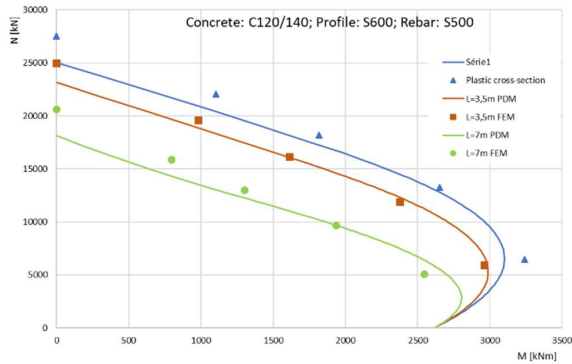
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



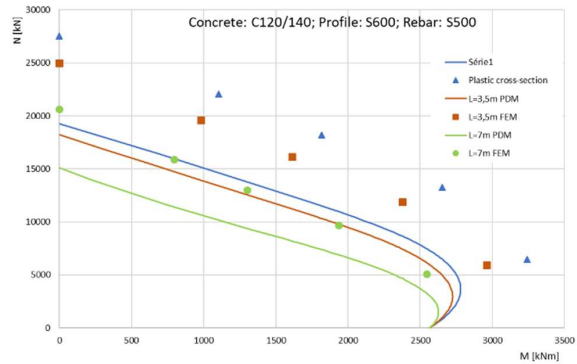
PEC 3 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA500	600	120	4 ϕ 20	500	46	50	0.98%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	27510	0	91%	70%	91%	91%
	22060	1103	94%	72%	94%	94%
	18180	1818	96%	75%	96%	96%
	13270	2654	99%	84%	99%	99%
L=3,5 m	24990	0	93%	73%	92%	93%
	19610	980.5	97%	76%	96%	95%
	16120	1612	100%	79%	99%	97%
	11880	2376	102%	84%	101%	97%
L=7 m	20620	0	88%	73%	85%	88%
	15890	794.5	92%	77%	89%	88%
	12990	1299	96%	80%	93%	92%
	9680	1936	100%	84%	97%	94%
δ			0.46	0.59	0.46	0.46
λ	3.5 [m]		0.40	0.35	0.42	0.59
	7 [m]		0.79	0.70	0.83	1.00

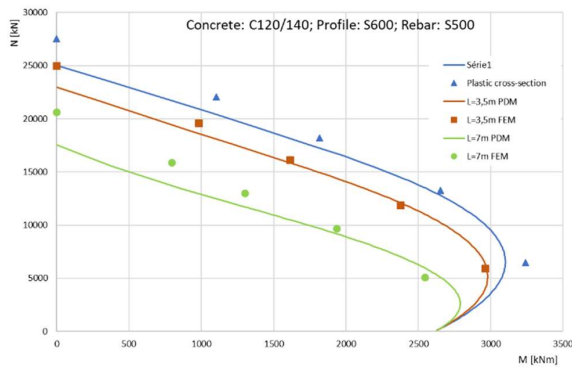
a) No material reduction



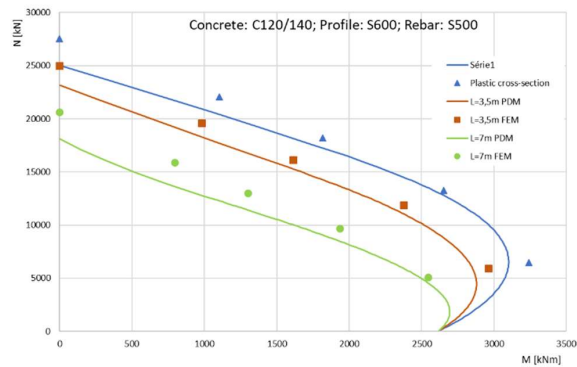
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard

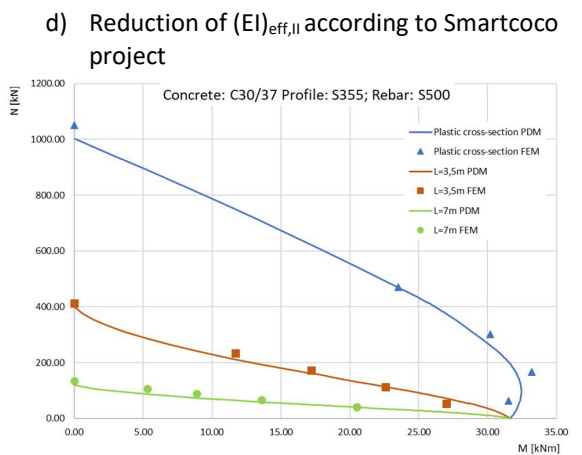
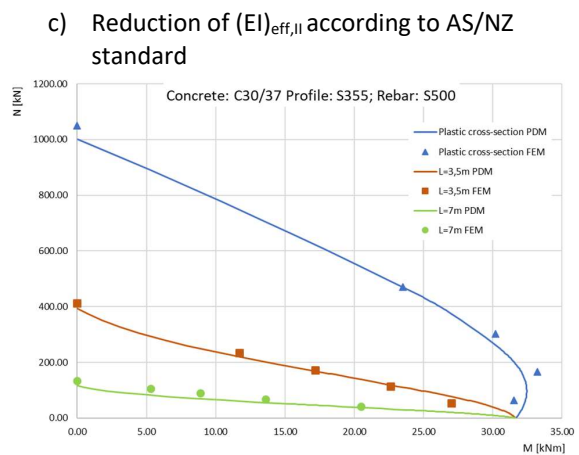
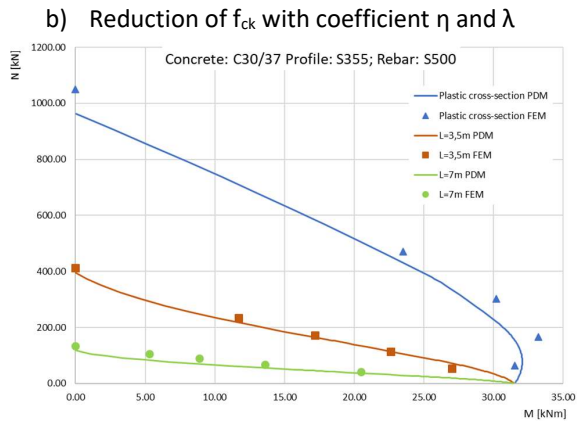
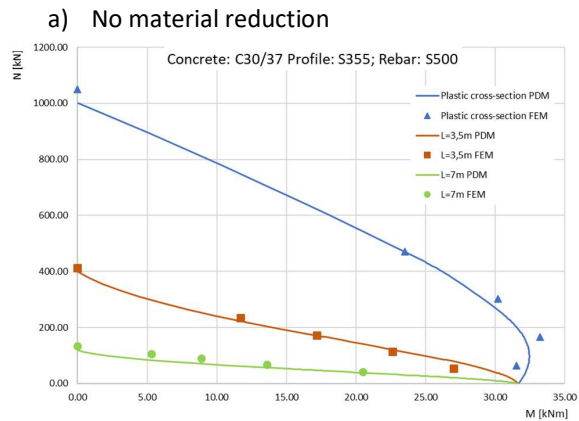


d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



PEC 4 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA100	355	30	4 ϕ 8	500	30	20	2.65%

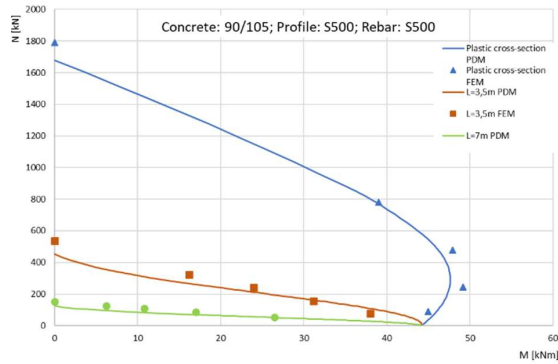
	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	1050	0	95%	92%	95%	95%
	470	23.5	100%	96%	100%	100%
	302	30.2	97%	94%	97%	97%
	166	33.2				
L=3,5 m	412	0	97%	96%	96%	97%
	234	11.7	97%	95%	96%	94%
	172	17.2	100%	98%	99%	97%
	113	22.6	102%	100%	102%	100%
L=7 m	134	0	89%	89%	88%	89%
	106	5.3	83%	82%	81%	85%
	89	8.9	84%	83%	83%	87%
	68	13.6	89%	88%	88%	92%
δ			0.71	0.74	0.71	0.71
λ	3.5 [m]		1.36	1.33	1.37	1.45
	7 [m]		2.71	2.66	2.74	2.60



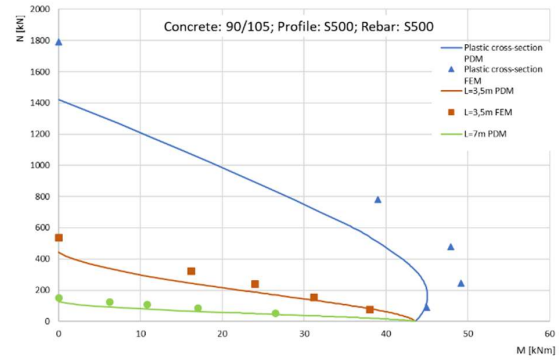
PEC 4 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA100	500	90	4 ϕ 8	500	30	20	2.65%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	1790	0	94%	79%	94%	94%
	781	39.05	99%	87%	99%	99%
	479	47.9				
	246	49.2				
L=3,5 m	538	0	84%	82%	82%	84%
	324	16.2	87%	83%	85%	86%
	240	24	93%	87%	91%	92%
	156	31.2	102%	95%	100%	100%
L=7 m	152	0	84%	83%	81%	84%
	125	6.25	80%	78%	77%	82%
	108	10.8	82%	79%	79%	84%
	85	17	87%	83%	85%	89%
δ			0.59	0.70	0.59	0.59
λ	3.5 [m]		1.72	1.59	1.76	1.79
	7 [m]		3.44	3.17	3.51	3.30

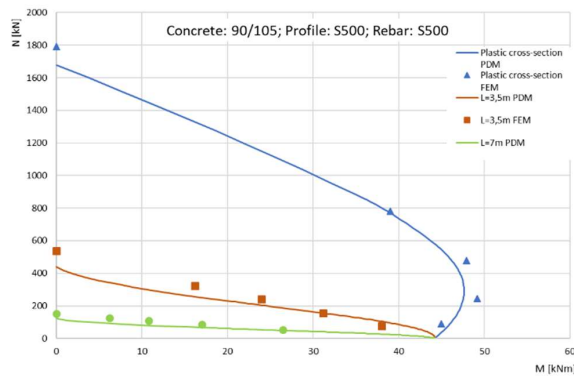
a) No material reduction



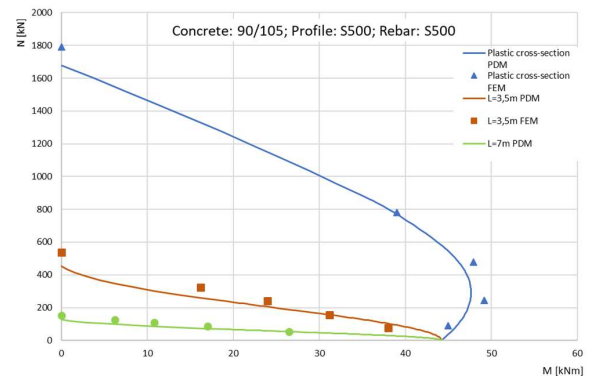
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



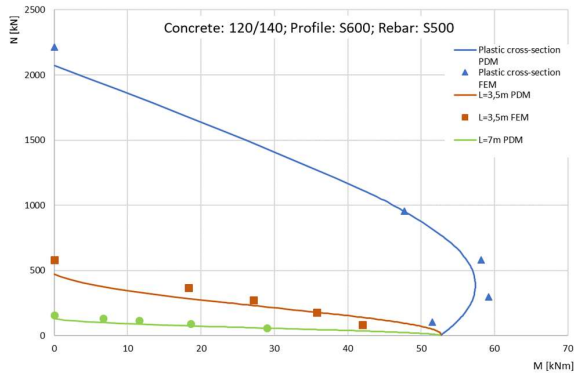
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



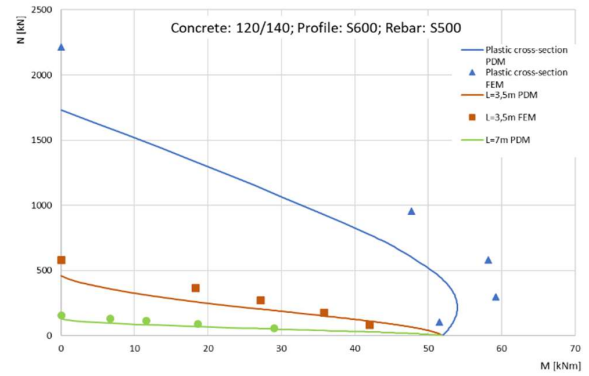
PEC 4 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA100	600	120	4 ϕ 8	500	30	20	2.65%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	2215	0	93%	78%	93%	93%
	955	47.75	100%	87%	100%	100%
	582	58.2				
	296	59.2				
L=3,5 m	582	0	81%	79%	78%	81%
	366	18.3	83%	79%	81%	83%
	272	27.2	91%	84%	88%	90%
	179	35.8	100%	93%	98%	100%
L=7 m	158	0	83%	82%	79%	83%
	133	6.65	78%	76%	75%	81%
	116	11.6	80%	77%	77%	83%
	93	18.6	85%	81%	83%	89%
δ			0.58	0.69	0.58	0.58
λ	3.5 [m]		1.90	1.74	1.95	1.97
	7 [m]		3.80	3.47	3.89	3.62

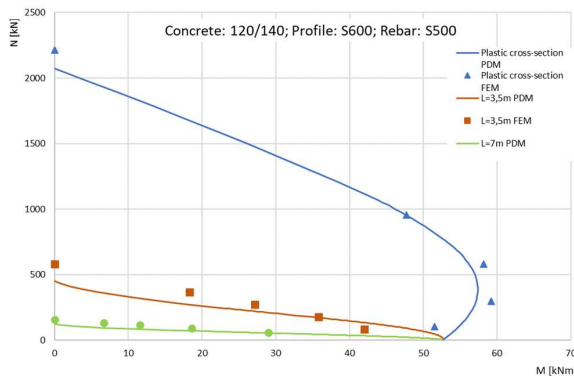
a) No material reduction



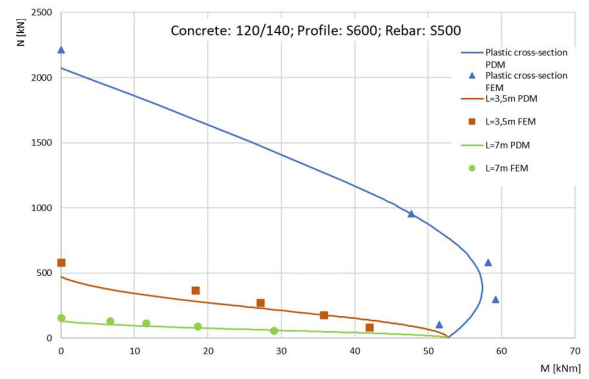
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



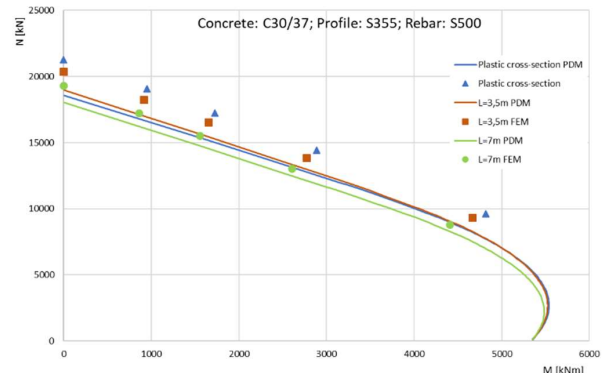
PEC 5 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA1000	355	30	4 ϕ 28	500	60	65	0.94%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	21260	0	94%	87%	94%	94%
	19070	953.5	95%	88%	95%	95%
	17250	1725	96%	89%	96%	96%
	14440	2888	97%	91%	97%	97%
L=3,5 m	20360	0	100%	93%	100%	100%
	18250	912.5	101%	94%	100%	100%
	16530	1653	101%	95%	101%	100%
	13860	2772	102%	96%	102%	101%
L=7 m	19310	0	100%	93%	100%	100%
	17220	861	101%	95%	101%	99%
	15540	1554	102%	96%	102%	99%
	13030	2606	104%	97%	103%	99%
δ			0.60	0.65	0.60	0.60
λ	3.5 [m]		0.14	0.14	0.15	0.32
	7 [m]		0.29	0.28	0.29	0.55

a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



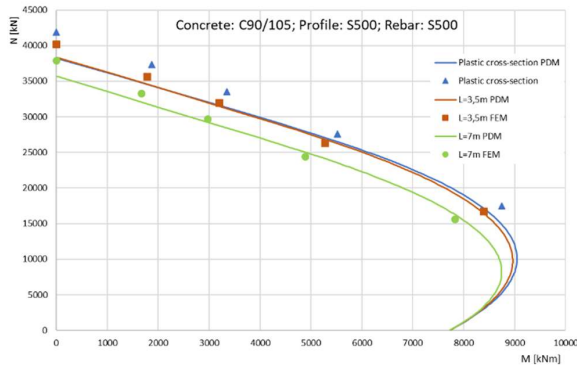
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



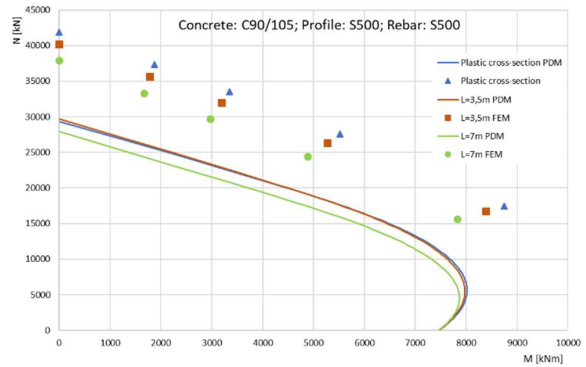
PEC 5 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA1000	500	90	4 ϕ 28	500	60	65	0.94%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	41880	0	91%	70%	91%	91%
	37370	1868.5	93%	71%	93%	93%
	33530	3353	94%	73%	94%	94%
	27610	5522	97%	76%	97%	97%
L=3,5 m	40180	0	95%	74%	95%	95%
	35650	1782.5	97%	75%	97%	97%
	31960	3196	99%	77%	99%	98%
	26350	5270	101%	79%	101%	100%
L=7 m	37930	0	94%	74%	93%	94%
	33300	1665	97%	76%	96%	95%
	29740	2974	99%	77%	98%	96%
	24440	4888	102%	80%	101%	98%
	δ		0.44	0.58	0.44	0.44
λ	3.5 [m]		0.19	0.17	0.20	0.34
	7 [m]		0.38	0.33	0.40	0.59

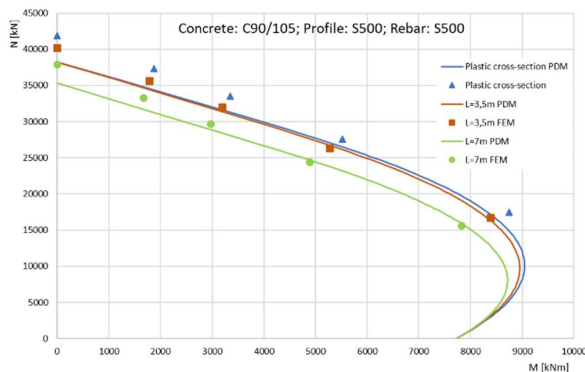
a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



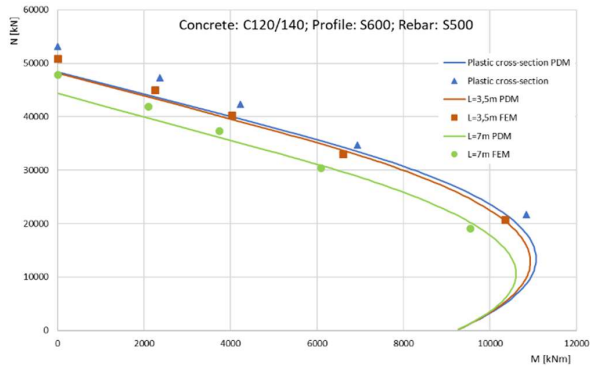
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



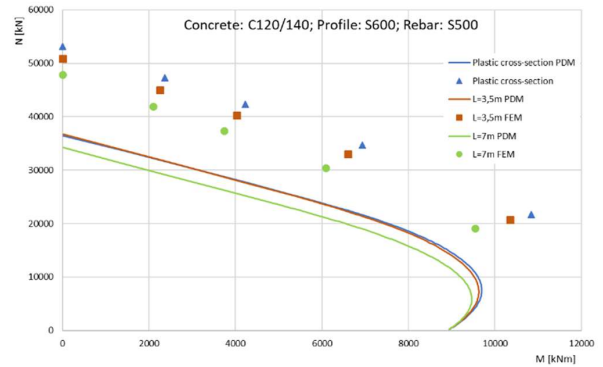
PEC 5 - Strong axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA1000	600	120	4 ϕ 28	500	60	65	0.94%

	SAFIR values		Ratio PDM/FEM			
	N [kN]	M [kNm]	A	B	C	D
Plastic cross-section	53120	0	91%	69%	91%	91%
	47280	2364	93%	70%	93%	93%
	42320	4232	95%	72%	95%	95%
	34680	6936	98%	75%	98%	98%
L=3,5 m	50880	0	95%	72%	94%	95%
	45040	2252	97%	74%	96%	96%
	40280	4028	98%	75%	98%	97%
	33020	6604	101%	78%	101%	100%
L=7 m	47860	0	93%	72%	91%	93%
	41910	2095.5	95%	74%	94%	94%
	37350	3735	97%	76%	96%	95%
	30460	6092	101%	79%	100%	97%
	δ		0.42	0.56	0.42	0.42
	λ	3.5 [m]	0.21	0.18	0.22	0.36
		7 [m]	0.42	0.36	0.44	0.63

a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project

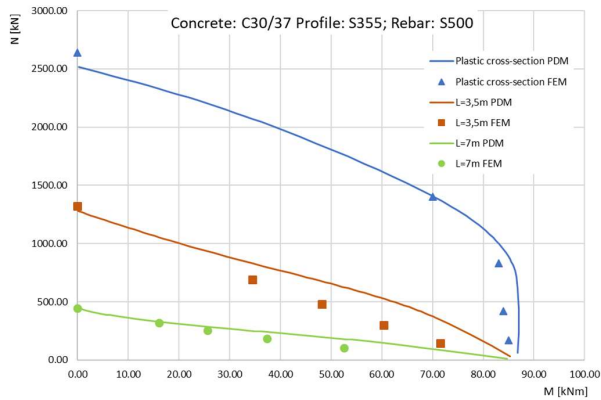


APPENDIX B. SENSITIVITY STUDY RESULTS

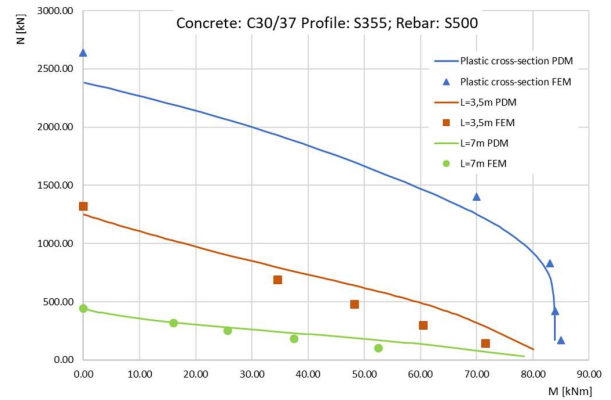
PEC 1 - Weak axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA180	355	30	4 ϕ 14	500	37	27.5	2.33%

		A	B	C	D
δ		0.61	0.64	0.61	0.61
λ	3.5 [m]	1.12	1.09	1.18	1.60
	7 [m]	2.25	2.19	2.37	2.52

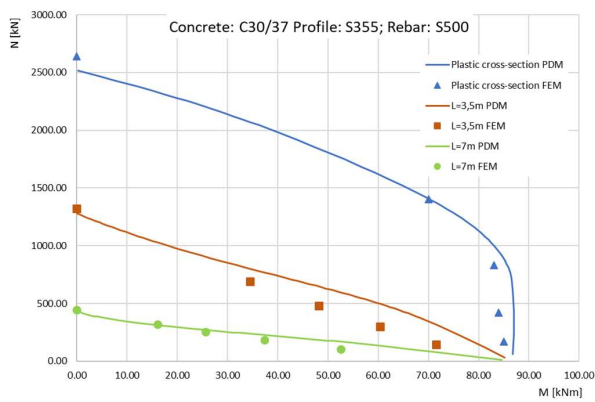
a) No material reduction



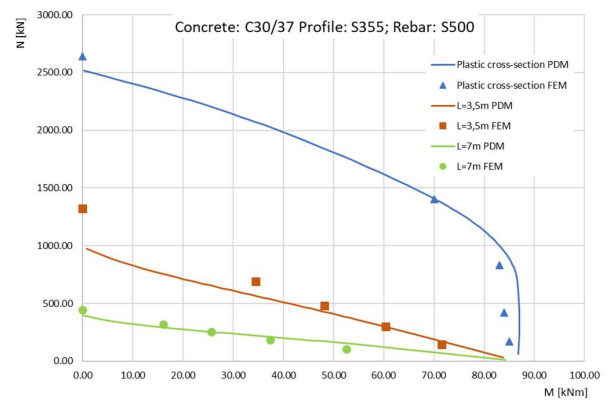
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard

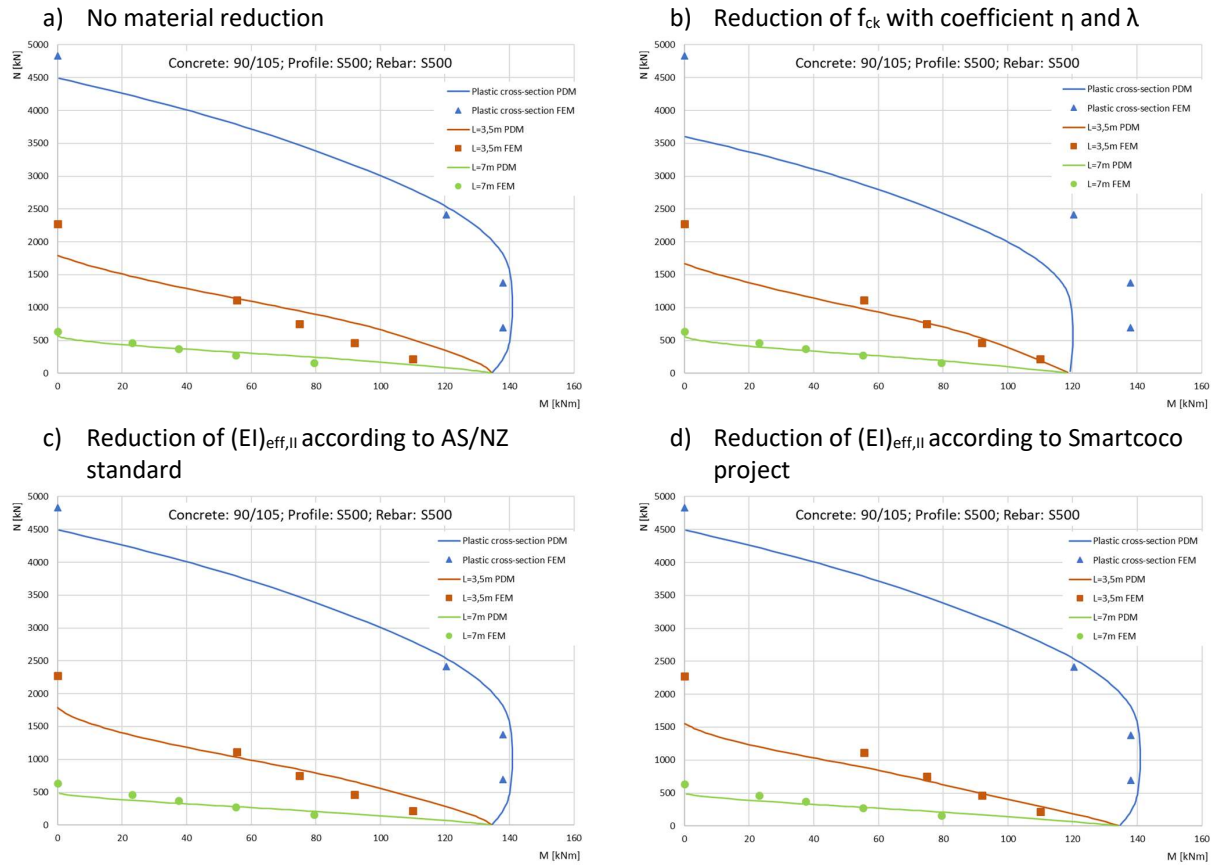


d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



PEC 1 - Weak axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA180	500	90	4 ϕ 14	500	37	27.5	2.33%

		A	B	C	D
δ		0.40	0.52	0.40	0.40
λ	3.5 [m]	1.27	1.12	1.41	1.64
	7 [m]	2.54	2.23	2.81	2.78

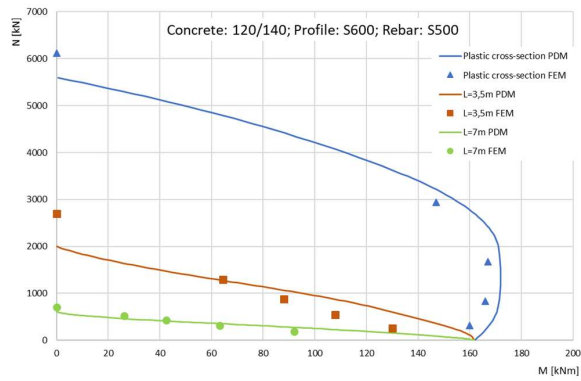


APPENDIX B. SENSITIVITY STUDY RESULTS

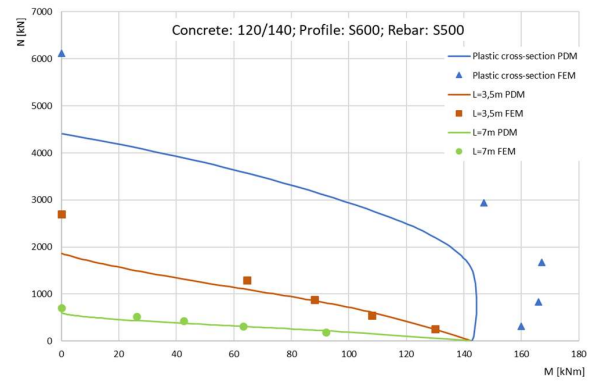
PEC 1 - Weak axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA180	600	120	4 ϕ 14	500	37	27.5	2.33%

		A	B	C	D
δ		0.46	0.59	0.46	0.46
λ	3.5 [m]	1.49	1.32	1.67	1.89
	7 [m]	2.97	2.64	3.33	3.21

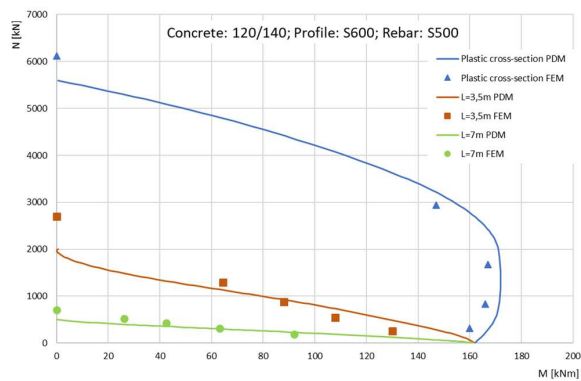
a) No material reduction



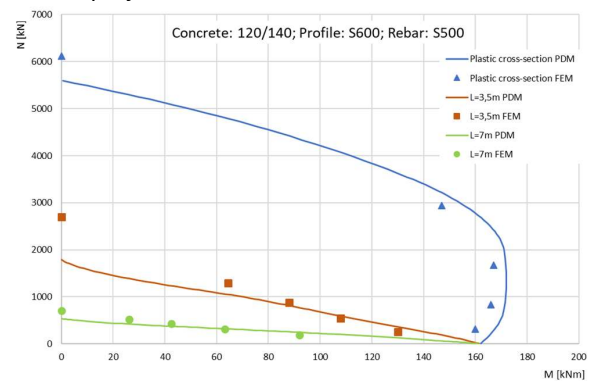
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



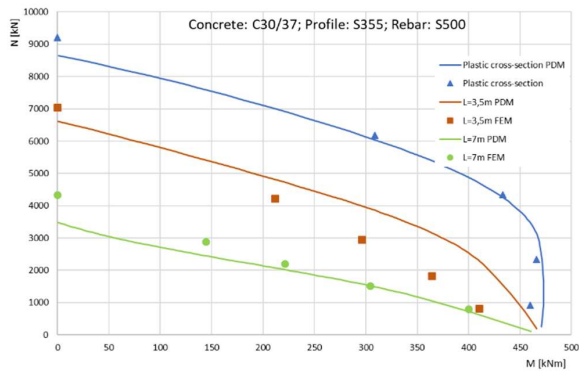
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



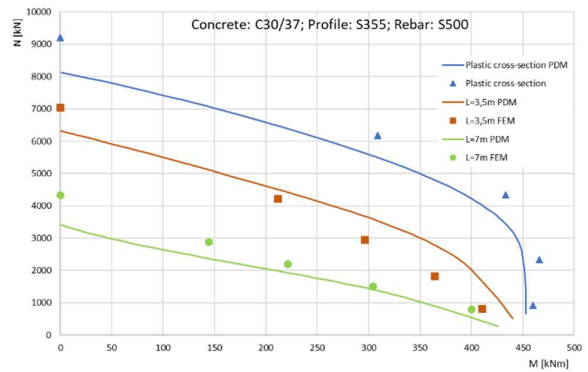
PEC 2 - Weak axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA400	355	30	4 ϕ 20	500	46	55	1.24%

		A	B	C	D
δ		0.63	0.67	0.63	0.63
λ	3.5 [m]	0.68	0.66	0.72	1.18
	7 [m]	1.36	1.31	1.44	1.80

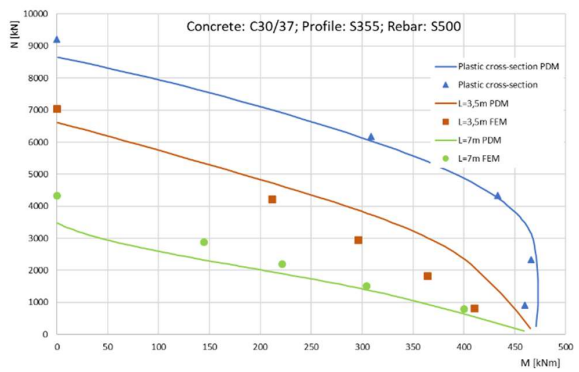
a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project

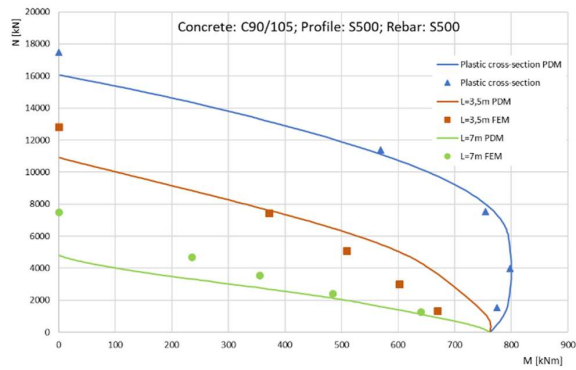


APPENDIX B. SENSITIVITY STUDY RESULTS

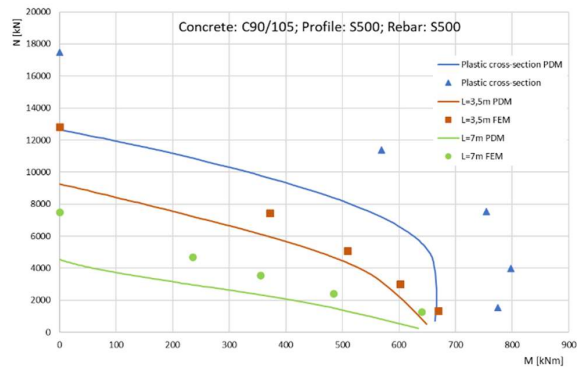
PEC 2 - Weak axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA400	500	90	4 ϕ 20	500	46	55	1.24%

		A	B	C	D
δ		0.48	0.60	0.48	0.48
λ	3.5 [m]	0.83	0.74	0.94	1.26
	7 [m]	1.67	1.48	1.87	2.09

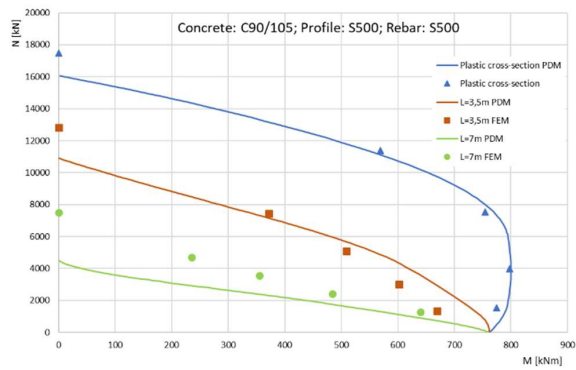
a) No material reduction



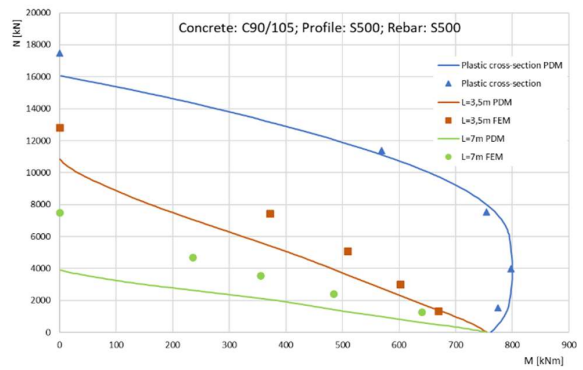
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



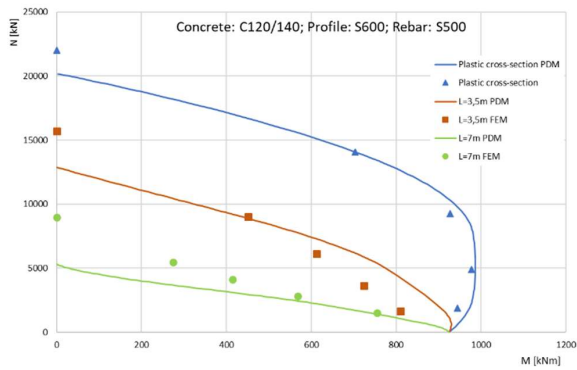
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



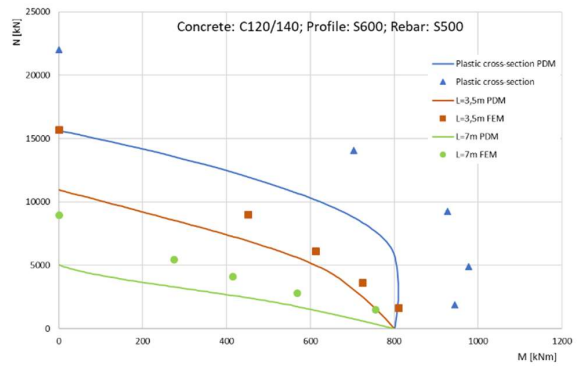
PEC 2 - Weak axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA400	600	120	4 ϕ 20	500	46	55	1.24%

		A	B	C	D
δ		0.45	0.59	0.45	0.45
λ	3.5 [m]	0.91	0.80	1.03	1.33
	7 [m]	1.81	1.59	2.07	2.21

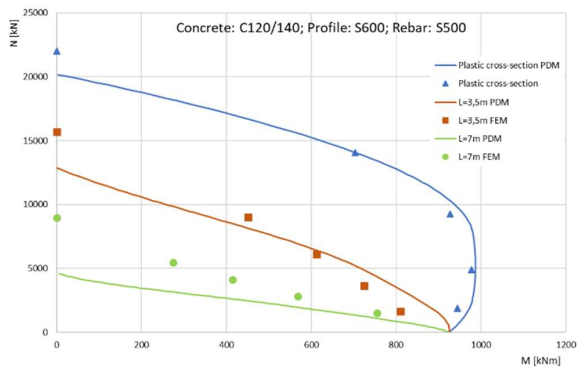
a) No material reduction



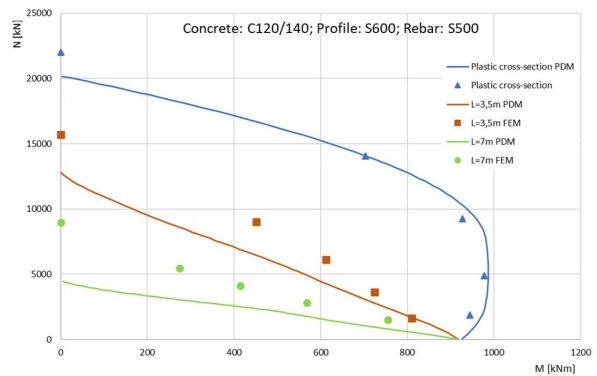
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



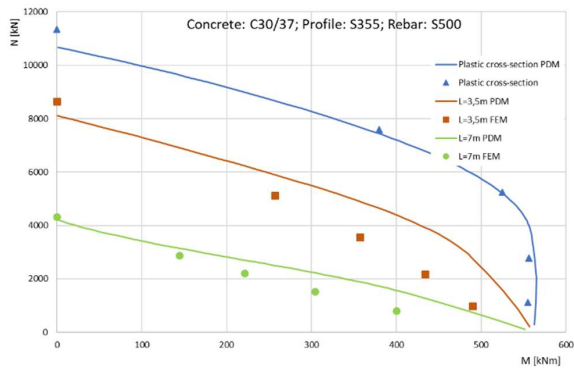
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



PEC 3 - Weak axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA500	355	30	4 ϕ 20	500	46	50	0.98%

		A	B	C	D
δ		0.64	0.68	0.64	0.64
λ	3.5 [m]	0.69	0.67	0.74	1.20
	7 [m]	1.38	1.33	1.47	1.84

a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



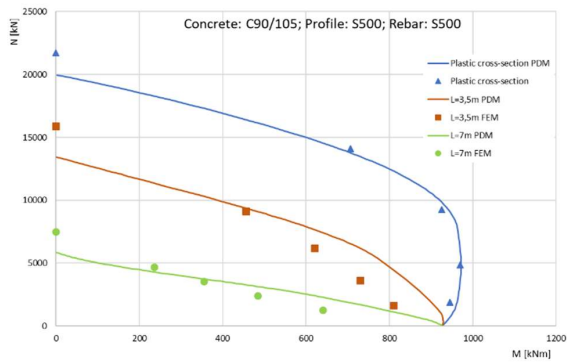
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



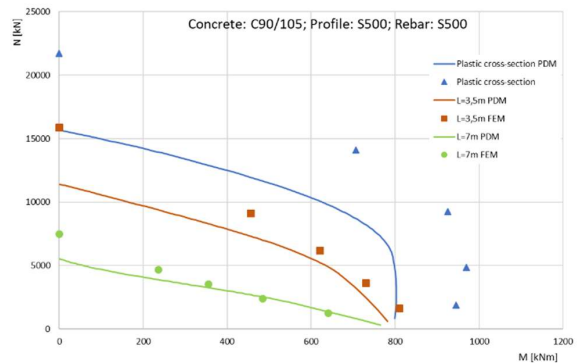
PEC 3 - Weak axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA500	500	90	4 ϕ 20	500	46	50	0.98%

		A	B	C	D
δ		0.48	0.61	0.48	0.48
λ	3.5 [m]	0.85	0.75	0.96	1.29
	7 [m]	1.69	1.50	1.91	2.13

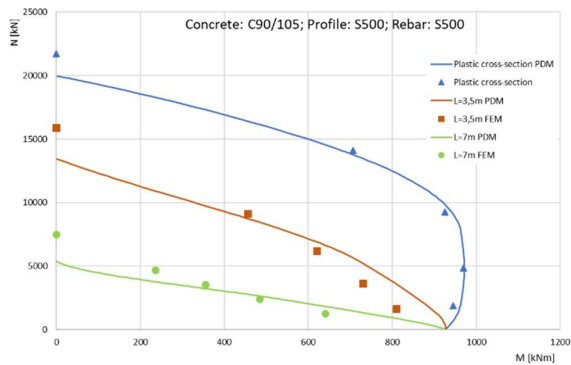
a) No material reduction



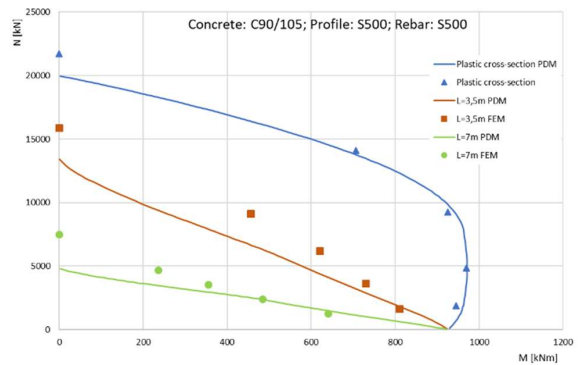
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project

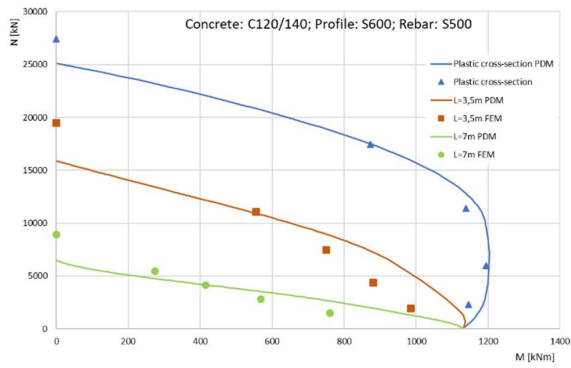


APPENDIX B. SENSITIVITY STUDY RESULTS

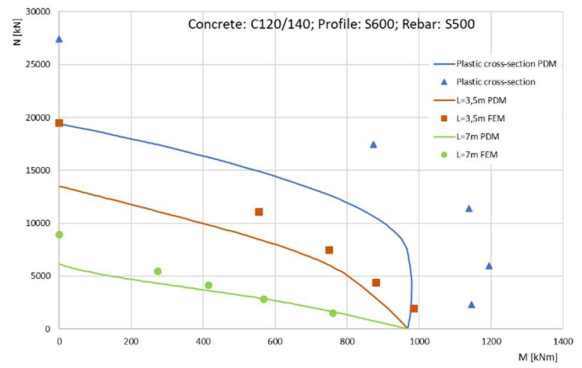
PEC 3 - Weak axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA500	600	120	4 ϕ 20	500	46	50	0.98%

		A	B	C	D
δ		0.46	0.59	0.46	0.46
λ	3.5 [m]	0.92	0.81	1.05	1.36
	7 [m]	1.83	1.61	2.11	2.25

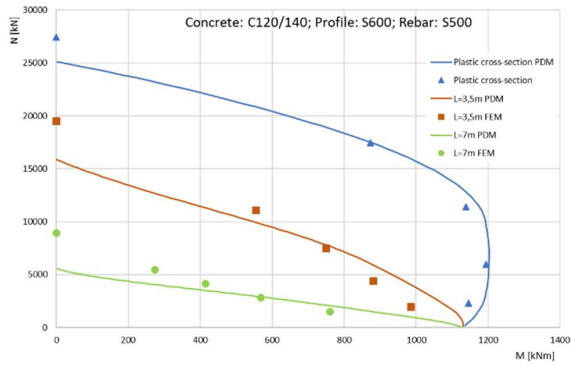
a) No material reduction



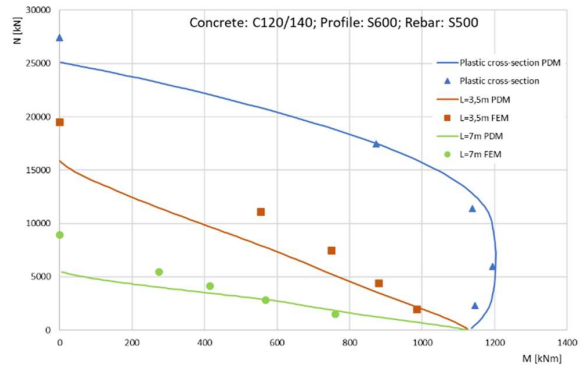
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



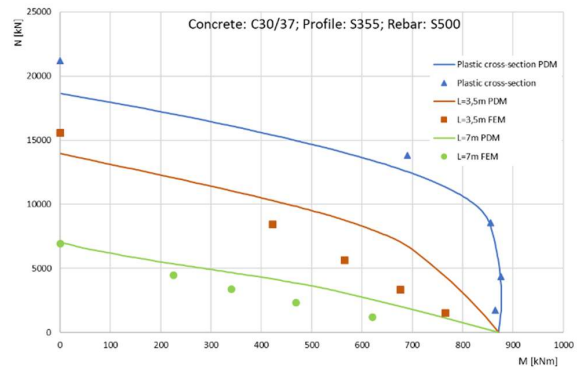
PEC 5 - Weak axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA1000	355	30	4 ϕ 28	500	60	65	0.94%

		A	B	C	D
δ		0.60	0.65	0.60	0.60
λ	3.5 [m]	0.76	0.74	0.84	1.39
	7 [m]	1.53	1.47	1.68	2.13

a) No material reduction



b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



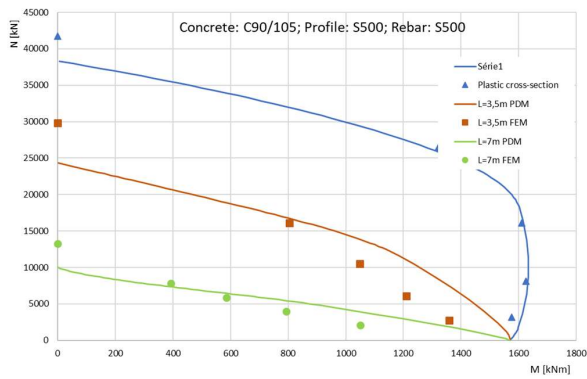
d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



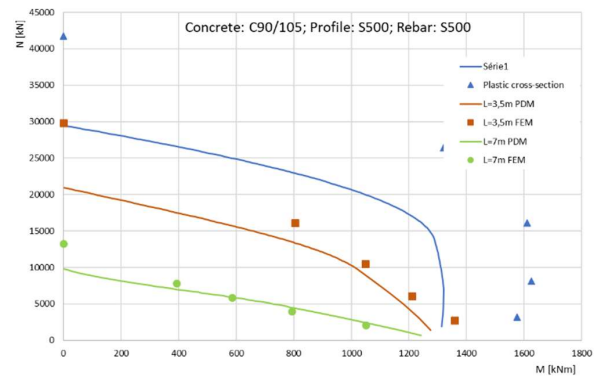
PEC 5 - Weak axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA1000	500	90	4 ϕ 28	500	60	65	0.94%

		A	B	C	D
δ		0.44	0.58	0.44	0.44
λ	3.5 [m]	0.92	0.80	1.08	1.47
	7 [m]	1.83	1.61	2.16	2.40

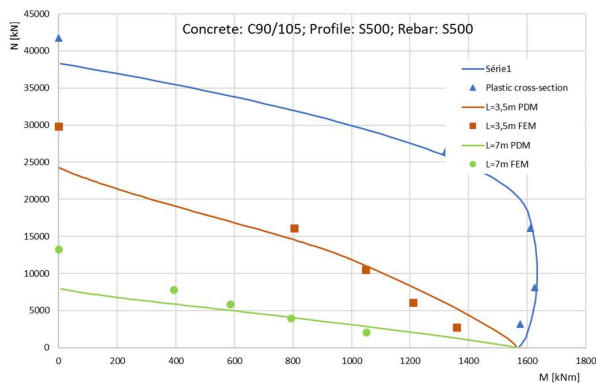
a) No material reduction



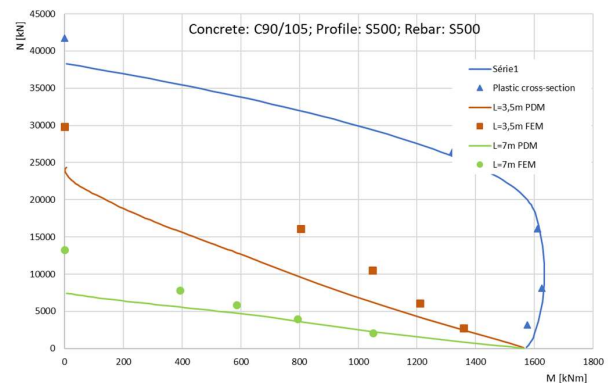
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project

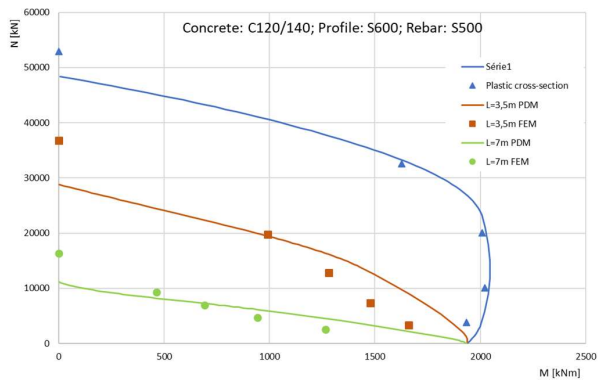


APPENDIX B. SENSITIVITY STUDY RESULTS

PEC 5 - Weak axis							
Profile		Concrete	Reinforcement				
Type	f_{yk} [MPa]	f_{ck} [MPa]	Rebars	f_{sk} [MPa]	c_y [mm]	c_z [mm]	ρ [%]
HEA1000	600	120	4 ϕ 28	500	60	65	0.94%

		A	B	C	D
δ		0.42	0.56	0.42	0.42
λ	3.5 [m]	0.99	0.86	1.19	1.53
	7 [m]	1.97	1.72	2.37	2.49

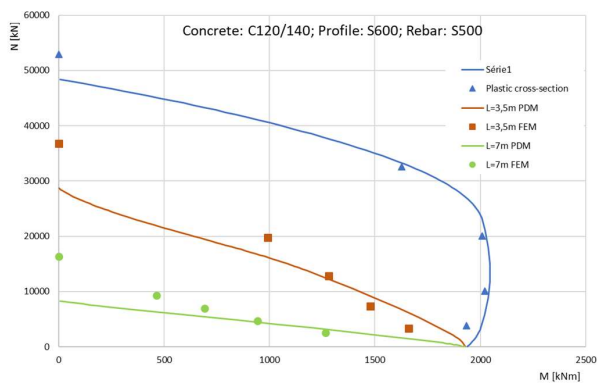
a) No material reduction



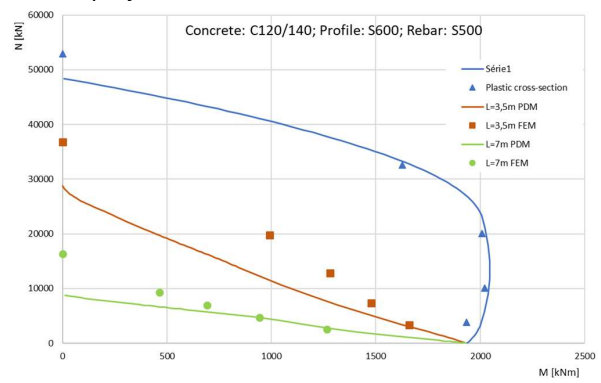
b) Reduction of f_{ck} with coefficient η and λ



c) Reduction of $(EI)_{eff,II}$ according to AS/NZ standard



d) Reduction of $(EI)_{eff,II}$ according to Smartcoco project



B.2.1 Box plot for high-strength materials : test 3

Resistance of cross-section

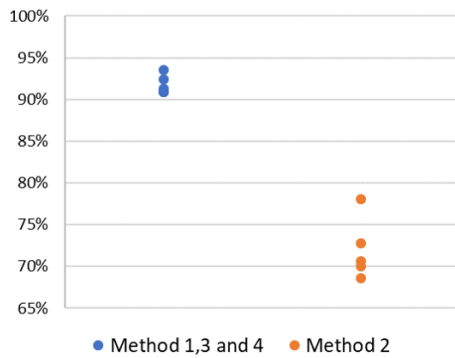


Figure B.12: Point A: Distribution of the ratios for the plastic cross-section resistance for test 3.

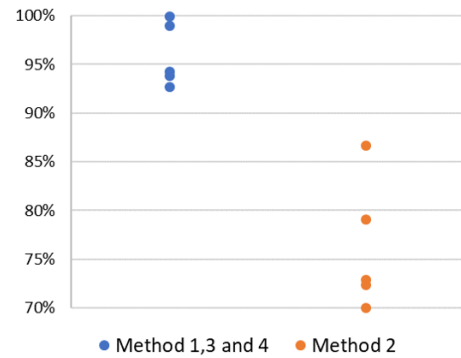


Figure B.13: Point B: Distribution of the ratios for the plastic cross-section resistance for test 3.

Column of 3,5 m long

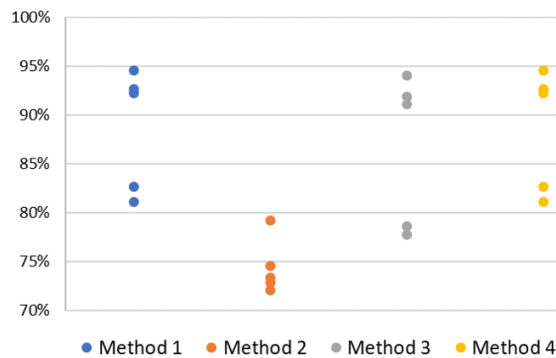


Figure B.14: Point A: Distribution of the ratios for the column of 3,5 m long for test 3.

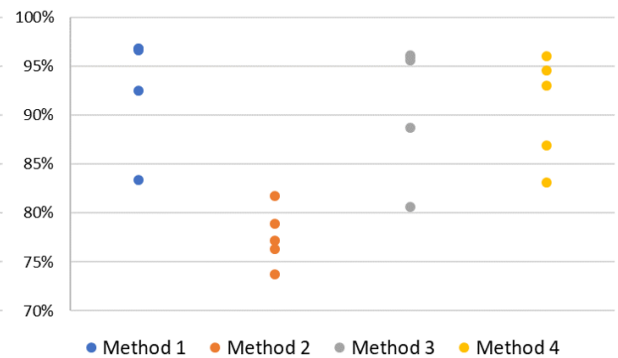


Figure B.15: Point B: Distribution of the ratios for the column of 3,5 m long for test 3.

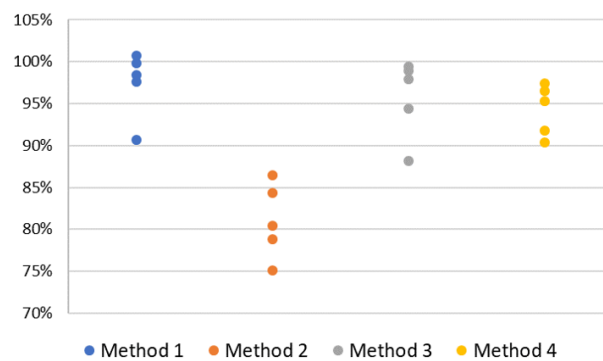


Figure B.16: Point C: Distribution of the ratios for the column of 3,5 m long for test 3.

Column of 7 m long

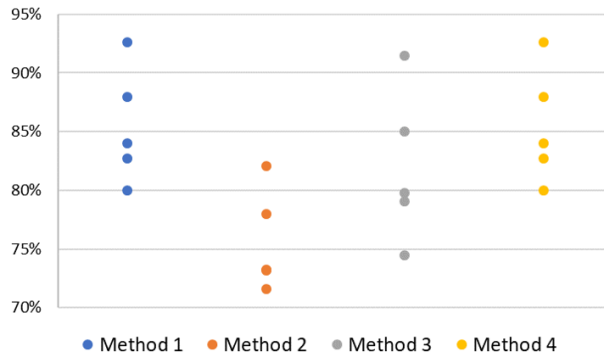


Figure B.17: Point A: Distribution of the ratios for the column of 7 m long for test 3.

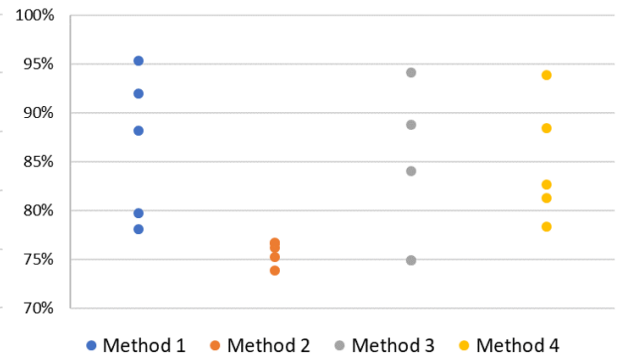


Figure B.18: Point B: Distribution of the ratios for the column of 7 m long for test 3.

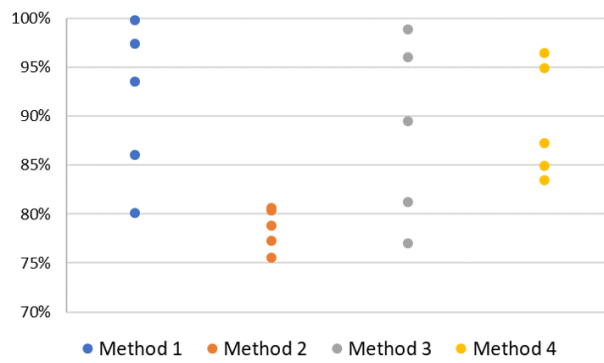


Figure B.19: Point C: Distribution of the ratios for the column of 7 m long for test 3.

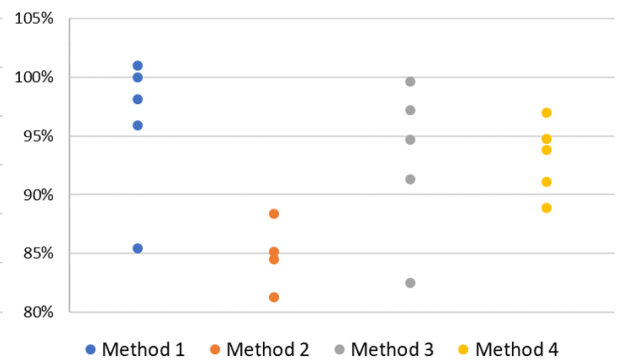


Figure B.20: Point D: Distribution of the ratios for the column of 7 m long for test 3.

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