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Chapter I

Design Philosophy and the Eurocodes

I- INTRODUCTION:

1.1 Scope:

With the development both technology and digital computing, professionals in construction are increasingly inclined to build ***taller and lighter***. This of course requires the combination of the ***concrete material*** as well as the ***metal frame elements***. The construction of buildings and civil engineering structures using composite construction elements first calls for the ***adaptation*** of known ***calculation techniques*** as well as the adaptation of the ***regulations and safety rules***. One of the regulations on which we will base ourselves in this chapter is obviously the Eurocode 4 that could, with certain precautions, be extended to the case of pathologies in other words repair and rehabilitation.

It is known that structural members that are made up of two or more different materials defined as ***composite elements***. Composite elements have a tremendous benefit. This is shown by the properties of each material which can be combined to form a ***single unit*** that performs better overall than its separate constituent parts. The most common form of composite element in construction is a ***steel-concrete composite***, however, other types of composites include; ***steel-timber***, ***timber-concrete***, ***plastic-concrete***, and so on.

As a material, concrete works well in compression, but it has less resistance in tension. Steel, however, is very strong in tension, even when used only in relatively small amounts. Steel-concrete composite elements use ***concrete's compressive strength*** alongside ***steel's resistance to tension***, and when tied together this ***results in a highly efficient*** and lightweight unit that is commonly used for structures such as multi-story buildings and bridges. *5.2.2. Challenges of mixed construction*

1.2. General Advantages of Composite Construction:

Composite steel-concrete construction is competitive in terms of overall cost. This is especially true for multi-story buildings with large spans between columns (over 12m) - fast and simple (shoring is not essential)

1.3. Mechanical, Economic and Architectural Advantages:

Compared to reinforced or pre-stressed concrete construction, one can notice a reduction in the weight of the structure with identical loading, an increase in the bending rigidity of the floor or beam and a reduction in the thickness of the floors (reduction in total height of the building with the same number of floors). The construction requirements are met regarding speed, very few scaffolding in service, very few wooden formwork used, and many more prefabricated elements,…

Compared to metal construction one can notice a significant improvement in fire resistance, composite beams have spans from 8 to 15m. They present also a monolithic aspect and a rigidity in its plane of a composite floor slab, which allows the transfer of horizontal forces (due to earthquakes for example) towards the elements ensuring the vertical stability of the building.

1.4. Main disadvantages:

- A very difficult site organization: storage, assembly and delivery areas, management and control of prefabricated elements, more advanced handling, sharing of cranes,
- A greater assembly and construction complexity: skilled workers and work team,
- Fixing connectors requires a lot of time, difficult if soldering (workers' experience, very good quality to ensure, etc.). The welding operation can be facilitated, if predisposition taken in the factory or if profiled sheet are used.

II- BASICS:

2. 1 - Fundamental Requirements:

In design, the engineers and professionals of constructions must take into account ***several aspects*** regarding the ***design and the serviceability*** of the structure. We have to take care of the ***random nature of the loads***, the ***variability of the materials***, and the ***problems that occur during construction***. This is all done to ***reduce the probability failure or unserviceability*** of the structure during its to an ***acceptably low level***. This is done of course through ***regulations and codes*** that stipulate that a structure must be calculated and produced so that:

1- It remains fit for use with an acceptable probability for the service for which it is performed.
2- It can resist with acceptable reliability for the loads for which it is calculated during execution and in service.

2. 2 – Limit States and Project Situations:

2.2.1 Limit States:

A limit state is a state beyond which the structure no longer meets the requirements for which it is designed. There are two (2) limit states: **U**ltimate **L**imit **S**tate (U.L.S.) and the **S**ervice **L**imit **S**tate (SLS).

The ultimate limit states are associated with the ruin of the structure, they include:

1. The loss of balance of the structure or one of its parts,
2. The ruin of the structure or one of its elements.

The service limit states correspond to the specified operating criteria that cannot be exceeded, they include:

1. Deformations and curvature affecting the appearance or operation of the construction, or causing damage to non-structural elements,

2. Vibrations bothering the occupants, damaging the building or its contents.

2.2.2 Design Type Situations or "Project Situations":

1. Persistent or durable situations, corresponding to normal conditions,
2. Transient situations , for example, during construction,
3. Accidental situations such as fire or earthquake.

2.3- Actions:

In the parts 1.1 of Eurocodes 2, 3 and 4 each one of these have a chapter "Basis of design" dedicated to the definitions, classifications and principles of limit state design. They are set out in details in these parts. This mainly emphasis on design of structures for buildings.

An action (F) is a force (load) applied to the structure (direct action) or an imposed deformation (indirect action) such as support displacements, or thermal effect actions, which are classified mainly according to their variation over time:

- Permanent Actions (G), such as self-weight of a construction or a structure, sometimes called also 'dead load',
- Variable actions (Q), sometimes called 'live load'
- Imposed load (I), such as Wind (Wn) or Snow (Sn),
- Temperature load (T),
- Accidental Actions (A) such as impact load from a vehicle.

2.4- Determination of the values of actions:

2.4.1 Characteristic values:

The ***characteristic values*** are fixed by ***regulations and codes***: In Eurocode 1, in other codes, or by the customer if the minimum requirements laid down in the standards are met within the codes in use. In this case, the ***permanent actions*** are represented and ***specified by a characteristic value G_k***. The term '***characteristic***' implies a ***defined fractile*** of an ***assumed or supposed statistical distribution*** of the action. The distribution of the action is modeled as a ***random variable***. For permanent loads it is usually the ***mean value*** i.e. 50% fractile. ***Four (4) representatives values represent the variable loads in this case.***

- Characteristic (Q_k) , normally the lower 5% fractile,
- Combination ($\psi_0 Q_K$), for use where the action is assumed to accompany the design value of another variable action,
- Frequent ($\psi_1 Q_k$),
- Quasi-Permanent ($\psi_2 Q_K$).

It should be noted also that the values of the ***combination factors ψ_0, ψ_1 and ψ_2 are all less than 1.0***. These values are given in the relevant Part of Eurocode 1. As an example, for imposed loads on the floors of offices, category B, they are 0.7, 0.5 and 0.3, respectively.

Design values of actions are, in general: $F_\Delta = \gamma_G G_k$ and in particular:

$$G_\Delta = \gamma_G G_K \quad \text{...} \mathbf{1.1}$$

$$Q_d = \gamma_K Q_K \text{ or } Q_d = \gamma_Q \psi_i Q_K \quad \text{................................} \mathbf{1.2}$$

Where: γ_G *and* γ_Q are partial safety factors for the actions (Eurocode 1). These factors depend on the limit state considered and if the action tend to increase or decrease which is commonly known as favorable or unfavorable action effects. The values used in this course are given in the table 1.1 below.

Tab 1.1 Values of γ_G *and* γ_Q for persistent design situation

Action	Permanent		variable	
	Unfavourable	Favourable	Unfavourable	Favourable
Ultimate **L**imit **S**tates	1.35	1.35	1.5	0
Serviceability **L**imit **S**tates	1.0	1.0	1.0	0

2.4.2 Effect of Actions :

The effects of actions are the response of the structure or the elements of the structure to these actions:

$$\boldsymbol{E_d = E(F_d)} \text{ .... } \mathbf{(1.3)}$$

Where the function $\boldsymbol{E}$ represents the response of the structural analysis. The effect is defined as an internal force or moment sometimes denoted $\boldsymbol{S_d}$ and verification for an ultimate limit state that consists of verifying or checking that:

$$\boldsymbol{S_d \le R_d} \text{ or } \boldsymbol{E_d \le R_d} \text{ ....} \mathbf{(1.4)}$$

Where $\boldsymbol{R_d}$ is the relevant design resistance of a cross section of a member of the structure considered.

2.4.3 Resistances:

The resistances (R_d) are calculated using design values of the properties of materials, (X_d) given by the following expression:

$$\boldsymbol{X_d = \frac{X_K}{\gamma_M}} \text{ .... } \mathbf{(1.5)}$$

Where $\boldsymbol{X_K}$ is a characteristic value of the property and $\boldsymbol{\gamma_M}$ is the partial factor of safety for the property in question. The characteristic value is typically a 5% lower fractile (i.e. for compressive concrete strength). If the statistical distribution is not well established it is the replaced by a nominal value (i.e. the yield strength of the structural steel) that is so chosen for use in design replacing X_k .

Tab 1.2 Values of γ_M for properties of materials and resistances

Material	Steel	Reinforced Steel	Profiled Sheeting	Concrete	Shear Connection
Property	f_y	f_{sk}	f_{yk}	f_{ck}	f_u
Symbol for γ_M	γ_a	γ_{sk}	γ_{yk}	γ_c	γ_v
Ultimate Limit States	1.10	1.15	1.10	1.5	1.25
Serviceability Limit States	1.0	1.0	1.0	1.0 or 1.3	1.0

2. 5- Combinations of Actions:

The Eurocodes treat systematically a problem or a subject using the many empirical procedures or methods developed and used in the past. The main principals for ultimate limit states are:

- Permanent actions are always present in all combinations,
- Each variable action is meant to be the principal or leading action (complete design value) and is combined with the lower combination values of other relevant variable actions,
- The design action effect is the most unfavourable of those calculated by this process.

Let us assume that a bending moment M_d is applied on a member of a structure. It is of course influenced let us say by its own weight (G), by an imposed vertical load (Q_1) and a wind load (Q_2). The fundamental combinations for verification for persistent design situations are then:

$$\gamma G_K + \gamma_{Q1} Q_{K,1} + \gamma_{Q2} \psi_{o,2} Q_{K,2} \text{ .... } \textbf{(1.6)}$$

And

$$\gamma G_K + \gamma_{Q1} \psi_{0,1} Q_{K,1} + \gamma_{Q2} Q_{K,2} \text{ .... } \textbf{(1.7)}$$

2. 5. 1. Ultimate Limit State:

In practice, the different combination given above will be written taking into account to different charges according to the Eurocode as follow:

a) **Ultimate Basic Limit State** (sustainable and transient project situations):

- 1.35 **G** (1) + 1.5 **Q**1(2) + [1.5x 0.87 **I** + 1.5x0.87 **Sn** + 1.5x 0.67 (1.2 **Wn)**] (3)

(1) If **G** is a favorable action, we will replace **1.35** by **1.0**

(2) **Q1** (basic variable action) can be **I** or **Sn** or (1.2 **Wn**) or **T**

(3) The variable action taken as basic variable action must not appear in the sum in square brackets.

Example:

Suppose two (2) variable actions: an operational overload **I** and the wind action Wn. The combinations of these two actions to be considered will be:

Basic variable action I

- 1.35 **G** + 1.5 **I** + 1.5x0.67 (1.2 **Wn**), **G** unfavorable
- **G** + 1.5 **I** + 1.5x0.67 (1.2 **Wn**), **G** favorable

Basic variable action Wn

- 1.35 **G** + 1.5 (1.2 **Wn**) + 1.5x0.87 **I**, **G** unfavorable
- **G** + 1.5 (1.2 **Wn**) + 1.5x0.87 **I**, **G** favorable

For building structures, the above combinations can be replaced by:

- **G** + 1.5 **Qi**
- **G** + 1.35 (**Q1** + **Q2** +…)

b) Ultimate Accidental Limit State

- **G** + **A** + [**I** or **Sn** or 0.2 (1.2 **Wn**)] + [**Sn** + **I**] (1)

(1) The variable action taken as basic variable action must not appear in the sum in square brackets.

2. 3. 2. Service Limit State:

In this section, we will limit ourselves to rare combinations:

- **G** + **Q**1 (1) + [0.87 I +.87 **Sn** + 0.67 (1.2 **Wn**)] (2)

(1) **Q1** (basic variable action) can be **I** or **Sn** or (1.2 **Wn**) or **T**

(2) The variable action taken as a basic variable action must not appear in the summation in square brackets.

For building structures, the above combinations can be replaced by:

- **G** + **Qi**
- **G** + 0.9 (**Q1** + **Q2** + ...)

III – CHARACTERISTIC OF MATERIALS:

This paragraph deals with the mechanical characteristics of the various materials that should be used for composite construction. Partial safety factors will also be cited.

3.1- Concrete:

3.1.1 Introduction

The strength and deformation characteristics for normal weight and lightweight concrete are given in Eurocode 2. The compressive concrete strengths used in the design rules in according to Eurocode 4 are based on cylinder strengths. Strength classes are defined for normal weight concrete as *Cx/y* and for lightweight concrete as *LCx/y*, where x and y are the characteristic cylinder and cube compressive strengths respectively. For example, C25/30 denotes a normal weight concrete with a characteristic cylinder strength of 25 N/mm^2 and a corresponding cube strength of 30 N/mm^2.

3.**1.2 Concrete resistance classes** (normal density): they are defined as mentioned below:

- f_{ck}: Characteristic resistance to compression on a cylinder measured at 28 days.
- f_{ctm}: Characteristic tensile strength on a cylinder measured at 28 days.
- E_{ctm}: Secant elasticity modulus [to be taken into account for actions with short-term effects]

Tab.1.4 strength and deformation characteristic for concrete (EC2)

	Strength classes for concrete														Analytical relation / Explanation
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck} + 8$ (MPa)
f_{ctm} (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0	$f_{ctm} = 0{,}30 \times f_{ck}^{(2/3)} \leq$ C50/60 $f_{ctm} = 2{,}12 \cdot \ln(1 + (f_{cm}/10)) >$ C50/60
$f_{ctk,\,0,05}$ (MPa)	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5	$f_{ctk,\,0,05} = 0{,}7 \times f_{ctm}$ 5% fractile
$f_{clk,\,0,95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6	$f_{clk,\,0,95} = 1{,}3 \times f_{ctm}$ 95% fractile
E_{cm} (GPa)	27	29	30	31	33	34	35	36	37	38	39	41	42	44	$E_{cm} = 22\,[(f_{cm})/10]^{0,3}$ (f_{cm} in MPa)
ε_{c1} (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8	see Figure 3.2 ε_{c1} (‰) $= 0{,}7\, f_{cm}^{0,31} < 2{,}8$
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8	see Figure 3.2 for $f_{ck} \geq 50$ Mpa ε_{cu1} (‰) $= 2{,}8 + 27\,[(98 - f_{cm})/100]^4$
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2,5	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{c2} (‰) $= 2{,}0 + 0{,}085\,(f_{ck} - 50)^{0,53}$
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{cu2} (‰) $= 2{,}6 + 35\,[(90 - f_{cm})/100]^4$
n	2,0									1,75	1,6	1,45	1,4	1,4	for $f_{ck} \geq 50$ Mpa $n = 1{,}4 + 23{,}4\,[(90 - f_{cm})/100]^4$
ε_{c3} (‰)	1,75									1,8	1,9	2,0	2,2	2,3	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{c3} (‰) $= 1{,}75 + 0{,}55\,[(f_{ck} - 50)/40]$
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{cu3} (‰) $= 2{,}6 + 35\,[(90 - f_{ck})/100]^4$

3.1.3 Concept of steel-concrete equivalence coefficient:

The determination of the characteristics of sections of composed beams homogenized with respect to concrete (area of homogenized section, moment of homogenized geometric inertia, ...)

$$\alpha = \frac{E_{cm}}{E_s} \quad \text{.......................................} \quad (1.8)$$

with E_S: longitudinal modulus of elasticity of structural steel, $E_S = 210000\ MPa$

$$f_{cm} = f_{ck} + 8 \quad \text{.......................................} \quad (1.9)$$

$$E_{cm} = 22000\left(\frac{f_{cm}}{10}\right)^{0.3} \quad \text{..............................} \quad (1.10)$$

3.2- Reinforcing Steel:

Concerning the reinforcement steel, one should take into consideration the following properties as it is indicated in the Eurocode 2.

- Method of manufacture,
- Density considered,
- Types of steel Steel grades _ ductility class (3 classes see annex C of EC2),
- f_{tk} Ultimate characteristic resistance in tension,
- ε_{uk} Unit elongation (corresponding to the achievement of this resistance)
- $F_{eE}400$ and $F_{eE}500$ (yield strength of 400 and 500 N / mm²)
- Longitudinal elasticity modulus Es varies between *190* and *200 kN/mm²*, but for simplicity, it is better to take it equal to $Ea = 210\ kN/mm^2$
- Elastic limits: stress-strain diagram
- Thermal expansion
- Yield strength fyk (in characteristic value for the 5% quantile)

3.3- Structural steel (Profiles…)

For composed structures in addition to concrete and reinforcement, steel elements are used to make these structures. These structural elements are cold or hot rolled steel, the characteristics of which are given in table 1.3 for steels of grades Fe 360, Fe 430, and Fe 510.

Tab.1.3 Characteristic of structural elements

Grade	Thickness t(mm)			
	t≤40mm		40mm≤t≤100mm	
	Fy (N/mm²)	Fy (N/mm²)	Fy (N/mm²)	Fy (N/mm²)
Fe 360	235	360	215	340
Fe 430	275	430	255	460
Fe 510	355	510	335	490

The values thus indicated in the table above can be used as characteristic values.

3.1- Design values of certain properties:

For steels covered by Eurocode 3, the following values must be taken into account in the calculations:

- Longitudinal Modulus of Elasticity: Ea = 210.000 N/mm^2

- Shear modulus: Ga = E / 2 (1+ νa)

- Poisson's ratio: νa = 03

- Density: ρa = 7850 kg /m^3

IV- Characteristics of the Cross Sections:

The effective width of concrete flanges should be detennined in accordance with the following provisions. When elastic global analysis is used, assume a constant effective width over the whole of each span. This value may be taken as the value b_{eff1} at mid-span for a span supported at both ends, and the value $b_{eff,2}$ at the support for a cantilever.

At mid-span or an internal support, the total effective width b_{eff}, see Figure 5.1, may be determined as:

$$\boldsymbol{b_{eff} = b_0 + \sum \beta_i b_{ei}} \quad \textbf{(1.8)}$$

Where:

- ***bo*** is the distance between the centers of the outstand shear connectors;
- ***bei*** is the value of the effective width of the concrete flange on each side of the web and taken as ***Lc /8*** but not greater than the geometric width ***bi*** . The value ***bi*** should be taken as the distance from the outstand shear connector to a point mid-way between adjacent webs, measured at mid-depth of the concrete flange, except that at a free edge ***bj*** is the distance to the free edge. The length ***Lc*** should be taken as the approximate distance between points of zero bending moment. For typical continuous composite beams, where a moment envelope various load arrangements governs the design, and for cantilevers, ***Lc*** may be assumed to be as shown in Figure 1.1.

(6) The effective width at an end support may be determined as:

$$b_{eff} = b_0 + \sum \beta_i \, b_{ei} \quad (1.9)$$

With:

$$\beta_i = (0.55 + 0.25 L_C / b_{ei} \leq 1.0 \quad (1.10)$$

Where:

bei is the effective width, see (5), of the end span at mid-span and ***Lc*** is the equivalent span of the end span according to Figure 1.1.

- The distribution of the effective width between supports and mid span regions lllay be assumed to be as shown in Figure 1.1.
- Where in buildings the bending moment distribution is influenced by the resistance or the rotational stiffness of a joint, this should be considered in the determination of the length L_C.
- For analysis of building structures, b_0 may be taken as zero and *bi* measured from the center of the web.

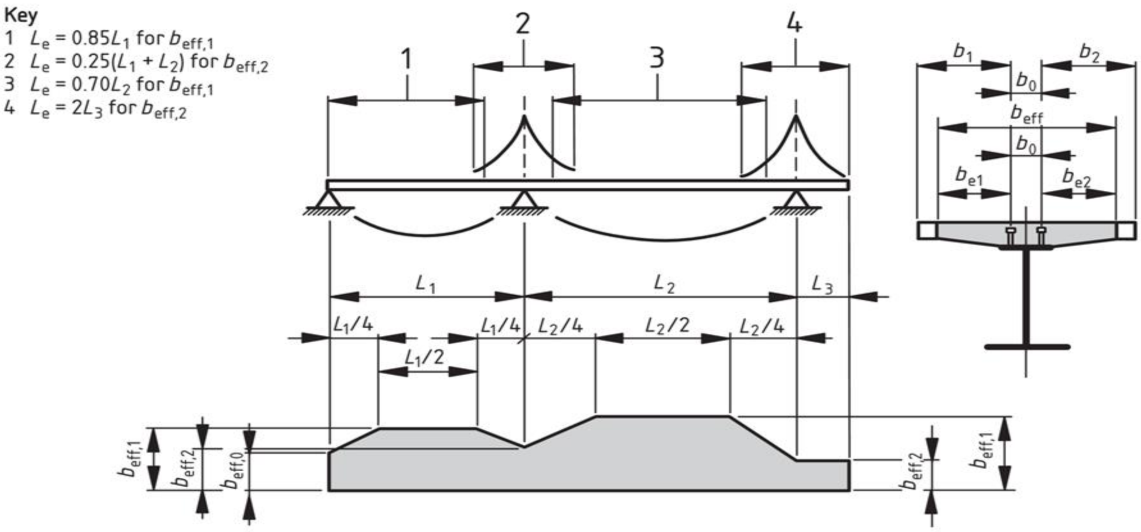

Fig.1.1: Equivalent spans, for effective width of concrete flange