Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 74 (1996)

Rubrik: Parallel Session 2B: Comparisons and application

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Comparisons and Application

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Examples of the Application of Eurocode 1

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Summary

This paper presents a series of trial calculations to illustrate the authors' interpretation of the requirements in Eurocode 1 for calculating the design loadings on structures and the load combinations to give the worst effects. The results of these calculations are compared with current design practice in Ireland and it is found that adoption of the factors given in Eurocode 1 will lead to different design loads for buildings. These differences include greater variable roof loading for both unlimited and restricted access, increased and more uniform loading intensities and higher wind pressures on walls while, in the design of foundations, the introduction of EC1 with Cases B and C will result in the need, certainly initially, to check both Cases B and C.

1. Introduction

The purpose of this paper is to investigate, by means of a number of examples (some based on sample problems by O'Brien & Dixon (1995)), the requirements of Eurocode 1 (EC1) for calculating the design loads on structures and the design of foundations. The examples form the basis for a comparison of the requirements of EC1 with those of the British standards which are currently used in Ireland to calculate the loadings on buildings and to design foundations.

2.4.4 Eccentrically Loaded Square Pad Foundation
The third foundation example is an eccentrically loaded square pad foundation with a 300 mm square column which provides a central vertical load of 300 kN and a horizontal load of 75 kN at a height of 4 m above the base of the foundation. The foundation widths and M_{max} values obtained for this foundation are given in Table 1. These results show that, for this eccentrically loaded foundation, both the size and the strength are governed by Case B.

2.4.5 Discussion
The calculated maximum widths and bending moments for the above foundation examples show that, using the factors given in Table 9.2 of EC1 - Part 1, it cannot be assumed in designs involving the strength of the ground and structural materials that Case B will always govern the strength of the member and Case C will always govern the size of the member. These examples have shown that the cases which control the size and strength of a foundation depend on the geometry of the problem and the nature of the loading. Until there has been more experience with the use of these cases to discover which is the relevant case in any design situation, it will be necessary, as required by EC1, to check that designs satisfy both cases.

3. Conclusions

The results of these examples show that, when compared with traditional practice in Ireland, adoption of the factors given in Eurocode 1 will lead to different design loads for buildings. These differences will include greater variable roof loading for both unlimited and restricted access, increased and more uniform loading intensities and higher wind pressures on walls. In the design of foundations, the introduction of EC1 with Cases B and C will result in the need, certainly initially, to check both Cases B and C.

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2.4.2 *Centrally Loaded Square Pad Foundation*

A square pad foundation supports a 200 mm square column which provides a central vertical load of 300 kN. The calculated foundation widths for Cases B and C, without any rounding-up as would normally occur in practice, are 0.83 m and 0.97 m respectively, as shown in Table 1. Thus the maximum foundation width of 0.97 m is obtained for Case C. Using this larger width, the maximum bending moments in the foundation at the face of the column, M_{max} for Cases B and C are 31.9 kNm/m and 23.6 kNm/m respectively, as shown in Table 1, and so the M_{max} value is obtained for Case B. Thus, for the ultimate limit state, the size of this foundation is governed by Case C and the strength by Case B. If the column is increased to 400 mm, then the M_{max} value occurs for Case C, rather than for Case B, as shown by the values in Table 1. For zero column width, i.e. for a concentrated load, M_{max} in kNm/m is independent of the foundation width chosen, whether the Case B or C widths, being equal to the Case B design load divided by 8.

	Square Pad Foundation V = 300 kN				Strip Foundation V = 400 kN/m		Square Pad Foundation V = 300 kN H = 75kN at h = 4m	
	Case B		Case C		Case B	Case C	Case B	Case C
Foundation width, b (m)	(0.83)		**0.97**		(1.04)	**1.36**	**3.72**	(3.59)
Design width (m)	0.97		0.97		1.36	1.36	3.72	3.72
Column/wall width (mm)	200	400	200	400	300	300	300	300
M_{max} (kNm/m)	**31.9**	11.6	23.6	**12.9**	**55.7**	41.3	**63.6**	44.9

Table 1: Calculated widths and bending moments for the foundation examples

Foundation settlements have not been considered in the above calculations. The traditional method for ensuring that foundation settlements are not excessive has been to use a global factor of safety of 3 and unfactored loads which, for this example, yields a foundation width of 1.16 m. This is greater than the design value of 0.97m obtained above using the factors in Table 9.2 of EC1 - Part 1 for the ultimate limit state. The M_{max} value for structural design of the foundation is determined using this width and a linear bearing pressure to balance the factored loads for the ultimate limit state. This approach yields an M_{max} value of 36.0 kNm/m which again is greater than the design value obtained above. Thus, if the ultimate limit state is the design criterion, this example indicates that using EC1 will result in foundations that are smaller and weaker than those obtained by the traditional methods. For comparison, in the design of embedded retaining walls using the factors in EC1, a number of investigators, e.g. Orr (1993), have found that these factors result in retaining walls that are smaller but stronger than those obtained by the current design methods.

2.4.3 *Centrally Loaded Strip Foundation*

The second foundation example is a long strip foundation which supports a 300 mm thick wall providing a central vertical load of 400 kN/m. The result of the calculations for this example are also given in Table 1 and show that, for this foundation, the maximum foundation width (1.36 m) is obtained for Case C and the M_{max} value (41.0 kNm/m) is obtained for Case B. Thus, for this example, as for the centrally loaded square foundation with the 200 mm column, the size is governed by Case C and the strength by Case B.

be multiplied by a factor of 0.85 to allow for non-simultaneous action between building faces and by a dynamic augmentation factor of 1.02. Hence the total building force is:

$$0.85(1.02)(0.726)(0.76 + 0.26)(0.82)(35x9) = 166 \text{ kN}$$

which is a force 5% in excess of that found using EC1. The internal wind pressure is found from the product of the dynamic wind pressure, 0.726 kN/m^2, an internal pressure coefficient of -0.3 and a size effect factor of 0.71 giving a total pressure of -0.16 kN/m^2. The net wall pressure is the difference between the external and internal pressures:

$$(0.726x0.76x0.82) - (-0.16) = 0.61 \text{ kN/m}^2$$

That this value is 50% in excess of the wall pressure calculated in accordance with EC1 must surely be a matter for concern. The previous British standard, CP3, Chapter V, gives a total building force of 152 kN and a wall pressure of 0.51 kN/m^2.

2.4 Foundation Design

2.4.1 Geotechnical Design and Cases A, B and C

Eurocode 1 requires that designs must satisfy three cases, A, B and C, with different sets of partial factors for the actions and material properties for each case given in Table 9.2. Case A is concerned mainly with the stability of structures where the strength of the ground or structural materials is of minor importance and so is not normally relevant for buildings. Case B is the standard case for designs involving the strength of structural materials and has been used in the examples above. In Case B the partial factor on permanent actions is 1.35. In many geotechnical design situations, such as bearing resistance and slope stability, the weight of the soil may be both an action and a contribution to the resistance. It is largely for this reason that permanent actions are not factored in traditional geotechnical designs and that Case C, with a factor of unity on permanent actions and corresponding factors on soil strength parameters, has been introduced in Eurocode 1.

While Case B governs most structural designs and Case C most geotechnical designs, there are situations where this does not hold. To investigate this, three examples of foundations are given which demonstrate that the size of the foundation and the maximum bending moment can be governed by different combinations of Cases B and C. The examples are all for foundations founded at a depth of 1 m in a deposit of dry sand with $\phi' = 30°$ and unit weight equal to 18 kN/m^3. The foundation widths and maximum bending moments in the foundations are calculated for the ultimate limit state using the factors for Cases B and C given in EC1 - Part 1, Table 9.2 and the bearing resistance equations and factors given in EC7 (ENV 1997-1:1994).

The foundation size is first calculated for Cases B and C separately using the load and soil strength factors for each case. The case giving the larger width governs the design of the foundation size. The larger width is then used as the basis for determining the maximum bending moment for structural design of the foundation with both Case B and C loadings and assuming linear(uniform) bearing pressures beneath the foundation. If Cases B and C were treated totally separately, with the Case B width used to calculate the Case B bending moment and similarly for Case C, a smaller design bending moment would generally be obtained.

and:

$$w_e \quad = 1.71\text{x}(-0.3)\text{x}0.276 \quad = \quad -0.14 \text{ kN/m}^2$$

This gives a total N-S wind force on the building of (0.36 + 0.14)(35x9) = 158 kN.

2.3.2 Wall Wind Pressure

As only one wall is being considered, the internal pressure coefficient, c_{pi} must be calculated. Because of the uniform distribution of openings around the perimeter, the opening ratio, μ, for the south wall is:

$$\mu = (\textit{length of walls other than south wall})/(\textit{total perimeter length})$$

$$= \frac{35 + 14 + 14}{35+14+35+14} \quad = \quad 0.64$$

In the graph of Fig. 6.10.2.9, c_{pi} varies linearly with μ. Interpolating gives:

$$c_{pi} = 0.8 - 1.3\left[\frac{\mu - 0.1}{0.9 - 0.1}\right] = 0.8 - 1.3\left[\frac{0.64 - 0.1}{0.8}\right] \quad = \quad -0.078$$

The mean height of the window openings is assumed to equal the mean building height of 4.5 m. As this is less than the minimum height, z_{min}, the roughness coefficient is:

$$c_r(z_i) = c_r(z_{min}) = k_r\, Ln(z_{min}/z_o) = 0.22\, Ln(8/0.3) = 0.722$$

The exposure coefficient at the mean height of the window openings is then:

$$c_e(z_i) = 0.722^2[1 + 7\text{x}0.22/0.722] = 1.63$$

Thus the internal wind pressure is:

$$w_i = c_e(z_i)c_{pi}q_{ref} = 1.63(-0.078)(0.276) = -0.035 \text{ kN/m}^2$$

Hence, according to EC1, the total wind pressure on the south wall is:

$$w_e - w_i = 0.36 - (-0.035) = 0.40 \text{ kN/m}^2$$

2.3.3 Comparison with National (British Standard) Code

The British standard, BS6399:Part 2 (1995), is similar in many respects to EC1 - Part 2.3 but is perhaps more complex. The basic wind speed for London specified in this standard is the same as the Eurocode value at 21 m/s and reduction factors, similar in definition to EC1, are taken for this example as unity. Taking the 'standard' approach gives a 'terrain and building' factor which increases the effective wind speed to 34.4 m/s and implies a dynamic wind pressure of 0.726 kN/m^2. External pressure coefficients, dependent on building geometry, are 0.76 and -0.26 for the south and north walls respectively and a size effect factor for external pressure is 0.82. The product of dynamic wind pressure, external pressure coefficient and size effect factor gives a wind pressure on each wall. However, to convert these to a total building force, the net pressure must

2.3 Wind Loading

The building illustrated in Fig. 1 is located at sea level in a suburban area outside London, 10 km from the sea. In order to check the capacity of the structure to resist applied horizontal forces, the total wind force in the N-S direction is required. In addition, the maximum wind pressure on the south wall is required for a check of the capacity of masonry wall panels. The interior of the building is open-plan and windows and doors are located uniformly around the perimeter.

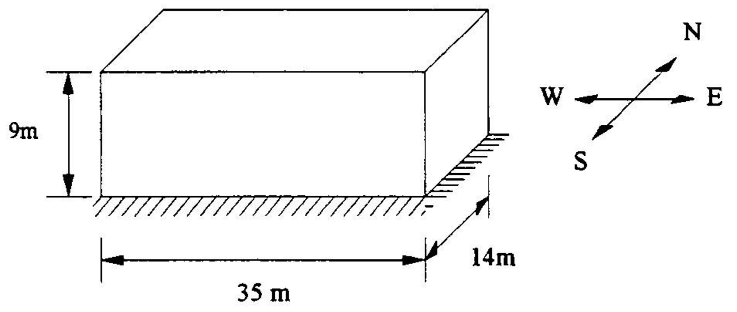

Fig. 1. Rectangular building

2.3.1 Total Wind Force on the Building

Taking the factors relating to direction (c_{DIR}), seasonal variation (c_{TEM}) and altitude (c_{ALT}) as unity, the reference wind velocity can be found directly from Fig. 6.7.2 in EC1 - Part 2.3 (ENV 1991-2.3:1993) and is taken for London to be 21 m/s. Hence the reference wind pressure is:

$$q_{ref} = 0.5\rho v_{ref}^2 = 0.5\text{x}1.25\text{x}21^2 = 276 \text{ N/m}^2$$

Taking a roughness category of 3 in Table 6.8.1, the terrain factor, $k_r = 0.22$, the roughness length, $z_o = 0.3$ m and the minimum height, $z_{min} = 8$ m. Hence the roughness coefficient at the top of the building is:

$$c_r(9) = k_r\,Ln(z/z_o) = 0.22\,Ln(9/0.3) = 0.748$$

Taking a topography factor of unity, the exposure coefficient becomes:

$$c_e(9) = c_r^2(9)c_t^2(9)\left[1+\frac{7k_r}{c_r(9)c_t(9)}\right] = 0.748^2[1 + 7\text{x}0.22/0.748] = 1.71$$

Referring to Fig. 1, the ratio of building depth to height, d/h is 14/9 = 1.56. The external pressure coefficient for the front face, Zone D, is then interpolated from Table 6.10.2.2:

$$c_{pe} = 0.8 - 0.067(d/h - 1) = 0.763$$

while the corresponding coefficient for the back face, Zone E, is -0.3. The resulting wind pressures on the front and back faces of the building are respectively:

$$w_e = c_e(z_e)c_{pe}q_{ref} = 1.71\text{x}0.763\text{x}0.276 = 0.36 \text{ kN/m}^2$$

$$Q_k = 0.75 + 0.46 = 1.21 \text{ kN/m}^2$$

2.1.3 Discussion

The current British standard for loading on buildings, BS6399:Part 3 (1988) is similar to EC1 in its requirements for snow load. However it was common practice in the past among many Irish designers to use a notional value of 0.75 kN/m^2 for the **total** variable loading due to occupancy **and** snow. Further, the part of the roof being used as an escape route would have been designed for a floor loading of only 1.5 kN/m^2. Hence the total variable loadings would have been 1.5 kN/m^2 and 0.75 kN/m^2 for parts with unlimited and restricted access respectively compared with the much greater values of 3.46 kN/m^2 and 1.21 kN/m^2. required by EC1.

2.2 Ultimate Limit State Design Loading Intensities

The minimum and maximum ULS design loading intensities are required for the roof in the previous example for the transient design situation.

2.2.1 Minimum and Maximum ULS Loading Intensities

Using the unfactored loadings obtained above and EC1 - Part 1 (ENV 1991-1:1994), the total ULS loading is given by the combination equation:

$$\gamma_g 4.88 + \gamma_q(3.0 + \psi_o 0.46)$$

From EC1 - Part 1, Tables 9.2 and 9.3, the upper and lower limits on this total design loading are:

$$\text{Maximum} = 1.35(4.88) + 1.5[3.0 + 0.6(0.46)] = 11.50 \text{ kN/m}^2$$
$$\text{Minimum} = 1.0(4.88) + 0[3.0 + 0.6(0.46)] = 4.88 \text{ kN/m}^2$$

2.2.2 Discussion

For an example such as this, there is no equivalent in the British standards for the ψ_o combination factor. In current Irish (or UK) practice, ψ_o would be taken by default as unity.

Another difference in the design approach arises from footnote 3 to Table 9.2 of EC1 - Part 1 which states that all permanent actions from one source are multiplied by 1.35 if the total resulting action effect is unfavourable. Hence if, for example, a continuous beam were being analysed according to EC1, as the self weight is all from the same source, the minimum factors in this situation are $\gamma_g = 1.35$ and $\gamma_q = 0$ resulting in a minimum loading of:

$$\text{Minimum} = 1.35(4.88) + 0[3.0 + 0.6(0.46)] = 6.59 \text{ kN/m}^2$$

However, using the British standards, factors of $\gamma_g = 1.0$ and $\gamma_q = 0$ would be applied to alternate spans resulting in a minimum loading of 4.88 kN/m^2.

2. Examples

2.1 Roof Loading

A hotel is to be constructed in County Wicklow, Ireland, 150 m above sea level. Part of the roof is to be used as a fire escape route. It is made of a reinforced concrete slab of thickness 175 mm which is covered in a lightweight sealant. The roof is sloped at 1:10 to facilitate the runoff of water. The design permanent and variable gravity loading intensities are required. The vertical loading due to wind does not govern the roof design.

2.1.1 Total Loading on Roof Parts with Unlimited Access
The permanent gravity loading consists of the self weight plus an allowance for ceilings and services:

Slab self weight = (25)(0.175) =	4.38 kN/m^2
Ceilings and services =	0.50 kN/m^2
Total permanent gravity load, G_k =	4.88 kN/m^2

When a roof is accessible, the variable loading due to occupancy should equal the loading for the area from which there is access. For this example, access will be through the stairs. From EC1 - Part 2.1 (ENV 1991-2.1:1993) the imposed variable gravity loading for a stairs in a residential buildings is 3.0 kN/m^2. This, together with the snow loading, constitutes the total variable gravity loading. From the snow load contour map in EC1 - Part 2.1, the basic snow load for County Wicklow (south of Dublin) is, s_b = 0.5 kN/m^2. The characteristic snow load is calculated from the equation:

$$s_k = s_b + (0.09 + 0.1s_b)(A - 100)/100$$

where A is the altitude which, in this case, is 150 m. Hence:

$$s_k = 0.5 + (0.09 + 0.1x0.5)(150 - 100)/100 = 0.57 \text{ kN/m}^2$$

From EC1 - Part 2.1, a slope of 1 in 10 (or 6°) implies a shape coefficient, μ_i of 0.8. Hence, assuming unit exposure and thermal coefficients, the design snow load intensity is obtained from:

$$s = \mu_i C_e C_t s_k = (0.8)(1)(1)(0.57) = 0.46 \text{ kN/m}^2$$

Thus the total variable gravity loading, $Q_k = 3.0 + 0.46 = 3.46$ kN/m^2.

2.1.2 Variable Loading on Roof Parts with Restricted Access
The escape route for this building is limited to one portion of the roof, with access to the remaining area closed off by a masonry wall. The permanent gravity loading for the escape route is unchanged at 4.88 kN/m^2. As there is restricted access to the rest of the roof, a loading intensity of 0.75 kN/m^2 can be adopted for variable loading due to occupancy. Hence the total variable gravity loading is:

A Relative Comparison of Actions and Strength in Four Concrete Building Design Codes

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Summary

Concrete building design codes from USA, Britain, Egypt, and the Eurocodes are considered. Comparisons of the provisions for actions (loads), and for resistance(strength) of sections in flexure are carried out. Several parameters are considered including variable actions for residential buildings, offices, shops, and different material strengths. Issue and consequences of mixing actions from one code & resistance from another code are also discussed.

1. Introduction

Structural Design codes of different countries provide the engineers with data and procedures for design of the structural components. Differences, sometimes large differences, could be noticed between the codes in the data given for actions, in the provisions for evaluating resistance of sections, and also in other code requirements for durability, detailing,...The paper presents a quantitative comparison of four concrete building codes. Actions and resistances are evaluated and compared for several cases.

Scope of Work: The design codes and load codes considered are ACI 318-89 and ASCE 7-88 from USA, BS 8110 and BS 648/BS 6399 from Britain, EC1 and EC2 from European Community, and Egyptian code of practice for the design of reinforced concrete structures (ECOP 89) and code for Loads (ECOPL 93). The following parameters are considered in the study: *i)* Permanent actions (dead loads) and Variable actions of buildings (live loads), *ii)* Types of building occupancy for variable actions: residential, offices and shops, *iii)* Action effects: flexure and longitudinal force, *iv)* Structural elements: beams and axially loaded short columns (briefly), *v)* Limit states: ultimate limit state, *vi)* Steel yield strength f_{yk} = 360, 500 N/mm^2 and concrete cylinder strength f_{ck} = 25, 40 N/mm^2.

2. Basis for Comparison of the Four Considered Codes

Consider a beam in a typical one way slab construction, e.g. beam *b1* shown in *Fig. 1.* If this beam is designed, for example according to the ACI code, it is required that at failure (assuming b_1 is a singly reinforced beam):

$$(1.4w_D + 1.7w_L)\ \frac{Bl^2}{8} \leq \phi\ \rho\ f_y(1-0.59\frac{\rho\ f_y}{f'_c})\ bd^2$$

$$(1.4w_D + 1.7w_L)\ C_1 \leq \mu\ bd^2$$

$$bd^2 \geq \frac{(1.4w_D + 1.7w_L)}{\mu}\ C_1 \qquad (1)$$

In equation 1, C_1 is a function of the structural system, and the area supported by the beam. C_1 does not, in most cases, differ from one code to the other. Numerator of Equ. (1) is a function of of the live load & the load factors given in the codes, and also of weight of structural and non-structural elements. Denominator of Equ. (1) is a function (f'_c, f_y, ρ), which are selected by the designer, and a function of the resistance model given in the code (stress-strain relations, limit strain, stress block shape, partial safety factors for materials γ_m). Equations similar to Equ.(1) could be written for different codes and for different load effects.

Evaluating the numerator of Equ. (1), a comparison of the ultimate design loads in different codes can be done. This is described in Sec. 3, & in Tables 1,2,3. Evaluating the denominator of Equ. (1), a quantitative comparison of the ultimate moment of resistance, as given by the different codes, can be done. Details are given in Sec. 4, and in Table 4. Above comparisons are useful, but they are not sufficient. Comparison of codes should include both action and resistance. This could be achieved using Equ. (1) as described below.

Consider two codes: code 1, and code 2. Using Equ. (1), bd^2 is evaluated for both codes (in terms of C_1). Then, the ratio of bd^2 for code 1 to bd^2 for code 2 is evaluated (C_1 is eliminated). If this ratio is larger than 1, then code 1 is more conservative (or less economic) than code 2, and vice versa. Repeating above process for several cases could give an idea on the economy of concrete structures as designed according to different codes. Examples are given in Sec. 5 and Table 6.

3. Actions in The Four Considered Codes

Table 1 presents some values of variable actions (LL) specified for different types of building occupancy. Notice, for example, large differences in live load intensities given for balconies, large differences for corridors in residential bldgs., & small differences for stair loads in shops.

Table 2 presents above values (LL) combined with permanent actions (DL), each multiplied by relevant load factor for ultimate limit state, i.e., Table 2 presents the evaluation of numerator of Equation 2. Following assumptions are made for evaluating items in Table 2: *i)* DL, LL are applied to the same area, *ii)* The lower value of DL intensities (3 kN/m^2) correspond to DL in thin slab or void slab construction plus the flooring weight, and the higher value (7 kN/m^2) correspond to dead loads in thick slab constructions plus the flooring weight.

Use	Code	Floors kN/m²	Corridors kN/m²	Stairs kN/m²	Balconies kN/m²
Residential	ACI 318-89	1.9	4.8	4.8	4.8
	EC2	2.0	2.0	3.0	4.0
	BS 8110	1.5	4.0	1.5	1.5 [b]
	ECOP 89	2.0	2.0 ⊕	3.0	3.0
Offices	ACI 318-89	2.4	4.8	4.8	4.8
	EC2	3.0	3.0	3.0	3.0
	BS 8110	2.5	4.0	4.0	2.5 [b]
	ECOP 89	2.5	2.5 ⊕	4.0	4.0
Shops	ACI 318-89	4.8	4.8	4.8	4.8
	EC2	5.0	5.0	5.0	5.0
	BS 8110	4.0	4.0	4.0	4.0 [b]
	ECOP 89	5.0 [a]	5.0 [a]	5.0 [a]	5.0 [a]

a- The variable action intensity for warehouses & stores is given by ≥ 10.0 *kN/m²* (according to the stored materials).
b- Imposed Load to be same as that on floor to which access is given.
⊕ This value is assumed to be same as that of floors.

Table 1. Values of Variable Action Intensities for Different Types of Building's Occupancy in Four Different Codes

Use	Dead Load kN/m²	ACI / EC2	EC2 / EC2	BS 8110 / EC2	ECOP 89 / EC2	EC2 Value kN/m²
Residential (Floors)		*0.95*	*1.0*	*0.75*	*1.00*	***2.00***
	3.00	1.05 **	1.0	0.94	1.06	7.05 ⊕
	4.00	1.05	1.0	0.95	1.07	8.40
	7.00	1.05	1.0	0.98	1.08	12.45
Residential (Balconies)		*1.20*	*1.0*	*0.375*	*0.75*	***4.00***
	3.00	1.23	1.0	0.66	0.90	10.05
	4.00	1.21	1.0	0.70	0.91	11.40
	7.00	1.16	1.0	0.79	0.97	15.45
Offices (Floors)		*0.80*	*1.0*	*0.833*	*0.833*	***3.00***
	3.00	0.97	1.0	0.96	0.96	8.55
	4.00	0.98	1.0	0.97	0.97	9.90
	7.00	0.99	1.0	0.99	1.02	13.95

** $1.05 = \frac{1.4\,D_{ACI} + 1.7\,L_{ACI}}{1.35\,D_{EC2} + 1.5\,L_{EC2}}$ ⊕ $7.05 = 1.35\,D_{EC2} + 1.5\,L_{EC2}$

Notes: 1- The values written in bold italic font represent the Variable Action intensity according to EC2, Values are taken from *Table 1.*
2- The values written in italic font represent relative Variable Action intensity with respect to EC2.
3- Columns 3,4,5,6 give relative values with respect to EC2

Table 2. Comparison of Ultimate Loads for Different Types of Building's Occupancy in Four Different Codes

ACI 318-89			Dead Loads D		Live Loads L		Wind Loads W	
	Case	Loads Considered	Max*	Min**	Max*	Min**	Max*	Min**
	1	D, L	1.4	0.9!	1.7	0	-	-
	2	D, L, W	0.75x1.4	0.75x1.4!	0.75x1.7	0.75x1.4!	0.75x1.7	0.75x1.7!
	3	D, W	0.75x1.4	0.9	-	-	0.75x1.7	1.3

* Loads increase load effect under consideration
** Loads decrease load effect under consideration
! This value is assumed by the authors

EC2			Permanent Loads G_K		Variable Imposed Loads Q_K		Wind Loads W_K
	Case	Loads Considered	Adverse	Beneficial	Adverse	Beneficial	
	1	G_K, Q_K	1.35	1.00	1.50	0	-
	2	G_K, Q_K, W_K	1.35	1.00	1.35	0	1.35
	3	G_K, W_K	1.35	1.00	-	-	1.50

(Simplified Combination Rules With Only One Variable Action)

BS 8110			Dead Loads G_K		Live Loads Q_K		Wind Loads W_K
	Case	Loads Considered	Adverse	Beneficial	Adverse	Beneficial	
	1	G_K, Q_K	1.4	1.0	1.6	0	-
	2	G_K, Q_K, W_K	1.2	1.2	1.2	1.2	1.2
	3	G_K, W_K	1.4	1.0	-	-	1.4

ECOP 89			Dead Loads D		Live Loads L		Wind Loads W
	Case	Loads Considered	Adverse	Beneficial	Adverse	Beneficial	
	1	D, L *	1.4	0.9	1.6	0	-
	2	D, L, W	0.8 x 1.4	0.8 x 1.4	0.8 x 1.6	0.8 x 1.6	0.8 x 1.6
	3	D, W	1.4	0.9	-	-	1.3

* For cases when live loads does not exceed 0.75 the dead loads, the ultimate load U becomes, $U = 1.5\,(D + L)$

Table 3. Partial Safety Factors for Actions at The Ultimate Limit State According to Four Different Codes.

Table 2 gives examples of values of the ultimate loads (DL, LL) for the codes considered in this study, evaluated with respect to ultimate load of EC2. The last column gives the values of ultimate loads for the EC2 in kN/m^2. The following general observations could be made concerning cases considered: *i)* ACI gives higher values of ult. loads for floors and balconies of residential buildings, and values near the average for office floors, *ii)* BS code gives lower values of ultimate loads when compared with the other three codes. This may be due to the lower values of variable action intensities in this code, *iii)* The differences between ultimate loads in the four codes decrease, in general, with the increase of the value of DL.

4. Resistance of Reinforced Concrete Sections in Flexure and Axial Loads

Flexural Resistance: The ultimate moments of resistance for a singly reinforced sections are given in Table 4. Parameters considered are shown in the table. Concerning characteristic concrete cylinder strength, and also steel strength, it should be mentioned that the used values may not correspond to the specific grades of the codes considered. However, since our interest here is to compare ultimate moments of resistance according to provisions of different codes, the same material strength should be used. It should be mentioned also that most information in ECOP 89 are for concrete cube strength up to f_{cu} = 30 N/mm². For the sake of the comparative study, used values of concrete strength used in the study are assumed to be applicable. Table 4 presents the relative values of the ultimate moment of resistance with respect to EC2. The last column gives the values of M_u for EC2 in terms of bd^2 (units of *N,mm*). The following observations could be made:

f_{ck} N/mm^2	f_{yk} N/mm^2	ρ %	Values Relative to EC2 Code				EC2 Value
			ACI / EC2	EC2 / EC2	BS 8110 / EC2	ECOP 89 / EC2	bd^2 N/mm^2
25	360	0.5	1.05	1.0	1.00	1.00	1.48
25	360	1.0	1.06	1.0	1.00	1.00	2.78
25	360	1.5	1.08	1.0	1.00	1.00	3.92
25	500	0.5	1.05	1.0	1.00	1.00	2.01
25	500	1.0	1.08	1.0	1.00	1.00	3.68
25	500	2.0	1.14	1.0	1.00	1.00	6.03
40	360	0.5	1.04	1.0	1.00	1.00	1.51
40	360	1.0	1.05	1.0	1.00	1.00	2.91
40	360	2.0	1.07	1.0	1.00	1.00	5.40
40	500	0.5	1.05	1.0	1.00	1.00	2.07
40	500	1.0	1.06	1.0	1.00	1.00	3.93
40	500	2.0	1.09	1.0	1.00	1.00	7.03

Notes: 1 - The values shown in the last column should be multiplied by bd^2 in (mm) to obtain the ultimate moment of resistance of the sec. in (N.mm).
2 - Columns 4,5,6,7 give relative values with respect to EC2.
3 - The above values are derived for under reinforced sections ($\rho < \rho_{balanced}$).

Table 4. Comparison of Ultimate Moment of Resistance of Singly Reinforced Concrete Sections in Four Codes

i) The ultimate moments of resistance are observed to be 4% to 14% higher for the ACI than for the EC2, BS 8110, ECOP 89. This difference increases slightly with the increase of (ρ).
ii) The values of ultimate moment of resistance of singly under reinforced concrete sections, M_u, are the same for EC2, BS 8110, ECOP 89. This is because, for the cases considered, the three codes use the same equivalent concrete block, & the same material partial safety factors.

Axial Resistance: Table 5 presents a comparison of the ultimate axial strength of columns, P_u. The columns are considered to be short, effect of buckling neglected. For the design of axially loaded short columns according to EC2, the following quotation is taken from Ref. 4, pp. 247. " For EC2 code, to avoid the necessity of considering slenderness effects, limit the story height to least lateral dimension of the columns to 12. Allow for bending effects by increasing the axial load by 25 to 50 percent. Working in terms of axial load only, the design ultimate load capacity of section is: $N_{ud} = \alpha . f_{cd} . A_c + f_{yd} . A_s$, N_{ud} = ult. value of applied axial force, with: $\alpha = 0.85$, $f_{cd} = f_{ck}/1.5$, $f_{yd} = f_{yk}/1.15$ $N_{ud} = 0.57 . f_{ck} . A_c + 0.87 . f_{yk} . A_s$ "

f_{ck} N/mm²	f_{yk} N/mm²	ρ %	Values Relative to EC2 Code: ACI / EC2	EC2 / EC2	BS 8110 / EC2	ECOP 89 / EC2	EC2 Value $\frac{P_u}{bd}$ N/mm²
25	500	1.0	0.78	1.0	0.87	0.77	18.60
25	500	3.0	0.73	1.0	0.87	0.77	27.30
40	500	1.0	0.80	1.0	0.87	0.77	27.15
40	500	3.0	0.75	1.0	0.87	0.77	35.85

Notes: 1- The values shown in the last column should be multiplied by the cross section dim. in (mm) to obtain the ultimate strength of column (Newton).
2- Columns 4,5,6,7 give relative values with respect to EC2.

Table 5. Comparison of Ultimate Strength of Axially Loaded Short Columns in Four Codes

5. Comparison of Codes Considering both Actions and Resistance

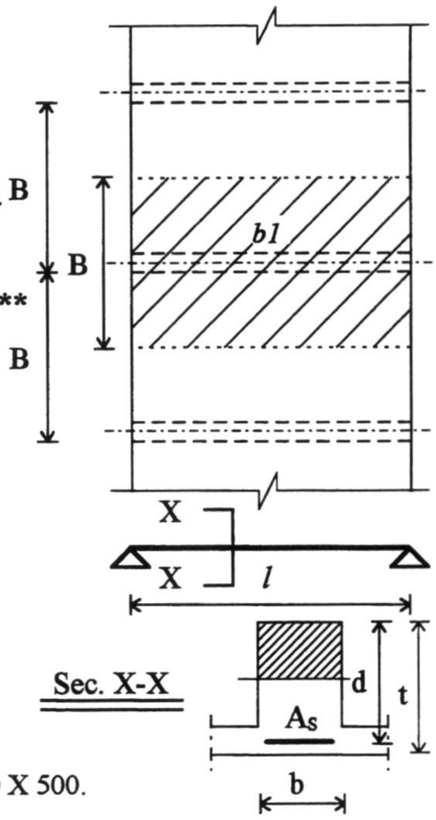
Section 3 presented a comparison between variable actions, and variable actions combined with permanent actions. Section 4 presented a comparison between ultimate resistance of concrete sections. These comparisons could be useful. They showed differences and similarities between codes. However, a better comparison between codes must involve both actions and resistance. For that purpose, three examples are given in the following.

Figure 1: shows the ultimate load effect and the ultimate section resistance of a singly reinforced beam. Data on dimensions & material properties are shown in figure. It is noted that beam dimensions of (t=450mm, b=200mm) satisfy the requirements of ACI, EC2, and BS codes, however they are unsafe for design using ECOP. This could be attributed ,partly, to the fact that the code uses a relatively higher partial safety factor for loads, equal to 1.5 for both DL & LL, when the value of the variable action does not exceed 0.75 the value of the DL.

Figure 2: The left four columns of Fig. 2 show the quantities of reinforcement needed for a singly reinforced beam, as computed according to the four codes considered in the study. Beam dimensions are t=450mm, b=200mm, f_{yk} = 500 N/mm², f_{ck} = 25N/mm².

Data For The Design Example:

- ⇨ Beam (*b1*) in a Residential Building floor.
- ⇨ Assume (*b1*) is a simple beam, $l > 2B$.
- ⇨ Assume (*b1*) is an inverted beam (rect. sec.).
- ⇨ Permanent Action = 7 kN/m^2 (from beam & slab).
- ⇨ $f_{ck} = 25\ N/mm^2$ ⇨ $f_{yk} = 500\ N/mm^2$ ⇨ $\rho = 1\ \%$
- ⇨ B = 2.7 *m* ⇨ $l = 5.5\ m$
- ⇨ b = 200 *mm* ⇨ t = 450 *mm* (except for ECOP)**



Summary of Results:

Code	Ult. Sec. Resistance *kN.m*	Ult. Action Effect *kN.m*
ACI 318-89	140.03	133.03
EC2	129.57	127.11
BS 8110	129.57	124.55
ECOP 89*	129.57	137.83
ECOP 89**	162.25	137.83

* Sec. does not satisfy ECOP. ** Increase sec. dim. to 200 X 500.

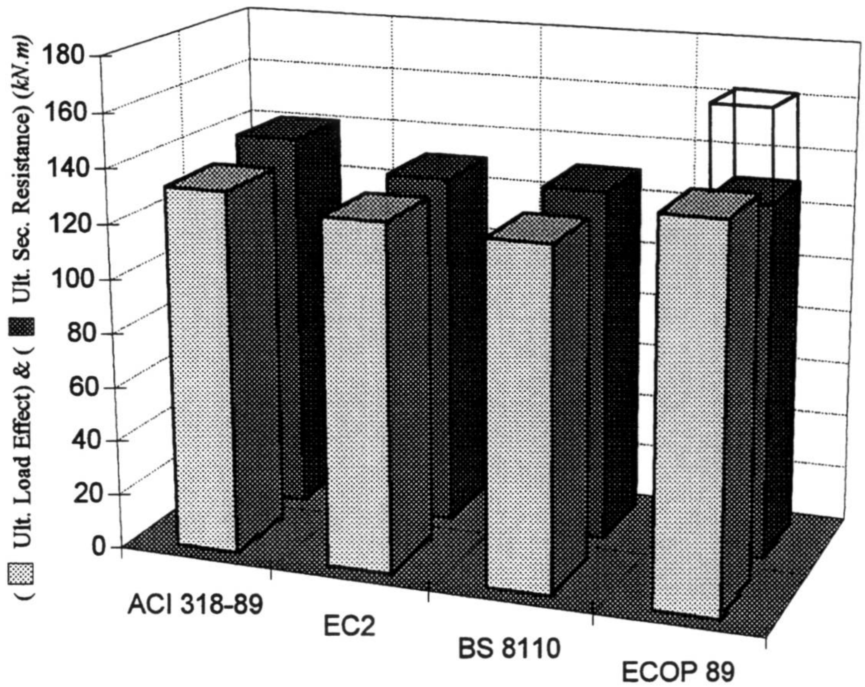


Fig. 1. Example to Show The Relation Between Ultimate Action Effect and Ultimate Section Resistance in Four Different International Codes.

Use	Materials N/mm^2	Permanent Load kN/m^2	ρ %	ACI / EC2	EC2 / EC2	BS 8110 / EC2	ECOP 89 / EC2
Residential (Floors)	$f_{ck} = 25$ $f_{yk} = 360$	3	0.5	1.004	1.0	0.936*	1.064**
		3	1.5	0.971	1.0	0.936	1.064
		7	0.5	0.997	1.0	0.980	1.084
		7	1.5	0.964	1.0	0.980	1.084
	$f_{ck} = 25$ $f_{yk} = 500$	3	0.5	0.998	1.0	0.936	1.064
		3	1.5	0.949	1.0	0.936	1.064
		7	0.5	0.991	1.0	0.980	1.084
		7	1.5	0.942	1.0	0.980	1.084
Offices (Floors)	$f_{ck} = 40$ $f_{yk} = 360$	3	0.5	0.928	1.0	0.959	0.965
		3	1.5	0.910	1.0	0.959	0.965
		7	0.5	0.953	1.0	0.989	1.022
		7	1.5	0.935	1.0	0.989	1.022
	$f_{ck} = 40$ $f_{yk} = 500$	3	0.5	0.925	1.0	0.959	0.959
		3	1.5	0.899	1.0	0.959	0.959
		7	0.5	0.950	1.0	0.989	1.022
		7	1.5	0.924	1.0	0.989	1.022

Note: *:it is assumed that* $f_{cu}\,(BS) = f_{cu}\,(ECOP) = 1.25\,f_{ck}\,(EC2) = 1.25\,f'_c\,(ACI)$

Examples: $* \; 0.936 = \dfrac{bd^2 \text{ (for BS 8110 Code)}}{bd^2 \text{ (for EC2 Code)}}$ $** \; 1.064 = \dfrac{bd^2 \text{ (for ECOP 89 Code)}}{bd^2 \text{ (for EC2 Code)}}$

The values of bd^2 for different codes are as follows:

ACI CODE : $bd^2 = \dfrac{(1.4D + 1.7L)\,Bl^2/8}{\phi\,\rho\,f_y\,(1 - 0.59\rho\,f_y/f'_c)}$

EC2 CODE : $bd^2 = \dfrac{(1.35G_K + 1.5Q_K)\,Bl^2/8}{0.87\,\rho\,f_{yk}\,(1 - 0.7787\rho\,f_{yk}/f_{ck})}$

BS 8110 CODE : $bd^2 = \dfrac{(1.4G_K + 1.6Q_K)\,Bl^2/8}{0.87\,\rho\,f_y\,(1 - 0.623\rho\,f_y/f_{cu})}$

ECOP 89 CODE :case 1, $L > 0.75D$, $bd^2 = \dfrac{(1.4D + 1.6L)\,Bl^2/8}{0.87\,\rho\,f_y\,(1 - 0.623\rho\,f_y/f_{cu})}$

case 2, $L \leq 0.75D$, $bd^2 = \dfrac{1.5\,(D + L)\,Bl^2/8}{0.87\,\rho\,f_y\,(1 - 0.623\rho\,f_y/f_{cu})}$

Table 6. Comparison of The Relative Values of (bd^2) for Singly Under Reinforced Concrete Sections according to Four Different Concrete Codes.

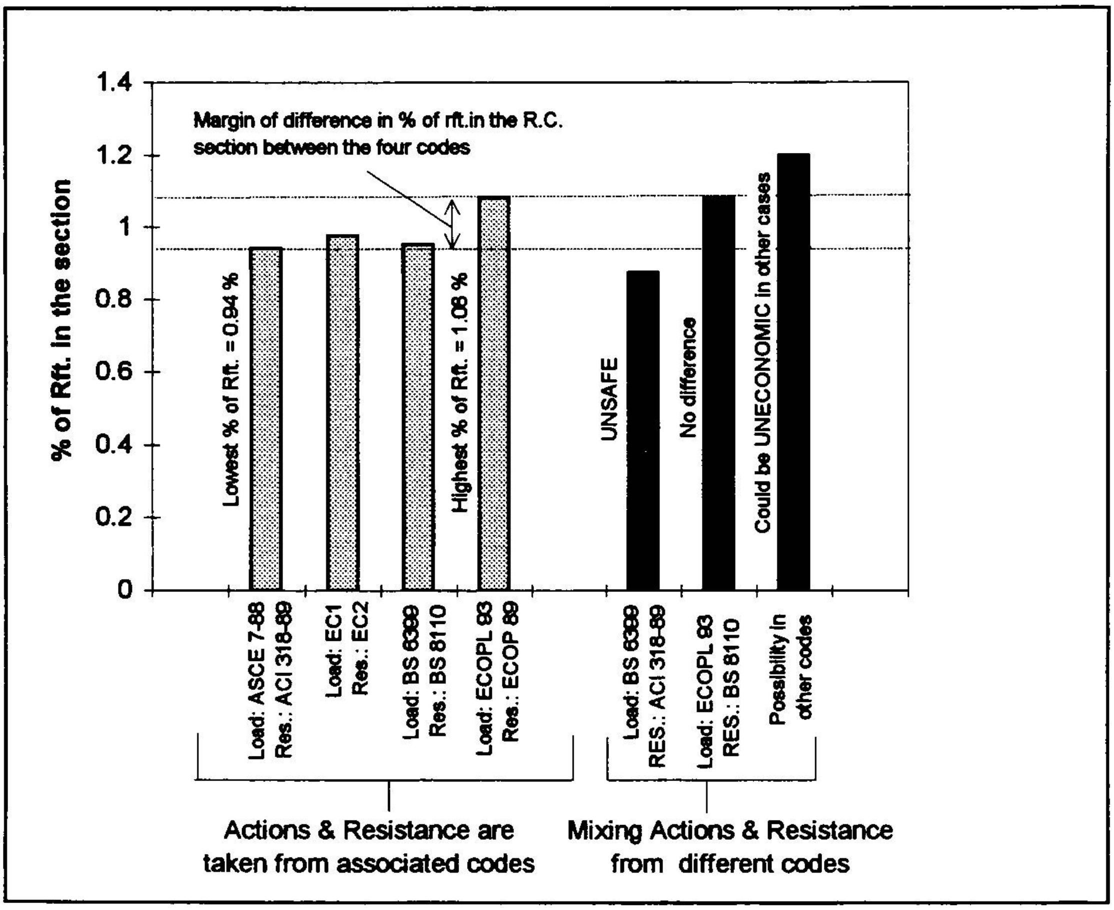


Fig. 2. Comparison of the Reinforcement Ratio [%] for a Singly Reinforced Beam Section in Flexure Calculated by Associated Codes & Mixed Codes

<u>Table 6:</u> In sec. 2, it was shown that evaluating bd^2 for different codes using Equ. (1), could be a measure of the economy of concrete structures designed according to these codes. Table 6 gives the relative values of (bd^2) of ACI 318-89, BS 8110, ECOP 89 with respect to EC2 for singly under reinforced concrete rectangular sections. Parameters considered are given in Table 6. It is noted that two intensities are considered for permanent actions to represent floors with different thickness, and also two types of building occupancy are considered. As an example concerning Table 6, (ECOP 89/EC2) = 1.064. This means that (bd^2 according to ECOP) = 1.064 (bd^2 according to EC2), i.e., for this case considered in Table 6, and considering the values of variable actions, the load factors, and the resistance models of the four codes, a concrete section designed according to Egyptian code requires slightly more materials than Eurocode, in the ratio 1.064 : 1.

6. Consequences of Mixing Design Codes

Mixing codes, i.e. using actions from one code and resistances from another code, is illegal. However, in some instances or in some regions which do not have their own codes or specifications, the practice of mixing codes is followed. Not only this is illegal, but it could be unsafe or uneconomic as shown in the last three columns of Fig. 2. For example, when using the ultimate loads from BS code and calculating the ultimate section resistance using ACI code, a lower steel reinforcement value is obtained. This structure could be unsafe.

7. Conclusions

Four concrete building design codes, and the corresponding codes for actions are considered. For the cases considered in the paper, the following conclusions can be made:
Actions: (1) Concerning variable actions, large differences in the variable actions intensities are observed in some cases, Table 1. (2) When variable actions are combined with permanent actions, the difference is still observed. However, the difference decreases with the increase of permanent action to variable action ratio, Table 2. (3) The ACI code gives higher values of ult. loads when compared with the other three codes. The EC code gives values of ult. loads near the average of the codes considered. The BS code gives lower values of ultimate loads when compared with the other three codes.
Resistances: (4) The ultimate moments of resistance are to some extent higher for the ACI than for the EC2, BS 8110, ECOP 89 codes. This difference increases slightly with the increase of steel content, Table 4. (5) The values of the ultimate moment of resistance of singly under reinforced concrete sections, M_u, are the same for EC2, BS 8110, ECOP 89.
Actions and resistances: (6) It is interesting to note that, in some cases, the ACI code gives higher ultimate action effects & higher ultimate section resistance than the EC2 & BS codes, however, it gives lower values of reinforcement, Fig. 2. (7) Beams designed by ACI and BS codes (Table 6) could be slightly more economic than those designed by EC2 and ECOP. (8) Using actions from one code & resistances from another code could lead to unsafe design.

8 Acknowledgment

The authors wish to express their sincere gratitude to Prof. Dr. Sabri Samaan, Professor and Former Head, Structural Eng. Department, Cairo University, for his comments on the paper.

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IABSE COLLOQUIUM

ISO OFFSHORE STRUCTURES STANDARD

BACKGROUND SCOPE AND CONTENT

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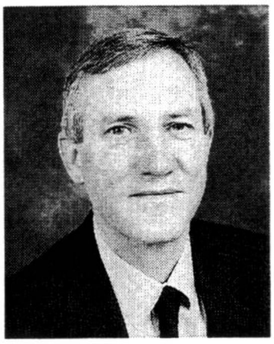

Richard Snell, born in 1947, obtained a degree in Civil Engineering from Leeds University. He is the Chairman of the ISO Offshore Structures Standard Committee and also of the Oil Industry Exploration and Production Forum Structures Sub-Committee.

1. Background to Standardisation in the Oil Industry

The Oil Industry has a long tradition of co-operation between Oil Companies on Engineering Standards. Although the Companies are competing on the acquisition of oil acreage and the efficient development of oil fields engineering integrity is recognised as a common interest and not an area of competition. A problem for one company has an impact on the reputation of the whole of the industry. This recognition also extends to the Contractors and Consultants supporting the Oil Companies who also participate in the preparation of Standards.

The Oil Industry first developed in the USA and a significant number of the principal Oil Companies and Contractors are US based. The American Petroleum Institute (API) has formed the central administrative organisation for Oil Industry Standards since 1923. Although the API is US based the principal non US companies are members and actively participate in the technical work. For a long time the API Standards have been the main Standards addressing the Industries specialised needs and are used worldwide. Where the API Standards do not address a routine Oil Industry topic other Standards have become de-facto Standards and are used by the Industry on a worldwide basis. One example of this is the Norwegian Standard NS 3473 for the design of Concrete Structures.

The first offshore structures were built less than 50 years ago. Initially they were in very shallow water and used pier type technology. In response to an Industry need to move into deeper water the API Offshore Structures Recommended Practice RP2A was developed and first issued in 1969. The pace of technology development and the effort that the Industry has put into this aspect of standardisation has led to 21 updates of this Practice in 25 years. A comparison of figures 1 and 2, 1970 and 1995 vintage structures respectively, shows the improvement that can be attributed to this updating of Industry practice.

Oilfield developments in industrially developed areas outside the USA, notably in N W Europe from the mid 1970's, led to the creation of regulations defining legal and technical requirements for some specific countries. The Oil Industry continued to design primarily to the API Standards with supplementary considerations for country requirements.

The initiative by the EEC European Standards Committee to produce a suite of European Standards and make them mandatory within the EEC provided the impetus for the Oil Industry to re-consider their approach to Standards and to conclude that in the long term a stand alone approach was not appropriate. The Industry, as its operations are worldwide, decided to put its future Standardisation effort into developing Standards under the administration of the International Standards Organisation and in 1989 devised under ISO Technical Committee 67 "Materials and Equipment for Petroleum and Natural Gas Industries" a structure of 7 sub-committees to address the principal technical areas within an oilfield. ISO TC 67 Sub-committee 7 was given a brief to prepare an ISO Offshore Structures Standard.

2. Scope of the ISO Offshore Structures Standard

The scope of work as defined by ISO is:-

Standardisation of:

Offshore Structures used in the Production and Storage of Petroleum and Natural Gas.

Procedures for the Assessment of the Site Specific Application of Mobile Offshore Drilling and Accommodation Units for the Petroleum and Natural Gas Industry.

The includes the sub-structure, station keeping and topsides structure for:-

- Fixed Steel and Concrete Platforms
- Semi-submersibles
- Ship type structures acting as stationary production or storage vessels.
- Tension Leg Platforms
- Spar Buoys
- Arctic Structures

- and the mooring assessment for site specific application of semi-submersibles and strength and stability assessment for jack-up drilling or accommodation units.

To date the Industry has concentrated most of its technical development effort on Fixed Structures of which some 7000 have been built around the world. Floating Production and Storage Structures are comparatively recent and not many have been built. Of those built to date most have been for short life fields in benign environments and have used converted ship or semi submersible hulls. Long life developments in deepwater and/or harsh environments require specific design considerations and conventional solutions may prove inappropriate.

As an example in a recent study BP has identified the technical, cost and schedule merits of concrete for hull construction.

In undertaking this task TC 67/SC 7 also has to:-

- Harmonise the API design provisions with the latest technology and with regional regulatory design criteria.
- Allow the use of a worldwide spread of national or regional standards for non-oil industry specific issues such as design of conventional components, materials, weld quality, corrosion etc.
- Allow variation in serviceability levels according to criticality as assessed by regional practice.
- Continue to provide the highly detailed primarily informative technical content normally in API Standards, considered essential to our Industry, within the essentially normative format of ISO Standards.
- Take into account existing ISO, IMO and Classification Society documents.
- Whilst making reference to other national and international design provisions for discrete components, which may vary in the reliability of the product, maintain and acceptable consistency in the reliability and serviceability of the overall structure.

It is an understatement to say that the breadth of this scope, the pace of technical development, the complexity that internationalisation of reference standards and hence the variability that it creates and the number of areas where new technology has to be developed presents a challenge.

3. Content of the Standard

The content of the ISO Offshore Structures Standard will address issues specific to the Oil Industry and make reference to other Standards for aspects not unique to our Industry and adequately addressed elsewhere. In general topics such as the design of concrete and conventional steel sections, materials, testing, fabrication and corrosion are handled by reference.

Topics addressed in detail in the Offshore |Structures Standard are:-

- Derivation of oceanographic design criteria.
- Marine soils investigation.
- Procedure for derivation of site seismicity and seismic design.
- Procedure for calculation of environmental loads.
- Extreme, persistent, transient, accidental and serviceability design limit states.
- Loads and load combinations.
- Load Factors and Resistance Factors for the design of substructure components.
- Performance service levels.
- Load Factors and loads for topsides design.
- Strength equations and material resistance factors for large diameter tubular structures.
- Large diameter pile design provisions.

- Guidance on analysis and modelling.
- Materials and fabrication requirements.
- Design procedures for hydrocarbon fire and blast events.
- Whole life cost issues.
- Re-assessment of existing structures.
- Hull requirements in addition to Class Rules for Production or Storage duty.
- Design procedures for Tension Leg Platforms.
- Design procedures for a spread mooring system including system considerations and integration with marine risers.
- Jack-Up platform strength and stability assessment procedures.
- Weight Control.
- Good practice experience that contributes to safe operation.

4. Format of the Standard

The format of the Standard, recognising the difficulty of using a 1000+ page document, has been planned the meet the likely needs of designers and the requirement to allow for future updates of discrete sections without re-printing the whole document.

Part 1 General Requirements
General Annexes to Part 1 on Metocean, Foundation, Seismic and Topsides
Regional Annexes to Part 1 identifying regional provisions and regulations

Part 2 Fixed Steel Structures
Informative Annex

Part 3 Fixed Concrete Structures
Informative Annex

Part 4 Floating Structures
Sub-parts for each structural form, moorings and risers.

Part 5 Site Specific Application of MODU's
Sub-parts for Semi-submersibles and jack-ups.

Figures 3 and 4 give an indication of the document format.

5. Programme

Part 1 is complete and should be published as ISO Standard 13819 in late 1995. It is intended that Parts 2 and 3 be completed by ISO TC 67/SC 7 and forwarded to ISO for ballot by the end of 1997. Part 4 should reach this stage by the end of 1998.

6. Principal Design Considerations for an Offshore Structure

The driving criteria for the in place design of a fixed offshore structure are the water depth, the wind/wave/current loading and the configuration and mass of the topsides. The primary requirement is adequate strength and durability. For a floating structure motions, stability, buoyancy and station keeping are also fundamental requirements.

6.1 Environmental Actions

For most structures the designer has to determine the design environmental events from first principles. This requires ownership or access to data which can be used to generate long term extreme criteria in terms of wave height, period, crest elevation, wind speed, and the tidal, surge, surface shear etc components of current. All these have seasonal and directional variation. Individual extremes for each element are combined to predict directional design events, normally with a return period of 100 years.

The classical method of deriving the extreme design load is to predict the environment and then derive loads using a well proven and calibrated procedure based on the Morrison equation. As structures move into deeper water and become more dynamically sensitive the more sophisticated designers are calculating structure response to a population of severe storms, extrapolating to a 100 year response and then deriving an associated set of environmental inputs for detailed component design. The environmental load factor varies on a regional basis depending on the environmental conditions. The variability associated with a tropical storm (hurricane) environment is greater than that associated with extra tropical areas such as N W Europe. Typically a load factor of 1.35 is used on a 100 year return period event. The associated material resistance factor is 0.85.

For a floating structure the extreme event may not be associated with the most extreme conditions. Situations where the current is acting in a different direction to the waves may well provide the most extreme mooring forces. Hull forces on a monhull are driven at the margins more by wave steepness than extreme wave height. A semi-submersible is often most vulnerable to quartering seas when the wave action is trying to alternatively part then close the hulls. Design predictions are normally made using model testing to confirm and calibrate analyses. The limit state design procedure is stretched when addressing highly compliant structures. Applying a load factor other than 1.0 to heave, pitch, roll, surge, sway and yaw motions is meaningless.

6.2 Transient Actions

Whilst the overall design of and offshore installation is governed by global loads many of the individual components are likely to be governed by transient actions. The actions arising from lifting or skidding a large structure onto a transport barge, transit to site and launch or lift into place are structure specific and whilst general load factors will be provided it is acknowledged that they are subject to individual Project situations.

Typically the variance in potential loads as a structure is skidded along beams and onto a transport barge will depend on:-

- Vertical stiffness of piles supporting the launch beams.
- Barge stiffness.
- Structure stiffness.
- Barge ballast system control.

In transit the variance in loads is a function of the roll and pitch motions of the barge. These can be controlled within reason by accepting weather limitations on the tow but roll angles of 15 degrees are generally considered for harsh condition tows.

The variance in potential loads in the slings and lifting structure during a heavy offshore lift is governed by crane barge control. There are essentially three very large masses. A typical scenario would be a 120,000T crane barge, a 30,000T transit barge with a 10,000T module to be lifted from it. For defined control capabilities load factors for the slings and lifting structure have been developed.

6.3 Accidental Actions

Typical accidental actions would comprise impact by a ship, hydrocarbon fire and/or blast, dropping of a heavy object and helicopter heavy landing.

For vessel impact the structure would be expected to withstand the impact energy associated with low speed manoeuvring of a supply vessel. It is not feasible to design to withstand a large commercial vessel at normal transit speed thus collision management measures are used as the primary means of defence. A load factor of 1 is used on energy absorbed.

Hydrocarbon incidents are addressed by a risk based design approach. The objective being to limit the risk of fatality of any individual working offshore. Normally this translates into limiting the spread of damage and ensuring personnel can be evacuated. It is reasonably practicable to design to withstand overpressures of 1.5 bar and fires of up to 1200 degrees centigrade for a limited duration. In so doing the limit states are related to deflections which might cause supplementary hydrocarbon releases rather than load.

6.4 Serviceability

To satisfy operational requirements a number of serviceability limits in terms of deflection of plant support structure have to be provided. These are often expensive if obtained by stiffness of the primary structure and may be achieved by local substructures for discrete plant items.

The serviceability of the overall structure is primarily an issue of fatigue and ductility requirements. A typical N W Europe structure would see around 5 million wave cycles per annum and have a life of 20 years. Specific design requirements for an offshore structure are provide in the Standard but the underlying technology is not unique to our Industry. Where we are potentially setting a first is in the harmonisation of fatigue curves and material performance requirements on a worldwide basis. Serviceability for fatigue is normally expressed in multiples of the design life with differing requirements according to in service inspectability. The fatigue durability of highly loaded tubular joint fabrications are sensitive to fabrication tolerances and weld quality thus the Standard gives very specific guidance on these aspects.

In addition to material considerations ductility of the whole structure as a system is normally calculated by a non linear "push over" analysis where the design load is factored up in successive analyses with failed components removed in a realistic manner to determine the Reserve Strength. This approach is particularly relevant when considering an innovative geometry and when re-assessing an existing structure or one that has suffered some damage.

7. Summary

The Oil Industry is preparing an ISO Offshore Structures Standard incorporating the knowledge held within the Industry. It is a substantial task in that it will have a large number of structural forms and controlling criteria with regional variations for worldwide application. The Standard references other existing Codes for all no Oil Industry specific issues.

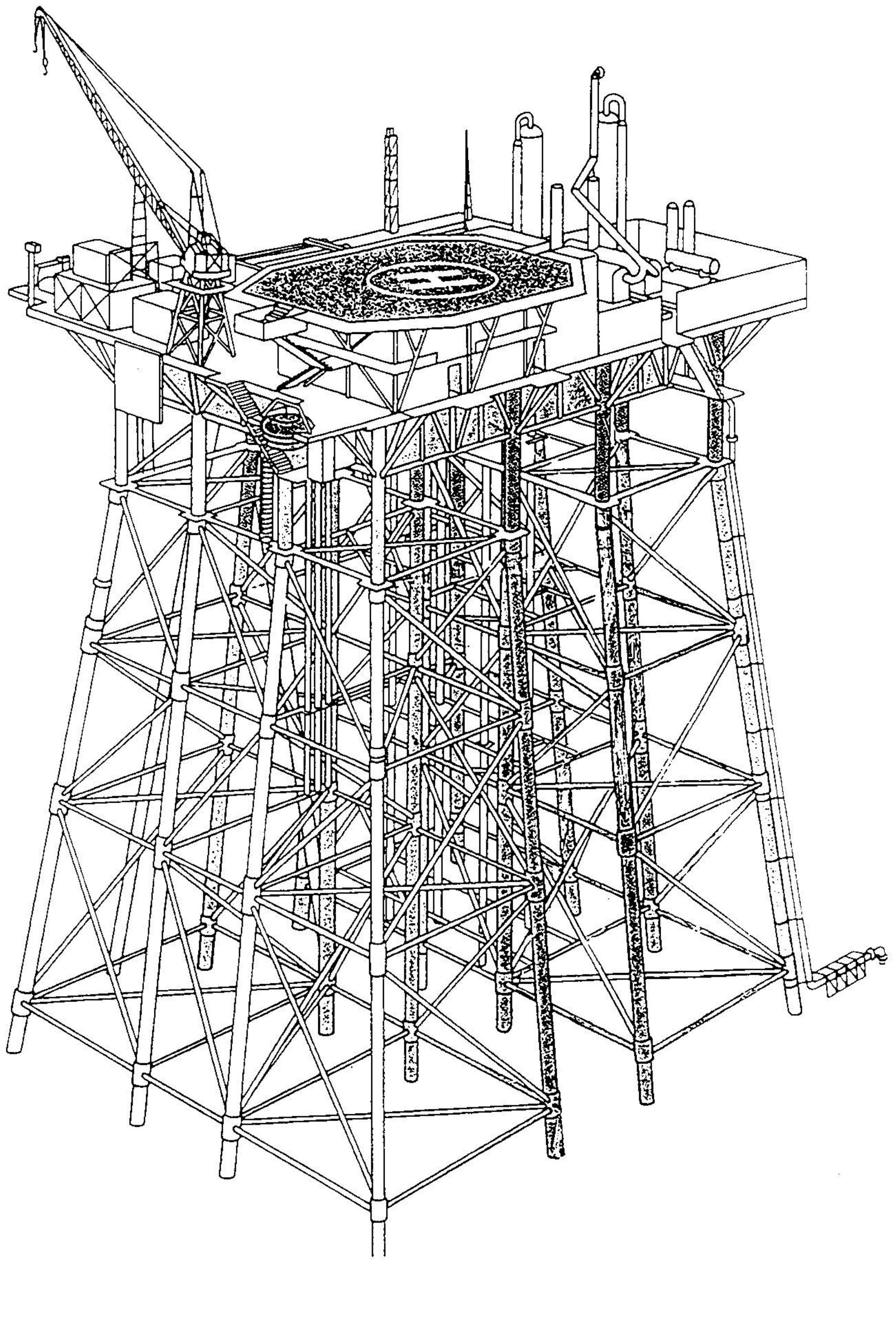

FIGURE 1

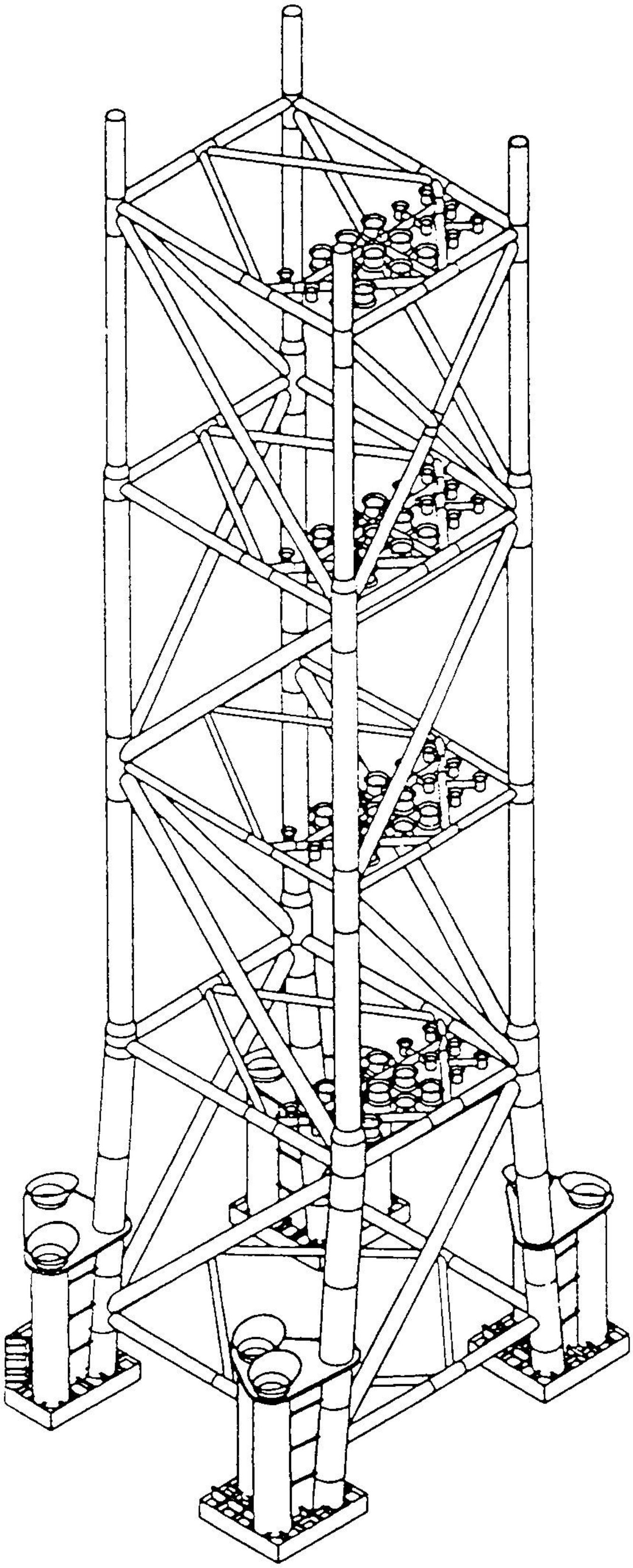

FIGURE 2

ISO Standard for Offshore Structures

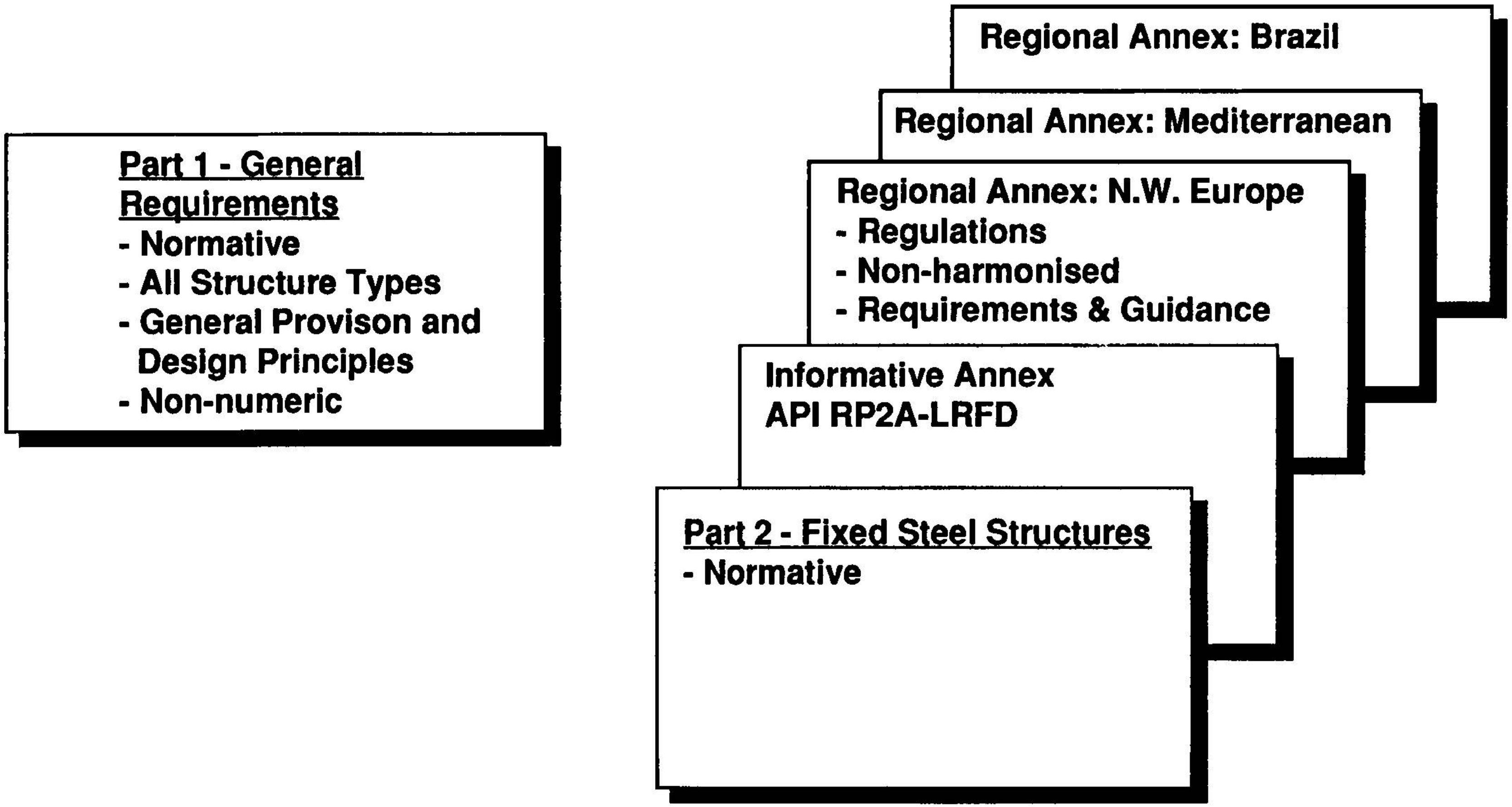

FIGURE 3

ISO Standard for Offshore Structures

Regional Annex

Informative Annex
to Part 2

Part 2

FIXED STEEL STRUCTURES

- Normative
- Whole Life-Cycle Design
- Harmonised Technical Requirements
- Loads
- Resistances Foundation
- Substructure
- Topsides Structure

Regional Annex

Informative Annex
to Part 3

Part 3

FIXED CONCRETE STRUCTURES

- Normative
- Whole Life-Cycle Design
- Harmonised Technical Requirements
- Loads
- Resistances Foundaton
- Substructure
- Topsides Structure

Regional Annex

Informative Annex
to Part 4

Part 4

FLOATING STRUCTURES

- Normative
- Whole Life-Cycle Design

Regional Annex

Informative Annexes

Further Parts

- Arctic Structures
- MODU Site-specific assessments

FIGURE 4

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Comparison with Russian Code

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Vladimir Raizer, born 1936 get his civil engineering degree in 1958 in Moscow State Civil Engineering University,his candidat of sci degree in 1962, doctorial degree in 1971 in TSNIISK member of JASS, JSO TK/98,published 5 books, more then 100 articles in structural mechanics and theory of reliability.

Summary

The aim of this manuscript consists of analysis of Eurucode 1,Part 1 Basis of Design (ENV1991,Part 1) from the position of russian engineering expirience and comparison with Russian Building Code (GOST 27751-88) "Building Structures and Foundation Bases.General Principles of Design".The observations are given for all sections of ENV1.

1.Section 1.General remarks.

This ENV 1 was elaborated on the base of ISO ST 2394 and one saw no difference in principle between them. But ENV1 is intended for practical use.It is important to pay attention that in former USSR the General Building Code based upon limit state design method in the form of partial factors was in action since 1955.Some years later it was performed in SMEA standart (SMEA ST-384-76) and was used in former SMEA countries.
One can note that as in ENV1 and as in GOST there is formulation lacking in precision for general conception of codifing procedures.The rules for requirements formulation to structures are ruther wordy.It is prescribed that a structure shall be designed in a such a way that it will during intended life with appropriate degrees of reliability to fulfill its function.For realisation of this prescribtion the method of partial factors is proposed.In this method a verification of limit state condition is caring into practice in one only point of state space.In this point all initial values will take design values.
So that a design with appropriate degrees of reliability is now only unattainable wish though in the main text and in the annexes there are so many general reasonings.It was proposed that a choice of partial factors values can be maked in such a manner that satisfaction of design enquality will quarantee a necessary reliability level.
Bat this assumption remains as hypothesis only.
It was said in Annex A that most of partial factors in ENV1 have been received on the base of constructional expirience and all necessary calibration procedures in the main text are absent.

In 1994-1995 in the process of elaborating of new version of Russian Building Code it was stated that partial factors method can't guarantee a design with given level of reliability.
GOST and ENV1- the designing requirements determined and precribed rules of analysis.In the same time requirements to the results of constructional work are almost absent.
But as a building structure is erecting for guarantee of function for technological process therefore a main requirement for structure must be consumer's requirements.
It was stated in Russia in 1994 the conception of consumer's requirements codification as a next step in new version of GOST.

2.Section 2 Requirements.

There are some problems with design working life.There are not such indications in GOST.
The procedures of partial factor method does not include time as design variable value and so that analysis of structures is fulfilling without considering working life.It is useful to note also that a working life is not defining by the class of structures as in ENVI but with technological process and will be determined by custumer.

3.Section 3 Limit states.

It is possible to state that the definition of design code making procedures are done more legible in ENV1 then in GOST.In GOST this procedure called "Limit state design method" that differed from "Partial factor method" in ENV1.But a conception of limit states then there is precise border between safe and unsafe regions can be consider in ENV1 as a general base for development of probabalistic method.

4.Section 4.Action and enviromental influences.

Introduction of two characteristic values-upper and lower for permanent actions in ENV1 gains an advantage over GOST.According to the GOST two design values can be determinating for these actions.The lower values are introducing then their unloaded influence is taken into account.

5.Section 5 Material properties.

Two characteristic values for properties introduction in ENV1 seems to be unnecessary.
There is no practical use in upper values.If it is necessary two design values can be introduced,for soil for example.

6.Section 6 Geometrical data.

This section consists no necessary information.In GOST it is absent.

7.Section 7.Modelling for structural analysis and resistance.

For this section one remark can be written.The point 7.2(2) affirmes that it is necessary to take into consideration the influence of deformation scheme of structure if it result in an increase of the load effect by more them 10%.But for numerical estimation of this increasing one must analyse the structure taking into account deformation scheme .

8.Section 8.Design assisted by testing.

It is very important to include this section into main document.There is not such section in GOST.

9.Section 9 Verification by the partial factor method.

This section is basic in ENV 1.In the partial factor method,it is verified that,in all relevant design situations,the limit states are not exceeded when design values for actions,material properties and geometrical data are used in the design models".The same deterministic approach is used in GOST.This approach does'nt allow to design a structure with given level of reliability.It is impossible to find a correspondence between numerical values of partial factors and level of reliability.From this condition it seems that there are too many partial factors in ENV1.It is impossible to combine incompatibility - to create the probabalistic baye under deterministic partial factor method.

10.Annexes.

There are four Annexes in ENV 1.All of them are informative and very useful for code making procedures Annex A present an approach for probabalistic base of partial factors method.but proposed procedures are possible for structure which consist of one element under one.But there is no answer about multyelements structures and load combination factors.

11.Conclusion.

Analysis of ENV1 shows that its theoretical base and code making procedures corresponds to GOST but in these both documents one can't find the result from reliability theorie and new approach in code making procedures based upon consumer requirements which have been proposed by EEC UN.But this problem is for new generation of building codes.

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