JOURNAL OF THEORETICAL
AND APPLIED MECHANICS
2, **38**, 2000

DESIGN TEMPLATES – A NEW TOOL FOR COMPUTER-AIDED DESIGN OF STRUCTURAL ELEMENTS

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A design template is defined as a scheme of computer-aided design process which could be verified and modified in particular cases by a designer. Global loads have to be determined and analysis is performed for each arrangement of unit loads separately. Thus, the influence matrices are determined and also: matrix of interaction of load effects and matrix of combination of simultaneous loads. New matrix procedures are introduced in order to identify the most unfavourable load arrangements and then – the most unfavourable load combinations and load effect interactions. The design templates are exemplified for portal steel frames subject to permanent loads, variable actions and second-order horizontal forces due to sway of the columns. Complex influence coefficients are introduced and amplified sway of the frame is taken into consideration. Corrections of some unsound clauses of the Eurocodes 1 and 3 are suggested.

Key words: computer-aided design, load combination, load effect interaction, steel frames, sway amplification

1. General remarks

New methodology of structural design has been elaborated for last three years (Murzewski, 1997, 1998, 1999). It consists in application of so called design templates, which are different from computer programs. The difference is that the design procedures can be easily verified and modified by the designer himself. Computer programs are useful in structural analysis but they can hardly help in design of structural elements. Standard specifications change too frequently and the program upgrading cannot follow in due time. Furthermore, international cooperation and parallel validity of national and European draft standards in building involve different design rules. The Polish certification procedures require formal verification by the authorised units. That is why the design algorithms have to be up-dated every time and revealed to verification.

The new methodology of design takes advantage of the well-known mathematical programs; like, Mathcad, Mathematica etc. Matrix calculus enables one to consider thousands of load cases in the design of one structural element. This has not been feasible in conventional calculations when designer's intuition not always could indicate the most dangerous load case. The logical value "1" (true) or "0" (false) at the end of the template indicates whether the structural element is safe or unsafe. An additional line or column of numerical results of the modules may by useful if errors in data or equations are suspected.

The following three modules create a design template for the structural elements like cross-sections, members, joints:

Module 1 – load specification, action arrangements, their influence and global analysis of the structural system

Module 2 – resistance of structural elements and interaction problems between the internal moments and forces

Module 3 – identification of extreme effects for load variants, interactions, combinations and imperfections.

Every module has three blocks:

- Block of constants
- Block of variables
- Design algorithm.

All data necessary to solve a problem should be arranged as it is shown in Table 1.

Table 1

	Module 1 Structural analysis	Module 2 Resistance characteristics	Module 3 Reliability verification
Block of constants (independent of results)	Dimensions: **L**, **a** of the structural system and its elements. Variable actions $F_i = Q, Sn, W, \dots$ in two or more variants $v_i = 0, 1, \dots$	Material properties $f, E, \dots$ (steel, concrete etc.) and γ_M. Stiffener spacing l_j, a_j for buckling modes $j = m, n, v, \dots$	Design working time t_d. Reliability differentiation γ_n. Variable action factors γ_i. Frame imperfection ϕ_o.
Block of variables (dependent on design results)	Permanent load $F_0 = G$ from Mod.2 and its load factors $\gamma_{sup}, \gamma_{inf}$. Optional architectural details. Stiffness $EA, EI, \dots$[1] Residual moments and forces[2]	Cross-section dimensions $h, b, \dots$ and its properties $A, S_y, I, \dots$ Eccentricity $e_n = M_S/N_S$ from Module 3. Class of the cross-section. Imperfection factors σ_j.	The amplified sway ϕ[3] Identification of extreme: – variants v_i for each F_i, – load combinations c, – load effect interactions e.
Design algorithm (formulae and results)	Vector of global forces $\boldsymbol{F}$. Equations of equilibrium and compatibility for unit loads. Influence matrices $\{c_{jv}\}_i$ for: – internal forces $M_S, N_S, V_S, \dots$ – deformations $\Delta, \phi, \delta, \dots$	Instability factors χ_m, χ_n, χ_v Reference resistance M_R or N_R Cores r_j and inverse cores r'_j. Interaction matrices (**rs**): – (**rm**) for the cross-section, – (**rn**) for the structural member.	The exact eccentricity e_n. Safety of design $S_{eq} \leqslant R = 1$ and economy checks $S'_{eq} \leqslant R = 0$ for smaller cross-sections. Deformation and/or crack[4] checking $\phi \leqslant 1/C = ?$ etc.

1) if the temperature effects are taken into consideration
2) if the elastic bending moments are modified according to EC3/5.2.1.3
3) mandatory for sway frames
4) not applicable for steel structures.

An example of simple design template for portal frames (Fig.1) is presented in next sections. There are:

- load arrangements v_i and influence coefficients $\{c_{jv}\}_i$ for normed loads (Module 1)
- steel I or H cross-section characteristics, buckling factors χ_j and M_S-N_S-V_S interactions (Module 2)
- most unfavourable variants, interactions and combinations of load effects selected for reliability checks (Module 3).

A semi-probabilistic design method of partial factors is applied according to European prestandards EC1 (acronym for Eurocode 1, 1993) and EC3 (acronym for Eurocode 3, 1992). A black triangle ▶ will mark the Author's critical remarks and suggested corrections of the Eurocodes.

2. Module 1 – structural analysis

Every independent action F_i is characterised by:

- Absolute value of the global force F_i [kN], $i = 0, 1, 2, ...$ which is equal to the sum of all vertical forces for the first load arrangement, but in the case of wind to the sum of horizontal forces
- Arrangement of real distributed and concentrated forces with weight factors $\boldsymbol{F}/F_i$ defined in several variants $v = 0, 1, 2, ...$ $\boldsymbol{F}/F_i$ are normed so that the sum of the forces is 100% in the first arrangement
- Equivalent forces $H_i = P_i\phi$ proportional to the relative vertical components P_i of the loads F_i but horizontal. An amplified sway ϕ shall be taken into global analysis of the frame.

2.1. Load arrangements

Specific weights [kN/m^3] of structural materials are inserted to the Module 1 block of constants. There are also lengths and other geometrical quantities which are necessary to determine the dead load $F_0 = G$. The unit values of variable loads F_i [kN/m^2], $i \geqslant 1$, are taken from standards.

The cross-section area A [m^2] is necessary for the self-weight evaluation of the structural member while the elastic stiffness EI [kNm^2] is required for the temperature effect analysis. The values will be included to the block of variables of Module 1 from the Module 2 by means of the clipboard. Their trial values will be improved in an iterative process of calculations.

Table 2 presents the normed load arrangements for an exemplary portal frame (Fig.1). Separate structural analyses must be performed for each variable load arrangement v_i in the third block; however, a single structural analysis is sufficient to evaluate the effects of permanent load for the characteristic value G, and proportional upper design value (G with the load factor γ_{sup}) and lower design value (G with load factor γ_{inf}).

Table 2

Normed load arrangements	$v = 0$	$v = 1$	$v = 2$
$i = 0$ Dead load	characteristic value $G = 1$	upper design value $G = \gamma_{sup}$	lower design value $G = \gamma_{inf}$
$i = 1$ Live load (e.g. cranes)	absent 0	the crane to the left $Q = 1$	the crane to the right $Q = 1$
$i = 2$ Snow	not at all 0	more snow on the left side of the roof $Sn = 1$	more snow on the right side of the roof $Sn = 1$
$i = 3$ Temperature	temperature of erection 0	in summer $T > 0$	in winter $T < 0$
$i = 4$ Wind	calm 0	from the left $W = 1$	from the right $W = 1$

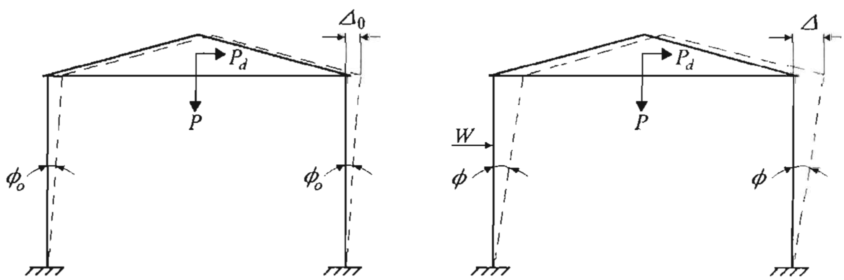

Fig. 1. Frame with the equivalent imperfection ϕ_o and amplified sway ϕ

2.2. Global analysis

Within the framework of the second order theory the normed loads are applied to the perfect elastic frame without out-of-plumb of columns. First, the analysis of elastic properties is carried out for the real loads $\boldsymbol{F}/F_i$ then it is done for the horizontal second order forces $\boldsymbol{H}/F_i$. Explicit formulae for portal frames and other simple structural systems may be found in design manuals. They must be copied to the template. The same template may be used in other design projects and new formulae will replace the old ones in the computer memory. If a redundant structural system cannot be solved by means of explicit formulae, special computer programs may be applied but the results have to be inserted in a matrix form to the environment of the design template. The vector of global forces F_i will result from calculations of Module 1 as well as the nested influence array $\{c_{vj}\}_i$ which transforms the applied loads F_i into load effects S_j. Elements of the nested matrix **c** of influence coefficients $\{c_{jv}\}_i$ are not scalar but they are represented by 2D matrices $\{c_{jv}\}$ for each action F_i.

2.3. Complex influence matrices

Nested (three-dimensional) arrays **a** and **b** are the components of nested influence array **c**. It has a complex form because the amplified sway ϕ is not known a priori. Finally, the elements of matrices **a** and **b** with the same subscripts will be added but now they are treated separately

$$\{c_{jv}\}_i = \{a_{jv}\}_i + \{b_{jv}\}_i\,\phi \tag{2.1}$$

where

- $\{a_{vj}\}_i$ – nested array which changes the normed loads $\boldsymbol{F}_i/F_i$ into their effects
- $\{b_{vj}\}_i$ – nested array which changes the second order forces $\boldsymbol{H}_i/F_i$ into their effects.

The following notation has been introduced (Table 2, Fig.1):

- i takes the values $0, 1, 2, 3, 4$ for independent loads; i.e. $F_0 = G$ (time-constant load), $F_1 = Q$ (imposed load), $F_2 = Sn$ (snow), $F_3 = T$ (temperature), $F_4 = W$ (wind action)
- v takes the values $0, 1, 2$ for load arrangements (variants): the characteristic, upper and lower design values of G; but $v_i = 0$ if the variable action is absent and $v_i = 1$ or 2 if two positive variants are actual

- j takes the values $0, 1, 2$ for load effects; in another notation: $j = m$ for bending moment M_S, $j = n$ for axial force N_S, $j = v$ for shear force V_S, $j = \phi$ for column sway, $j = \delta$ for beam deflection.

The Eurocode 3 and the Polish design standard PN-90/B-03200 recommend iterative procedure to amplify the initial imperfection ϕ_o of frames which are classified as the sway frames. The classification of frames is cumbersome. That is why we better treat each frame as a sway frame in the computer-aided analysis. The amplified sway ϕ will be determined exactly in Module 3 for selected load arrangements, load combinations and load effect interactions. The iterative procedure may be faster if a preliminary value $\phi > \phi_o$ is taken at the starting point.

► It has been shown (Murzewski, 1992) that neither iteration of sway values ϕ nor approximate amplification factors (EC3/5.2.6) would be necessary if the exact stiffness EI of the columns (from Module 2) and combination values of loads $(\psi\gamma F)_{ic}$ (from Module 3) were known in advance. A new explicit formula has been derived

$$\phi = \frac{EI\phi_o \operatorname{sgn}\phi_o + \sum a_{\phi_i}(\psi\gamma F)_i}{EI - \sum b_{\phi_i}(\psi\gamma F)_i} \tag{2.2}$$

where the matrix $(\boldsymbol{\psi\gamma}\mathsf{F})$ is reduced to a vector $(\boldsymbol{\psi\gamma F})$ for the selected load combination $c = 0, 1, 2, ...$ Both influence matrices (the first order a_ϕ and the second order one b_ϕ) are reduced to vectors $\boldsymbol{a}_{\phi_i}$ and $\boldsymbol{b}_{\phi_i}$, respectively, for selected load arrangements v_i.

3. Module 2 – resistance characteristics

The characteristic values of yield strength $f_k = f_y$ with the partial safety factor $\gamma_M = 1.1$ and ultimate tensile strength f_u with $\gamma_{M2} = 1.25$ are inserted into the block of constants. The free lengths l_y, l_z and spacings a_j of stiffeners necessary for stability verification are also there. A trial value of eccentricity e_n is inserted to the block of variables. There are also the cross-section dimensions, e.g. h, b, t_w, t_f, r_w (Fig.2) in the case of a HEB section. Algorithms for evaluation of the resistance R_j in simple design cases and interaction relations between the load effect components $S_j = M_S$, N_S, V_S are given in the third block.

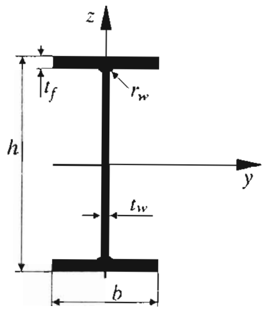

Fig. 2. I cross-section of a steel member

3.1. Equivalent load effects

If the bending moment M_S occurs simultaneously with the axial force N_S and/or the shear force V_S, the ultimate limit state of cross-section will be reached earlier than in the case of a single M_S action. A new concept of equivalent load effect S_{eq} will help to check the ultimate limit state of structural element in the case of interacting moment and forces

$$S_{eq} \leqslant R \tag{3.1}$$

where

- R – reference resistance; let it be the bending resistance $R = M_R$ for cross-section design and compression resistance $R = N_R$ for member design
- S_{eq} – equivalent load effect; bending moment $S_{eq} = M_{eq}$ for cross-section design and equivalent compression force $S_{eq} = N_{eq}$ for member design.

Both structural elements: cross sections and members are treated separately. The difference is that the buckling factors χ_j and an equivalent uniform moment factor μ are introduced to member design.

▶ If the equivalent load effects M_{eq} and N_{eq} are linear functions of loads F_i, matrix calculus may be applied and interaction matrices may be defined. They help to transform vector $\boldsymbol{F}$ into a scalar S_{eq}. Another design philosophy is presented in Eurocodes and most national standards. The reduced resistance moments are defined: M_{N_R} – for simultaneous action of bending moment and axial force, M_{V_R} – in the case of moment and shear force interaction. They will not be used any more.

3.2. Cross-section design

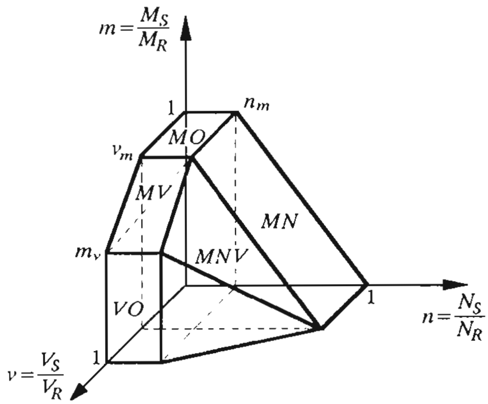

Fig. 3. Piece-wise linearised interaction surface

The standard interaction formulae MN, MV and NV may be represented in a non-dimensional coordinate system (Fig.3)

$$m = \frac{M_S}{M_R} \qquad n = \frac{N_S}{N_R} \qquad v = \frac{V_S}{V_R} \tag{3.2}$$

where the resistance values M_R, N_R, V_R are defined for simple design cases as follows

$$M_R = \frac{W_y f_k}{\gamma_M} \qquad N_R = \frac{A_n f_k}{\gamma_M} \qquad V_R = \frac{A_v f_k}{\sqrt{3}\gamma_M} \tag{3.3}$$

where $W_y = W_{pl}$ for the cross-sections of Class 1 and Class 2 or $W_y = W_{el}$ for the cross-section of Class 3 (EC3/5.4.5), the cross-sections of Class 4 are not taken into consideration in this paper; $A_N = A$ if $N_S > 0$ (compression) or $A_N = \min(A, A_{net} f_u/\gamma_{M2})$ if $N_S < 0$ (tension) (EC3/5.4.3&4); $A_V \approx ht_w$ – shear area ≈ full section of the web for any class of the cross-section (EC3/5.4.6). Subscript d denoting the design value is not added to the main symbol in this paper.

The Eurocode 3 allows one to apply plastic global analysis only to the structures the members of which have cross-sections of Class 1 although it recommends the plastic resistance moment W_{pl} also for the cross-section of Class 1 and Class 2. A little more conservative rule has been accepted by the Polish design standard. Both standards admit elastic global analysis in all cases.

► The Author does not object to the apparently contradictory elastic global analysis and plastic resistance moduli in bending of cross-sections of Class 1. He understands that the residual stresses and stiff cross-sections allow for reaching the upper yield limit $f_{yu} > f_y$ before full redistribution of bending moments at the ultimate limit state occurs. However, the author would advise to take different shear areas: $A_{V_{pl}} = A_V$ for Class 1 or Class 2 and another value $A_{V_{el}} = It_w/S_{I/2}$ for the cross-sections of Class 3, respectively. The proportion $A_{V_{el}}/A_{V_{pl}}$ will be exactly the same as it is in the case of bending section moduli W_{el} and W_{pl}

$$\frac{A_{V_{el}}}{A_{V_{pl}}} \approx \frac{It_w}{S_{I/2}ht_w} = \frac{W_{el}}{W_{pl}} \tag{3.4}$$

The standard interaction formulae have empirical background and they have been simplified in order to make the design easier.

The MN interaction diagram (EC3/5.4.8) is piece-wise linear for the standard rolled I sections

$$m = \begin{cases} 1 & \text{for} \quad n \leqslant n_m \\ \dfrac{1-n}{1-n_m} & \text{for} \quad n > n_m \end{cases} \tag{3.5}$$

where $n_m = 0.5 + bt_f/A$ is the coordinate of the MN diagram corner for the cross sections of Class 1 or Class 2, but $n_m = 1$ for the cross-sections of Class 3.

The MV interaction diagram (EC3/5.4.7) is parabolic in the range $0.5 < v < 1$

$$m = \begin{cases} 1 & \text{for} \quad v \leqslant v_m \\ 1-(1-m_v)(2v-1)^2 & \text{for} \quad v > v_m \end{cases} \tag{3.6}$$

where $m_v = 1 - A_V^2/(4t_wW_y)$ and $v_m = 0.5$ are the coordinates of the MV diagram corners.

► Secant linearization of nonlinear interaction equations has been advised (Murzewski, 1997). The parabolic MV interaction curve, Eq (3.6), in the range $0.5 < v < 1$ may be approximated by the secant committing an error less than 5% on the safe side

$$0.5m + (1-m_v)v = 1 - 0.5m_v \tag{3.7}$$

Secant linearisation of the limit state locus is always conservative. Inaccuracy can be as small as we wish thanks to further fragmentation of the curves. The clause EC3/5.4.9 defines some interaction rules for bending, shear and axial force which are not compatible with EC3/5.4.7. That is why the more detailed but uncertain eight equations for the M_S-N_S-V_S interaction (Murzewski, 1999) are not presented in this paper.

Let six linear interaction equations define the ultimate limit state locus (Fig.3). The non-dimensional interaction matrix $\mathbf{m}$ represents its geometrical shape. The coefficients m_{je} of linear equations (3.8) of the polyhedron in the interaction ranges $e = MO,\ MN,\ MV,\ NV,\ MNV$ create the non-dimensional interaction matrix $\mathbf{m}$ (Eq $(3.8)_2$)

$$m_{0e} M_S + m_{1e} N_S + m_{2e} V_S = M_R$$

$$\mathbf{m} = \begin{bmatrix} 1 & 1-n_m & \dfrac{1-v_m}{1-m_v v_m} & 0 & \dfrac{(1-n_m)(1-v_m)}{D} & 0 \\ 0 & 1 & 0 & \dfrac{1-v_m}{1-n_v v_m} & \dfrac{1-v_m}{D} & 0 \\ 0 & 0 & \dfrac{1-m_v}{1-m_v v_m} & \dfrac{1-n_m}{1-n_v v_m} & \dfrac{(1-m_v)(1-n_m)}{D} & 1 \end{bmatrix} \tag{3.8}$$
$$\qquad\quad \underbrace{}_{MO}\quad \underbrace{}_{MN}\quad \underbrace{}_{MV}\quad \underbrace{}_{NV}\quad \underbrace{}_{MNV}\quad \underbrace{}_{VO}$$

where $M_R =$ const is the reference resistance equal to the design resistance moment in simple bending and

$$D = 1 - (m_v - m_v n_m + n_m) v_m \qquad\qquad v_m = 0.5$$

A specific interaction matrix $(\mathbf{rm})$ relative to a selected cross-section is defined as the product of the cross-section core vector $\boldsymbol{r}$ and non-dimensional interaction matrix $\mathbf{m}$. The elements of the vector $\boldsymbol{r}$ are defined not only for MN but also for MV interaction ranges

$$(\mathbf{rm}) = \mathrm{diag}[\boldsymbol{r}] \cdot \mathbf{m} \qquad\qquad \boldsymbol{r} = \begin{bmatrix} 1 \\ \dfrac{W_y}{A_N} \\ \dfrac{\sqrt{3}W_y}{A_V} \end{bmatrix} \tag{3.9}$$

The transposed specific interaction matrix $(\mathbf{rm}) = (\mathbf{rm})^\top$ will be multiplied by the nested influence matrix $\{c_{jv}\}_i$ and the product will define effective influence coefficients $\{(crm)_{ev}\}_i$. The subscript j will disappear in the matrix multiplication.

3.3. Member design

The member design rules are different from those for the cross-section if there are compressive stresses and instability must be taken into consideration. The reduction factors χ_j for member design are defined in Eurocode 3

$\chi_0 = \chi_{LT}$ for beam buckling (EC3/5.5.2)
$\chi_1 = \chi$ for column buckling (EC3/5.5.1)
$\chi_2 = \sqrt{3}\,\tau_{be}/f_y$ for shear buckling in the "simple" post-critical method (EC3/5.6.3).

Each reduction factor χ_j depends on the so called "non-dimensional" slenderness $\overline{\lambda}_j$, proportional to the member slenderness $\lambda_j = l/i$.

▶ In author's opinion (Murzewski, 1992) the "non-dimensional" slenderness specified in the clause EC3/5.5.1 should be corrected

$$\widetilde{\lambda}_j = \frac{\lambda_j}{\pi}\sqrt{\frac{1.21 f_k}{E}} \qquad \text{instead of} \qquad \overline{\lambda}_j = \frac{\lambda_j}{\pi}\sqrt{\frac{f_k}{E}} \tag{3.10}$$

The reason for correction is that at least equal safety factor should be applied to median resistance of very slender columns $N_{el} = \pi^2 EA/\lambda^2$ (if $\lambda \to \infty$) as it is for the median plastic resistance of thick columns $N_{pl} = f_m A$ (if $\lambda \to 0$). The difference between the median value f_m of yield strength and the nominal value f_k is about 21% for structural steel according to the representative statistical estimates for structural steel (Murzewski, 1989) but the EC elastic modulus $E = 210\,\text{GPa}$ is close to its median value. The correction (3.10) has been introduced to the Polish design standard. Better agreement would be with the value $E = 205\,\text{GPa}$ (Polish standard) or $E = 200\,\text{GPa}$ (American standard).

The interaction curve MN for bending and axial compression can be concave because of the second order deflection of the member. Secant linearisation would be unsafe in this case. The nonlinear and mathematically non-homogeneous equation is recommended as the limit state criterion by EC3/5.5.4

$$\frac{M_S}{\chi_0 M_R}\Big(1 - \frac{\mu_1 N_S}{\chi_1 N_R}\Big) + \frac{N_S}{\chi_1 N_R} \leqslant 1 \tag{3.11}$$

where the correction factor μ_1 can be either positive or nonpositive. It is defined as a complicated function of geometric properties of the member.

▶ Equation (3.11) can be reduced to a linear equation by the algebraic transformation

$$\Big(\frac{M_S}{\chi_0 M_R} + \frac{N_S}{\chi_1 N_R}\Big)\mu_{mn} \leqslant 1 \tag{3.12}$$

where

$$\mu_{mn} = \frac{1}{2} + \sqrt{\frac{1}{4} - \frac{\mu_1 e_n}{r_1(1+e_n)^2}}$$

Such a transformation is possible also for the Polish MN interaction formula (Murzewski, 1997). The advantage of the form (3.12) is that the eccentricity ratio $e_n = M_S\chi_1/(N_S\chi_0)$ need neither partial factors γ_i, nor combination factors ψ_i provided that the loading process is simple (proportional). If a safety condition is defined in the symbolic form (3.1), both the left-hand side N_{eq} of and the right-hand side N_R of the inequality $N_{eq} < N_R$ will be separated thanks to this correction.

Interaction between the bending moment M_S and shear force V_S for members is similar to that for cross-sections (EC3/5.6.7). The nondimensional interaction matrix (Eq (3.8)) may define also the shape $\mathbf{n}$ of the limit state locus of members as well in the three-dimensional space m-n-v (Fig.3). Since the scalar axial force N_{eq} has been defined as the equivalent action effect S_{eq} for member design, the inverse section cores $r'_j = r_n/r_j$ are introduced with the buckling factors χ_j and the second-order effect factor μ_{mn}. So, the specific interaction matrix $(\mathbf{rn})$ is defined for member design as follows

$$\boldsymbol{r}' = \begin{bmatrix} \dfrac{\mu_{mn}A}{\chi_0 W_y} \\ \dfrac{\mu_{mn}}{\chi_1} \\ \dfrac{\sqrt{3}A}{\chi_2 A_V} \end{bmatrix} \tag{3.13}$$

hence $(\mathbf{rn}) = \mathrm{diag}[\boldsymbol{r}'] \cdot \mathbf{n}$.

Then, the specific interaction matrix transpose $(\mathbf{rn})^{\top}$ is multiplied by the nested influence matrix $\mathbf{c}$ (Eq (2.1))

$$\{(crn)_{ev}\}_i = (rn)_{ej}\{c_{jv}\}_i \tag{3.14}$$

4. Module 3 – reliability verification

The intended lifetime of the structure t_d and partial safety factors γ_i for actions F_i are inserted to Module 3 block of constants. The γ_i values may be corrected by factor γ_n depending on the accepted reliability class. The equivalent geometric imperfection, defined as the initial sway ϕ_o (EC3/5.3.4), is also in the block of constants; however, a guess value of amplified sway ϕ is inserted to the block of variables. The value ϕ shall be corrected by means of the non-iterative formula (2.2). The actual sway ϕ will change definitely the complex influence coefficients (2.1) into the real numbers $\{c_{jv}\}_i$. The nested unit load array **c** has been multiplied in Module 2 by the specific interaction matrix (**rs**) = either (**rm**) or (**rn**) for any load F_i

$$\{(crs)_{ev}\}_i = (rs)_{ej}\{c_{jv}\}_i \tag{4.1}$$

The nested array (**crs**) means either (**crm**) in the case of cross-section design or (**crn**) in the case of member design like the equivalent load effect S_{eq} has meant either equivalent moment M_{eq} or equivalent force N_{eq}.

4.1. Combination matrix

Ferry-Borges and Castanheta (1971) defined a stationary stochastic model of actions where the loads F_i were ordered with respect to the numbers $\nu_i = t_{ref}/\theta_i$ of repetitions during a specified reference period t_{ref}. Random values F_i remain constant during elementary time periods θ_i, $i = 1, 2, ..., n$, (Fig.4). There are 2^{n-1} possible combinations in this model with n being the number of independent variable loads.

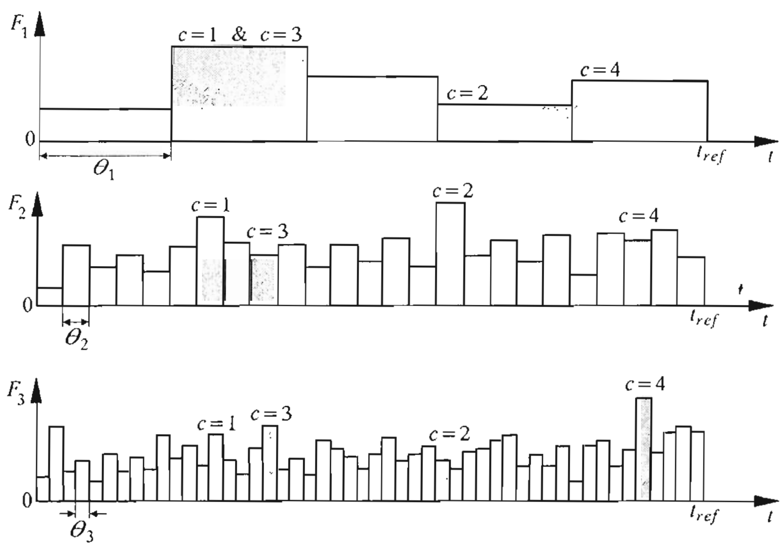

Fig. 4. Regular sequences of three random actions F_i, which are constant during the periods θ_i

A simplified rule was proposed by Turkstra (1972) where one variable action F_c, $c = 1, 2, ..., n$, was dominant and the values of other non-dominant actions F_i, $i \neq c$, were reduced to their point-in-time intensities. Thus only n combinations have to be taken into consideration. A similar combination rule is recommended by Eurocode 1 but the elementary period θ_i are taken instead of the point-in-time values. The reference period is $t_{ref} = 50$ years and the combination factors are $\psi = 0.7$ for the imposed loads and $\psi = 0.6$ for climatic actions at the ultimate limit states according to the Eurocode 1 (Table 3). Combination factors ψ of the Eurocode 1 yield lower estimates of combined loads than the Ferry-Borges and Castanheta model predictions. Therefore they are unsafe.

Table 3

ψ_{ic}	1	2	3	4
Q	1	0.7	0.7	0.7
S	0.6	1	0.6	0.6
T	0.6	0.6	1	0.6
W	0.6	0.6	0.6	1

► A new combination rule (Table 4) has been derived (Murzewski, 1996) which gives safe upper bound estimates of combined action effect. The elements of the new combination matrix are

$$\psi_{ic} = \begin{cases} 1 - v_i \ln(50/\theta_i) & \text{if} \quad i < c \\ 1 - v_i \ln(50/\theta_c) & \text{if} \quad i > c \\ 1 & \text{if} \quad i = c \end{cases} \tag{4.2}$$

where

c – subsequent numbers of possible dominant variables, $c = 1, 2, ..., n$

v_i – Gumbel coefficients of variation of the ith action maximal in the reference period t_{ref}.

Table 4

ψ_{ic}	1	2	3	4
Q	1	0.7	0.7	0.7
S	0.7	1	0.6	0.6
T	0.7	0.6	1	0.6
W	0.7	0.6	0.6	1

The coefficients v_i and elementary periods θ_i have been identified so (Murzewski, 1999) that the same values $\psi_{ic} = 0.7$ and 0.6 would appear in the combination matrix (Table 2). The new matrix $\boldsymbol{\psi}$ is symmetric. If the intended lifetime td is different than $t_{ref} = 50$ years, the non-dominant combination factors ψ_{ic} will be the same but the combination factor ψ_{ii} of dominant loads will be different from 1, namely

$$\psi_{ii} = 1 + v_i \ln \frac{t_d}{50} \tag{4.3}$$

Eq (4.3) has been derived assuming of the Gumbel probability distribution.

The elements of the combination matrix $\boldsymbol{\psi}$ are multiplied by the elements of the design load vector $(\boldsymbol{\gamma F}) = \gamma_i \cdot \boldsymbol{F}_i$ and they give the combination load matrix $(\boldsymbol{\psi\gamma}\mathsf{F})$ necessary for the ultimate limit states verification

$$(\boldsymbol{\psi\gamma}\mathsf{F}) = \mathrm{diag}[(\boldsymbol{\gamma F})] \cdot \boldsymbol{\psi} \tag{4.4}$$

4.2. Extreme load effects

Solution to so-called the first extremum problem helps to identify the most unfavourable variant v_i for every load F_i, $i = 0, 1, 2, ..., n$ one after another in each interaction range $e = 0, 1, 2, ...$ So, either nested array **(crs)** ((**crm**) or (**crn**)) will be reduced to two conventional matrices (**maxcrs**) or (**mincrs**). The maximum effect of any variable load F_i, $i \geqslant 1$, will be non-negative and the minimum effect of this variable load will be non-positive thanks to the zero variants $v_i = 0$, $i \geqslant 1$

$$(maxcrs)_{ei} = \max_v \{(crs)_{ev}\}_i \qquad \text{and} \qquad (mincrs)_{ei} = \min_v \{(crs)_{ev}\}_i \tag{4.5}$$

In the example under consideration, we have obtained the two matrices, (**maxcrs**) and (**mincrs**), with the elements $(maxcrs)_{ei}$ and $(mincrs)_{ei}$, respectively, for five independent actions $F_i = G$, Q, Sn, T, W (Table 2) and the interaction ranges $e = M0$, MN, MV, $V0$, VN, MNV (Fig.3).

The extreme influence matrices (**maxcrs**) and (**mincrs**) for the selected load variants are multiplied by the combination load matrix $(\boldsymbol{\psi\gamma}\mathsf{F})$. The matrix product of the combination load matrix $(\boldsymbol{\psi\gamma}\mathsf{F})$ and the extreme influence matrix (**crs**) will give the equivalent load effect matrices

$$(\mathbf{maxS}) = (\mathbf{maxcrs}) \cdot (\boldsymbol{\psi\gamma}\mathsf{F}) \qquad \text{and} \qquad (\mathbf{minS}) = (\mathbf{mincrs}) \cdot (\boldsymbol{\psi\gamma}\mathsf{F}) \tag{4.6}$$

The load index i disappears as a result of the matrix multiplication (4.6). The maximal equivalent load effects $(maxS)_{ec}$ and minimal equivalent load effects $(minS)_{ec}$ are derived for each load effect interaction range $e = 0, 1, 2, ...$ and each load combination case $c = 0, 1, 2, ...$

Solution to the second extremum problem helps to determine the most unfavourable effective load S_{eq} among all combinations c and interactions e. The two scalar extreme values $(MaxS)$ and $(MinS)$ are determined

$$(MaxS) = \max_{e,c}\{(maxS)_{ec}\} \qquad \text{and} \qquad (MinS) = \min_{e,c}\{(minS)_{ec}\} \tag{4.7}$$

The absolute maximum of equivalent moment M_{eq} is actual for design of bisymmetrical steel cross-sections

$$M_{eq} = \max\{(MaxS), |(MinS)|\} \tag{4.8}$$

and the absolute value of minimal equivalent force N_{eq} (compression) is actual for member design

$$N_{eq} = |(MinS)| \tag{4.9}$$

Thus the equivalent load effect S_{eq} is determined in either design problem. The critical load case (v_i, c, e) can be additionally recognised. The result S_{eq} shall be adjusted to the exact amplified sway ϕ. The preliminary value of ϕ shall be corrected in the block of variables for either design case.

Finally, the ultimate limit states of cross-section and the structural member are verified

$$M_{eq} \leqslant M_R =? \qquad \text{and} \qquad N_{eq} \leqslant N_R =? \tag{4.10}$$

If we get 1 on the right-hand side of either criterion (4.10), the safety will be all right; however, economy of design shall be still checked in such a way that some smaller sections shall be taken into account and criteria (4.10) will be not satisfied, i.e. symbol 0 should appear on the right-hand side of at least one criterion.

All critical cross sections and structural members can be verified using the same or a modified design template. In addition, separate calculations have to be made in order to verify the limit states of serviceability: Maximum values of frame sway $|\phi|$ and beam deflection $|\delta|$ have to be verified for characteristic loads F_i (without γ_i factors) and suitable combination rule from Eurocode 1 (EC1/9.5.2)

$$\max_{v,i,c}(\Delta\phi) = \max_{v,i,c}|\phi| - \phi_o \leqslant \phi_{max} \qquad \text{and} \qquad \max_{v,i,c}|\delta| \leqslant \delta_{max} \text{ or } \delta_2 \tag{4.11}$$

The limiting values ϕ_{max} and δ_{max}, δ_2 are specified in Eurocode 3 (EC3/4.2.2).

5. Final remarks

This paper may be helpful to see what is Design Template in the author's understanding. The idea of design templates is such that a designer can modify it or include some parts of previous templates. The author hopes that the design templates will give good solutions for codified design when semi-probabilistic method of partial factors has been introduced and new standards are implemented. Professional structural design computer programs are usually obsolete except for elastic analysis programs which do not change too frequently. Academic teachers and professional designers may learn only few conventions of a chosen mathematical computer program unless they have used them before. A great difference between laborious traditional calculations and the new computer-aided approach is obvious. Using the computer templates one must take into consideration the following items:

- Computer-aided structural design will be easily verifiable if all constants, variables and algorithms are written in environment of the mathematical program (e.g., Mathcad) in due order

- No numerical results must be revealed for confirmation of structural reliability but they would be helpful to discover errors, to check economic and technological aspects

- Thousands of load cases, load combinations etc. for each element may be checked if matrix calculus and new extreme value procedures are applied

- Once prepared templates may be saved in the computer memory; they may be included entirely or partly to next design projects.

Design rules and algorithms of Eurocode 1 and Eurocode 3 have been taken to exemplify the design template. The Eurocodes are not yet mandatory even in the countries of the European Community. Therefore some discussions and corrections (marked with ►) may be useful to perfect the final version of the European design standards.

Acknowledgements

Most research was financially supported by the State Committee for Scientific Research (KBN) under grant 7T07E04010. Discussions with Colleagues from the Chair for Reliability and Design of Metal Structures of Cracow University of Technology helped to develop the design template concept.

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Szablony projektowania – nowe narzędzie komputerowych obliczeń elementów konstrukcji

Streszczenie

Szablon projektowania definiuje się jako wcześniej przygotowany układ obliczeń komputerowych, który może być sprawdzany i modyfikowany w poszczególnych przypadkach przez projektanta, co nie jest możliwe przy korzystaniu z komputerowych programów projektowania. Ustalić trzeba najpierw obciążenia globalne i wykonać obliczenia statyczne dla każdego wariantu obciążeń jednostkowych z osobna. W wyniku obliczeń statycznych formułuje się macierze blokowe wpływu obciążeń. Określa się

następnie macierze współdziałania sił wewnętrznych i macierze kombinacji jednoczesnych obciążeń. Stosuje się nowe procedury macierzowe w celu wyszukania najbardziej niekorzystnych wariantów dla każdego niezależnego obciążenia z osobna, a następnie – najbardziej niekorzystnego przypadku współdziałania sił wewnętrznych i kombinacji obciążeń. Przykładowy szablon projektowania dotyczy stalowej ramy portalowej pod obciążeniem stałym i działaniami zmiennymi oraz siłami drugiego rzędu wynikającymi z przechyłu słupów. Na początku obliczeń iteracyjnych określa się zespolone współczynniki wpływu obciążeń i wstawia próbną wartość dopełnionego kąta przechyłu. Proponuje się przy tym poprawki pewnych nieścisłych wzorów Eurokodu 1 i 3.

Manuscript received October 27, 1999; accepted for print January 11, 2000