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NEW RESULTS IN THE FIELD OF STRUCTURAL OPTIMIZATION

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Summary

Mass and cost reduction of structures needs structural optimization. Structural optimization has three main parts, structural analysis, synthesis and engineering evaluation. During the four decades at the University of Miskolc, where optimization studies have been performed, we have made the optimization of the following structures: crane girders, dogbone sections, silos, stiffened and cellular plates, tubular structures, sandwich and composite structures, etc. The fabrication cost calculation is very important for the optimum sizes. The optimization techniques used for optimization tend from mathematical methods to sequential quadratic programming, using automatic derivation. Expert systems, genetic algorithms and neural networks are new promising fields of optimization. In this paper we show some new examples in structural optimization.

1. Optimum design of silos

Silos are used for many engineering purposes. An elevated silo consists of the following structural parts: roof, circular cylindrical bin, transition ringbeam, conical hopper and supporting columns. The optimum design problem of silos is characterized by some specialities as follows. The structure is determined by two main dimensions i.e. the height H and the radius R of *the circular cylindrical bin* (Fig.1). The bin consists of several horizontal courses. The width of these courses is determined by the available plate width (e.g. 1500 mm). The effect of a sudden temperature change as well as the dynamic filling and emptying effects are taken into account by multiplying factors. The local buckling of the cylindrical shell of variable thickness should be checked for two effects: a) for vertical compressive stresses due to the full load, b) for wind pressure

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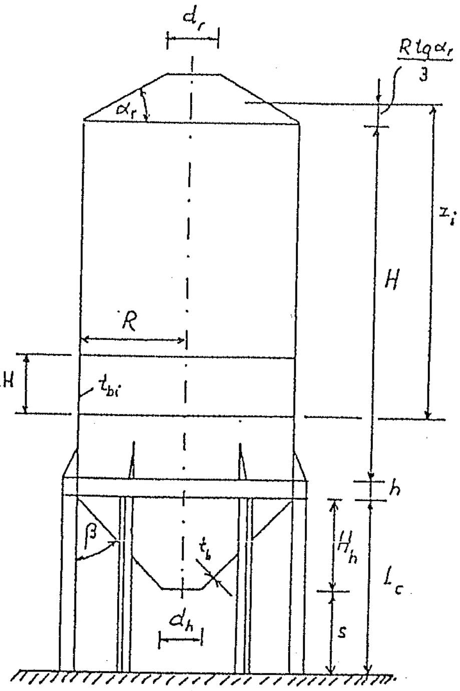


Figure. 1. Welded steel silo

acting on the empty silo.

The loads acting on *the transition ringbeam* cause compression, bending and shear in two planes and torsion. Since the open section beams have very small torsional stiffness, it is advantageous to use welded box ringbeams (Farkas, Jármai [1]). The ringbeam is optimized using the constraints on stress and local buckling of component plates.

The uniform thickness of *the conical hopper* can be calculated from the stress constraint. The slope angle of the hopper is determined by the friction angle of the stored material.

The number of *the supporting columns n* may vary in a range determined by the service conditions, i.e. by the prescribed distance between columns needed for lorries. The columns may have a square or circular hollow section, which is designed using the constraints on overall and local buckling.

Table 1. K/k_m (kg) values for four silos of equal storage capacity of 500 m3 for k_f/k_m=1

Closed formulae have been derived for the optimal dimensions of SHS or CHS columns. The objective function (K) is formulated as a cost function, which includes the material (K_m) and fabrication (K_f) cost, k_m and k_f are the specific material and fabrication costs. For material Fe 360 and Fe 510 type steels are used with yield stress 235 and 355 MPa, respectively. Results in **Table 1** show that the highest H/R ratio gives the minimum cost.

R(m)	4.25	3.50	3.15	2.90
H(m)	7.50	12.00	15.00	18.00
H/R	1.76	3.43	4.76	6.20
roof	3769	2597	2073	1779
bin	8853	11627	13240	14295
ringbeam	6101	4943	4170	3597
hopper	4356	3065	2583	2169
columns	2681	2231	2068	1952
total	25760	24463	24134	23792

2. Optimum design of sandwich beams with fiber reinforced composite

The poor damping capacity of aluminium beams can be improved by using a rubber layer. The disadvantage of such sandwich beams was the relatively high dynamic deflection due to the shear deformation of the rubber layer. For the present study our aim was to decrease this large deflection by using fiber-reinforced plastic (FRP) layers. To investigate the static and dynamic behaviour of sandwich beams constructed from aluminium square hollow section (SHS) and rubber and FRP layers, we have used 3 specimens as shown in Fig.2. Static bending tests and vibration damping measurements serve to describe the most important characteristics of the investigated models. For the calculations the static and dynamic bending theory of sandwich beams with thick faces is applied.

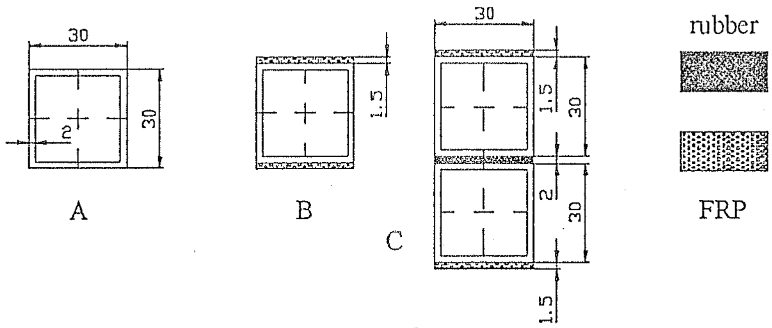


Figure 2. Tested specimens

In the static tests the maximum deflection is measured at the midspan of simply supported beams loaded by a concentrated force at midspan. Three specimens have been manufactured and tested (Fig.2).

In order to determine the eigenfrequencies and vibration damping or loss factors the Brüel-Kjaer vibration measuring devices have been used in our laboratory. The loss factors are obtained by using the half-power bandwidth method. The formula for the loss factor at the ith eigenfrequency f_i is $\eta_i = \Delta f / f_i$ where Δf is the frequency bandwidth. Table 2 gives the measured results.

Table 2.

Measurement results: eigenfrequencies and loss factors

Specimen	A			B			C		
I	1	2	3	1	2	3	1	2	3
f_i (Hz)	32	196	536	33	200	543	52	255	648
η_i	0.0125	0.0028	0.0015	0.012	0.0032	0.0028	0.052	0.057	0.053

The static bending stiffness of a SHS aluminium beam can be increased significantly by using FRP layers. This increase was in the case of our investigated specimens about 35%, without any increase in weight. The FRP layers do not increase the vibration damping, the loss factor is only about 1%. The static behaviour of a SHS profile with FRP layers can be calculated by the reduced bending stiffness. The damping can be increased significantly by applying a rubber layer of high damping capacity [2,3]. In our case the loss factor has been quadrupled (comparison between specimens A and C). Due to a soft rubber layer the static bending stiffness decreased by 52%. The static and dynamic behaviour of specimen C can be calculated with sufficient accuracy by the bending theory of sandwich beams with thick faces. The optimization is made taking into account the face beams stresses, the local buckling of the web plate, the deflection and the damping factor of the beam.

3. Welded cellular plates for ship deck panels

The investigated cellular plates consist of two face sheets and some longitudinal ribs of square hollow section (SHS) welded between them using arc-spot welding technology. The cellular deck panels are subject to axial compression and a transverse load causing bending. In the optimization procedure the dimensions and number of longitudinal SHS ribs as well as the thickness of face sheets are sought which minimize the cost function and fulfil the design constraints. The width and length of the three-span panel are known. The cost function contains the material and fabrication cost. The design constraints relate to the stress due to compression and bending and to the eigenfrequency of the structure.

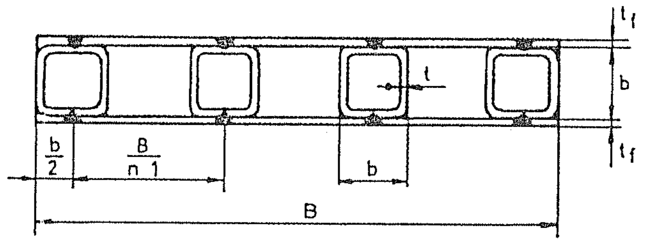


Figure 3. Cellular plate of the ship deck panel with tubular stiffeners

Constraints on eigenfrequency [4], stability due to compression and bending and stress constraint for the upper face sheet should be taken into account [5]. A serviceability constraint can be defined expressing that the first eigenfrequency of a simply supported bent beam of span length L should be larger than a prescribed value. According to Eurocode 3 [6], the stress

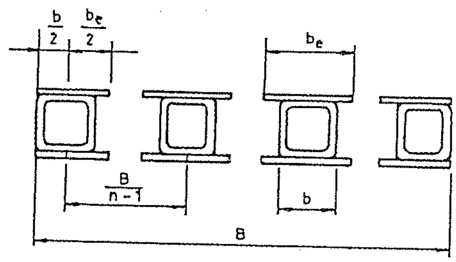


Figure 4. Effective cross-section for compression

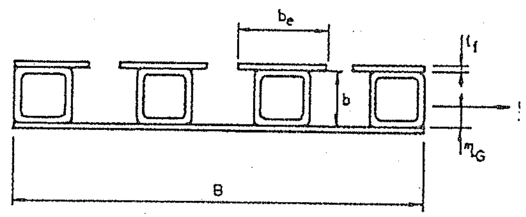


Figure 5. Effective cross-section for bending

constraint should be defined for a section of class 4. To obtain the effective cross-section, the effective width of face sheets should be calculated according to EC3.
For bending another asymmetric effective cross-section should be taken into account as shown in Fig.5.

The numerical data are as follows: f_0 = 18 Hz, $E = 2.1*10^5$ MPa, B = 2000, L = 2250 mm, $\Theta_d = 3$, $\rho = 7850$ kg/m^3 = $7.85*10^{-6}$ kg/mm^3, $m_{add} = 2*50$ = 100 kg/m = 0.1 kg/mm, p = 3.5 kN/m^2 = $3.5*10^{-3}$ N/mm^2, $\psi = 1.4$, $\sigma = N/A_{eff} = 150$ MPa. The computational results are summarized in Tables 3.

The optimum number of ribs is larger for minimum weight design (k_f/k_m = 0) i.e. n=8 for f_y = 235 and n = 6 for f_y = 355 MPa, than for minimum cost design (k_f/k_m = 1 or 2) i.e. n = 7 for f_y = 235 and n = 6 or 5 for f_y = 355 MPa. The optimum number of ribs depends on f_y.. Cost savings of 14-18% can be achieved using steel of yield stress 355 instead of 235 MPa. The cost difference between the best and worst solution for f_y = 235

MPa and $k_f/k_m = 2$ is 100(2747-1683)/1683 = 63%, which emphasizes the importance of structural optimization. Calculations show that the stability and stress constraints are in most cases active and the eigenfrequency constraint is passive [7].

Table 3.

Optimization results for $f_y = 235$ MPa: number of ribs n, optimum dimensions in mm and K/k_m-values in kg for cost in function of the ratio k_f/k_m

k_f/k_m	n	t_f	b	t	K/k_m
	4	8	120	3	1987
	5	4	120	3	1212
	6	4	30	2	916
0	7	2.5	40	2	639
	8	2	50	2	582
	9	2	50	2	602
	10	2	50	2	621
	4	8	120	3	2367
	5	4	120	3	1620
	6	4	30	2	1331
1	7	2.5	40	2	1161
	8	2	50	2	1232
	9	2.5	40	2	1306
	4	8	120	3	2747
	5	4	120	3	2027
	6	4	30	2	1745
2	7	2.5	40	2	1683
	8	2.5	40	2	1813
	9	2.5	40	2	1943

4. Fabrication cost calculations

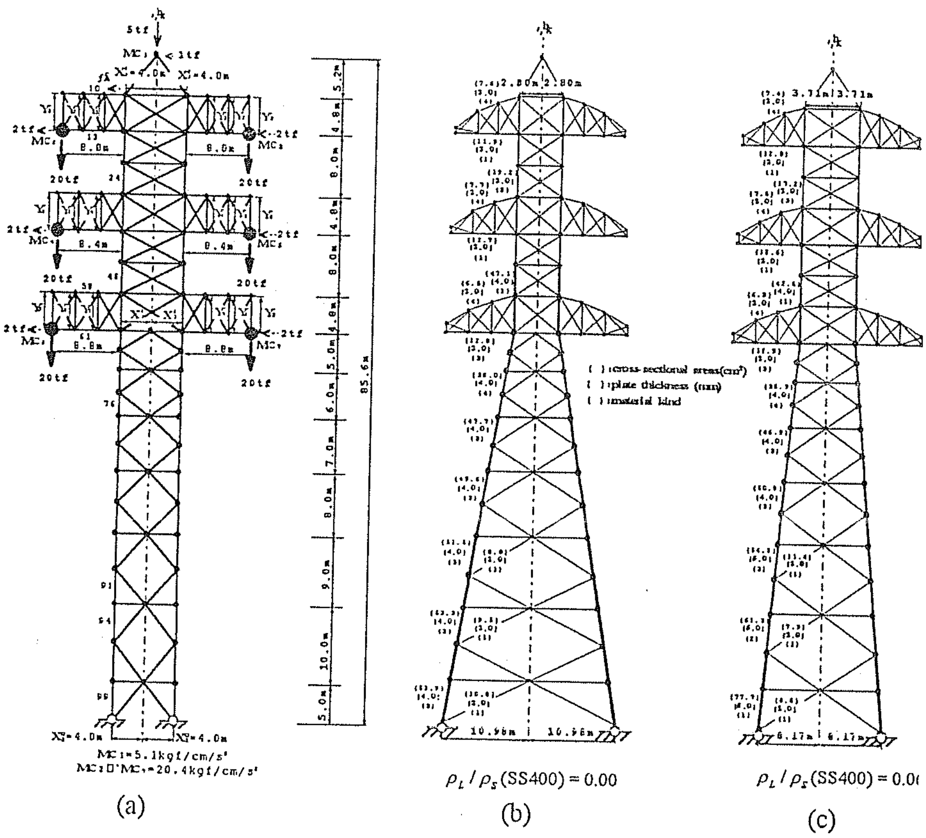
In the early stage of optimization the mass of the structures has been minimized. Nowadays there are also some optimization techniques, which can not handle complicated cost functions. To get an economic structure in the period of increasing fabrication costs, one should take into account as many elements of costs as possible. The cost of a structure is the sum of the material, fabrication, transportation, erection and maintenance costs. The fabrication cost elements are the welding-, cutting-, preparation-, assembly-, tacking-, painting costs etc. It is very difficult to obtain such cost factors, which are valid all over the world. If we choose times, as the basic data of fabrication phases, we can handle this problem. The fabrication time depends on the technological level of the country and the manufacturer, but it is much closer to the real process to calculate with. After computing the necessary time for each fabrication phase one can multiply it by a specific cost factor, which can represent the development level differences [3,9].

Using the COSTCOMP [8] program we can calculate the welding times. Times are usually general, but costs are different in various countries. Introducing the fabrication and material specific cost ratio k_f/k_m between 0 and 2 kg/min, it is possible to build the cost function from the times and to work out optimization in different economic conditions. Examples are shown applications for design of welded box beams and stiffened plates. The fabrication cost percents for welded box beam and stiffened plate are 29 - 35 and 46 - 71% of the total costs, respectively, thus they can have a significant effect on optimum dimensions. The discrete optima depend on the manufacturers, on the k_f/k_m ratio and the welding technologies.

5. System optimization of configuration, material and sizing variables for truss structures

An efficient optimal design method for truss structures has been developed to determine the optimum values to be used for the coordinates of all panel points, cross-sectional dimensions and discrete material kinds of all member elements simultaneously satisfying stress and displacement constraints due to static and earthquake loads [14,15,16,17]. In the optimum design problem, the objective function is the total construction cost of truss structure considering not only the cost of truss members but also the cost of land at construction site. The stress, displacement and slenderness ratio constraints specified in the Japanese Specifications for Highway Bridges are considered as behaviour and side constraints. The stresses of member elements and displacement of structure due to earthquake loads are calculated by a response spectrum method.

At the first stage of the optimum design process, the maximum allowable stress and



ρ_L / ρ_S(SS400) = 0.00 ρ_L / ρ_S(SS400) = 0.0(

(a) (b) (c)

Figure 6.: Initial 193-bar transmission tower truss and optimum solutions for ρ_L / ρ_c(SS400) = 0.00 *and* 0.06

cross-sectional dimensions (areas) relationships of the member elements are introduced for the available material kinds by using the suboptimization concept [13]. Then the primary optimum design problem expressed in terms of configuration, material and sizing variables is transformed into an approximate subproblem of convex and separable form by using mixed direct/reciprocal design variables and the sensitivities of behaviour constraints with respect to the primary design variables. The approximate subproblem is solved by a two-stage minimization process incorporating with dual method for the improvement of structural shape and member sizes, and discrete sensitivity analysis for the improvement of material kinds to be used for each member elements.

The numerical design examples of minimum-cost design problems of 193-bar transmission tower truss are shown in *Figure 6*. In this optimum design problem X_1, X_2, Y_1, Y_9 are considered as the shape design variables. All member elements are assumed to be made of circular steel pipes. By applying the suboptimization concept [13,17], the cross-sectional dimensions (diameter and plate thickness) and the maximum allowable stress relationships for SS400, SM490, SM490Y, SM570 steel kinds, where each steel kinds are termed as material number 1, 2, 3 and 4, respectively in *Figure 6*, are introduced for member elements. *Figure 6*.(b) and (c) show the optimum solutions for $\rho_L / \rho_s(SS400)$=0.00 and 0.06, respectively, where ρ_L and $\rho_s(SS400)$ are the unit costs of land at construction site and that of SS400 steel (material kind 1), respectively. As the ratio $\rho_L / \rho_s(SS400)$ increases the optimum value of X_1 becomes smaller and X_2 becomes larger, and the distributions of optimum material kinds and optimum cross-sectional sizes (areas) are affected considerably by the unit price of land at construction site.

By inspecting a number of numerical results, it has been confirmed that the proposed optimal synthesis method can determine the optimum configuration of structure, discrete material kinds and cross-sectional dimensions of member elements of large scale transmission tower truss subjected to static and earthquake loads quite efficiently and reliably within 15 iterations.

6. Optimization of nonlinear frame structures based on energy principles

New optimum design methods for plane frame structures with linear or arbitrary nonlinear stress-strain relationship materials have been developed by Ohkubo et al. [20,21,22]. The linear and nonlinear structural analysis problem is solved unificatively and efficiently as the complementary energy minimization problem in which the actual member end forces of plane frame structures can be determined by minimizing the total complementary energy of structure with respect to member end forces subject to equilibrium equations at the free nodes [18,19]. The necessary conditions for structural analysis problem are derived from the stationary conditions of the Lagrangian function of the analysis problem, namely complementary energy minimization problem. Then the primary optimum design problem is reformulated considering both the primary design constraints (stress and displacement constraints) and the necessary conditions for analysis problem. In the reformulation of optimum design problem, member end forces of all member elements and displacements at free nodes are also dealt with as the design variables in addition to the primary design variables (cross-sectional areas A for truss structures and flange plate thicknesses t of box sections for frame structures). The reformulated optimum design problem is solved by using a linear approximation concept and the gradient projection method, however, no behaviour sensitivity is necessary to use in the optimization process.

The rigorousness, efficiency and reliability of the proposed method are demonstrated by showing the design examples of volume minimization problems of truss and frame structures and comparing the optimum solutions with ones obtained by the dual method (DUAL-S) in which the behaviour sensitivities are used. In the design examples of truss structures, the allowable stress of the material, allowable displacement at free nodes and the weight per unit volume are set, respectively, at 2500kgf/cm^2, and 58.4% of those by DUAL-S method. 100.0cm and 7850kgf/m^3, and the nonlinear material shown in *Figure*

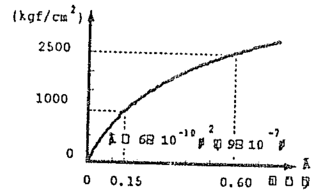


Figure 7.: Nonlinear material

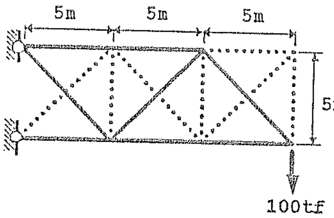


Figure 8.: Optimum member arrangement of 15-bar truss

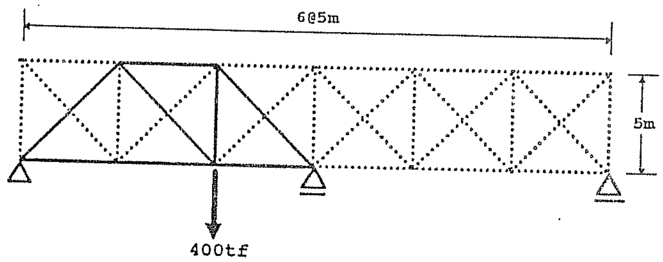


Figure 9.: Optimum member arrangement of 31-bar truss

7 is used. The initial and optimum member arrangements for 15-bar and 31-bar trusses are, respectively, shown in *Figures 8* and *9*. The optimum member arrangements of 15-bar and 31-bar trusses obtained by the proposed method correspond to those obtained by DUAL-S method, and the values of objective functions by the proposed method are quite similar to those by DUAL-S method. The computation times to obtain the optimum

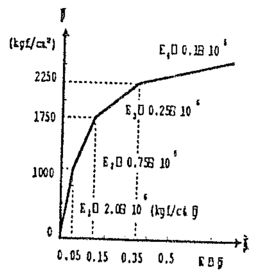


Figure 10.: Nonlinear material

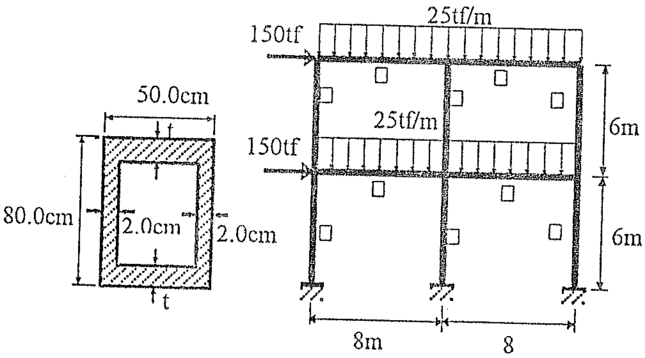


Figure 11.: 2-bay 2-story plane frame and cross-section of a member element

Table 4.
Comparison of optimum solutions obtained by proposed method and DUAL-S for 2-bay 2-story plane frame shown in Figure 11

Method	Proposed Method		DUAL-S	
No. of mem. element	t_{opt} (cm)[1]	σ_{max}(kgf/cm²)[2]	t_{opt} (cm)[1]	σ_{max}(kgf/cm²)[2]
1	1.33	-2327.2	2.96	-2288.5
3	5.41	-2491.4	5.46	-2464.8
5	3.20	-2420.3	2.76	-2380.1
7	0.10	-2350.7	0.34	-2331.9
9	1.30	-1887.9	0.10	-2073.0
W(kgf)	41.82		41.62	
λ_{max}(cm)	11.53		11.48	
Active constraints	$\sigma_2,\sigma_3,\sigma_4,\lambda$		$\sigma_2,\sigma_3,\sigma_4,\lambda$	
Iteration	27		8	
CPU time(ratio)[3]	281.7 (0.44)		644.4 (1.0)	

1)Optimum plate thickness 2) Maximum stress of member element
3) Ratio of computation time of proposed method to that of DUAL-S

solutions for 15-bar and 31-bar trusses by the proposed method are, respectively, 51.0%.

The optimum solutions of 2-bay 2-story plane frame shown in *Figure 11* with nonlinear material in *Figure 10* are summarized in *Table 4.* In this optimum design problem allowable stress of the material, allowable displacement at free nodes and the weight per unit volume are set, respectively, at 2500kgf/cm^2, 11.5cm and 7850kgf/m^3. Both stress and displacement constraints are active at the optimum solution. Slight differences in the set of optimum plate thickness t_{opt} and maximum stress σ_{max} are observed in the both optimum solutions, however, the difference in both minimum weights (the values of objective functions) W is only 0.48%. The computation time by the proposed method is 44% of that by DUAL-S.

From various numerical design examples, it has been clarified that the proposed optimum design method can determine the optimum sizes of cross section of frame structures with nonlinear materials more efficiently and reliably compared with the dual method. For the reason that the proposed optimum design methods are developed on the basis of energy principle, the design methods are unified approaches and can solve systematically the optimum design problems of truss and frame structures with any types of linear or nonlinear stress-strain relationship material.

7. An intelligent multicriteria fuzzy optimum design method for large-scale structural systems

For practical multicriteria fuzzy optimum design problems an efficient, systematic and generalized approach has been developed by using classification of design variables as common design variables and objective-oriented design variables, suboptimization concept, introduction of measure membership functions for relative evaluation of all objective functions and fuzzy decision-making techniques [23,24].

The practical prestressed concrete three-span continuous bridge system shown in *Figure 12* is considered for the design example. The bridge has a total length of 200m and a width of 14m. In this optimum design problem the total expected cost of the bridge system after an earthquake f_e to be minimised, and the aesthetics of the bridge system f_a to be maximised are considered as the primary objective functions. From the results of many previous works, it is assumed that the total expected cost f_e is evaluated by $f_e = f_t + P_f C_o$ where f_t is the total construction cost of the bridge system and C_o is the cost of demolition after collapse as well as all other expenses directly or indirectly incurred due to the failure of the bridge system. P_f is the probability of failure of the bridge system due to an earthquake. For the reason that the safety parameter ϕ_s affects the probability of failure of the bridge system after an earthquake directly, we defined the safety parameter ϕ_s as a design parameter. Since the span ratio *Sr* (l_1/l_2) and girder height at the interior support *H* significantly affect two objective functions f_e and f_a, *Sr* and *H* are dealt with as the common design variables. The other side, the prestressing forces, tendon eccentricities of parabolic prestressing and thickness of the bottom slab of box section are dealt with as the design variables in the suboptimization of the construction cost of superstructure. In the suboptimisation of the construction cost of the RC piers, the reinforcement areas in each pier segment shown in *Figure 12* are dealt with as the design variables. In the suboptimisation of the construction cost of the RC pile foundations, the numbers of RC piles in the directions of the bridge axis and its perpendicular direction, P_x and P_y, respectively, the diameter and interval of piles, *D* and *S*, in each pile foundation are dealt with as the design variables (see *Figure 12*).

The following discrete values of the common design variables *Sr* and *H*, which define the comparable ranges in the practical design problem, are considered as *Sr* = 0.5, 0.61, 0.75, 0.93 and *H* = 4.5m, 5.0m, 5.5m, 6.0m, 6.5m, 7.0m, 7.5m, 8.0m, 8.5m. For the safety parameter, following discrete values are taken into account ϕ_s = 1.0, 1.2, 1.4, 1.6, 1.8. The suboptimisation process of the superstructure is conducted for all discrete combinations of *Sr* and *H*, and that of the substructure is conducted for all discrete combinations of *Sr*, *H* and ϕ_s. Collecting the minimum total construction costs of bridge system with respect to *H* for all discrete combinations of *Sr* and ϕ_s, the relationships between minimum total construction cost of bridge system and *H* for every discrete *Sr* and ϕ_s are introduced.

We have estimated C_o in equation f_e to be 172870141 x 50 units, which is 50 times of

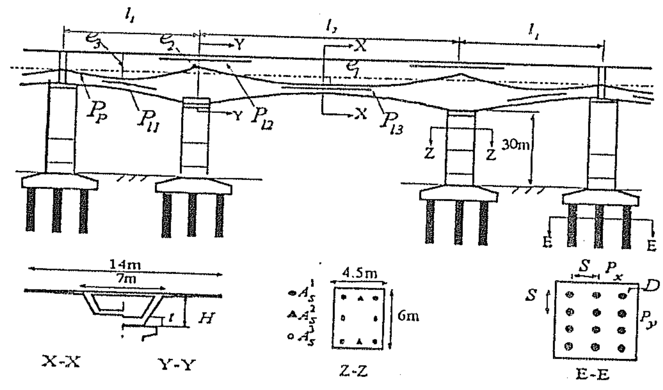


Figure 12.: A three-span continuous prestressed concrete bridge system and design variables

the average value of minimum total construction cost for all discrete combinations of Sr and H with $\phi_s = 1.0$. The minimum total expected costs of the bridge system for every discrete combinations of Sr, H and ϕ_s are arrived at by substituting the suboptimised minimum total construction cost f_t and P_f into equation f_e. By arranging the minimum total expected costs of bridge system obtained by equation f_e with respect to H for all discrete combinations of Sr and ϕ_s, the relationships between minimum total expected cost of bridge system and H for every discrete combinations of Sr and ϕ_s are introduced.

The normalised weighted factors, W_e and W_a, of the membership functions for the minimum total expected cost and the aesthetics, respectively, are assumed to be W_e=0.7 and W_a =0.3. The weighted maximum membership value $\mu_{kj,opt}$, and the corresponding optimum girder height $H_{kj,opt}$, for the k-th span ratio Sr_k and the j-th safety parameter ϕ_{sj}, is given by using a weighted operator method. By arranging the weighted maximum membership values $\mu_{k,j,opt}$ (k=1,..,4) (j=1,...,5), we obtain the relationships between the weighted maximum membership values $\mu_{k,j,opt}(H_{k,j,opt}, Sr_k, \phi_{sj})$ (k=1,...,4) and the span ratio Sr for every discrete safety parameter ϕ_{sj} (j=1,..,5). The optimum span ratio $Sr_{j,\,opt}$ is determined as $Sr_{2,\,opt}$=0.66 corresponding to a maximum weighted membership value of $\mu_{2,opt} = 0.847$. The optimum span ratios $Sr_{j,opt}$ for every discrete safety parameter $\phi_{s,j}$ (j=1,..,5) are obtained using the same process.

Arranging the maximum weighted membership values $\mu_{j,opt}$ (j=1,...,5) and the corresponding safety parameter ϕ_{sj} (j=1,...,5), we introduce the relationship between the maximum membership values $\mu_{j,opt}(\phi_{sj})$ and the safety parameter ϕ_s. The global optimum safety parameter $\phi^g_{s,opt}$ is determined as ϕ_s=1.44 with a maximum value of μ_{opt} = 0.97. The global optimum span ratio is determined by applying a backward interpolation process to the relationships between the weighted maximum membership value and the span ratio derived earlier. For $\phi^g_{s,opt}$=1.44, the two discrete ϕ_{sj}, namely $\phi_{s,3}$ =1.4 and $\phi_{s,4}$ =1.6, which are nearest to $\phi^g_{s,opt}$ give rise to the global optimum span ratio of Sr^g_{opt}=0.621. Using the derived relationships between the weighted maximum membership value and the girder height for $\phi_{s,3}$ =1.4, $\phi_{s,4}$ =1.6 and Sr_2 =0.61, Sr_3 =0.75 which are nearest to $\phi^g_{s,opt}$=1.44 and Sr^g_{opt} =0.621, the global optimum girder height $H^g_{\rm opt} = 6.92$ m is determined. The exact global optimum values of the prestressing forces, tendon eccentricities of parabolic prestressing and thickness of the bottom slab of box section for the global optimum ϕ_s, Sr and H are, respectively, determined by the suboptimization process of the superstructure and substructure.

From the numerical example it can be said that the global optimum solution can be determined rationally and systematically for any convex or nonconvex multicriteria fuzzy optimisation problem by using the proposed design method.

8. New Promising Areas

Expert systems, genetic algorithms and neural networks are new promising fields of optimization. Optimization algorithms can be built into expert system which help to find the best solution of a problem [10,11]. Neural networks can be used for function approximation. If the function-evaluation is heavy, then this approximation can be very powerful [12]. Genetic algorithms are very useful finding the solution of nonconvex optimization problems.

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