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Optimal design using genetic algorithm with nonlinear inelastic analysis

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Abstract. An optimal design method in cooperated with nonlinear inelastic analysis is presented. The proposed nonlinear inelastic method overcomes the difficulties due to incompatibility between the elastic global analysis and the limit state member design in the conventional LRFD method. The genetic algorithm used is a procedure based on Darwinian notions of survival of the fittest, where selection, crossover, and mutation operators are used to look for high performance ones among sections in the database. They are satisfied with the constraint functions and give the lightest weight to the structure. The objective function taken is the total weight of the steel structure and the constraint functions are load-carrying capacity, serviceability, and ductility requirement. Case studies of a planar portal frame, a space two-story frame, and a three-dimensional steel arch bridge are presented.

Keywords: nonlinear inelastic analysis; optimal design; genetic algorithm.

1. Introduction

In the current engineering practice, the interaction between a structural system and its members is represented by the effective length factor. The effective length method generally provides a good design of framed structures. However, despite its popular use in the past and present as a basis for design, the approach has its major limitations. The first of these is that it does not give an accurate indication of the factor against failure, because it does not consider the interaction of strength and stability between the member and structural system in a direct manner. It is well-recognized fact that the actual failure mode of the structural system often does not have any resemblance whatsoever to the elastic buckling mode of the structural system that is the basis for the determination of the effective length factor K. The second and perhaps the most serious limitation is probably the rationale of the

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current two-stage process in design: elastic analysis is used to determine the forces acting on each member of a structural system, whereas inelastic analysis is used to determine the strength of each member treated as an isolated member. There is no verification of the compatibility between the isolated member and the member as part of a frame. The individual member strength equations as specified in specifications are unconcerned with system compatibility. As a result, there is no explicit guarantee that all members will sustain their design loads under the geometric configuration imposed by the frame work.

In order to overcome the difficulties of the conventional approach, nonlinear inelastic analysis should be directly performed (Chen and Kim 1997). With the current available computing technology with advancement in computer hardware and software, it is feasible to employ nonlinear inelastic analysis techniques for direct frame design. The nonlinear inelastic analysis, based on simple refinements of the elastic-plastic hinge model, has been proposed by Chen and Kim (1997) for plane frame analysis. Nonlinear inelastic analysis methods for the three-dimensional structure have been developed by Orbison (1982), Prakash and Powell (1993), Liew and Tang (1998). In recent work by Kim *et al.* (2001), Kim and Choi (2001), the nonlinear inelastic analysis based on refinement of the elastic-plastic hinge model has been extended for the space structure.

A genetic algorithm is an optimal design technique introduced by Holland (1975). Recently, several genetic algorithms (GA) and simulated annealing (SA) have been developed by Rajeev and Krishnammorthy (1992), Lin and Hajela (1992), May and Balling (1992), and Pantelides and Tzan (1997). These methods, however, cannot verify the compatibility between the isolated member and the member as part of a frame. The individual member strength equations as specified in specifications are unconcerned with system compatibility. As a result, there is no explicit guarantee that all members will sustain their design loads under the geometric configuration imposed by the frame work.

In this paper, a genetic algorithm and a section increment method are combined with nonlinear inelastic analysis to perform an optimal design. The genetic algorithm used is a procedure based on Darwinian notions of survival of the fittest, where selection, crossover, and mutation operators are used to look for high performance ones among sections in the database. In the section increment method, a member with the largest unit value (calculated by LRFD interaction equations) is replaced one by one with an adjacent larger member selected in the database. The results of the optimal design using genetic algorithm are compared with those of the section increment method. The weight of a steel frame is taken as an objective function. Load-carrying capacity, serviceability, and ductility requirement are used as constraint functions. Although the genetic algorithm applied herein is not new, its combination with a computationally efficient advanced analysis leads to a powerful automated design program for the daily engineering design of space steel structures.

2. Practical nonlinear inelastic analysis

To capture second-order (large displacement) effects, stability functions are used to minimize modeling and solution time. Since stability functions use only one beam-column element to define the second-order effect of an individual member, they are an efficient and economical method of performing a frame analysis (Kim and Chen 1996a, Kim and Chen 1996b). The force-displacement equation using the stability functions may be extended for three-dimensional beam-column element as (Kim *et al.* 2001, Kim and Choi 2001)

$$\begin{Bmatrix} P \\ M_{yA} \\ M_{yB} \\ M_{zA} \\ M_{zB} \\ T \end{Bmatrix} = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & 0 & 0 & 0 \\ 0 & S_1\frac{EI_y}{L} & S_2\frac{EI_y}{L} & 0 & 0 & 0 \\ 0 & S_2\frac{EI_y}{L} & S_1\frac{EI_y}{L} & 0 & 0 & 0 \\ 0 & 0 & 0 & S_3\frac{EI_z}{L} & S_4\frac{EI_z}{L} & 0 \\ 0 & 0 & 0 & S_4\frac{EI_z}{L} & S_3\frac{EI_z}{L} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{GJ}{L} \end{bmatrix} \begin{Bmatrix} \delta \\ \theta_{yA} \\ \theta_{yB} \\ \theta_{zA} \\ \theta_{zB} \\ \phi \end{Bmatrix} \tag{1}$$

where P, M_{yA}, M_{yB}, M_{zA}, M_{zB} and T are axial force, end moments with respect to y and z axes and torsion, respectively. δ, θ_{yA}, θ_{yB}, θ_{zA}, θ_{zB} and ϕ are the axial displacement, the joint rotations, and the angle of twist. S_1, S_2, S_3 and S_4 are the stability functions with respect to y and z axes, respectively. The stability functions given by Eq. (1) may be written as

$$S_1 = \begin{cases} \dfrac{\pi\sqrt{\rho_y}\sin(\pi\sqrt{\rho_y}) - \pi^2\rho\cos(\pi\sqrt{\rho_y})}{2 - 2\cos(\pi\sqrt{\rho_y}) - \pi\sqrt{\rho_y}\sin(\pi\sqrt{\rho_y})} & if \quad P < 0 \\ \dfrac{\pi^2\rho_y\cosh(\pi\sqrt{\rho_y}) - \pi\sqrt{\rho_y}\sinh(\pi\sqrt{\rho_y})}{2 - 2\cosh(\pi\sqrt{\rho_y}) + \pi\sqrt{\rho_y}\sinh(\pi\sqrt{\rho_y})} & if \quad P > 0 \end{cases} \tag{2a}$$

$$S_2 = \begin{cases} \dfrac{\pi^2\rho_y - \pi\sqrt{\rho_y}\sin(\pi\sqrt{\rho_y})}{2 - 2\cos(\pi\sqrt{\rho_y}) - \pi\sqrt{\rho_y}\sin(\pi\sqrt{\rho_y})} & if \quad P < 0 \\ \dfrac{\pi\sqrt{\rho_y}\sinh(\pi\sqrt{\rho_y}) - \pi^2\rho_y}{2 - 2\cosh(\pi\sqrt{\rho_y}) + \pi\sqrt{\rho_y}\sin(\pi\sqrt{\rho_y})} & if \quad P > 0 \end{cases} \tag{2b}$$

$$S_3 = \begin{cases} \dfrac{\pi\sqrt{\rho_z}\sin(\pi\sqrt{\rho_z}) - \pi^2\rho\cos(\pi\sqrt{\rho_z})}{2 - 2\cos(\pi\sqrt{\rho_z}) - \pi\sqrt{\rho_z}\sin(\pi\sqrt{\rho_z})} & if \quad P < 0 \\ \dfrac{\pi^2\rho\cosh(\pi\sqrt{\rho_z}) - \pi\sqrt{\rho_z}\sinh(\pi\sqrt{\rho_z})}{2 - 2\cosh(\pi\sqrt{\rho_z}) + \pi\sqrt{\rho_z}\sinh(\pi\sqrt{\rho_z})} & if \quad P > 0 \end{cases} \tag{2c}$$

$$S_4 = \begin{cases} \dfrac{\pi^2\rho_z - \pi\sqrt{\rho_z}\sin(\pi\sqrt{\rho_z})}{2 - 2\cos(\pi\sqrt{\rho_z}) - \pi\sqrt{\rho_z}\sin(\pi\sqrt{\rho_z})} & if \quad P < 0 \\ \dfrac{\pi\sqrt{\rho_z}\sinh(\pi\sqrt{\rho_z}) - \pi^2\rho_z}{2 - 2\cosh(\pi\sqrt{\rho_z}) + \pi\sqrt{\rho_z}\sin(\pi\sqrt{\rho_z})} & if \quad P > 0 \end{cases} \tag{2d}$$

where $\rho_y = P/(\pi^2 EI_y/L^2)$, $\rho_z = P/(\pi^2 EI_z/L^2)$ and P is positive in tension.

The Column Research Council(CRC) tangent modulus concept is used to account for gradual yielding (due to residual stresses) along the length of axially loaded members between plastic hinges. From Chen and Lui (1986), the CRC E_t is written as

$$E_t = 1.0E \qquad \text{for} \quad P \le 0.5P_y \tag{3a}$$

$$E_t = 4\frac{P}{P_y}E\left(1 - \frac{P}{P_y}\right) \qquad \text{for} \quad P > 0.5P_y \tag{3b}$$

where P_y is yield force.

The tangent modulus model is suitable for the member subjected to axial force, but not adequate for cases of both axial force and bending moment. A gradual stiffness degradation model for a plastic hinge is required to represent the partial plastification effects associated with bending. When softening plastic hinges are active at both ends of an element, the force-deflection equation may be expressed as

$$\begin{Bmatrix} P \\ M_{yA} \\ M_{yB} \\ M_{zA} \\ M_{zB} \\ T \end{Bmatrix} = \begin{bmatrix} \frac{E_t A}{L} & 0 & 0 & 0 & 0 & 0 \\ 0 & k_{iiy} & k_{ijy} & 0 & 0 & 0 \\ 0 & k_{ijy} & k_{jjy} & 0 & 0 & 0 \\ 0 & 0 & 0 & k_{iiz} & k_{ijz} & 0 \\ 0 & 0 & 0 & k_{ijz} & k_{jjz} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{GJ}{L} \end{bmatrix} \begin{Bmatrix} \delta \\ \theta_{yA} \\ \theta_{yB} \\ \theta_{zA} \\ \theta_{zB} \\ \phi \end{Bmatrix} \tag{4}$$

where

$$k_{iiy} = \eta_A\left(S_1 - \frac{S_2^2}{S_1}(1-\eta_B)\right)\frac{E_t I_y}{L} \tag{5a}$$

$$k_{ijy} = \eta_A \eta_B S_2 \frac{E_t I_y}{L} \tag{5b}$$

$$k_{jjy} = \eta_B\left(S_1 - \frac{S_2^2}{S_1}(1-\eta_A)\right)\frac{E_t I_y}{L} \tag{5c}$$

$$k_{iiz} = \eta_A\left(S_3 - \frac{S_4^2}{S_3}(1-\eta_B)\right)\frac{E_t I_z}{L} \tag{5d}$$

$$k_{ijz} = \eta_A \eta_B S_4 \frac{E_t I_z}{L} \tag{5e}$$

$$k_{jjz} = \eta_B\left(S_3 - \frac{S_4^2}{S_3}(1-\eta_A)\right)\frac{E_t I_z}{L} \tag{5f}$$

The terms η_A and η_B is a scalar parameter that allows for gradual inelastic stiffness reduction of the element associated with plastification at end A and B. This term is equal to 1.0 when the element is elastic, and zero when a plastic hinge is formed. The parameter η is assumed to vary according to the parabolic function:

$$\eta = 1.0 \qquad \text{for} \qquad \alpha \leq 0.5 \tag{6a}$$

$$\eta = 4\alpha(1 - \alpha) \qquad \text{for} \qquad \alpha > 0.5 \tag{6b}$$

where α is a force-state parameter that measures the magnitude of axial force and bending moment at the element end. The term α may be expressed by AISC-LRFD and Orbison *et al*., respectively (Kim *et al*. 2000). A three dimensional column and a six-story steel frame were verified to show validity of the nonlinear inelastic analysis presented above (Kim and Choi 2001).

Geometric imperfection should be modeled in analysis. For braced frames, member out-of-straightness, rather than frame out-of-plumbness, needs to be used for geometric imperfections. The geometric imperfection of L_c/1,000 is adopted, where L_c is length of column. For unbraced frames, frame out-of-plumbness needs to be used for geometric imperfections. The geometric imperfection of L_c/500 is adopted. For the detail of the geometric imperfection refer to ECCS (1984 1991), AS (1990), CSA (1989, 1994), and AISC (1993).

3. Optimal design

3.1 Genetic algorithm

The genetic algorithm was introduced by Holland (1975) at the Michigan university. It is based on the theory of natural evolution and the law of genetic. Fig. 1 shows a procedure of the genetic algorithm. First, the control parameters are selected by designers. The parameters are composed of number of chromosome, crossover rate, mutation rate, number of maximum generation, number of individual, and number of population. The genetic algorithm generates populations using the selected control parameter. A population denoting the whole section types of a structure is composed of individual representing one section type of a structure. A population of the first generation is selected by evaluating fitness of the initial populations. Then, the new populations of the next generation are reproduced by manipulating the chromosomes of the selected individuals. The chromosome is the binary number of '0' or '1' whose string is the individual. Crossover and mutation operators are applied to the reproduced populations. The crossover and mutation rate for obtaining good convergence are determined by designers. A population of the second generation is selected by evaluating fitness of the populations. This procedure is repeated by reaching the number of maximum generation.

Fig. 2 shows an optimal design procedure using the genetic algorithm. First, the sections as the first generation are selected. The total weight, serviceability, and load-carrying capacity of the structure composed of the selected sections are evaluated. If the structure is not satisfied with the serviceability or the load-carrying capacity, the sections of the second generation are assigned. If the structure of the new sections is satisfied with the serviceability and the load-carrying capacity of the structure, the weight of the structure composed of the new sections is reserved as the base weight.

Next, the new members of the third generation are assigned. If the structure of the third sections is

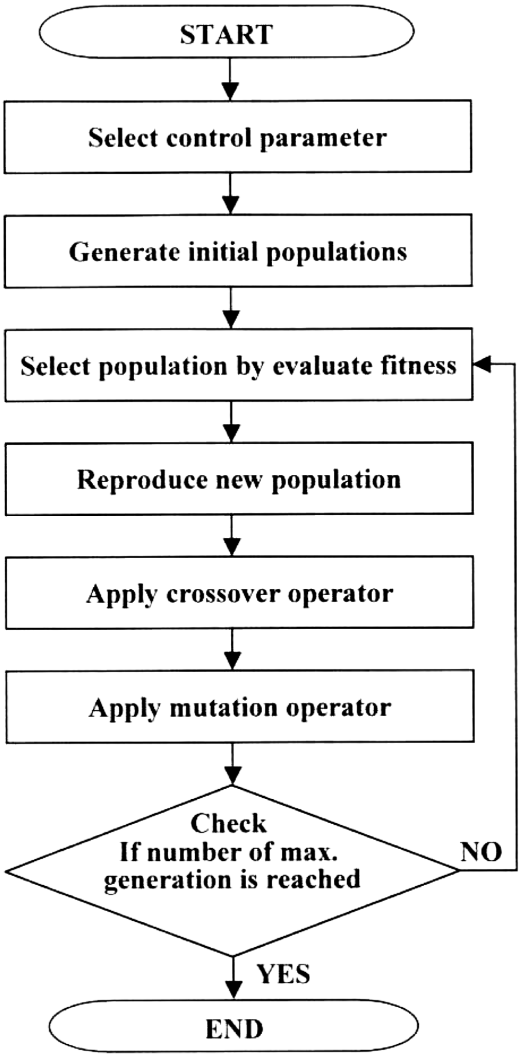

Fig. 1 Procedure of genetic algorithm

satisfied with the serviceability and the load-carrying capacity, and the weight of the structure composed of the sections is lighter than that of the structure assigned by the second generation, the weight of the third generation is reserved as the new base value. If the structure of the third sections are not satisfied with the objective function, the serviceability, or the load-carrying capacity of the structure, the forth sections are selected. These routines are repeated by reaching the number of maximum generation assigned by the designer. Finally, the lightest sections satisfying the serviceability and the load-carrying capacity are selected.

3.2 Section increment method

Fig. 3 shows a procedure of the section increment method. The lightest sections are assigned to the initial members. If the initial members are not satisfied with the constraint functions of the structure,

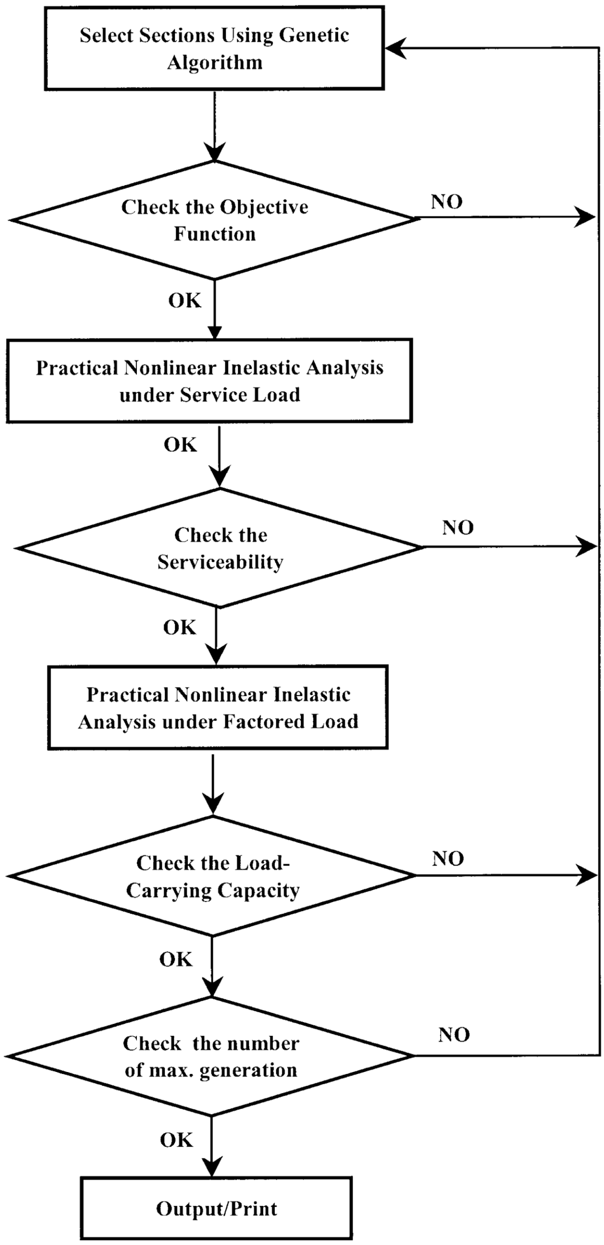

Fig. 2 Optimal design procedure using genetic algorithm in cooperated with nonlinear inelastic analysis

they are increased one by one according to the following two rules. In the first rule, the shallowest section among the various ones with the same weight is selected in order to minimize the difference of strength and stiffness in weak and strong axis. For example, W10X30, W12X30, and W14X30 are the sequence to be selected according to the rule. In the second rule, if the same weight section does not exist, a slightly heavier one is selected. If W14X30 selected by the first rule is not satisfied with the constraint functions, it is replaced by W8X31 according to the second rule. If W8X31 is not satisfied

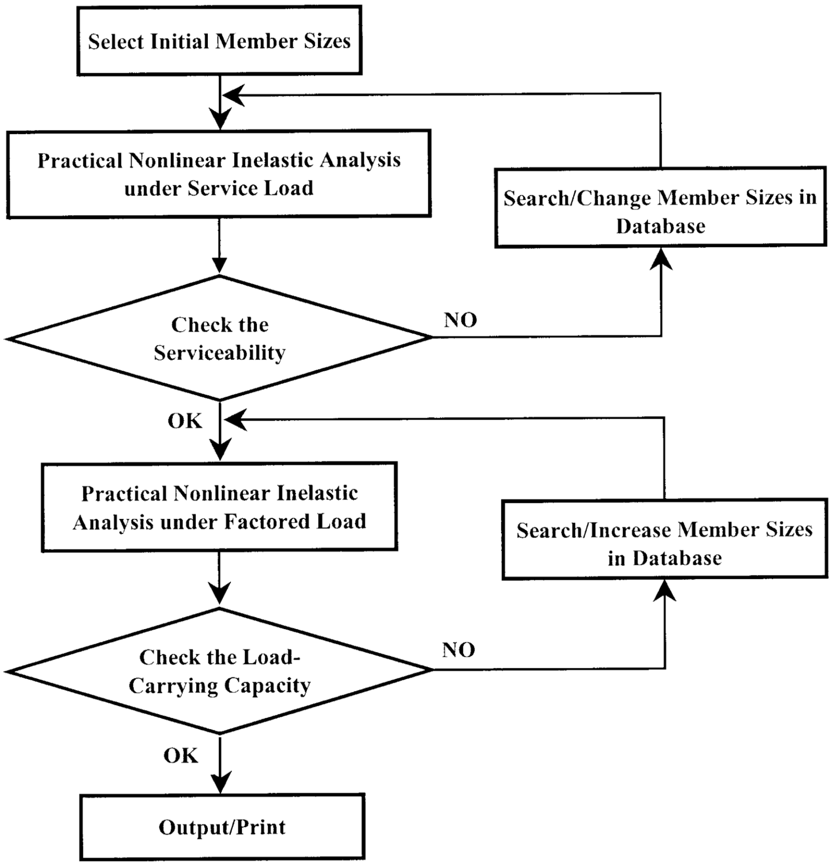

Fig. 3 Optimal design procedure using section increment method in cooperated with nonlinear inelastic analysis

with the constraint functions, it is replaced by W16X31 corresponding to the first rule.

Nonlinear inelastic analysis is performed in order to check the serviceability of the structure subjected to service loads. If the beam is not satisfied with the constraint function of deflection, it is replaced with an adjacent larger one selected in the database. If the column is not satisfied with the constraint function of inter-story drift, it is replaced with an adjacent larger one. This routine is repeated until the serviceability conditions are satisfied. Next, nonlinear inelastic analysis is performed in order to check the load-carrying capacity of the structure subjected to factored loads. The value a of each member is calculated by the LRFD interaction Eqs. (7a) and (7b). A member with the largest value a is replaced one by one with an adjacent larger member selected in the database. The database used in this paper embodies W shape listed in the AISC-LRFD Specification. This routine is repeated until the load-carrying capacity of a structural system is equal to or greater than the factored load.

$$\alpha = \frac{P}{\phi_c P_n} + \frac{8}{9}\frac{M_y}{\phi_b M_{ny}} + \frac{8}{9}\frac{M_z}{\phi_b M_{nz}} \qquad \text{for} \qquad \frac{P}{\phi_c P_n} \geq 0.2 \tag{7a}$$

$$\alpha = \frac{P}{2\phi_c P_n} + \frac{M_y}{\phi_b M_{ny}} + \frac{M_z}{\phi_b M_{nz}} \qquad \text{for} \qquad \frac{P}{\phi_c P_n} < 0.2 \tag{7b}$$

where ϕ_c and ϕ_b are resistance factors for compression and flexure. P, M_y and M_z are axial force, end moments with respect to y and z axes, respectively. M_{ny} and M_{nz} are nominal flexure strength. In this paper, the plastic moment as nominal flexure strength is used because a compact section and adequate lateral supports are assumed. P_n is nominal axial compressive strength determined as

$$P_n = 0.658^{\lambda_c^2} F_y A \qquad \text{for} \quad \lambda_c \le 1.50 \tag{8a}$$

$$P_n = \frac{0.877 F_y A}{\lambda_c^2} \qquad \text{for} \quad \lambda_c > 1.50 \tag{8b}$$

for which λ_c is

$$\lambda_c = \frac{KL}{\pi r}\sqrt{\frac{F_y}{E}} \tag{9}$$

where F_y is yield stress. A and L are the gross cross sectional area and the unbraced length. r is the radius of gyration about the plane of buckling. E is Young's modulus. K is the effective length factor calculated approximately by Dumonteil (1992):

For the braced frame

$$K = \frac{3G_A G_B + 1.4(G_A + G_B) + 0.64}{3G_A G_B + 2.0(G_A + G_B) + 1.28} \tag{10a}$$

For the unbraced frame

$$K = \sqrt{\frac{1.6G_A G_B + 4.0(G_A + G_B) + 7.5}{G_A + G_B + 7.5}} \tag{10b}$$

in which G_A and G_B are the column-to-beam stiffness ratios at the column ends.

3.3 Objective function

The objective function taken is the weight of a structure, which is expressed as

$$OBJ = \rho\left(\sum_{i=1}^{NB}(V_b)_i + \sum_{j=1}^{NC}(V_c)_j\right) \tag{11}$$

where ρ is the unit weight. NB and NC are the total number of beams and columns, respectively. $(V_b)_i$ and $(V_c)_j$ are the volume of the i-th beam and the j-th column.

3.4 Constraint function

Load-carrying capacities, serviceability, and ductility requirement are used as constraint functions.

3.4.1 Load-carrying capacity

Nonlinear inelastic analysis follows the format of Load and Resistance Factor Design. In AISC-

LRFD (1993), the factored load effect does not exceed the factored nominal resistance of structure. Two kinds of factors are used: one is applied to loads, the other to resistances. The LRFD has the format

$$\sum \gamma_k Q_k \leq \phi R_n \tag{12}$$

where R_n is nominal resistance of the structural member, Q_k is force effect, ϕ is resistance factor, and γ_k is load factor corresponding to Q_k.

The main difference between the current LRFD method and the nonlinear inelastic analysis method is that the right side of Eq. (12), ϕR_n in the LRFD method is the resistance or strength of a component member of a structural system, but in the nonlinear inelastic analysis method, it represents the resistance or the load-carrying capacity of the whole structural system. In the nonlinear inelastic analysis method, the load-carrying capacity is obtained from applying incremental loads until a structural system reaches its strength limit state such as yielding or buckling. The left-hand side of Eq. (12), $\sum \gamma_k Q_k$ represents the member forces in the LRFD method, while it presents the applied load on the structural system in the nonlinear inelastic analysis method.

The proposed method evaluates a system-level resistance which is different from AISC-LRFD specification doing member level resistance factors. AISC-LRFD (1993) specifies the resistance factors of 0.85, 0.90, and 0.90 for compression, tension, and flexural strength of a member, respectively. When a structural system collapses by member buckling, the resistance factor of 0.85 is used since the dominant behavior is compression. When a structural system of a frame collapses by member yielding, the resistance factor of 0.9 is used since the dominant behavior is tension. When a structural system of a frame collapses by forming plastic mechanism, the resistance factor of 0.9 is used since the dominant behavior is flexure. AASHTO-LRFD(1998) specifies the resistance factors of 0.90, 0.95, and 1.0 for compression, tension, and flexural strength, respectively. The constraint function associated with load-carrying capacity is written as

$$G(1) = \phi R - \sum \gamma_k Q_k \geq 0 \tag{13}$$

where ϕR is the load-carrying capacity of a structure, and $\sum \gamma_k Q_k$ is the applied factored load on the structure.

3.4.2 Serviceability

Based on the studies by the ASCE Ad Hoc Committee (1986) and by Ellingwood (1989), the interstory drift limit for story height (H) is selected as $H/300$ for wind load. The deflection limit for girder with the span length (L) is selected as $L/360$ for service load. The constraint functions of interstory drift and deflection are written as

$$G(2) = \frac{H_i}{300} - (\Delta_{cv})_i \geq 0 \tag{14}$$

$$G(3) = \frac{L_i}{360} - (\Delta_{bv})_i \geq 0 \tag{15}$$

where $G(2)$ and $G(3)$ are constraint functions on limiting of interstory drift and deflection, respectively. H_i and $(\Delta_{cv})_i$ are the story height and the interstory drift of the i-th story. L_i and$(\Delta_{bv})_i$ are the length and the deflection of the i-th beam. The serviceability limit for steel bridges with the span length (L) is equal to $L/800$ for vehicular load and $L/1000$ for pedestrian load in AASHTO-LRFD Specification(1998).

3.4.3 Ductility requirement

Adequate rotation capacity is required for members to develop their full plastic moment capacity. This is achieved when members are adequately braced and their cross-sections are compact. The limits for lateral unbraced lengths and compact sections are explicitly defined in AISC-LRFD (1993). Based on AISC-LRFD Table B5.1, the constraint functions of ductility requirement are written as

$$G(4) = \frac{300 r_{yi}}{\sqrt{F_y}} - L_{bi} \geq 0 \tag{16}$$

$$G(5) = \frac{65}{\sqrt{F_y}} - \left(\frac{b_f}{2t_f}\right)_i \geq 0 \tag{17}$$

$$G(6) = \frac{640}{\sqrt{F_y}} - \left(\frac{h_c}{t_w}\right)_i \geq 0 \tag{18}$$

where $G(4)$ is a constraint function on limitation of unbraced length to avoid lateral instability. $G(5)$ and $G(6)$ are constraint functions on limitation of width-thickness ratio for flanges and web to avoid local buckling. r_{yi} and L_{bi} are radius of gyration about the weak axis and unbraced length of i-th member, respectively. b_f and t_f are the width and thickness of flange, respectively. t_w is the thickness of web. h_c is the depth of web clear of fillets. F_y is the yield stress of steel.

The limits for lateral unbraced lengths and compact sections for steel bridges are explicitly defined in AASHTO-LRFD. Based on AASHTO-LRFD, the constraint functions of ductility requirement are written as

$$G(4) = \left[0.124 - 0.0759\left(\frac{M_l}{M_p}\right)\right]\left[\frac{r_y E}{F_y}\right] \geq L_b \tag{19}$$

$$G(5) = 0.382\sqrt{\frac{E}{F_y}} \geq \frac{b_f}{2t_f} \tag{20}$$

$$G(6) = 3.76\sqrt{\frac{E}{F_y}} \geq \frac{2D_{cp}}{t_w} \tag{21}$$

where $G(4)$ is a constraint function on limitation of unbraced length to avoid lateral instability. $G(5)$ and $G(6)$ are constraint functions on limitation of width-thickness ratio for flanges and web to avoid local buckling. M_l is the lower moment due to the factored loading at either end of the unbraced length. M_p is plastic moment. r_y and L_b are minimum radius of gyration and unbraced length, respectively. b_f, t_f, and t_w are flange width, flange thickness, and web thickness, respectively. D_{cp}, E, and F_y are web depth, Young's modulus, and the yield stress of steel, respectively.

4. Design example

4.1 Planar portal frame

Fig. 4 shows a planar portal frame. The population, i.e., the group of the section types, is shown in

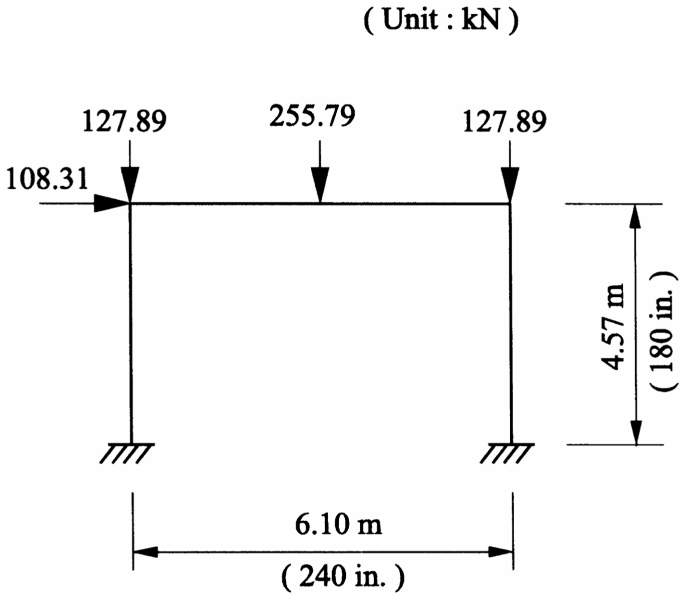

Fig. 4 Planar portal frame

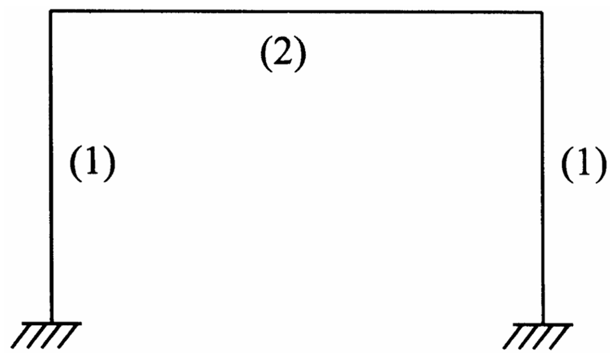

Fig. 5. Group of section types of planar portal frame

Fig. 5. The yield stress used was 250 MPa (36 ksi) and Young's modulus was 200,000 MPa (29,000 ksi). Design loads were dead load of 54.75 kN/m (3.75 kips/ft), live load of 36.50 kN/m (2.50 kips/ft), and wind load of 83.3 kN (18.72 kips). The load factors used were 1.2 for dead load (D), 0.5 for live load (L), and 1.3 for wind load (W). Factored design loads were converted into equivalent concentrated loads as shown in Fig. 4. The out of plumbness was assumed to be $\psi = 2/1{,}000$.

In case of the genetic algorithm, the number of individual, i.e., the number of section type of the planar portal frame used was two. The crossover rate from 0.1 to 1.0, the mutation rate from 0.1 to 1.0, and the number of population from 5 to 50 were tested. The good convergence was shown in using the crossover rate of 0.2, the mutation rate of 0.9, and the number of group of 10. The number of maximum generation used was 5000. To use large number of maximum generation can reduce the total weight of the structure during optimal design process. The computational efficiency is not deteriorated by analyzing the updated structure only when it is lighter than a current lightest weight of the structure. In case of the section increment method, the initial member sizes used were W4×13, one of the lightest sections. The member sizes were increased one by one according to the two rules mentioned before until the structure was satisfied with the constraint functions.

Table 1 shows the comparison of the optimal designs using the genetic algorithm and the section increment method. The frame encountered ultimate state when the applied load ratios reached 1.24 for the genetic algorithm and 1.26 for the section increment method. Since the frame collapsed by forming plastic mechanism, the system resistance factor of 0.9 was used. The ultimate load ratios (λ) resulted in 1.116 (=1.24 × 0.9) in the genetic algorithm and 1.134 (=1.26 × 0.9) in the section increment method, which were greater than 1.0, and the section sizes of the system were adequate. The structure designed by using the genetic algorithm was 23.5% lighter than that by the section increment method. The genetic algorithm can provide better optimal design results since it investigates the wide range of design variable whereas the section increment method does not.

The lateral drifts for the wind loads of $1.0W$ were calculated as $H/383$ in case of the genetic algorithm and $H/695$ in the section increment method. These met the inter-story drift limit of $H/300$. The deflections of the beam for the service live loads of $1.0L$ were calculated as $H/1091$ and $H/1139$ in the genetic algorithm and the section increment method, respectively. These satisfied the deflection limit of $L/360$. The load-displacement curves of the planar portal frame using the genetic algorithm (GA) and the section increment method (SIM) are shown in Fig. 6. The sequences of the formation of plastic hinges in the planar portal frame are shown in Fig. 7.

Table 1 Optimal design of planar portal frame

Design variable	Genetic Algorithm	Section Increment Method
	Section sizes	Section sizes
1	W18×35	W24×62
2	W16×45	W18×35
Total weight	1,960 lb (8,718 N)	2,563 lb (11,401 N)
Load-carrying capacity	1.24	1.26

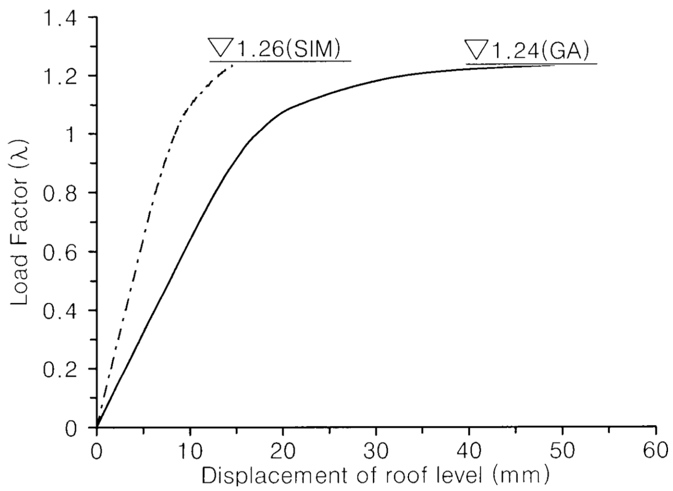

Fig. 6 Load-displacement of planar portal frame

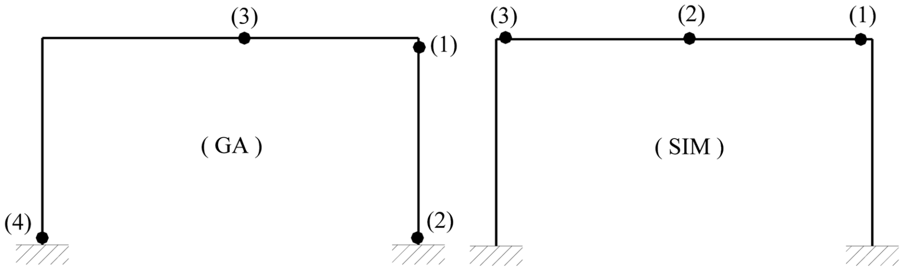

Fig. 7 Sequences of the formation of plastic hinges in the planar portal frame

4.2 Space two-story frame

Fig. 8 shows a space two-story frame. The group of the section types is shown in Fig. 9. The yield stress used was 250 MPa (36 ksi) and Young's modulus was 200,000 MPa (29,000 ksi). The load conditions were given as follows: (1) Roof loads : Dead load = 10.8 kN/m^2 (225 psf) and Live load = 7.2 kN/m^2 (150 psf). (2) Floor loads : Dead load = 13.2 kN/m^2 (275 psf) and Live load = 12.0 kN/m^2 (250 psf). (3) Wind load = 7.2 kN/m^2 (150 psf). The load combination used was 1.2D+0.5L+1.3W. Factored design loads were converted into equivalent concentrated loads as shown in Fig. 8. The out of plumbness was assumed to be $\psi = 2/1{,}000$.

In case of the genetic algorithm, the number of individual, i.e., the number of section type of the space two-story frame used was ten. The crossover rate of 0.2, the mutation rate of 0.9, and the number of group of 10 were used. The number of maximum generation used was 20000. In case of the section increment method, initial member sizes used were W4 × 13, one of the lightest sections.

Table 2 shows the comparison of the optimal designs by using the genetic algorithm and the section increment method. The frame encountered ultimate state when the applied load ratios reached 1.93 in the genetic algorithm and 1.19 in the section increment method. The system resistance factor of 0.85

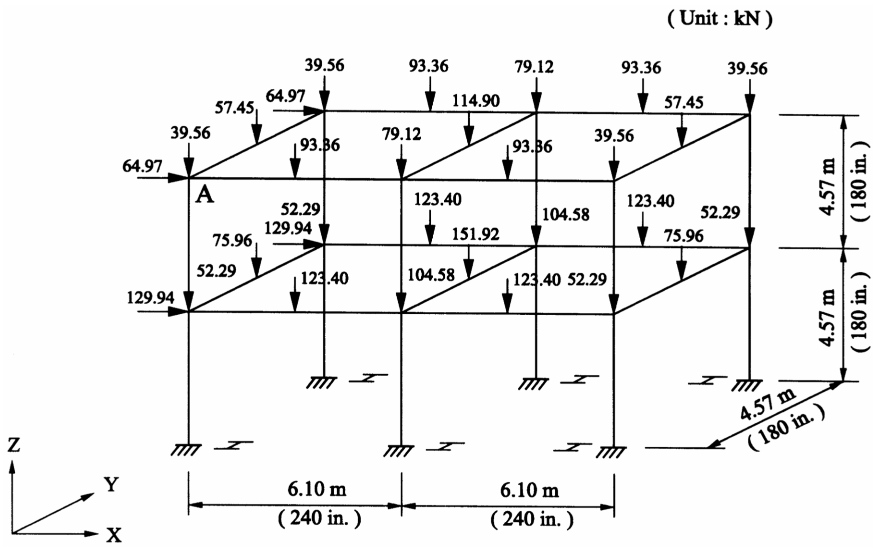

Fig. 8 Space two-story frame

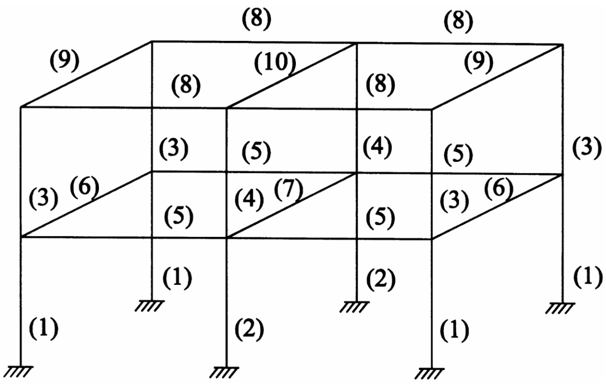

Fig. 9 Design variables of space two-story frame

Table 2 Optimal design of space two-story frame

Design variable	Genetic Algorithm	Section Increment Method
	Section sizes	Section sizes
1	W14×145	W40×192
2	W16×67	W40×192
3	W10×112	W40×167
4	W12×65	W40×192
5	W24×68	W16×31
6	W16×31	W12×14
7	W12×30	W12×22
8	W10×68	W14×22
9	W8×18	W8×13
10	W12×30	W12×16
Total weight	32,727 lb (145,576 N)	38,792 lb (172,555 N)
Load-carrying capacity	1.93	1.19

was used since the frame collapsed by buckling. The ultimate load ratios (λ) resulted in 1.641 (=1.93×0.85) in the genetic algorithm and 1.012 (=1.19×0.85) in the section increment method, which were greater than 1.0, and the section sizes of the system were adequate. The structure designed by using the genetic algorithm was 15.6% lighter than that by the section increment method. The lateral drifts for the wind loads of $1.0W$ were calculated as $H/331$ in case of the genetic algorithm and $H/351$ in the section increment method. These met the inter-story drift limit of $H/300$. The deflections of the beam for the service live loads of $1.0L$ were calculated as $H/1900$ and $H/1814$ in the genetic algorithm and the section increment method. These satisfied the deflection limit of $L/360$. The load-displacement curves of the two-story frame using the genetic algorithm (GA) and the section increment method (SIM) are shown in Fig. 10. The sequences of the formation of plastic hinges in the space two-story frame are shown in Fig. 11.

4.3 Three dimensional steel arch bridge

Fig. 12 shows a steel arch bridge, which is 7.32 m (24 ft) wide and 61.0 m (200 ft) long. The group of

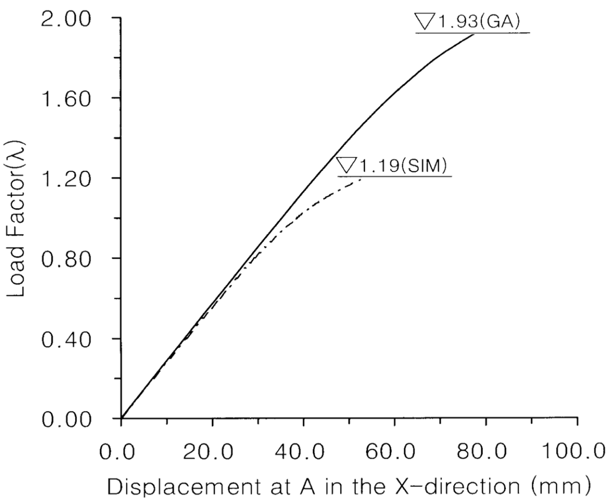

Fig. 10 Load-displacement of space two-story frame

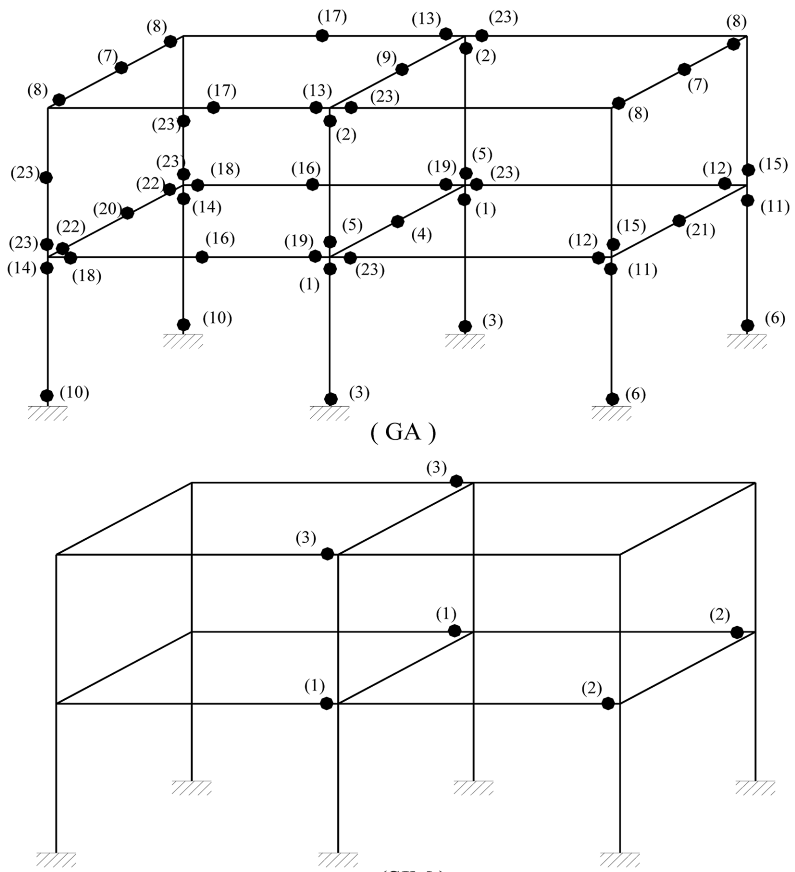

Fig. 11 Sequences of the formation of plastic hinges in the space two-story frame

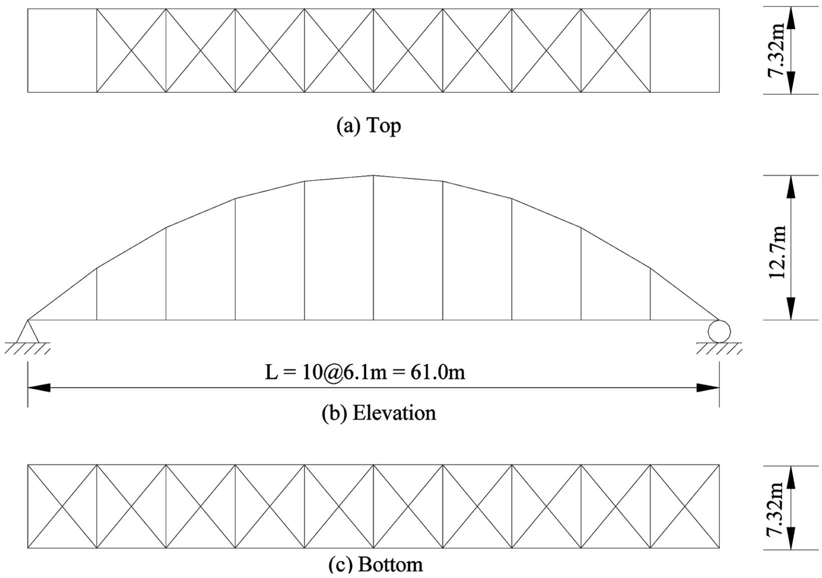

Fig. 12 3-D steel arch bridge

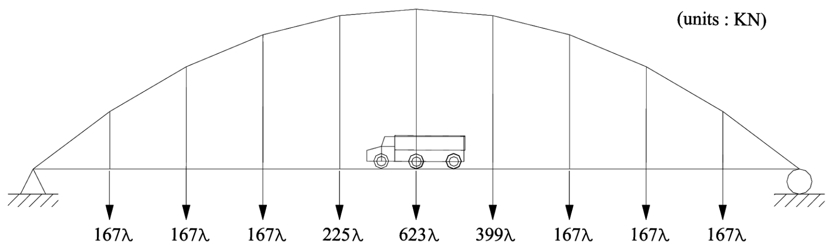

Fig. 13 Factored load

the section types for arch rib, tie, vertical member, and bracing were assigned to 1, 2, 3, and 4, respectively. The yield stress used was 250 MPa (36 ksi) and Young's modulus was 200,000 MPa (290,000 ksi). The dead load, live load, and impact load specified in AASHTO-LRFD (1998) were considered as design loads. The concentrated dead loads and live loads of HS-20 were applied on each joint. The load factors of 1.25 for the dead load, 1.75 for the live load, and 0.30 for the impact load were used. Fig. 13 shows the design load considering the load factor.

In case of the genetic algorithm, the number of individual, i.e., the number of the section type of the 3-D steel arch bridge used was four. The crossover rate of 0.2, the mutation rate of 0.9, and the number of group of 10 were used. The number of maximum generation used was 2000. Table 3 shows the optimal design by using the genetic algorithm. It is shown that the optimal design process using the genetic algorithm works well in the 3-D steel arch bridge which is composed of truss and frame members. The comparison between genetic algorithm and the section increment method for 3-D steel arch bridge is not regarded as necessary any more since the genetic algorithm gives much better results than the section increment method in the examples of 2-D and 3-D frames. The steel arch bridge encountered ultimate state when the applied load ratio reached 1.39. The system resistance factor of

Table 3 Optimal design of 3-D steel arch bridge

Design variable	Genetic Algorithm
	Section sizes
1	TS20×12×5/8
2	W27×84
3	W16×57
4	W6×12
Total weight	141,227 lb (628,206 N)
Load-carrying capacity	1.39

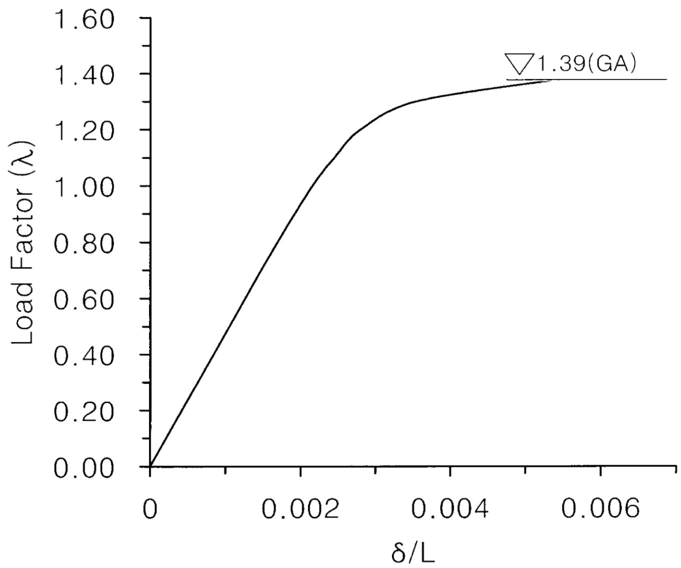

Fig. 14 Load-dispalcement of 3-D steel arch bridge at midspan

1.00 was used since the frame collapsed by forming plastic mechanism. The ultimate load ratio (λ) resulted in 1.39 (=1.39 × 1.00), which was greater than 1.0 and the member sizes of the system were adequate. The maximum deflection by the service load was calculated as 42 mm (1.66 in) at mid-span. The deflection ratio was L/1452, which satisfied the deflection limit of L/800. The load-displacement curve of the 3-D steel arch bridge is shown in Fig. 14.

5. Conclusions

In this paper, an optimal design using the genetic algorithm and the section increment method in cooperated with a nonlinear inelastic analysis is developed. The summaries and conclusions of this study are as follows.

1. The proposed method can practically account for all key factors influencing behavior of frame members and truss members: gradual yielding associated with flexure; residual stresses; and geometric nonlinearity.

2. The practical nonlinear inelastic analysis overcomes the difficulties due to incompatibility between the elastic analysis of the structural system and the limit state member design in the conventional LRFD method.
3. The genetic algorithm and the section increment method in cooperated with nonlinear inelastic analysis are used for optimal design. The objective function considered is the weight of the structure, and the constraint functions considered are load-carrying capacity, serviceability, and ductility requirements.
4. The planar portal frame designed by using the genetic algorithm was 23.5% lighter than that by the simple section increment method. The space two-story frame designed by using the genetic algorithm was 15.6% lighter than that by the section increment method. The genetic algorithm can provide better optimal design results since it investigates the wide range of design variable whereas the section increment method does not.
5. It is shown that the optimal design process using the genetic algorithm works well in the 3-D steel arch bridge which is composed of truss and frame members.
6. The practical nonlinear inelastic analysis and the genetic algorithm are combined for optimal design. This contribution will provide much benefit to practicing engineering.

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