Optimization of Eccentrically Braced Frame (EBF) Steel Structure Using Genetic Algorithm-SAP2000

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Abstract— **The optimization procedure for designing a steel frame is presented in this paper. The steel structure of a three storey and single portal bay eccentrically braced frame (EBF) is analyzed with the propose method. The objective function of the optimization is minimizing weight subjected to three constraints while AISC 2005 is used as preference in structural steel analysis. Nonlinear static analysis also performed to determine the ductility of optimized structure. The final result of optimization is structure with total weight of 4984 kg and ductility of 3.1. It is concluded that GA-SAP2000 can optimize EBF structure with three constraints.**

Keywords- optimization; weight; steel structure; eccentric braced frame; genetic algorithms; AISC 2005

I. INTRODUCTION

SAP2000 program is one of the finite element analysis tool which already used for analyzing and modeling the structure based on some relevant code such as AISC-LRFD 2005 code. In this code, EBF steel structure is planned to meet several criteria such as planning link, diagonal brace, beam outside link, and columns [1, 2].

Genetic algorithm (GA) itself which is a member of the Evolutiossnary Algorithms (EA) is one approach to determine the global optimum is based on Darwin's theory [3, 4]. Common operators used in GA are initialization of population, evaluate population, selection, mating, crossover, mutation, stopping criterion and get results [5].

Since the commercial FEM (finite element method) program such as SAP2000 and genetic algorithms can be combined to obtain the structure automatically, then this combination would be good if it is used to solve optimization problems [5].

After obtained the results of the optimization process will be performance nonlinear static analysis to determine the ductility of the structure.

II. THEORIES

A. *Nonlinear Static Analysis*

Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. With the increase in the magnitude of the loading, weak links and failure modes of the structure are found. The loading is monotonic with the effects of the cyclic behavior and load reversals being estimated by using a modified monotonic force-deformation criteria and with damping approximations. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design [6].

The ATC-40 and FEMA 356 documents have developed modeling procedures, acceptance criteria and analysis procedures for pushover analysis. These documents define criteria for hinges used in this method. Two points labeled A, B, C, D and E are used to define force deflection behavior of the hinge and three points labeled IO, LS, and CP are labeled to define the acceptance criteria (IO, LS and CP stand for Immediately Occupation, Life Safety and Collapse Prevention, respectively) (see fig. 1). The FEMA 356 code is used in this paper for finding the target displacement of analyzed structures.

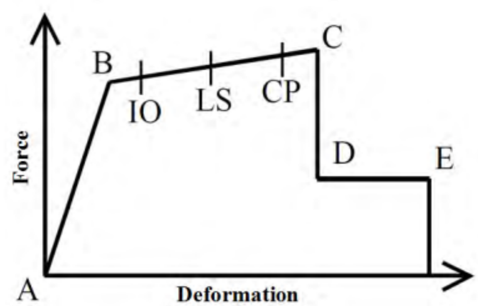

Figure. 1. Force-deformation for pushover hinge [6].

B. Eccentrically braced frames (EBF)

EBF is expected to withstand significant inelastic deformations in the links when subjected to the forces resulting from the motions of the design earthquake. The diagonal braces, columns, and beam segments outside of the links shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strainhardened links, except where permitted in this Section. EBF shall meet the requirements in Section 15 AISC-LRFD 2005 [7].

C. Failure Mechanisn Concept

Failure mechanism is expected in the structure if not unavoidable is the top of the first to collapse before the bottom had collapsed. So the bottom of column, beam, and brace planned stronger than the above. Strength elements can be seen from the large value of the cross-section area (A), profile's depth (H), elasticity modulus (E) and plastic modulus (Z) (with the same yield strength) [8].

D. SAP2000

SAP2000 program is a finite element analysis tool which already used for analyzing and modeling structure. SAP2000 could process or import the file input with extension MDB, XLS, TXT and SDB. SAP2000 also could export analysis result and design to files with extension XLS, TXT and SDB. After input file being opened, SAP2000 will run analysis, save the results and design of all members and create output file. From the output file, the required data such as stress and displacement along the frame as an indicator for the acceptance criteria can be obtained [5, 8 & 9].

E. GA and SAP2000

Since the structure is simple, GA procedures are processed in Single PC. Optimization problems are solved by using combination of SAP2000 and simple GA (see fig. 2) [5, 8 & 9].

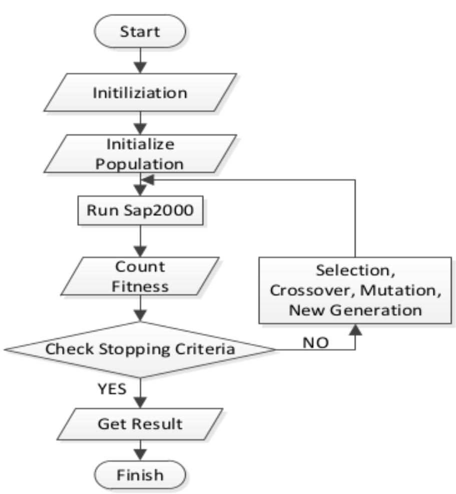

Figure. 2. Flowchart GA-SAP2000[5, 8 & 9].

After initial population is created, the program commands PC to 1) run SAP2000, 2) analyze input files, 3) design the input files, 3) close SAP2000. Each input file must have one output file. The message is to let PC to evaluate and calculate fitness value of each output file (see Fig. 2). Raw data for of drift calculation are taken from "Joint Displacements" table. Data for stress constraint calculation are taken from "Steel Design 1 – Summary Data AISC360-05-IBC2006" Sap2000 output file table. This iteration is processed until the generation reach 300. The specific generation number is used as the stopping criteria.

III. MATERIALS AND METHODS

Portal is optimized in this study based on cases in chapter 7.3.10 EBF design example in the book "Ductile Design of Steel Structures" and divided into nine elements (fig. 4). ASTM A572 Grade 50 steel with a specified yield strenght of 345 Mpa (50 ksi) was used for the beams, columns, and links. ASTM A500 Grade B square structural tube manufactured to a specified yield stength of 320 Mpa (46 ksi) was used for the eccentric braces. The portal works loads such as: gravity unfactored concentrated live loads (PL) of 100 kN (22,5 kips) and dead loads (PD) of 250 kN (56,2 kips) are applied to each column at each level, and uniformly distributed dead loads (QD) of 15 kN/m (1,11 kips/ft) and live loads (QL) 10 kN/m (0,74 kips/ft) are applied along the beams. Unfactored lateral seismic loads (PQ3, PQ2, and PQ1) of 259 kN (58 kips), 172,5 kN (39 kips), and 86,5 kN (19,4 kips) are aplied at the third, second, and first floor level of the frame (see fig. 3 for details).

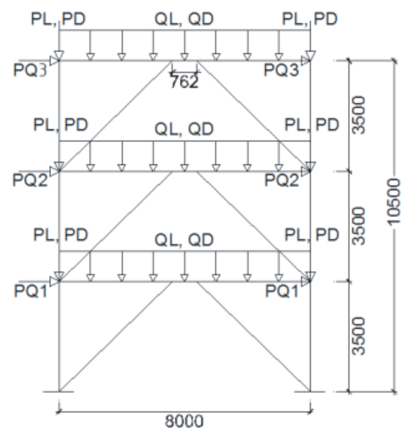

Figure. 3. Steel structure model and loading [10].

266 types of WF profile used for the beams and column, and 332 types of HSS profiles used for fence as available profiles are taken from SAP2000 database in this optimization. With a review of the 9 elements total possible configurations that can be generated is $6^{266} \times 3^{332}$.

The objective function is to minimize the weight of structure subjected to three constraints (stress constraint, displacement constraint, and and configuration of the profile properties):

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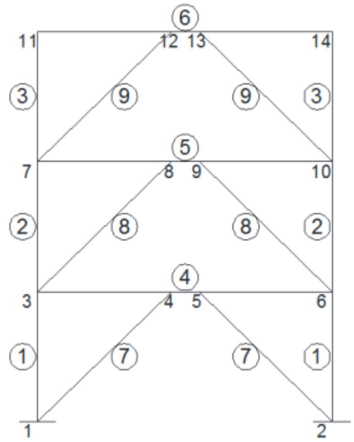

Figure. 4. Grouping of the portal [11].

$$Objfunc = Min(\sum \rho_i A_i L_i)(1+\sum re_i + \sum rj_j + \sum ct3_j + \sum bt3_k + \sum brt3_l + \sum cbt2_j + \sum bbrt2_m) \quad (1)$$

i = all element number (1 to 9), j = joint number at outside column (3, 7, and 11), k = beam element number (4, 5, and 6), l = brace element number (7, 8, and 9), m = joint number at brace (4, 8, and 12).

Where *Objfunc* is objective function, ρ is a unit weight, A is an area of cross section, L is a length of element, c = column, b = beam , br = brace, t3 = outside height (WF) or outside depth (HSS) profile, t2 = top flange width (WF) or outside width (HSS) profile, re_i is element capacity constraint, $re_i=0$ if $status_i$="No Messages"and $re_i=2^2$ if $status_i\neq$"No Messages", rj_j is displacement constraint, $rj_j=0$ if $drift_i<0{,}00875$and $rj_j=drift_i^2$ if $drift_i>0{,}00875$, $ct3_j$ is outside height profile column constraint, $ct3_j=0$ if $Rct3_j<1$and $ct3_j= Rct3_j^2$ if $Rct3_j>1$, $bt3_k$ is outside height profile beam constraint, $bt3_k=0$ if $Rbt3_k<1$and $bt3_k= Rbt3_k^2$ if $Rbt3_k>1$, $brt3_l$ is outside depth profile brace constraint, $brt3_l=0$ if $Rbrt3_l<1$and $brt3_l= Rbrt3_l^2$ if $Rbrt3_l>1$, $cbt2_j$ is top flange width profile of beam and column constraint, $cbt2_j=0$ if $Rcbt2_j<1$and $cbt2_j= Rcbt2_j^2$ if $Rcbt2_j>1$, $bbrt2_m$ is top flange width profile of beam and brace constraint, $bbrt2_m=0$ if $Rbbrt2_m<1$and $bbrt2_m= Rbbrt2_m^2$ if $Rbbrt2_m>1$.

For displacement constraint, the interstory drift is limited to 0,0025 times the storey height. For stress constraints, the column status in the output file SAP2000 "Steel Design 1 - Summary Data - AISC360-05-IBC2006" should contain "no message".

For the outside height profile column constraint, ratio at every joint must under 1:

$$Rct3_j = \frac{t3\ column}{t3\ of\ column\ at\ lower\ column} < 1 \quad (2)$$

For the outside height profile beam constraint, ratio must under 1:

$$Rbt3_k = \frac{t3\ beam}{t3\ of\ beam\ at\ lower\ beam} < 1 \quad (3)$$

For the outside height profile brace constraint, ratio must under 1:

$$Rbrt3_l = \frac{t3\ brace}{t3\ of\ brace\ at\ lower\ brace} < 1 \quad (4)$$

For the top flange width of beam and column constraint, ratio must under 1:

$$Rcbt2_j = \frac{t2\ beam}{t2\ of\ column\ on\ same\ storey} < 1 \quad (5)$$

For the top flange width profile of beam and brace constraint, ratio must under 1:

$$Rbbrt2 = \frac{t2\ brace}{t2\ of\ beam\ on\ the\ same\ storey} < 1 \quad (6)$$

Where,

t3 = outside height (WF) or outside depth (HSS) profile

t2 = top flange width (WF) or outside width (HSS) profile

GA process is carried out with the parameters: 50 individuals, 300 generations, 0,8 crossover, mutation 0,005, crosses a cut point, the elitism of 25% and use the rest of the roulette wheel selection. Nonlinear static analysis is then used to test the optimized structure.

IV. RESULTS AND DISCUSSION

Optimization is successfully done with combination of GA and SAP2000. Individual results are obtained with the best fitness value from 300th generation with the weight of the structure is 4984,35 kg. The result profiles are shown in Table 1 and Fig. 7. Drift happened at the first storey is $6{,}256\times10^{-3}$ m. The second storey drift is $6{,}313\times10^{-3}$ m. And the third storey drift is $5{,}355\times10^{-3}$ m. All drift do not exceed the specified limit ($8{,}750\times10^{-3}$ m).

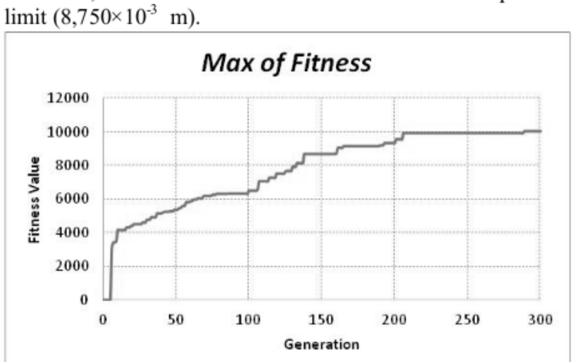

Figure. 5. Graph the highest fitness value of each generation.

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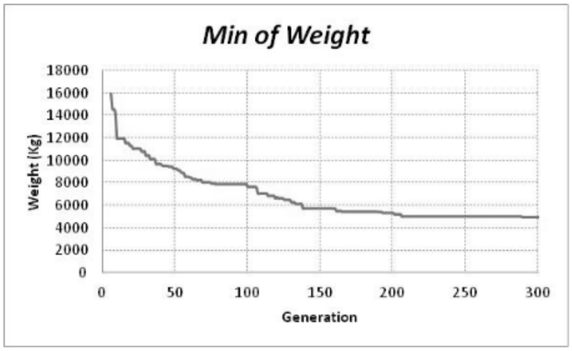

Figure. 6. Graph the lowest weight of each generation.

Table. 1. Optimization result

Tipe	Frame	Profil
Column	1	W16X77
	2	W12X50
	3	W12X45
Beam	4	W12X50
	5	W12X50
	6	W10X45
Brace	7	HSS10X8X.3125
	8	HSS10X8X.3125
	9	HSS7X7X.375

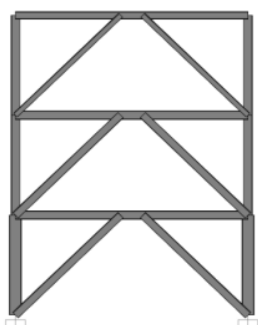

Figure. 7. Cross-sectional profile of portal.

Ratio of properties column profile between the floors, shown in fig. 8.

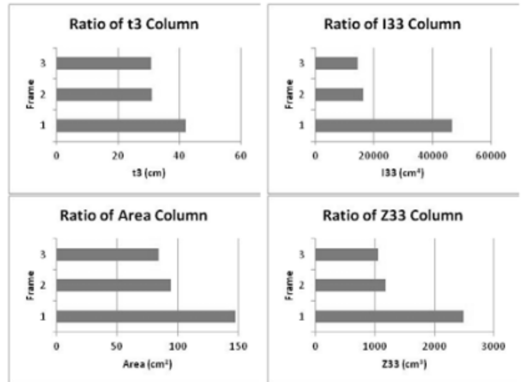

Figure. 8. Ratio of properties column profile.

In Fig. 8 it is shown that the higher the floor, column properties (t3, area, i33, and Z33) will be equal or smaller.

Ratio of properties beam profile between the floors is shown in fig. 9.

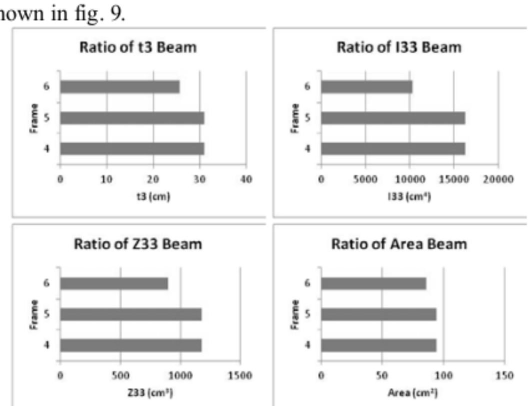

Figure. 9. Ratio of properties beam profile.

From fig. 9 it is shown that the higher the floor, beam properties (t3, area, i33, and Z33) are equal or smaller.

Ratio of properties brace profile between the floor, shown in fig. 10.

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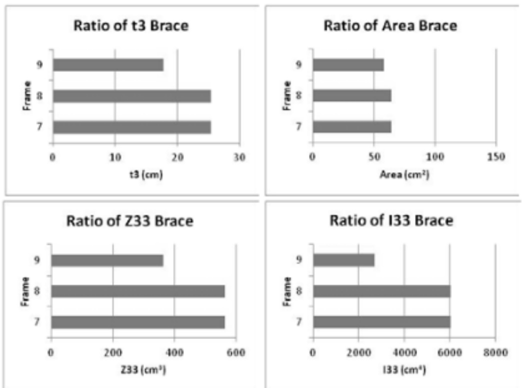

Figure. 10. Ratio of properties brace profile.

From Fig. 10 it is shown that the higher the floor, brace properties (t3, area, i33, and Z33) will be equal or smaller.

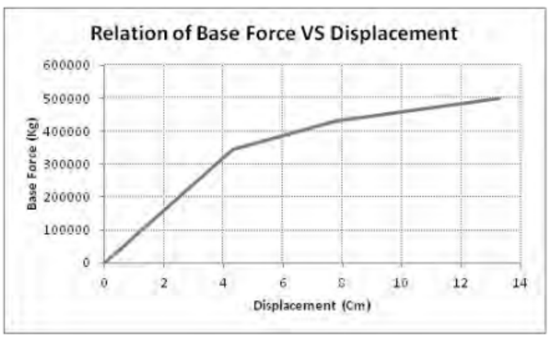

Figure. 11. Pushover curve.

Ductility is ratio between the maximum drift of the building structure as it reaches the condition of the verge of collapse (Δm) and drift of the structure when the first yielding in the building structure (Δy). The value of ductility (Δm/Δy) shown in fig. 11 is 3,16.

V. CONCLUSION

Optimization process with three constraints (stress elements, interstory drift, and cross-sectional configuration properties) is carried out successfully with weight is 4984 kg and ductility is 3,16, it is based on after the control of all constraints that used none exceeded. In the control elements of the structure is based on the AISC LRFD 2005, met all the requirements needed from the absence of portal elements that have overstressed, the absence of rotation of the link that exceed the limits, the absence of the element slenderness limit, all the elements required in the category (link profile and column category seismically compact, brace profile category compact), and others. Drift in all floors are not exceeding limitations. Likewise with height and width of profile configuration in all elements used in accordance with constraints. With use the height and width profile constraint are also available area, i33, and Z33 is uniform during the optimization process is complete (300 generations).

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