Framework to Explore the Design Space for Design of Tall Buildings

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ABSTRACT

Design of tall buildings is undergoing a resurgence that is driven by a variety of factors – economical growth, scarcity of land in urban areas, high land costs, increased population density, technological advancements and man's desire to build taller structures. Considerable research work has been done in the last two decades to meet this demand. Computer-based tools that help design engineers explore design alternatives are indispensable in tackling this complex problem. In addition, a framework that finds the near optimal design, adds value to this exploratory work. In this paper, we develop a general framework for the design optimization of buildings using sizing, shape, and topology design variables. Sizing optimization can be carried out using discrete design variables (from a database of available sections) or continuous design variables (cross-sectional dimensions of custom wide flange sections). Similarly, shape optimization can be carried out using either discrete or continuous design variables. And finally, topology optimization can be carried out using boolean design variables. Allowable stress design guidelines are used as constraints along with displacement, inter-story drift, total structural weight, and frequency constraints. The finite element model is made of three-dimensional beam elements. A typical function evaluation involves a linear, static analysis with multiple load cases, a linear, modal analysis to extract the lowest few eigenpairs, and a linear, buckling analysis to find the buckling capacity. An optimization toolbox that contains gradient-based and population-based optimizers, is a part of the framework. Numerical results how that the framework is capable of producing efficient designs effectively.

KEYWORDS

Tall buildings; finite element analysis; design optimization; sizing, shape and topology optimization.

INTRODUCTION

The demand for tall buildings continues to grow fueled by economic growth, scarcity of land in urban areas, high land cost, technological advances in materials, construction techniques and design principles, growing population in megacities, and above all, the ego to build the tallest building. From 2000 to 2013, the total number of 200-meter-plus buildings completions in existence increased from 261 to 830 – an astounding 318 percent (Safarik and Wood, 2014). The challenges in building higher and taller at a more economical level with due considerations to usability, safety and sustainability have kept alive the research interests in tall buildings design. Ali and Moon (2007) discuss the evolution of tall building's systems and the technological driving force behind tall building developments. They categorize interior structures as rigid frames, braced hinged frames, shear wall/hinged frames, shear wall-frame interaction system and outrigger structures, and exterior structures – tube, diagrid, space truss structures, spaceframes and exo-skeleton. Fawzia and Fatima (2010) discuss the use of belt truss and outrigger systems in controlling deflections. A three-dimensional finite element model of a 60-storey composite building is used to investigate the performance (deflection control) of the building subjected to wind loads. One, two and three outrigger levels are used to show that significant reduction in lateral deflections and inter-story drifts can be obtained in comparison to a model without any outrigger system. A method for determination of preliminary member sizes in the context of diagrid systems is presented in Moon et al. (2007). The developed methodology is used to size the diagrid members in building ranging from 20 to 60 stories. With an aspect ratio of 7, for 60-story diagrid structures, the optimal range of diagrid angle is from about 65^0 to 75^0. With an aspect ratio of 5, for 42-story buildings, the range is lower by 10^0 indicating that the angle is a function of the building dimensions. At the preliminary as well as the detailed design stages, the use of a designer guided system is indispensable. Design optimization tools provide such a system helping take over the mundane task of finding the best possible design once the problem formulation is set and the basic finite element and design models are established. Several researchers have addressed design optimization of building systems – cost-revenue conceptual design of high-rise buildings (Grierson and Khajehpour, 2002), multiple design criteria (Ng and Lam, 2005), evolutionary methods (Manickarajah et al., 2000; Kameshki and Saka, 2001; Kicinger et al., 2005), optimality criteria (Chan and Chui, 2005), optimum design of steel structures with openings using sizing, shape and topology design variables (Lagaros et al., 2006), numerical design optimization tools (Baker et al., 2008), preliminary design optimization (Jayachandran, 2009), and topology optimization (Liang et al., 2000; Stromberg et al., 2012). Through the use of principal stress trajectories, evolutionary structural optimization and gradient-based and population-based techniques, conceptual development of innovative structural and architectural topologies can take place [Baker et al., 2008]. Principal stress trajectories are described in terms of partial differential equations and then solved to yield diagrid systems. In another approach, gradient-based method is used to find the optimal shape of a tall building by minimizing the displacement at the top of the building when subjected to constant wind pressure. The structural elements are assumed to be located only on the exterior face of the tower when height, enclosed volume and base are constrained. For verification, a similar problem is also solved using genetic algorithm where the best solution is similar to that obtained by the gradient-based technique. Topology optimization provides another means of finding a better design. Liang et al. (2000) developed a methodology for finding the optimal layout of members in a bracing system subjected to multiple lateral loading conditions. The objective function is the weight and the performance constraint places an upper limit on the mean compliance of the structure. The unbraced (skeletal) structure is modeled with beam elements and the continuum between the beams and columns are modeled with plane stress elements. The final continuum topology is used as a guide for placing the bracing elements. Similarly, Stromberg et al. (2012) address the task of finding an efficient planar lateral bracing system via topology optimization that is based on combining beam and continuum finite elements where, though unstated, the final topology is a reasonable guess for the exploration of a diagrid system.

The main objective of this paper is to develop a framework for the optimal design of tall buildings. The framework allows for investigating the design efficiencies of different building systems, e.g. rigid frames, belt truss and outrigger systems, diagrid systems etc. The design framework starts with a general design problem formulation that allows for finding the sizing variables (essentially cross-sectional dimensions and properties), shape variables (location of structural members and joints), and topology variables (presence, absence and location of members). The performance and serviceability constraints include normal stress, shear stress, displacement, inter-story drifts, buckling, and natural frequencies. An optimization toolbox is tightly integrated with a function and gradient evaluation system that includes a finite element analysis system and computer code to compute the function and gradient values. The problem formulation and the evaluation of function and gradient values are discussed in the next section. This is followed by several design examples that show how the framework is used to investigate the design space.

OPTIMAL DESIGN PROBLEM FORMULATION

Design problems involving buildings can be posed several different ways. The formulations are motivated by the need to find optimal solutions at the preliminary design stage so that an exhaustive search of the design possibilities can take place accurately and efficiently. The objective is to design buildings with m stories having n structural steel members, assigned to p cross-section property groups $\left(p \le n\right)$. The skeletal framework is assumed to be made of steel.

Design Problem Formulation – Minimum Weight Design (MWD)

This is a minimum weight design problem and the problem is posed as follows.

Find $$\mathbf{x} = \left\{\mathbf{x}_c, \mathbf{b}\right\} \tag{1}$$

Minimize $$W\left(\mathbf{x}\right) = \sum_{i=1}^{n} A_i L_i \gamma_i \tag{2a}$$

Subject to $$\sigma_{\max,i}^{t,c} \le \sigma_a^{t,c} \quad (i = 1, 2, ...n) \tag{3}$$

$$\tau_{\max,i} \le \tau_a \quad (i = 1, 2, ...n) \tag{4}$$

$$\left(D_i\right)_{\max}^{T} \le D_a \tag{5}$$

$$\frac{D_j - D_{j-1}}{h_j} \le \left(D_{ij}\right)_a \quad (j = 1, 2, ..., m) \tag{6}$$

$$\frac{\pi^2 EI}{(kL)^2} \le (P_{cr})_a \tag{7}$$

$$\lambda^B \ge \lambda_a^B \tag{8}$$

$$\mathbf{x}_c^L \le \mathbf{x}_c \le \mathbf{x}_c^U \qquad (c = 1,2,..,p) \tag{9a}$$

$$b_l \in \{0,1\} \qquad (l = 1,2,...,r) \tag{9b}$$

where *W* is the total weight of all the structural elements, γ_i is the weight density of material, L_i is the length and A_i is the cross-sectional area of member *i*. Eqns. (3-4) are used to impose stress constraints where $\sigma_{\max,i}^t$, $\sigma_{\max,i}^c$, $\tau_{\max,i}$ are the maximum tensile, compressive and shear stress, and the subscript *a* denotes the allowable value. Eqn. (5) defines the constraints imposed on the maximum lateral drift $(D_i)_{\max}^T$ in longitudinal and transverse directions of the building. Eqn. (6) defines the inter-story drift constraints for the structure where D_j and D_{j-1} are the drifts of j^{th} and $(j-1)^{th}$ story respectively and h_j is the height of j^{th} story. Buckling constraints are imposed via Eqn. (7) in the form of Euler buckling and via Eqn. (8) in the form of overall buckling of the structure where $(P_{cr})_a$ is the allowable buckling capacity of the member and λ^B is the lowest eigenvalue from the buckling eigenvalue problem and λ_a^B is the allowable value. Eqn. (9a) is used to denote either discrete design variables (selected from a predetermined table of cross-sectional shapes) or continuous design variables as explained later. Eqn. (9b) denotes boolean design variables that are used to select or deselect bracing members $(r < n)$.

Finite Element Analysis
Finite element analysis is used during function evaluation necessary to compute the objective function and constraints. Three sets of linear algebraic and eigenproblems are solved as follows.

$$\mathbf{K}_{d\times d}\mathbf{D}_{d\times lc} = \mathbf{F}_{d\times lc} \tag{10}$$

$$\mathbf{K}_{d\times d}\mathbf{\Phi}_{d\times d} = \mathbf{\Lambda}_{d\times d}\mathbf{M}_{d\times d}\mathbf{\Phi}_{d\times d} \tag{11}$$

$$\mathbf{K}_{d\times d}\,\phi_{d\times 1} = \lambda^B \mathbf{K}_{d\times d}^{\sigma}\phi_{d\times 1} \tag{12}$$

where $\mathbf{K}_{d\times d}$, $\mathbf{M}_{d\times d}$ and $\mathbf{K}_{d\times d}^{\sigma}$ are the structure stiffness matrix, mass matrix and geometric stiffness matrix respectively. In addition, *d* is the total number of degrees-of-freedom in the finite element model, *lc* is the number of load cases, λ^B is the buckling load factor for the lowest mode. Eqn. (11) is typically solved in a smaller space as $\hat{\mathbf{K}}_{q\times q}\mathbf{\Phi}_{q\times q} = \mathbf{\Lambda}_{q\times q}\hat{\mathbf{M}}_{q\times q}\mathbf{\Phi}_{q\times q}$ since only the lowest few *q* eigenpairs are of interest $(q \ll d)$. Further explanations involving the problem formulation are presented next.

Performance Constraints
Performance-based design includes strength and serviceability requirements as discussed below.

Stress Constraints: Allowable Stress Design (ASD) requirements are imposed where the requirement is that the allowable strength of each structural component equals or exceeds the required strength. As per AISC Specification (2005), the allowable tensile/compressive stress for gross steel cross section is $0.6 f_y$ and the allowable shear stress for gross steel cross section is $0.4 f_y$ where f_y is the yield strength of the steel material. In the finite element analysis, the magnitude of the maximum beam element stresses are computed conservatively as follows (x-y-z denote the longitudinal axis and the two transverse directions, respectively) at the two ends and at the quarter-points of each beam finite element.

Normal stress:

$$\sigma_{\max}^{t} = \max\left(\frac{N_x}{A} + \frac{|M_y|}{S_y} + \frac{|M_z|}{S_z}, 0\right) \tag{13}$$

$$\sigma_{\max}^{c} = \min\left(\frac{N_x}{A} - \frac{|M_y|}{S_y} - \frac{|M_z|}{S_z}, 0\right) \tag{14}$$

Shear stress:

$$\tau^{y} = \frac{|V_y| Q_y}{I_y t_y} \qquad \tau^{z} = \frac{|V_z| Q_z}{I_z t_y} \qquad \tau^{T} = \frac{|T_x|}{T_J} \tag{15}$$

$$\tau_{\max} = \max\left\{\tau^{y} + \tau^{T}, \tau^{z} + \tau^{T}\right\} \tag{16}$$

where $\left\{A, S_y, S_z, Q_y, Q_z, t_y, t_z, I_y, I_z, T_J\right\}$ are the cross-sectional properties and dimensions, i.e. area, section moduli, first moments of the area, widths resisting shear, moments of inertia and torsional constant, respectively, $\{N_x, V_y, V_z\}$ are the normal and shear forces in the element's local x, y, z directions, and $\{T_x, M_y, M_z\}$ are the torsional and bending moments in the element's local x, y, z directions.

Displacement Constraints: Two types of displacement constraints are imposed – Eqns. (5) and (6). First, the displacements in the two transverse directions are limited to 1/600 to 1/400 of the total building height [ASCE, 1998]. Second, the inter-story drift is another serviceability criterion for design requirements is taken to be less than 1/500 of the story height [Ng and Lam, 2005].

Buckling Constraints: In addition of the strength requirements imposed via stress constraints, in performance based designs, structural instability must be prevented. Member buckling is controlled via the imposition of Euler buckling constraint (Eqn. (7)). Buckling behavior of the structure is determined by solving the eigenproblem shown in Eqn. (12) where λ^{B} is the buckling load factor that needs to be much greater than 1 to prevent buckling under the action of the applied loads, i.e. all load cases. Other instabilities such as buckling of flanges, torsional buckling etc. are not considered.

Design Variables

Selection of the design variables help define the problem as sizing, shape and/or topology design optimization problem [Rajan, 1995].

Sizing Variables: To meet the high strength requirements of the beams, braces and columns, we assume that custom sections are needed. Using continuous design variables, each element cross section is taken as a custom wide flange section and the web height of the section is used as the primary design variable. A group of heavy 130 wide flange sections from the AISC database was examined for generating relationships among the four dimensions of a typical wide flange section - web height, h_w, flange width, w_f, web thickness, t_w, and flange thickness, t_f. A sample relationship is shown in Fig. 1.

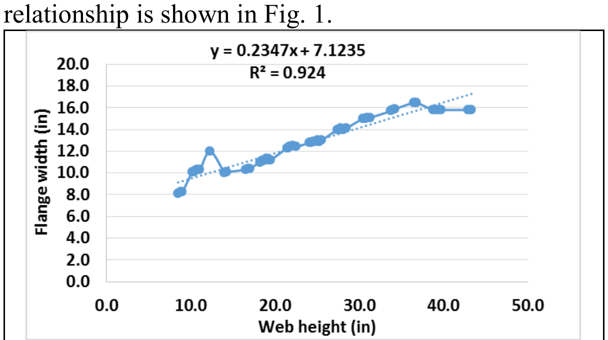


Figure 1. Flange width versus web height relationship

The linear relationships obtained from this exercise are as follows (the R^2 values for the three fits are 0.924, 0.956, 0.553) [Sirigiri et al., 2015]

$$w_f = 0.235 h_w + 7.12 \tag{17}$$

$$t_f = 0.492 t_w + 0.106 \tag{18}$$

$$t_f = 0.0406 w_f + 0.118 \tag{19}$$

where the implied units are inches. The low R^2 value for the flange thickness and flange width relationship is due to the wide range of available flange thicknesses (0.4"-0.8") for a given flange width (8.0"-17.0"). It should be noted that other commonly used cross-sectional shapes can be easily incorporated in the problem formulation, and that the wide-flange sections are used in this study to readily illustrate the design concepts.

Topology Variables: Diagonal bracing members are often used to brace steel frameworks to maintain lateral drifts within acceptable limits. The optimal layout design of bracing systems is a challenging task for structural designers because it involves a large number of possibilities for the arrangement of the members. In the absence of an efficient optimization technique, the selection of lateral bracing systems for multi-story steel frameworks is undertaken by a designer based on a trial-and-error process or previous design experiences. In topology optimization of a structure, boolean design variables are assigned to structural elements. The value of ***1*** for the variable implies that the element exists in the structure and ***0*** implies that element is removed from the structure [Rajan, 1995].

Finite Element Analysis

The FE analysis necessary to compute the function and gradient values in the problem formulations are carried out using the GS-USA© Frame3D program [Sirigiri and Rajan, 2013]. A small strain, small displacement, linear elastic finite element analysis is carried out by the computer program. The responses from the FE analysis that can be used in the problem formulations include nodal displacements, beam element nodal forces, beam element maximum tensile, compressive and shear stresses, thin plate/shell stresses and structural lowest frequencies. The frequency analysis is carried out using the stiffness and mass of the structural elements plus the mass of the nonstructural elements and a fraction (25%) of the live loads.

Numerical Optimization Techniques

The design optimization toolbox used for optimal design has several optimization techniques that can be invoked depending on the problem type.

Gradient-based Techniques: Gradient-based techniques are particularly useful when designing with continuous design variables and continuous and differentiable objective and constraint values. In particular, the Method of Feasible Directions (MFD) [Rajan et al., 2006] is used in this study. Typical problems with about 25-50 design variables can be solved in about 10-15 iterations involving less than a hundred function evaluations and about 10-15 gradient evaluations. The active set strategy is used in order to make the storage space and computations efficient.

Population-Based Global Optimization Techniques: When the design variables are not continuous (e.g. discrete, boolean), or the objective and constraint functions are not differentiable, or if there is a need to find several distinct local optima and possibly, the global optimum, population-based techniques are desirable. In particular, the Genetic Algorithm is used in this study [Rajan, 1995; Rajan and Nguyen, 2004]. Typical problems involving 25-50 design variables require several hundred function evaluations to yield high quality solutions.

In the next section, details of a case study are presented showing how the developed design optimization methodology is used for the design of tall buildings.

NUMERICAL RESULTS

The numerical results are obtained by executing the developed program, GS-USA© Frame3D, on a Dell Precision Workstation T5400 with Intel Xeon E5440-2.83 GHz processor, 8 GB RAM running Windows 7-64 bit operating system.

Design Optimization with Planar Frame Models

The developed design methodology is tested using a 40-story building that has a fs ft by s ft rectangular floor layout (Fig. 2) where s is the spacing between the frames and f is the number of frames in the (strong) y-direction. The height of the first floor containing the lobby is 16 ft. All the other floor heights are 13 ft. The total height of the building is 523 ft. The floor system consists of a composite metal deck slab (3" cellular steel deck with 2.5" concrete slab), supported on the steel joists. All the degree-of-freedom at the bottom of the columns are restrained. All the models with bracing elements considered in this study are assumed to be symmetrical.

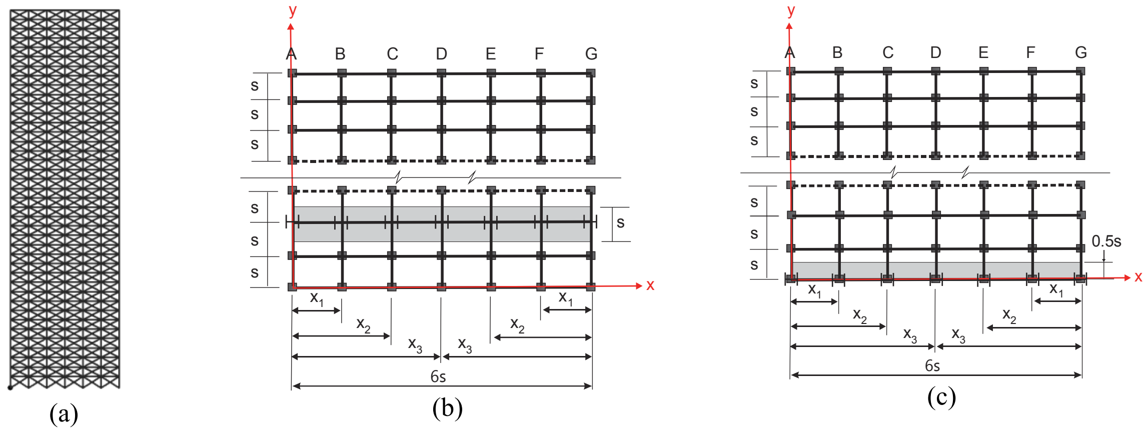


Figure 2. (a) Initial 40-floor planar frame layout (b) Typical floor plan (x-y plane; z: gravity direction) showing interior frame (c) End frame

In the rest of this section, the frame shown in Fig. 2(a) and corresponding to the interior frame with the shaded area in Fig. 2(b), is used during design optimization. Three different frames are considered with $s = 20', 25', 30'$ and are labeled 120-WF, 150-WF and 180-WF frames. There are 287 nodes, 1000 beam finite elements and 840 effective degrees-of-freedom in the initial planar finite element model.

Material Properties and Loads

The steel columns, beams and bracings of the building are assumed to be of grade A992/A992M. The floor slabs and elevator shaft walls are assumed to be of high strength reinforced concrete. Table 1 summarizes the material properties.

Table 1. Material Properties

Material	Structural elements	Mass Density	Elastic Modulus
Steel, Grade 992/A992M	Columns, beams, and bracings	$15.24\, slug/ft^3$	$29000\, ksi$
Concrete	Slab, curtain walls & elevator shaft walls	$0.39\, slug/ft^3$	$4600\, ksi$

The dead loads are due to the steel deck and floor finishes. Assuming that the building is for office use, the live load on the floors and roof are as per the specification in Table 4-1, ASCE-7-10 [ASCE, 2010]. Wind pressure loads are computed as per Chicago City Code Table 16 (13-52-310). Tributary area concept is used to compute the loading on the planar frame. Table 2 summarizes the key loading values.

Table 2. Load Values

Location	Item	Load
Floors	Dead load	144 psf
Floors	Live load	100 psf
Columns (ground level-top floor)	Wind load	20-31 psf

Allowable Stress Design (ASD) Load Combinations

In ASD, the combinations of service loads are evaluated for maximum stresses and compared to allowable stresses. ASCE-7-2010 combinations of loads are shown below.

1. LC 1: Dead load
2. LC 2: Dead load + Live load
3. LC 3: Dead load + 0.6 Wind Load
4. LC 4: Dead load + 0.75 Live Load + 0.75(0.6 Wind Load)

Design Variables

The floors were grouped together into 20 groups, 2 floors per group. The cross section properties of columns, beams and bracing members are varied along the height so that the properties are the same in any group. In this study, the two end columns and the interior columns are not treated differently and are assumed to have the same properties in a particular floor group. The web height of the wide flange section assigned for each of these groups is defined as a continuous sizing design variable with an upper limit as a function of the story height. Twenty belt trusses were created along the height of the planar frame by grouping the bracings of two floors together. A boolean value of ***1*** for L_1 indicates that the bracing exists along the entire floor of the building. The locations of the interior columns (along B and C) are allowed to vary. Equality constraints are used to impose symmetry so that columns B and E, and C and F are symmetrically placed about D. Thus, there are 62 continuous design variables and 20 boolean design variables in each design model as shown in Table 3.

Table 3. Description of Design Variables in Planar Frame Models

Variables	Variable type	Number	Description
C_1 to C_{10}	Continuous (Sizing)	20	Web height of column cross-section for the 20 groups of floors
LB_1 to LB_{10}	Continuous (Sizing)	20	Web height of longitudinal beam cross-section for the 20 groups of floors
BT_1 to BT_{10}	Continuous (Sizing)	20	Web height for belt truss bracing member cross-section for the 20 groups of floors
L_1 to L_{10}	Boolean (Topology)	20	Bracing pattern for belt trusses for the 20 groups of floors
x_1 and x_2	Continuous (Shape)	2	Location of the two interior columns. The end columns and the center column remain fixed.

For problem involving only continuous design variables, MFD was used for a maximum of 50 iterations with a relative and absolute convergence tolerance of 10^{-4}. Otherwise, GA was used with a population size of 500 for 50 generations.

Design Optimization Solutions

Initially, models were created with no bracing elements. These models were executed to find a baseline optimal design for comparative purposes. However, no feasible solution could be found for two of the three models - the 150' wide and the 180' wide frames. The addition of bracing elements via belt trusses produced acceptable designs in those two cases. It should be noted that the weight of each member was calculated center-to-center without accounting for connections. The values of some of the key structural parameters are shown in Table 4 (ton denotes 2000 pounds).

Table 4. Key Structural Parameters

	120-2D	120-2D-BT	150-2D-BT	180-2D-BT
Total weight of all beams, columns and bracing members, *ton*	865	860	1405	1935
Initial location of the interior columns (x_1, x_2, x_3) (see Fig. 2), *in*	(240, 480, 720)	(240, 480, 720)	(300, 600, 900)	(360, 720, 1080)
Final location of the interior columns (x_1, x_2, x_3) (see Fig. 2), *in*	(205, 449, 720)	(204, 448, 720)	(287, 587, 900)	(344, 700, 1080)
Lowest frequencies, *Hz*	(0.18, 0.45, 0.80)	(0.18, 0.45, 0.80)	(0.15, 0.64, 0.96)	(0.17, 0.72, 1.03)
Smallest Buckling Load Factor, λ^B	15.7	16.2	17.9	17.8

The optimal designs are summarized in Tables 5-6. Table 5 shows the weight distribution of the structural elements. The summary of the structural response is shown in Table 6. The normalized weight (weight per designed unit area) is computed by assuming a square building with 49 frames as

$$W_N = \frac{7W + 42W_B + 3W_X}{\text{Total covered area}} = \frac{7W + 42W_B + 3W_X}{40\left(36s^2\right)} \tag{20}$$

where W is the total weight on one frame (see Table 4), W_B is the weight of all the remaining beams and W_X is the weight of all the remaining bracing elements. The final shape and topologies of the optimal designs are shown in Fig. 3 along with the plots of the highest compressive stress (blue denotes highest value and red the lowest value) for LC 4, the governing load case.

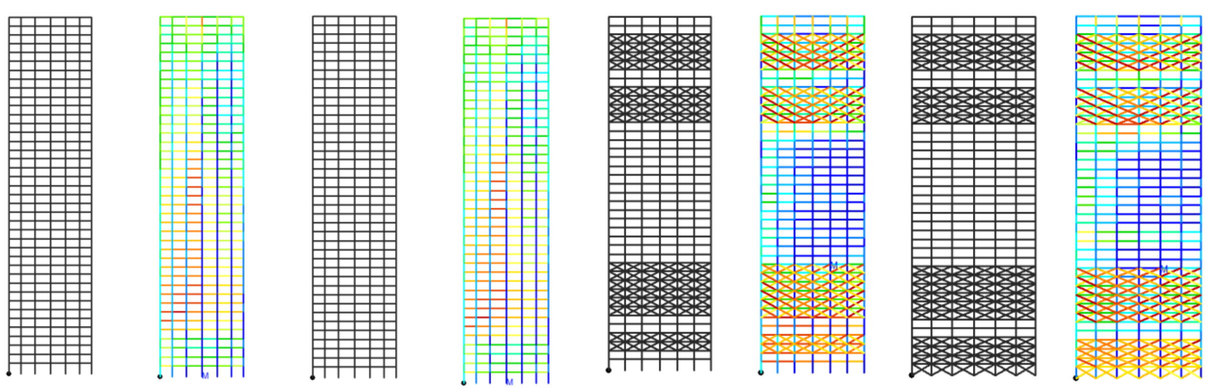
Discussion: As expected, increasing the span from s=20' to s=30' makes it much more challenging and difficult to obtain a feasible design. A simple moment-connected frame appears to be the optimal design when the span is relatively small (s=20') – both 120-2D model (solved via MFD) and 120-2D-BT model (solved via GA) lead to very similar final designs. However, when the span is increased, it becomes necessary to combine a rigidly-connected frame with belt trusses. Even with the use of these structural elements, the columns dictate the design requiring a relaxed upper bound on the web height. Interestingly enough, the columns with the highest stresses are the interior columns at the base of the belt trusses (Fig. 3(c)-(d)). However, a large number of members have their max. compressive (columns and beams) and tensile stress (beams) close to the limit of 30,000 psi for both LC 2 and LC 4 indicating that the design is near optimal and efficient. Displacement, shear stress, buckling (Euler and buckling load factor) and the lowest frequencies of the structure are not close to controlling the design. The lowest three frequencies are well spaced. The largest displacement value occurs on the top floor of the structure and is about 4% of the typical floor height. In every model, the interior columns move outward thereby reducing the outermost spans and increasing the interior spans. From an office space usage viewpoint, this may be desirable since larger uninterrupted spans are available for use as conference or meeting rooms. The normalized weight shows a wide range between 51.5-76.4 psf. It is possible that the 150-2D-BT and 180-2D-BT models can be improved (are not the global optimum solutions) and that with a better solution, a lower normalized weight can be found. We will discuss the difficulty of finding solutions in the Boolean-continuous design variable space later in the paper. The wall clock time (WCT) for one function evaluation is about 0.11 s so that the total WCT for a GA-runs (the belt truss models) is about 2800 s and about 250 s for the MFD-solved model (120-2D).

Table 5. Summary of the Weights for Various Best Designs
(Number in parenthesis denotes % of total weight)

Model	**Weight of Columns, W_C (ton)**	**Weight of Beams, W_B (ton)**	**Weight of Bracing, W_X (ton)**	**Total Weight, W (ton)**	**Normalized Weight, W_N (lb/ft^2)**
120-2D	485 (56)	380 (44)	0 (0)	865	76.4
120-2D-BT	485 (56)	375 (44)	0 (0)	860	76.4
150-2D-BT	690 (49)	380 (27)	335 (24)	1405	59.6
180-2D-BT	1040 (53)	440 (23)	455 (24)	1935	51.5

Table 6. Summary of the Maximum Structural Response
(Number in parenthesis denotes % of allowable value)

Model	**X-Disp. (in)**	**Tensile Stress (psi)**	**Comp. Stress (psi)**	**Shear Stress (psi)**
120-2D	6.1 (51)	22240 (74)	30000 (100)	7540 (38)
120-2D-BT	6.1 (51)	22360 (75)	30000 (100)	7540 (38)
150-2D-BT	5.9 (49)	30000 (100)	29800 (99)	9850 (49)
180-2D-BT	3.8 (32)	30000 (100)	29800 (99)	10300 (52)



120-2D **120-2D-BT** **150-2D-BT** **180-2D-BT**

Figure 3. Final topology of the best designs and compressive stress plot (LC 4)

Design Using Braced Tube - Diagrids

The building design problem from Section 3.1 is used in the context of braced tube-diagrid building form. To facilitate the automatic selection of the two end nodes for the bracing elements, a list of potential nodes was used for each element – these discrete design variables were added to the list of design variables (Eqn. (1)) as $\mathbf{x} = \{\mathbf{x}_c, \mathbf{b}, \mathbf{x}_d\}$. The new problem formulation has a total of 38 design variables. The C_1 to C_{10} design variables were applied to columns B, C, E, F. The LB_1 to LB_{10} design variables were applied to all beams not connected to the primary braced tube-diagrid system, and x_1 and x_2 design variables were used as before. Using information from a preliminary design study of this configuration, new design variables were introduced to handle the diagrid system - 4 continuous design variables were introduced for columns A, D and G, 4 continuous design variables for the longitudinal beams, 4 design variables for the bracing members, and 4 discrete design variables were introduced to take care of the end nodes of the bracing members. Possible bracing configurations are shown in Fig. 4 where the thick lines denote columns, beams and bracing members that are part of the diagrid system. Symmetry of the structure is enforced with the nodes of the bracing elements being placed on columns A, D and G as shown in Fig. 4. This approach is perhaps preferable to the use of continuum element-beam element model as those shown in previous research work [Liang et al., 2000; Stromberg et al., 2012] since the modeling and design approach fits in with the optimal design problem with no additional assumptions or limitations. The problem data are the same as those used in Section 3.1 The optimal design results are shown in Tables 7-9. The final shape and topology of the building is shown in Fig. 5.

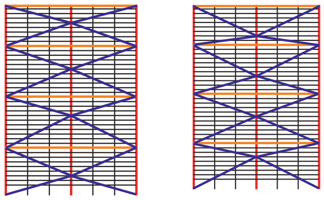


Figure 4. Two of the several possible bracing configurations

Table 7. Key Structural Parameters

	180-2D-D
Total weight of all beams, columns and bracing members, *ton*	2800
Initial location of the interior columns (x_1, x_2, x_3) (see Fig. 2), *in*	(360, 720, 1080)
Final location of the interior columns (x_1, x_2, x_3) (see Fig. 2), *in*	(289, 650, 1080)
Lowest frequencies, *Hz*	(0.40, 1.04, 1.69)
Smallest Buckling Load Factor, λ^B	45.5

Table 8. Summary of the Weights for Various Best Designs
(Number in parenthesis denotes % of total weight)

Model	**Weight of Columns, W_C (ton)**	**Weight of Beams, W_B (ton)**	**Weight of Bracing, W_X (ton)**	**Total Weight, W (ton)**	**Normalized Weight, W_N (lb/ft^2)**
180-2D-D	1130 (40.3)	655 (23.4)	1015 (36.3)	2800	77.4

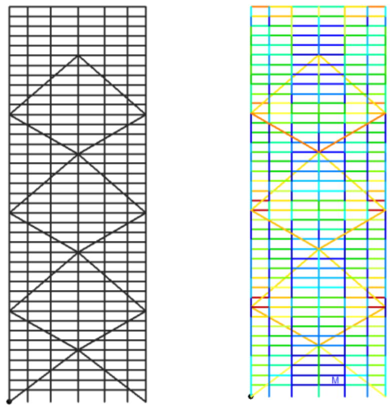
Table 9. Summary of the Maximum Structural Response
(Number in parenthesis denotes % of allowable value)

Model	**X-Disp. (in)**	**Tensile Stress (psi)**	**Comp. Stress (psi)**	**Shear Stress (psi)**
180-2D-D	0.92 (8)	29600 (99)	29700 (99)	9980 (50)

Discussion: Both the compressive/tensile stress and buckling load factor control the design. The reason why the latter controls the design is because of the extremely long bracing members. In practice one would attach the intermediate points in the bracing members to appropriate points in the intersecting columns and beams. Such connections are not generated in the model in the current research since this would involve finding the intersection of the beams and columns during the optimization run, thereby requiring new nodes and elements to be generated. Finally, it should be noted that the bracing members help reduce the lateral displacement from 3.8” to just 0.78”.

CONCLUSIONS

A simultaneous sizing, shape and topology optimization methodology is developed and used for the optimal design of tall buildings modeled as planar frames. The developed framework allows for investigating the design efficiencies of different building systems using planar finite element models. Continuous, discrete and Boolean design variables are used in the context of sizing, shape and topology optimization. The computed results yield important building parameters such as cross-sectional dimensions and properties, location of structural joints, and member layout via presence, absence and location of members. The performance and serviceability constraints include normal stress, shear stress, displacement, inter-story drifts, buckling, and natural frequencies.


(a) (b)
Figure 5. (a) Final design (b) Compressive stress plot (LC 4)

Results show that efficient planar models can be obtained that satisfy performance requirements in a reasonable amount of time. Finally, it should be noted that the results from a geometrically nonlinear analysis yielded almost the same results as that obtained from linear analysis justifying the use of linear analysis. While planar frames provide a wealth of knowledge, the understanding is limited by the limitations of the model. Buildings are three-dimensional and planar frames typically do not contain some of the important structural elements – floors, walls, specialized system such as tubes or cores etc. A tributary area approach in computing equivalent planar model loads is approximate. In addition, planar models do not account for behavior found only in three-dimensional systems such as member bending and shear about two axes, torsional moments, and torsional eigenmodes. A logical extension of the framework discussed in this paper is to build and use a three-dimensional finite element model for design optimization. One of the numerical optimization challenges is the difficulty in finding solutions in the Boolean-continuous design variable space encountered when sizing and shape continuous design variables are mixed with topology design variables that are either Boolean or discrete or both. The computational challenge is to reduce the overall compute time so as obtain solutions in a reasonable amount of time. One of the easily implementable strategies to improve the computational throughput is through parallel computations. In the gradient-based solutions, the computation of numerical derivatives and those during line search can be parallelized [Rajan et al., 2006]. The reduction in the objective function is typically much greater in the initial few iterations after which the reduction are relatively much smaller. Hence using a coarse convergence criteria can cut down on the computational cost without an unacceptable loss of accuracy. In population-based solutions, the parallelization is an embarrassingly parallel problem [Rajan and Nguyen, 2004]. The GA solutions required involving topology optimization required a larger probability of mutation. In this research, repeating chromosomes [Chen and Rajan, 2000] were not detected so as to avoid carrying out a previous function evaluation. Implementing this option would cut down the computational cost since later generations typically contain a large number of identical members.

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