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INTEGRATED TIME HISTORY ANALYSIS AND PERFORMANCE-BASED DESIGN OPTIMIZATION OF BASED-ISOLATED CONCRETE BUILDINGS

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SUMMARY

Base isolation has become a practical control system for protecting structures against seismic hazards. Most previous research studies on the design method for base-isolated structures have been focused on the design optimization of either the base isolation or the superstructure. It is necessary to optimize simultaneously both the base isolation and the superstructure as a whole in order to seek the most cost-efficient design for such structures. This paper presents an effective numerical optimization technique for the performance-based design of base-isolated concrete building structures under time history loading. Attempts have been made to automate the integrated nonlinear time history analysis and design optimization procedure and to minimize the total cost of the base-isolated building subject to multiple design performance criteria in terms of the story drift of the superstructure and the lateral displacement of the isolation system. In the optimal design problem formulation, the cost of the superstructure can be expressed in terms of concrete element sizes while assuming all these elements to be linear elastic under different levels of design earthquakes. However, the base-isolation is assumed to behave nonlinearly and its cost can be related to the effective horizontal stiffness of each isolator. To reduce computational effort, only critical peak drift responses over the entire time history are first identified and then included in the optimization process as design constraints. Using the principle of virtual work, the peak drift responses can be explicitly formulated and the integrated optimization problem can be solved by the Optimality Criteria method. The technique is capable of achieving the optimal balance between the costs of the superstructure and isolation systems whilst the seismic drift performance of the building under multiple levels of earthquake motions can be simultaneously considered.

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INTRODUCTION

Seismic isolation, an innovative seismic design approach, has been increasingly used in earthquake-prone regions for protecting structures against damage from earthquakes by limiting the earthquake attack, rather than resisting it [1-6]. Numerical studies reported by Shenton and Lin [7] show that better performances of base-isolated reinforced concrete (RC) frames can be achieved when compared to fixed-base frames. Unlike traditional fixed-base structures, a base-isolated structure requires the cost of an isolation system, but this additional cost can usually be offset by the cost saving in the superstructure since a well-designed base isolation system can largely reduce earthquake loading. More importantly, long-term savings are ensured due to better seismic performance by isolating the structure from seismic actions [3,8,9]. The most popular seismic isolation systems use elastomeric bearings which consist of rubber and steel plates with an energy dissipation mechanism. Since the base isolation system can maintain the superstructure in the elastic range during earthquakes, it is very applicable for some important buildings, such as nuclear power plants, historical buildings and laboratories with sensitive equipment, etc. There have been many successful examples of base-isolated construction in the United States, Chile, Indonesia, New Zealand, Italy, China and Japan [4].

Traditionally, not only are the superstructure and the isolation system designed separately in a building, but also the determination of the satisfactory dynamic responses fulfilled in both the superstructure and isolation system requires a highly iterative trial-and-error reanalysis and redesign process even with the aid of today's engineering computer software. Many research studies have been devoted to the optimal design of either the superstructure [10-12] or the base isolation system [3, 13-16]. Cheng and Li [17] stated that few researchers treated the integrated optimization design of the structure and the control system and emphasized that it is desirable to use all design resources of the structure and control system because of strong interaction between the two systems and therefore, simultaneous structure/control design is necessary to achieve ideal control with minimum cost. Therefore, it is intended in this research that both the structure and the base isolation, as a whole, are simultaneously optimized to achieve optimal performance based design.

In a conventionally designed structure, structural and non-structural damage caused by an earthquake excitation is mainly manifested through the control of story drift responses of the structure [6]. Not only has lateral drift been an important indicator that measures the level of damage to the structural and non-structural components of a building, but also the control of drift performance is one of the most challenging and difficult tasks in building design since lateral drift is a system design criterion that requires the consideration of all structural members in a building.

The seismic response of a base-isolated structure is usually investigated by two alternative approaches, i.e. making use of full nonlinear analysis (e.g., nonlinear time history analysis) or linearized techniques (e.g., response spectrum analysis) [4]. The dynamic time history analysis method has been widely used in the design of building structures. This technique can replicate and record the actual performance of a building under a design time history record. Although time history analysis provides the most comprehensive analysis of dynamic seismic responses, this method involves significantly greater computational effort. Generally, at least three representative earthquake motions must be considered to allow for uncertainty in the precision and frequency content of the seismic excitation at a site [20]. However, it is a rather difficult and tedious task to control all structural responses produced by various representative earthquake motions. Zou and Chan [18], and Zou [19] developed an efficient computer-based optimization technique for lateral drift design of isolated reinforced concrete buildings subject to spectrum loadings. Much effort is still needed to extend the current optimization technique to seismic design of isolated buildings under time history loadings.

This paper presents an effective optimization technique for the seismic drift performance design of isolated building structures subject to time history loadings. Attempts will be made to automate the integrated nonlinear time history analysis and design optimization process. It is required that the natural vibration period of a base-isolated structure reach a target period under specified earthquake loading and the lateral displacement of the isolation system be limited within the deformation capacity of individual isolators. It is also required that under specified earthquake loading, the interstory drift ratio of the superstructure be limited within an acceptable range. Using the principle of virtual work, the time history drift responses of a base-isolated RC building can be explicitly formulated. A rigorously derived Optimality Criteria (OC) method is developed to minimize the total construction cost of a base-isolated building structure subject to the interstory drift constraints of the base-isolated superstructure and effective stiffness constraints of the base isolation system.

SEISMIC DESIGN OPTIMIZATION PROBLEM

Optimal Design Problem

Isolation system

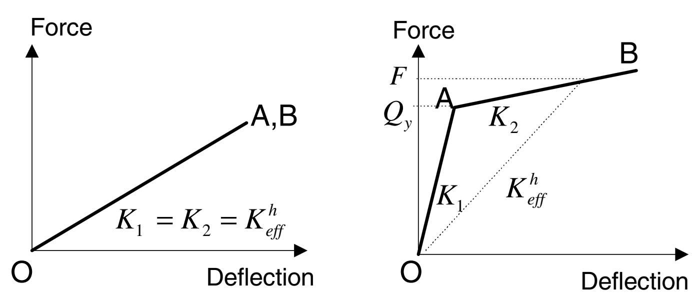
A base-isolated structural concrete building consists of a base isolation system and a concrete superstructure. The isolation system includes all individual linear and/or nonlinear isolators. As shown in Fig. 1(a), a linear isolator results in a linear relationship of force and deflection; whereas a nonlinear isolator, normally modeled by a bilinear curve, exhibits a nonlinear relationship of force and deflection. As shown in Fig. 1(b), K_1 is the elastic stiffness of the base isolator and K_2 is the post-yield stiffness which can be assumed to be linearly related to K_1 by the means of a statistic regression analysis such that $K_2 = n^h K_1$, where n^h defines the stiffness ratio of K_2 over K_1; Q_y is the yield strength of an isolator which can also be expressed as a function of K_1 such that $Q_y = f(K_1)$. As displayed in Fig. 1(c), each base isolator has the vertical stiffness, K^v, in the vertical direction and the horizontal effective stiffness, K^h_{eff}, in the transverse directions. The horizontal effective stiffness, K^h_{eff}, also called the secant stiffness, is defined to be the slope of a line from O to B as shown in Figs. 1(a) and 1(b). Summing up the individual horizontal effective stiffnesses, the lateral stiffness, K_B, of an isolation system having $b = 1, 2,.., N_b$ isolators can be expressed as

$$K_B = \sum_{b=1}^{N_b} K^h_{eff,b} \tag{1}$$

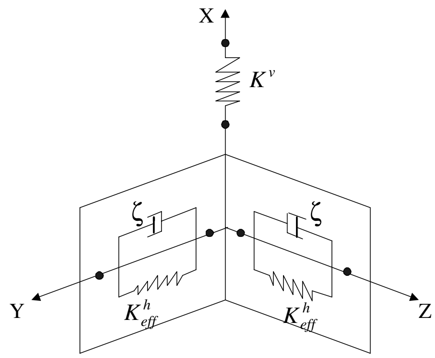
Referring to Fig.1(b), K^h_{eff} can be linearly related to the elastic stiffness K_1 as follows.

$$K^h_{eff} = \frac{n^h F K_1}{F + (n^h - 1) Q_y} \tag{2}$$

where F is the lateral internal force in an isolator along horizontal direction.



(a) Linear isolator (b) Nonlinear isolator



(c) Base isolator

Fig.1 Definition of Linear and Nonlinear Isolators

In general, the structure damping is essentially the damping of the base-isolated system, which is dependent on the damping of isolators [3, 8, 15, 20]. The total effective damping of the isolation system, ζ_B, can be expressed collectively as [5]

$$\zeta_B = \frac{1}{K_B}\sum_{b=1}^{N_b} K^h_{eff,b} \cdot \zeta_b \tag{3}$$

where ζ_b is the effective damping ratio of the individual *b*th isolator with an effective stiffness equal to $K^h_{eff,b}$. The total effective damping of the isolation system, ζ_B, will be used in the time history analysis.

Design variables

In this study, the number and the type of isolators (linear or nonlinear) are predefined in the isolation system and thus design variables are generally the vertical stiffness, K^v, and the horizontal effective stiffness, K^h_{eff}, of each base isolator. In general, the vertical stiffness K^v can be assumed to be linearly dependent on K^h_{eff} such that $K^v = n^v K^h_{eff}$, where n^v is the stiffness ratio which can be determined by statistical analysis. Since the horizontal effective stiffness, K^h_{eff}, can be related to the elastic stiffness, K_1, as shown in Eq. (2). As a result, K_1 can be considered as the primary design variable for an isolator.

If the topology of the superstructure is predefined, the structural element sizes can be usually taken as design variables of the superstructure. Given with a framework element of the six cross sectional properties, i.e., the axial area (A_X), two shear areas (A_Y and A_Z) and three moments of inertias (I_X, I_Y, I_Z), can be theoretically considered as basic design variables. However, structural elements for practical structures are generally not freely independent of each other. Assuming rectangular concrete elements, their sectional properties can be expressed in terms of the width (B) and depth (D) as basic design variables as follows.

$$A_X = B \cdot D\,; \qquad A_Y = A_Z = \frac{5}{6} B \cdot D \tag{4a, b}$$

$$I_Y = \frac{1}{12} B^3 \cdot D\,; \quad I_Z = \frac{1}{12} B \cdot D^3\,; \quad I_X = \left[\frac{1}{3} - 0.21\frac{B}{D}\left(1 - \frac{B^4}{12D^4}\right)\right] B^3 \cdot D \tag{4c, d, e}$$

Since the effect of member torsion is generally very small, one may simplify the torsional moment of inertia I_X to the following approximate expression

$$I_X = \kappa B^3 \cdot D \tag{4f}$$

κ denotes the torsional coefficient that depends on the ratio value of depth to width (i.e., D/B) of the element i. For thin wall sections where $D/B >> 1$, κ can be approximately equal to 0.3.

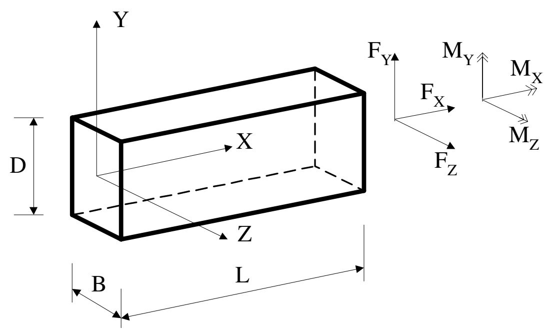


Fig. 2. Definition of Local Coordinate System for A Member

Objective function

For a base-isolated concrete building having b=1, 2,…, N_b base isolators and i=1, 2,…, N_i members, there is only one independent sizing variable (K_{1b}) for each base isolation and each concrete element generally has two independent sizing variables (B_i , D_i). The design optimization objective addressed herein is to minimize the total construction cost involving the costs of the base isolation system and the superstructure.

$$F(K_{1b}, B_i, D_i,) = \sum_{i=1}^{N_i} w_i B_i D_i + \sum_{b=1}^{N_b} f(K_{1b}) \tag{5}$$

where F represents the total construction cost; the first part of the cost is the concrete cost and w_i is the cost coefficient for the ith member of the superstructure; the second part is the isolation system cost and $f(K_{1b})$ represents the cost of the bth isolator, which can be assumed to relate to the horizontal stiffness of each base isolator, K_{1b} , as

$$f(K_{1b}) = m_1 K_{1b} + m_2 \tag{6}$$

where m_1 and m_2 are the cost coefficients of the bth base isolator, which can be obtained through a statistical investigation. Based on the discrete cost data provided from the manufacturer, the relationship between the cost and the elastic horizontal stiffness of each type of isolator can be established and expressed in the form of a linear function by regression analysis. As a result, the cost coefficients m_1 and m_2 given in Eq. (6) can then be found. In this study, the cost function of various types of base isolators was developed based on the data provided by Shantou Vibro Tech Industrial and Development Co. Ltd.

Target period and displacement constraint at base story

The aim of base isolation is to reduce the force imparted to the structure to such a level that no damage to the structural or nonstructural elements occurs. For this purpose, there is usually a good separation between the fixed-base period of vibration, T_f , and the base-isolated period of vibration, T_B for a building structure. Due to the low stiffness of the isolation system as compared to the superstructure, large lateral deformation is expected to concentrate at the base isolation level of a building such that the flexibility of the isolated structure is more affected by the base isolation whilst the superstructure behaves as essentially rigid. As a result, the natural period of the isolated structure can be assumed to be very much dominated by that of the isolation system [2, 5].

In common practice of base-isolation design, the design process starts with the preliminary design of a fixed-base structure. In general, it is desirable to devise a base isolation system that reduces the earthquake base shear by approximately at least three times of that of the fixed-base structure. Based on a given design spectrum for a building, the target base-isolated period of vibration, T_B , can be determined approximately by dividing the acceleration value of the fixed-base structure by a factor of three [20]. The UBC code [21] gives a recommendation on the amplitude of T_B that the isolation period of the structure may be greater than three times the elastic, fixed-base period of the structure above the isolation system under moderate earthquake loading.

Once the target period of the isolation system, T_B , is established, the main effort then is to devise the isolation system so as to achieve the target period. Under the condition that the isolation system damping is temporarily fixed, the target period, T_B , of the isolation system can be achieved by controlling the lateral seismic displacement at the base floor level. Specifically, in order to attain sufficient flexibility so as to lengthen the period of the base isolation system, the isolation system is required to deform to maintain a minimum displacement at the base floor level as follows:

$$u_0 \geq \delta_0^L \tag{7}$$

where u_0 is the lateral displacement at the top of the isolation system under the seismic loading; the subscript 0 represents the base floor level; δ_0^L is the specified minimum lateral displacement and can be given as

$$\delta_0^L = \Gamma^{(1)}\phi_0^{(1)} S_d \tag{8}$$

in which $\Gamma^{(1)}$ is the modal participation factor for the first mode; $\phi_0^{(1)}$ is the amplitude of the first mode at the base floor level; S_d is the spectral displacement for the first mode. Since the modal spectral displacement can be related to the modal spectral acceleration, the minimum lateral displacement, δ_0^L , given in Eq.(8) can be rewritten as

$$\delta_0^L = \Gamma^{(1)}\phi_0^{(1)} \left(\frac{T_B}{2\pi}\right)^2 RS_a(T_B, \zeta_B) \tag{9}$$

where T_B is the natural period of the base isolation system; $S_a(T_B, \zeta = 5\%)$ represents the modal spectral acceleration with 5% damping ratio; and R is a damping response reduction coefficient [20].

Since base isolators are very flexible under lateral forces, it is necessary to prevent them from failure in the event of severe earthquakes. In addition to the design requirement on the minimum target period of the base isolation system, it is needed to ensure that each individual isolator does not deform excessively beyond the shear deformation capacity of the isolator [21]. Therefore, the displacement of the isolation system at the base floor level, u_0 , should be limited within the least allowable deformation capacity of the base isolators of the isolation system as

$$u_0 \leq \delta_0^U = \min_b(\chi_b) \tag{10}$$

where δ_0^U is the upper bound displacement limit at the base floor level and χ_b is the allowable deformation limit for each base isolator..

Lateral interstory drift constraint at the superstructure
As stated earlier, the lateral interstory drift of a multistory building is an important parameter that measures the damage level of the building under earthquake loading. If the differential lateral displacement of two adjacent story levels exceeds certain acceptable limit, the building will be deemed not to satisfy the specified performance criterion. Therefore, a set of interstory drift constraints can be stated as follows.

$$\Delta u_j = u_j - u_{j-1} \leq \delta_j^U \qquad (j = 1, 2, ..., N_j) \tag{11}$$

where Δu_j is the interstory drift at two adjacent *j* and *j*-1 floor levels; δ_j^U is the corresponding interstory drift limit.

Element strength constraints
In addition to the checking of the drift response of an isolated building, each of the elements of the building muse be checked to ensure for adequate strength requirement as

$$\sigma_i \leq \sigma_i^U \qquad (i=1, 2, \ldots, N_i) \tag{12}$$

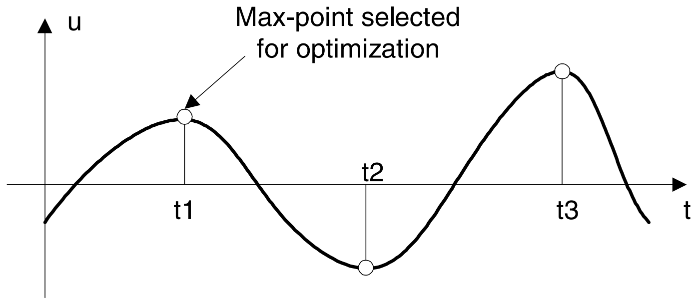
where σ_i represents a stress state for member *i* and σ_i^U denotes the corresponding allowable member strength. In order to reduce computational effort, the strength design of a member can be considered separately on a member by member manner and therefore not included as part of the system design constraints on lateral drift performance. In general, after each structural analysis, the strength sizes of each structural elements can be first sized in accordance with code requirements and these values are then taken as the lower size bound in the drift design optimization.

In order to facilitate numerical solutions of the design optimization problems, the implicit lateral drift constraint Eqs. (7), (10) and (11) should be expressed explicitly in terms of the design variables K_1, B_i and D_i.

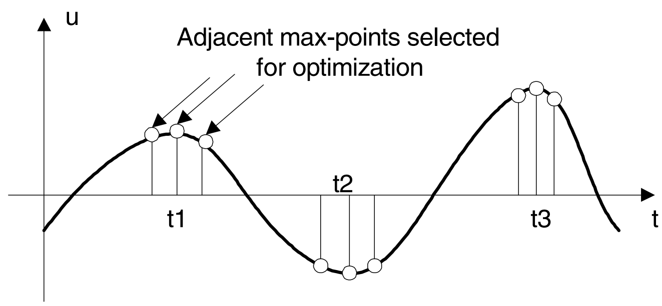
Formulation of Optimal Design
Lateral displacement formulation
In this study, the dynamic response considered is analyzed by the nonlinear time history method. The procedure of the time history analysis involves a step-by-step solution through a time domain to yield the dynamic response of a structure to a given time history of ground motion. Since it is generally necessary to consider at least three representative earthquake motions to check the dynamic response of a building, the control of lateral drift responses at all time intervals may result in an excessive number of constraints to be considered throughout the entire time history record. The consideration of all time history seismic drift responses requires an enormous amount of computational effort and, therefore, treatment with a vast number of time history responses is a challenging problem for most numerical optimization algorithms.

Various numerical techniques exist for treating such time-dependent design constraints [22]. The basic idea of these methods is to eliminate somehow the time parameter, *t*, from the optimization problem or in other words, a time-dependent problem is transformed into a time-independent one. One method is to replace the entire number of time history responses by a limited number of responses only at each of the local max-points along the time axis, as shown in Fig. 3(a). Since the max-points may not necessarily lie exactly at the grid points, another method in which the constraints in a range of grid points adjacent to the maximum points is recommended, as illustrated in Fig. 3(b).

In this study, critical drifts at max-points and their adjacent max-points are selected as drift constraints in the optimization, rather than considering the entire time history responses during the earthquake. In order to define the critical drift explicitly in terms of element sizing variables, the internal forces of the structure corresponding to the max-point or the adjacent max-point dynamic drifts are first identified and captured from the time history analysis. Then, the virtual internal forces of all members are found from the static analysis where a unit virtual load is applied to the structure at the location of and in the sense of the corresponding critical story displacement, u_j.



(a) Worst-case design formulation
(b)



(c) Constraints at grid points adjacent to maximum points

Fig. 3. Possible Treatments of A Dynamic Constraint

Based on the results of the time history analysis and the virtual load analysis, the maximum displacement responses at the critical points of time history can then be expressed by the principle of virtual work. Specifically, the total virtual work u_j^t (i.e., the *t*th time displacement at *j*th level of a concrete building) involving the virtual work done $u_{j,isolator}^t$ by the base isolation system and the virtual work done $u_{j,member}^t$ by structural members for the superstructure can be written as

$$u_j^t = u_{j,isolator}^t + u_{j,member}^t \tag{13}$$

where

$$u_{j,member}^t = \sum_{i=1}^{N_i} \int_o^{Li} \left(\frac{F_X^t f_{Xj}}{EA} + \frac{F_Y^t f_{Yj}}{GA_Y} + \frac{F_Z^t f_{Zj}}{GA_Z} + \frac{M_X^t m_{Xj}}{GI_X} + \frac{M_Y^t m_{Yj}}{EI_Y} + \frac{M_Z^t m_{Zj}}{EI_Z} \right)_i dx \tag{14}$$

$$u_{j,isolator}^t = \sum_{b=1}^{N_b} \left(\frac{F_X^t f_{Xj}}{K_{eff}^v} + \frac{F_Y^t f_{Yj}}{K_{eff}^h} + \frac{F_Z^t f_{Zj}}{K_{eff}^h} \right)_b \tag{15}$$

where L_i is the length of member *i*; E, G are the axial and shear elastic material moduli; A_X, A_Y, A_Z are the axial and shear areas for the cross-section; I_X, I_Y, I_Z are the torsional and flexural moments of inertia for the cross-section; F_X^t, F_Y^t, F_Z^t, M_X^t, M_Y^t, M_Z^t are the elements' internal forces and moments at the critical time point, t; f_{Xj}, f_{Yj}, f_{Zj}, m_{Xj}, m_{Yj}, m_{Zj} are the virtual element forces and moments due to a unit virtual load applied to the building at the location corresponding to the time history story displacement,

u_j^t. Note that the coordinate system and sign convention of member internal forces are depicted in Figs. 1(c) and 2.

Considering rectangular concrete elements with the width (B_i) and depth (D_i) taken as design variables and expressing the cross section properties in terms of B_i and D_i as shown in Eqs (4a-f), the critical time displacement $u_{j,member}^t$ shown in Eq. (14) can be simplified [18,19] as

$$u_{j,member}^t(B_i, D_i) = \sum_{i=1}^{N_i} \left(\frac{C_{1ij}^t}{B_i D_i} + \frac{C_{2ij}^t}{B_i D_i^3} + \frac{C_{3ij}^t}{B_i^3 D_i} \right) \tag{16}$$

where

$$C_{1ij}^t = \int_0^{L_i} \left(\frac{F_X^t f_{Xj}}{E} + \frac{F_Y^t f_{Yj} + F_Z^t f_{Zj}}{5G/6} \right)_i dx \tag{17a}$$

$$C_{2ij}^t = \int_0^{L_i} \left(\frac{M_Z^t m_{Zj}}{E/12} \right)_i dx \,; \qquad C_{3ij}^t = \int_0^{L_i} \left(\frac{M_X^t m_{Xj}}{G\kappa} + \frac{M_Y^t m_{Yj}}{E/12} \right)_i dx \tag{17b, c}$$

Considering cylindrical isolators with the horizontal elastic stiffness (K_{1b}) taken as a design variable, the displacement $u_{j,isolator}^t$ shown in Eq.(15) can be simplified as

$$u_{j,isolator}^t(K_{1b}) = \sum_{b=1}^{N_b} \left(\frac{C_{4bj}^t + C_{5bj}^t Q_y(K_{1b})}{K_{1b}} \right) \tag{18}$$

where

$$C_{4bj}^t = \frac{1}{n_b^v n^h} \cdot F_{Xb}^t f_{Xbj} + \frac{1}{n^h} \cdot (F_{Yb}^t f_{Ybj} + F_{Zb}^t f_{Zbj}) \tag{19a}$$

$$C_{5bj}^t = \frac{(n^h - 1)}{n_b^v n^h} \cdot \frac{F_{Xb}^t f_{Xbj}}{F_{Yb}^t} + \frac{n^h - 1}{n^h} \cdot (f_{Ybj} + f_{Zbj}) \tag{19b}$$

Based on Eq. (18), the lateral displacement u_0^t at the base floor level (i.e., $j = 0$) is given as

$$u_0^t(K_{1b}) = \sum_{b=1}^{N_b} \left(\frac{C_{4b0}^t + C_{5b0}^t Q_y(K_{1b})}{K_{1b}} \right) \tag{20}$$

Explicit formulation of the design problem

Upon the explicit drift Eq. (13) through Eqs. (16) to (20), the base-isolated structural design optimization problem can be expressed in terms of the design variables K_{1b} , B_i and D_i, as

Minimize:

$$F(K_{1b}, B_i, D_i) = \sum_{b=1}^{N_b} (m_1 K_{1b} + m_2) + \sum_{i=1}^{N_i} w_i B_i D_i \tag{21}$$

subject to:

1) for the base isolation system,

$$g_1(K_{1b}) = \frac{\delta_0^L}{u_0} = \delta_0^L \left[\sum_{b=1}^{N_b} \left(\frac{C_{4b0}^t + C_{5b0}^t Q_y(K_{1b})}{K_{1b}} \right) \right]^{-1} \le 1 \tag{22}$$

$$g_2(K_{1b}) = \frac{u_0}{\delta_0^U} = \frac{1}{\delta_0^U}\sum_{b=1}^{N_b}\left(\frac{C_{4b0}^t + C_{5b0}^t Q_y(K_{1b})}{K_{1b}}\right) \le 1 \tag{23}$$

2) for the superstructure system at various floor levels when $j = 1,2,...N_j$,

$$g_d(K_{1b}, B_i, D_i) = \frac{1}{\delta_j^U}\left[\sum_{b=1}^{N_b}\frac{\Delta C_{4bj}^t + \Delta C_{5bj}^t Q_y(K_{1b})}{K_{1b}} + \sum_{i=1}^{N_i}\left(\frac{\Delta C_{1ij}^t}{B_i D_i} + \frac{\Delta C_{2ij}^t}{B_i D_i^3} + \frac{\Delta C_{3ij}^t}{B_i^3 D_i}\right)\right] \le 1$$

$$(d = 3,4,...N_j + 2) \tag{24}$$

$$K_{1b}^L \le K_{1b} \le K_{1b}^U \quad (b = 1, 2,..., N_b) \tag{25a}$$

$$B_i^L \le B_i \le B_i^U \;;\; D_i^L \le D_i \le D_i^U \quad (i = 1, 2,..., N_i) \tag{25b, c}$$

Eq.(21) defines the total construction cost function F which consists of the cost of the superstructure system and the cost of the isolation system. Eqs.(22) and (23) define the lateral displacement constraints at the base floor level; the subscript d represents the dth design constraint ($d = 1,2,...N_j + 2$); δ_0^L is the allowable minimum displacement limit of the base story, determined by Eq. (9); δ_0^U is the allowable maximum displacement limit at the base floor level. Eq.(24) defines a set of interstory drift constraints for the superstructure; δ_j^U is the allowable jth story drift limit. Eqs. (25a, b and c) define the sizing constraints of design variables.

Once the design optimization problem is explicitly expressed in terms of design variables, the next task is to adopt a suitable method for solving the problem. The Optimality Criterion (OC) approach is adopted in this study due to its superior numerical efficiency for the design of large-scale building structures. When using the OC technique, it is necessary to reanalyse the structure after each design cycle and to reapply the continuous optimization OC process until the convergence to the minimum cost design is obtained. Details of the OC technique can be referred to the work of Zou [19].

PROCEDURE OF SEISMIC OPTIMAL DESIGN

The automated optimal design procedure for base-isolated, multi-story building structures is outlined as follows.

1. Assume initial member sizes of the superstructure, identify the initial number and size of base isolators for the isolation system, and determine their lower and upper size bounds.
2. Compute the total effective damping ratio, ζ_B, of the isolation system using by Eq. (3).
3. Determine the minimum and maximum displacement limits for the base isolation system at the base floor level by Eqs. (9) and (10).
4. Carry out the nonlinear time history analysis of the structure with the damping ratio, ζ_B, to identify the critical interstory drift responses.
5. Apply virtual loads to the structure and perform static virtual load analyses.
6. Establish the explicit formulation of the optimal design problem Eqs. (21)-(24) for the lateral drift constraints of the superstructure and the isolation system.
7. Apply the recursive OC optimization algorithm to determine the optimal total cost of the element sizes of the building structure and the base isolation system.
8. Check the convergence of the objective function Eq. (21). If the total costs of the base isolated structure for two successive design cycles are within 0.5%, the optimal design solution is deemed to converge and then the design is terminated; otherwise, return to Step 2 for the next design cycle.

CONCLUDING REMARKS

The seismic design optimization problem of base-isolated concrete building structures subject to time history loadings has been explicitly formulated by the principle of virtual work in terms of element sizing design variables. It is emphasized that the peak time history drift responses of the building framework are selected as drift constraints in the optimization and the lateral displacement of the isolation system is limited within the deformation capacity of individual isolators.

The proposed optimization approach integrates the dynamic analysis with a rigorously derived Optimality Criteria method, which is capable of controlling the optimal time history drift performance while achieving the optimal balance between the cost of the superstructure and the cost of the isolation system. It is believed that the integrated optimal design methodology will provide a powerful computer-based technique to automate and to optimize the seismic drift performance of base-isolated concrete building structures. Further research work is needed to demonstrate and validate the effectiveness of the integrated time history analysis and optimization methodology.

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