

DUAL BUCKLING RESTRAINED BRACED STEEL FRAMES FOR ENHANCED SEISMIC RESPONSE

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Abstract: This paper presents an analytical study on the seismic stability of multi-story concentrically braced steel frames. The building height varies from 8 to 24 stories and two braced frame systems are studied: conventional buckling restrained braced frames, and dual buckling restrained braced frames. The second system includes an elastic truss to mitigate soft-story response. The structures are designed according to the Canadian seismic provisions using the modal response spectrum analysis method, and their performance is examined under two types of ground motions for western Canada: crustal and sub-crustal events and subduction earthquakes. Incremental dynamic analysis is performed for both ensembles. The results show that conventional braced frame system is more vulnerable to dynamic instability than the dual system. Due to their long duration of shaking, subduction earthquakes are more critical than intra-plate events. Dual buckling restrained braced frames should be preferred for multi-story applications but they should be provided with suitable lateral strength to safely withstand ground motions produced by subduction earthquakes.

Keywords: multi-story buildings, braced steel frames, buckling-restrained braces, earthquakes, stability

1 INTRODUCTION

Concentrically braced steel frames (CBFs) in multi-story buildings are prone to soft-story response under strong seismic ground motions. This is mainly attributed to the poor hysteretic response of the steel bracing members under cyclic loading and the limited capacity of the system to redistribute the inelastic demand over the building height (Tremblay 2000, 2003). Taller buildings are more sensitive to story mechanisms due to the larger number of floors and the more pronounced contribution of the higher vibration modes to the response. Story behavior can lead to dynamic instability due to P-delta effects and building height limits have been introduced in the CSA-S16 Standard for the design of steel structures in Canada (CSA 2001) to prevent this phenomenon. CBFs exceeding these limits can still be used provided that the design engineer can demonstrate that the structure can exhibit a stable inelastic response when subjected to the design earthquake level.

Buckling restrained braces (BRBs) have recently been introduced in North America as a substitute for conventional steel braces in concentrically braced steel frames. As shown in Fig. 1a, these braces are typically made of a steel plate inserted in a steel tube filled with concrete. An unbonded material is used between the steel core and the concrete fill.

The steel plate is then prevented from buckling and can yield in compression and tension, which leads to a much more stable and symmetrical inelastic response under cyclic demand without strength degradation due to buckling (Watanabe et al. 1988; Tremblay et al. 1999; Iwata et al. 2000; Clark et al. 2000; Ko et al. 2002). Several analytical studies indicated that BRB frames such as the one illustrated in Fig. 1b can exhibit satisfactory performance in low-rise applications (Dasgupta et al. 2004; Fahnstock et al. 2003; Ko et al. 2002; Sabelli 2001, 2003; Tremblay et al. 1999, 2004). Fahnstock et al., Tremblay et al., Tremblay and Poncet (2004) and Mayes et al. (2004) however reported concentrations in the story drift demand, especially in taller buildings.

Tremblay (2003) proposed the dual buckling restrained braced frame system (Dual BRB) shown in Fig. 1c to mitigate this effect and ensure stable inelastic response. Compared to the common BRB frame of Fig. 1b, BRB members in the dual system are only used in one half of the bracing bent width, the braces in the second half of the bay width being designed to remain essentially elastic under a strong shaking. The edge columns and beam segments that connect to those stronger braces are also designed to avoid yielding and a continuous vertical elastic truss is formed by

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adding a central column to the bracing bent. The role of the vertical truss is to force inelastic demand to develop into a mode approaching the first vibration mode, thus preventing concentrated inelastic response over the building height and mobilizing the entire energy dissipation potential of the structure. In the Dual BRB frame system, additional steel tonnage is required to construct the elastic, stronger vertical truss but this is compensated to some extent by the fact that only half the number of BRB members is needed. A similar system has been used recently in Osaka, Japan for a 23-story building¹.

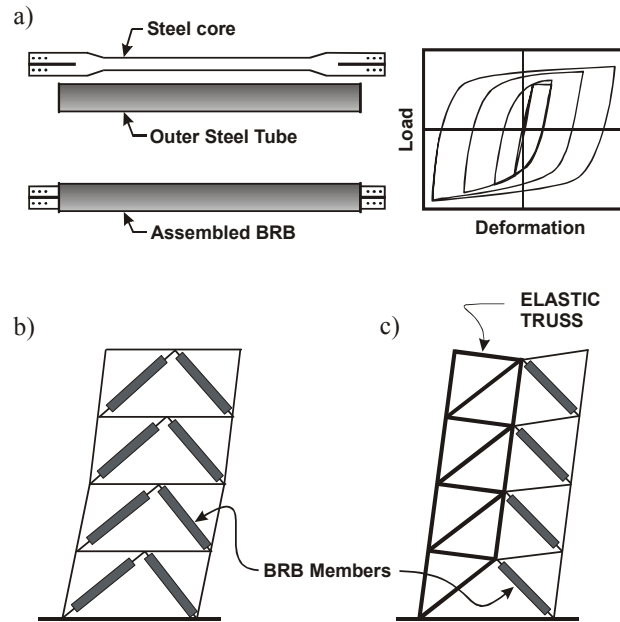


Fig. 1: Structural systems and brace hysteretic response: a) typical buckling restrained brace (BRB) member and hysteretic response; b) BRB frame; and c) Dual BRB frame.

Tremblay and Poncet (2004) confirmed the good performance of the Dual BRB system for building structures up to 16 stories in height when subjected to crustal and sub-crustal earthquakes. However, their study had several limitations that required further studies. For instance, the design of the structures was performed using the simple code equivalent static force procedure, the BRB hysteretic response was modeled assuming a simple bilinear hysteresis with 2% strain hardening, and inelasticity in the Dual BRB analytical model was restrained to the bracing members only. Recent studies also revealed that long duration motions generated by inter-plate earthquakes could be more detrimental to the stability of structures relying on hysteretic energy dissipation mechanisms (e.g., Tremblay 1998; Tremblay and Atkinson 2001; Tirca and Tremblay 2004). This type of seismic events is anticipated along the Cascadia

subduction zone on the west coast of the U.S. and Canada and it was felt that the performance of the Dual BRB frame system needed to be examined for such ground motions.

This paper presents an analytical study performed to examine the seismic response of 8- to 24-story buildings located in Victoria, B.C., a city located in the vicinity of the Cascadia subduction zone. For all buildings, both the BRB and the Dual BRB frame systems were studied. The structures were designed using the modal response spectrum analysis method and the BRB inelastic response was reproduced using a Ramberg-Osgood formulation including both kinematic and isotropic strain hardening properties. The structures were subjected to an ensemble of site representative ground motions from crustal and sub-crustal earthquakes as well as simulated ground motion time histories from M_w 8.5 subduction events.

2 BUILDINGS STUDIED

2.1 Description of the Buildings

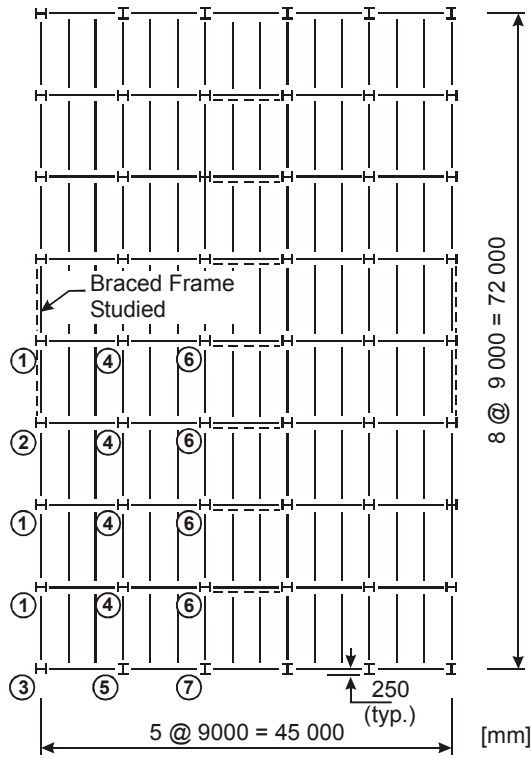
A total of five building heights were considered: 8, 12, 16, 20, and 24 stories. All structures were assumed to be regular in plan and elevation, with the rectangular foot print shown in Fig. 2. The story height at all floors, h_s , was set to 4.0 m. The response of the structures was examined in the direction parallel to the long walls. In this direction, lateral loads are resisted by two identical braced frames located along the exterior walls (see Fig. 2). As typically assumed in Canada and the U.S., all members of the gravity frame are considered as pin-connected at their ends and 100% of the lateral loads are resisted by the bracing bents. For simplicity, accidental in-plane torsional response was neglected and rigid floor diaphragm response was assumed in the design and analysis. The lateral loads were then distributed equally to the two braced frames and a 2D analytical model was used to evaluate the seismic response. Two-story column tiers were employed.

An elevation view of the two different braced frame systems is shown in Fig. 3 for the 8-story building. For both systems, the bracing configuration on each wall consisted of a pair of chevron bracing bents acting in parallel. This geometry combines the high efficiency of chevron bracing for story shear resistance and stiffness and the large overall flexural stiffness resulting from the doubled bracing bent width.

For the regular BRB system (Fig. 3a), four buckling restrained braces are used at each level in each braced frame, and yielding is expected to develop nearly simultaneously in all braces. Therefore, although individual bracing members can exhibit a stable cyclic inelastic behavior, soft-story response can still form as the all-pinned structure possesses limited capacity to force simultaneous brace yielding in adjacent floors after inelastic brace response has initiated at

¹ A. Wada. 2004. Personal communication

a given level. Only the continuity of the columns over two or three stories, as typically encountered in multi-story structures, can provide some restraint against variations in the story drift demand from one story to the next (Tremblay and Stierner 1994; Kimura et al. 2002).



Gravity loads:

Roof: Dead = 3.0 kPa
Live = 1.1 kPa
Floor: Dead = 3.5 kPa
Partitions = 1.0 kPa
Live = 3.8 kPa
Exterior walls = 1.5 kPa

Steel:

BRBs: $F_y = 290$ MPa
Other members:
 $F_y = 345$ MPa

Fig. 2: Plan view of the buildings studied.

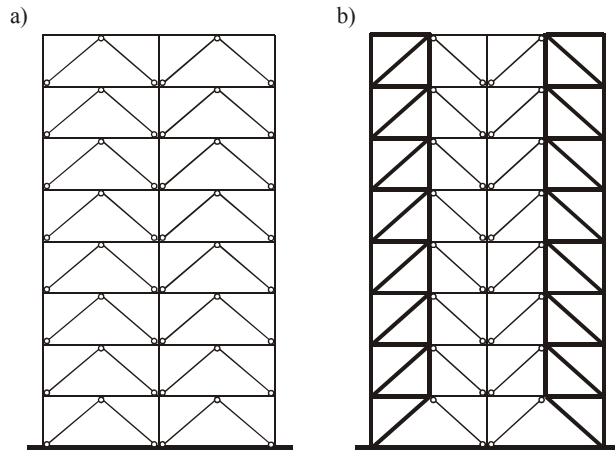


Fig. 3: Elevation of the 8-story: a) BRB system; b) Dual BRB system.

In the Dual BRB configuration, elastic trusses are provided in each braced bay to create the symmetrical pattern shown in Fig. 3b. One main difference compared to Japanese practice is the fact the beams of the gravity frames in the structures studied herein are pin-connected to the columns. In Japan, these beams are typically rigidly joined to the columns, providing elastic back-up lateral stiffness.

2.2 Design Seismic Loads

The buildings were assumed to be located in Victoria, B.C., Canada, and the design was performed according to the upcoming 2005 edition of the National Building Code of Canada (NBCC) (Heidebrecht 2003) and the CSA-S16-01 Standard. The gravity dead load (D), floor live loads (L), and roof snow loads (S) are given in Fig. 2. Earthquake loads are discussed in the next section. In the calculation of the column axial loads, the floor live load was multiplied by the reduction factor $F_r = 0.3 + (9.8/B)^{0.5}$, where B is the cumulated tributary floor area. When considering only gravity loading, the factored loads were obtained from the combinations $1.25D + 1.50L + 0.5S$ and $1.25D + 0.50L + 1.50S$. Load combinations involving earthquake loads (E) were: $1.0D + 0.5L + 0.25S + 1.0E$ or $1.0D - 1.0E$, the latter being used when earthquake and gravity load effects were opposed.

In the 2005 NBCC, the design lateral earthquake load, V , is given by:

$$V = S(T_a) M_v I_E W / (R_o R_d) \quad (1)$$

In this expression, T_a is the period used in design $T_a = 0.025 h_n$, where h_n is the building height. Alternatively, the period obtained from methods of mechanics can be used but that value must not exceed twice the value given by the empirical expression, i.e. $T_a = 0.05 h_n$. The design spectrum S is obtained from 2% in 50 years uniform hazard spectral acceleration ordinates prescribed for the site, $S_a(T_a)$, and the soil class. It is equal to $F_a S_a(0.2)$ for $T_a < 0.2$ s, the smaller of $F_v S_a(0.5)$ and $F_a S_a(0.2)$ at $T_a = 0.5$ s, $F_v S_a(1.0)$ at $T_a = 1.0$ s, $F_v S_a(2.0)$ at $T_a = 2.0$ s, and half this value at $T_a = 4.0$ s. For periods between 0.2 and 4.0 s, S is obtained by linear interpolation. In these expressions, F_a and F_v are respectively the acceleration-based and velocity-based site coefficients. The factor M_v accounts for higher mode effects on base shear, I_E is the importance factor, W is the seismic weight, and R_d and R_o are the force modifications factors related to the ductility and the overstrength of the lateral load resisting system, respectively.

The lateral load V need not exceed 2/3 of the value determined from Equation (1) with $T_a = 0.2$ s but must not be less than the value determined with $T_a = 2.0$ s.

For Victoria, the S_a values at periods of 0.2, 0.5, 1.0, and 2.0 s are respectively 1.2, 0.82, 0.38, and 0.18 g. Firm

ground condition (Site Class C) was assumed in the study, with $F_a = F_v = 1.0$. For braced steel frames built in the western part of Canada, $M_v = 1.0$ for $T_a < 1.0$ s and 1.5 for $T_a > 2.0$ s. For intermediate period values, the product $S \cdot M_v$ is obtained by linear interpolation. The buildings are of the normal importance category with $I_E = 1.0$ and the seismic weight W is given for each building in Table 1. The weight corresponds to the dead load, including half of the partition loads, plus 25% of the roof snow load. The values are for one building.

In the NBCC 2005, the ductility-related factor R_d varies from 1.0 to 5.0 for the less ductile (un-reinforced masonry) to the more ductile (ductile steel moment resisting frame) structural systems, respectively. Values for R_o range between 1.0 and 1.6, depending on the overstrength likely to be present in the system (Mitchell et al. 2003). No values have been specified yet for R_d and R_o for braced steel frames that include buckling restrained braces. For eccentrically braced steel frames, $R_d = 4.0$ is prescribed in NBCC, and the same value was proposed for BRB frames in view of the similitude between the hysteretic response of BRBs and links in eccentrically braced steel frames (Tremblay et al. 2004). Limited overstrength is anticipated in braced frames with BRB members as the core segment of the braces is sized to exactly match the force demand. That value was adopted herein but the influence of using a lower R_d value is also examined in the last section of the paper. A value of 1.3 was selected for R_o to reflect the extra capacity resulting from strain hardening response and the difference between nominal and factored resistances.

2.3 Analysis and Design of the BRB frames

In the NBCC, Equation (1) is intended to be used with the static equivalent lateral force procedure. This method assumes that the force V is distributed over the building height according to an inverted triangular pattern and a concentrated load at the roof level. For regular structures located in moderate and high seismic regions, as was the case here, this method is permitted only for structures less than 60 m in height and when T_a is shorter than 2.0 s. For other structures, the NBCC requires that a dynamic analysis procedure be performed, such as the modal response spectrum method. For that method, the design spectrum S is used to determine the modal contributions. To provide uniformity in seismic protection, the NBCC provisions also require that the results from dynamic analysis be scaled such that the dynamically determined base shear force is not be less than 80% of the value of V determined with Equation (1). The 20% difference in design seismic loads between the two methods reflects the fact that dynamic analysis simulates better the effects of earthquakes compared to the static procedure. For design engineers, this reduction is a strong

encouragement for using dynamic analysis methods.

In this study, a first design trial was obtained for the BRB structures using the equivalent lateral force procedure. Thereafter, modal response spectrum analysis was carried out to obtain a seismic demand reflecting more closely the elastic dynamic properties of the structures, and the design was modified accordingly. Successive modal analysis-design cycles were then performed until convergence was attained. In the process, the period T_a used for the calculation of V was initially set to $0.025 h_n$ and then subsequently modified using the period computed for the structure designed in the previous iteration step. For all BRB frames, the periods T_a was found to reach the upper limit of $0.05 h_n$ permitted in the NBCC and that limit therefore applied.

The values of T_a are given in Table 1, together with the base shear ratio V/W obtained at the end of the design iterative procedure. The base shear values in Table 1 are those obtained from dynamic analysis and then correspond to 80% of the earthquake lateral force given by equation (1). As shown, for the 12-story and taller buildings, the period T_a was longer than 2.0 s and the floor limit on V applied, resulting in the same V/W ratios (2.77%). Table 1 also gives the periods and cumulated participative masses for the first three modes of vibration for all structures.

Table 1: Design parameters and dynamic properties of the buildings studied.

n floors	8	12	16	20	24
W (MN)	114	172	231	289	348
T_a (s)	1.6	2.4	3.2	4.0	4.8
V/W (%)	4.00	2.77	2.77	2.77	2.77
<i>BRB</i>					
T_1 (s)	2.22	3.48	4.28	5.02	5.62
T_2 (s)	0.81	1.24	1.48	1.69	1.93
T_3 (s)	0.50	0.72	0.84	0.93	1.09
M_1 (%)	79	76	73	70	71
M_2 (%)	92	90	89	86	88
M_3 (%)	96	95	94	91	93
V_{max}/V	2.12	1.80	1.85	1.79	1.75
<i>Dual BRB</i>					
T_1 (s)	1.68	2.70	3.43	4.21	4.73
T_2 (s)	0.56	0.90	1.12	1.35	1.53
T_3 (s)	0.30	0.47	0.58	0.69	0.79
M_1 (%)	76	73	70	68	67
M_2 (%)	91	89	87	86	86
M_3 (%)	96	94	93	92	92
V_{max}/V	2.40	2.15	2.11	2.02	1.99

Figure 4a presents the vertical distribution of the story shear obtained from dynamic analysis. In all cases, the shear increases rapidly in the upper floors and then varies nearly linearly towards the base. In Fig. 4b, the story shears from dynamic and static analyses are compared. As implied by the

design procedure, the ratio is equal to 0.8 at the base of all structures. The figure then shows that the demand from dynamic analysis becomes relatively lower in the upper floors, especially at the top floor. It is only at the top two floors of the 8-story building that dynamic analysis is more severe than the static method.

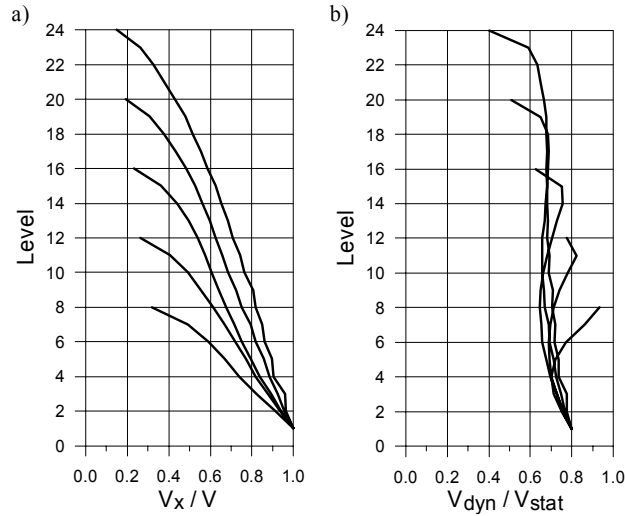


Fig. 4: Story shear forces: a) Design story shear forces from dynamic analysis; and b) Ratio of dynamic to static story shear forces.

The core plates of the BRB members were made with ASTM A36 steel. The cross-section area of the yielding core segment of the braces, A_c , was sized such that its factored axial resistance, P_{br} , was equal to the combined effects of gravity and earthquake loads. In this calculation, the resistance P_{br} was taken as $P_{br} = \phi A_c R_y F_y$, where ϕ is the resistance factor ($\phi = 0.9$ for steel) and $R_y F_y$ is a realistic estimate of the actual steel yield strength for the core plates. The use of $R_y F_y$ reflects current industry practice where the final dimensions of the brace cores are established at the fabrication stage, based on the actual yield strength of the material selected for the braces. A value of $R_y F_y = 290$ MPa was adopted herein to obtain representative brace sizes. The on-center length of the braces, L_w , is equal to 6021 mm and the length of the yielding core segment, L_c , was set equal to 3000 mm. As shown in Fig. 1a, the brace plate outside that segment has larger cross-section area. For analysis purposes, an equivalent BRB cross section area, $A_e = 1.465 A_c$, was assumed over the entire length L_w to give the same overall brace axial stiffness including the influence of the end segments.

Beams and column members in the bracing bents were selected from standard W shapes made of $F_y = 345$ MPa steel. A capacity design procedure was applied to ensure elastic response of these elements during the design level earthquake. These components were designed following applicable steel design standard requirements (CSA 2001) to

sustain member forces induced by gravity loads acting in combination with lateral load effects corresponding to the expected axial capacity of the BRB members, $P_u = \beta R_{sh} P_{by}$. In this expression, β accounts for the friction that can develop between the core and the fill material ($\beta = 1.10$), R_{sh} accounts for maximum strain hardening effects anticipated in the core ($R_{sh} = 1.25$), and P_{by} is the expected brace yield strength ($= A_c R_y F_y$). For the central column of the bracing bents, the vertical force components acting in the two BRB connecting at a given floor would cancel assuming both brace forces are equal. In practice, it is expected that differences will exist between the two braces. For instance, one brace will be in compression and develop a higher resistance due to frictional response compared to the tension brace. Upon yielding, strain hardening in the tension and compression braces may not be exactly the same. In design, a downward vertical load equal to 20% of the brace capacity P_{by} was therefore added at every floor level to account for this behavior. All columns were assumed to be made of two-story segments.

P-delta effects were neglected in the design process. As per NBCC, lateral deflections obtained from analysis were multiplied by $R_o R_d / I_E$ to obtain realistic estimates of the anticipated deflections, including inelasticity effects. For buildings of the normal importance category ($I_E = 1.0$), the resulting story drifts must be limited to $0.025 h_s$. In the design process, drifts were verified and were found to exceed the limit only for the 24-story BRB frame. For that building, the size of the outer columns of the bracing bents was increased in the lower floors to correct this problem, without inducing any additional story shear resistance to the structure. Design for wind loading in the direction studied was performed and it was found that strength and drift requirements under this loading condition were not critical, even for the taller buildings.

2.4 Design of the Dual BRB frames

The role of the elastic vertical truss in the Dual BRB system is to prevent concentration of inelastic demand in the braces along the building height. Therefore, member forces in this truss system depend on the stiffness of the elastic truss and on the variation between adjacent floors of the brace plastic deformations experienced under earthquake ground motions. These forces cannot be predicted using elastic static or dynamic analysis methods. They can only be evaluated from nonlinear time history analysis of the system under a representative ensemble of ground motions, and an iterative design approach involving several nonlinear dynamic analysis-design cycles would be needed to reach a satisfactory response with the minimum amount of steel.

In this study, a simplified design approach is proposed which could be easily implemented in practice to obtain a

preliminary design of the system. Nonlinear dynamic analysis would then be needed to validate and refine that preliminary solution. In this approach, the BRB members are first selected to meet the same strength requirements as in a regular BRB frame. The braces in the elastic truss are made of standard steel tubular or W shapes sized to resist an axial compression load equal to α times the yield strength P_{by} of the adjacent BRB member. The factor α must be large enough to prevent inelastic response. Since the demand on the elastic truss depends on the relative inelastic response of BRB members in several adjacent stories, the same elastic brace member can be used for a number of consecutive floors.

In this study, the BRB members selected for the BRB frames were used in the corresponding Dual BRB buildings, allowing direct comparison between the two systems. The same elastic brace was used for groups of four successive stories. That brace was designed at the lowest level of the group of four stories using $\alpha = 4.0$. The amplification factor of 4.0 corresponds to the R_d value adopted in the design of the frame, assuming that the maximum vertical shear in the elastic truss would be limited to the elastic frame response.

In the bracing bent, the central column located between the two rows of BRB members are designed following the procedure adopted for the BRB frame system. The exterior columns are designed for the combination of the gravity loads and the vertical component of the amplified axial brace loads, as shown in Fig. 5. In order to account for the fact that the likelihood that all braces above the level under consideration develop simultaneously their amplified axial load decreases with the number of stories, the “square root of the sum of the square” (SRSS) approach proposed by Redwood and Channagiri (1991) was implemented to cumulate the contribution of the brace loads from the roof to the level above the level under consideration. The intermediate column located between rows of elastic braces and BRB members are designed for earthquake loads equal to the vertical unbalanced forces the BRB braces yield in compression and the elastic braces approach yielding in tension (see Fig. 5). The beam segments that are part of the elastic trusses are designed to resist the horizontal component of the elastic brace yield tensile strength.

The objective of this design strategy is to ensure that yielding will be constrained to the braces of the elastic truss in case of localized excessive demand on the elastic truss system during an earthquake. The authors realize that this procedure is probably too conservative and does not necessary lead to the optimum solution for a given building. The adequateness of the method will be briefly discussed when examining the performance of the structures in the subsequent sections of the paper. However, since the main purpose of this project was to assess the potential for Dual

BRB frames to improve the seismic stability of multi-story braced frames, limited effort was devoted at this phase of the project on the development of optimum cost-effective design solutions for the elastic truss system. Refinements towards this direction are planned in the subsequent phases of the research.

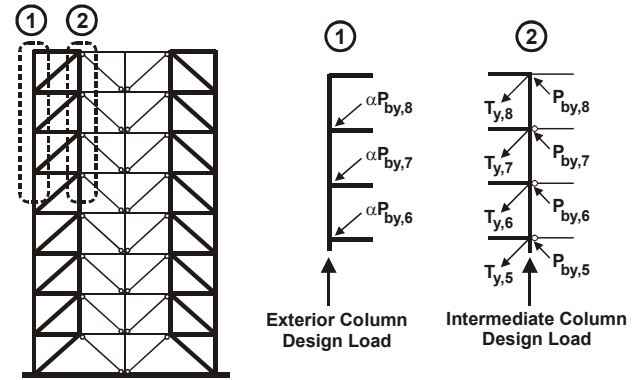


Fig. 5: Design of the exterior and intermediate columns in the Dual BRB frame system.

3 ANALYSIS PROCEDURE

3.1 Modeling Assumptions

Nonlinear time step dynamic analysis of the structures was performed using the RUAUMOKO computer program (Carr 2002). As indicated earlier, torsional response and in-plane diaphragm deformations were neglected, and a two-dimensional numerical model could be used to determine the seismic response of the symmetrical building structures. The model included one of the two bracing bents acting in the direction parallel to the long walls as well as the tributary leaning gravity columns, as illustrated in Fig. 6 for the 8-story BRB frame.

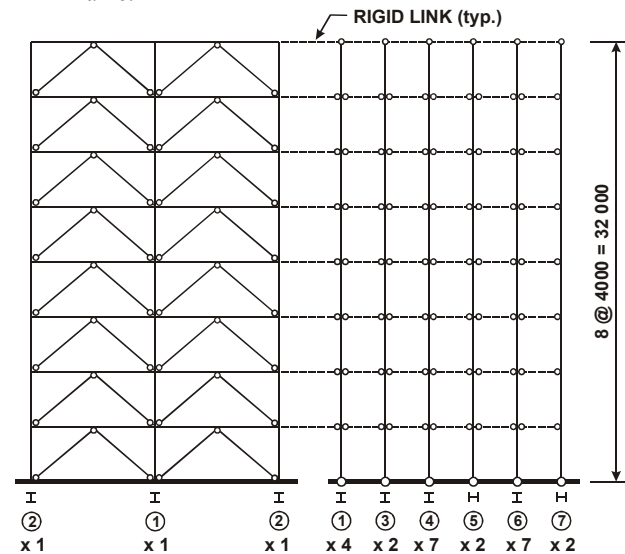


Fig. 6: Analytical model of the 8-story BRB frame with tributary gravity columns.

Buckling restrained braces were modeled using a symmetrical Ramberg-Osgood formulation that was modified to prevent off-sets of the forces in small amplitude cycles (Pyke model) and to include the isotropic/kinematic (I-K) strain hardening model proposed by Nakashima et al. (1995). The Ramberg-Osgood multiplier, α , was set equal to 1.0 for all braces. The weighting coefficient, β , and the Ramberg-Osgood factor, r , were adjusted to match the results from tests. The yield strength in tension was set equal to P_{by} defined earlier, and yielding in compression was adjusted to $1.10 P_{by}$ to account for frictional response. Figure 7 shows the correlation between the predicted hysteretic response and that measured during Test C1-1 by Tremblay et al. (2004). In this comparison, axial strains in the test brace core segment were used to obtain brace axial deformations for the brace geometry of the buildings studied. As shown, excellent correlation is obtained with the hysteretic model. The BRB members were considered as pin-connected to the other framing members.

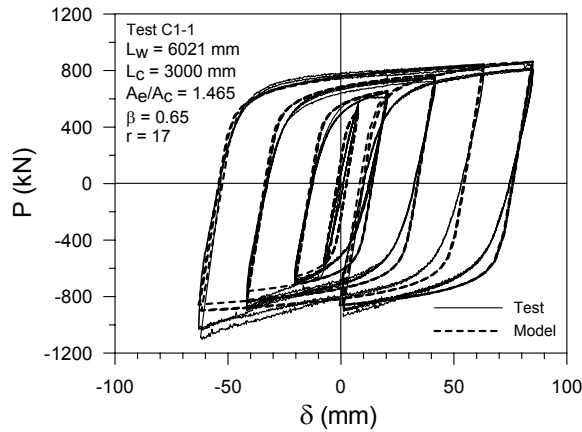


Fig. 7: Comparison between test and predicted buckling restrained brace hysteretic response.

Beams and columns, as well as the braces in the elastic trusses of the Dual BRB system, were modeled using frame elements with bilinear plastic hinges concentrated at their ends. These elements were also capable of axial yielding in tension and compression. The yielding strength in compression was set equal to the compression resistance of the members, including buckling effects. Perfectly rigid beam-to-column connections were assumed at all joints. The columns of the bracing bents were assumed to be fixed at their bases whereas pinned connections were considered at the base of the gravity columns. Splice connections in columns were assumed to be fully rigid in bending. At every level, the lateral displacements of the gravity columns were constrained to be the same as the lateral deformations of the bracing bents.

A Newmark constant acceleration integration scheme with a maximum time step of 0.001 s was used. Large

displacement analysis was performed with 100% of the dead load, 50% of the floor live load, and 25 % of the roof snow load applied on the structure. Rayleigh damping proportional to initial stiffness was used with 3% of critical damping in the first and third vibration modes.

3.2 Ground Motion Record Ensembles

Two earthquake record ensembles were used in the study. The first ensemble included four simulated and six historical ground motion time histories produced by intra-plate seismic events matching the two dominant magnitude-hypocentral distance scenarios for the Victoria region: **M**6.5 at 30 km and **M**7.2 at 70 km (Tremblay and Atkinson 2001). All motions are for firm soil conditions. The amplitude of the ground motions was adjusted to match, on average, the design spectra over the applicable period range. The description of the ground motions and the adjustment factors are given in Table 2. The 5% damped response spectra of the scaled records are compared to the design spectrum in Fig. 8a.

Table 2: Design parameters and dynamic properties of the buildings studied.

No.	Event	R (km)	Adjust. factor
S1	Simulated M_w 6.5, Trial #1	30	1.05
S2	Simulated M_w 6.5, Trial #4	30	1.05
S3	Simulated M_w 7.2, Trial #1	70	1.20
S4	Simulated M_w 7.2, Trial #2	70	1.20
S5	M_s 6.1 1984 Morgan Hill, San Ysidro, Gilroy #6, 90°	38	1.20
S6	M_w 6.7 1994 Northridge Castaic, Old Ridge Road 90°	44	0.80
S7	M_w 6.7 1994 Northridge Castaic, Old Ridge Road 0°	44	0.70
S8	M_w 7.0 1989 Loma Prieta Stanford Univ. 0°	54	1.30
S9	M_w 7.0 1989 Loma Prieta M_w 7.0, Presidio 90°	100	1.40
S10	M_w 7.1 1949 Western Wash. Olympia, Test Lab., 86°	76	1.80

In the second ensemble, four ground motion time histories were simulated for four possible rupture scenarios along the Cascadia subduction fault plane (Tremblay and Antkinson 2001). A finite-fault simulation method with multiple subsources distributed along the fault plane was used to represent **M**8.5 Cascadia earthquakes. That method has been tested against observed near-field ground motions at rock sites from many large earthquakes, including the **M** 8.0 1985 Michoacan, Mexico and the **M** 8.0 1985 Valparaíso, Chile earthquakes (Beresnev and Atkinson 1997, 1999). The records were scaled to match the site specific design spectrum representing the Cascadia subduction earthquake

hazard for Victoria (Fig. 8b) (Adams et al. 2003). Figures 8c and 8d provide examples of both types of ground motions.

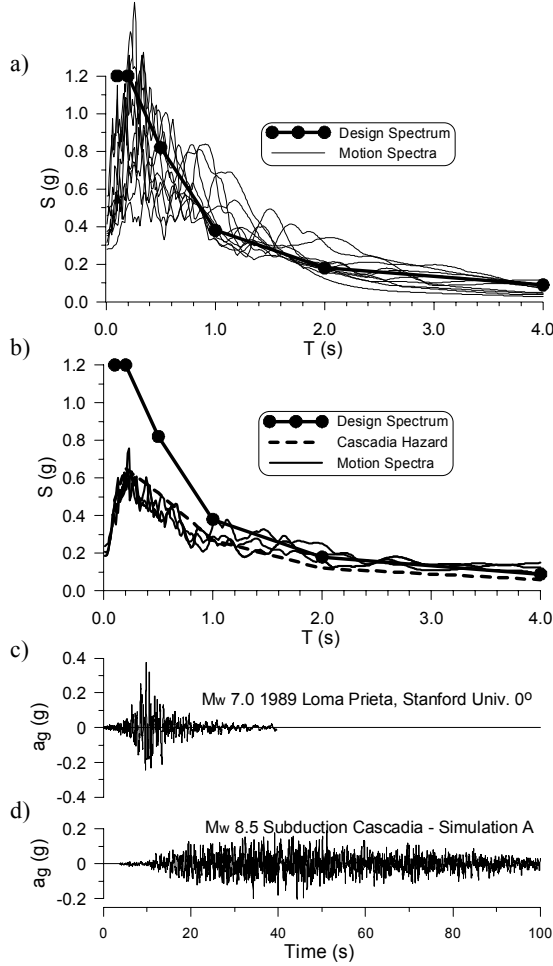


Fig. 8: Design UHS and scaled ($S_F = 1.0$) ground motions 5% damped acceleration spectra: a) Crustal and sub-crustal time histories; b) Subduction time histories; c) Typical crustal event; and d) Typical subduction event simulation.

3.3 Incremental Dynamic Analysis Procedure

The performance of the structures was first examined for the design earthquake level, i.e. under the ground motion time histories adjusted to match the design spectra, as discussed earlier. Several response parameters were examined for that ground motion level. Thereafter, an incremental dynamic analysis (IDA) procedure was applied to assess and compare the seismic performance of the various structural systems.

For the IDA procedure, the amplitude of the ground motion time histories was multiplied by a scaling factor, S_F , that was increased in successive 0.1 increments from $S_F = 0.2$ up to the collapse of the structures. It is noted that the case $S_F = 1.0$ corresponds to the design ground motion level. For each ground motion and scaling factor value, the maximum value of the peak story drift ratios ($\%h_s$) over the building height was retained and plotted against the scaling factor. Figure 9 shows an example of one set of 10 individual

IDA curves computed for a particular structure, as well as the corresponding 16%, 50% and 84% fractile IDA curves that were subsequently obtained from the individual curves by joining the fractile quantities computed at each scaling factor ordinate (Vamvatsikos and Cornell 2002a, 2002b). In the next section, the 84th fractile IDA curves are used to evaluate and compare the performance of the structures.

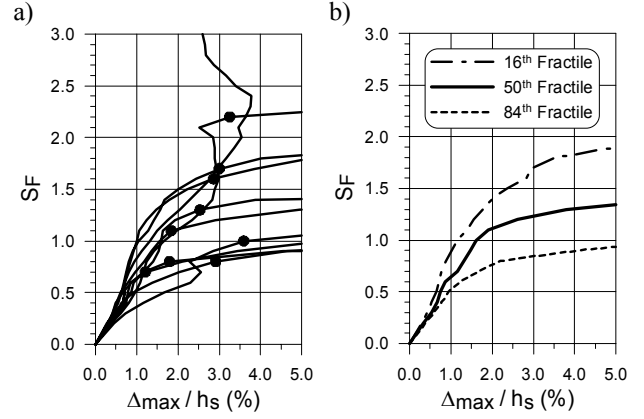


Fig. 9: Incremental dynamic analysis of the 16-story BRB frame under crustal and sub-crustal earthquakes: a) Individual IDA curves with global drift capacity; b) Fractile curves.

4 ANALYSIS RESULTS

4.1 Static Incremental Analysis

Nonlinear static incremental (push-over) analysis was performed before carrying out the time history analyses. The lateral loading pattern associated to the design story shear from modal analysis was used in these analyses. The lateral load-lateral roof deformation response for the 12- and 24-story structures is shown in Fig. 11. Figure 12 illustrates the lateral deformation patterns obtained from the same two buildings when the roof drift reached $0.01 h_n$ and $0.02 h_n$.

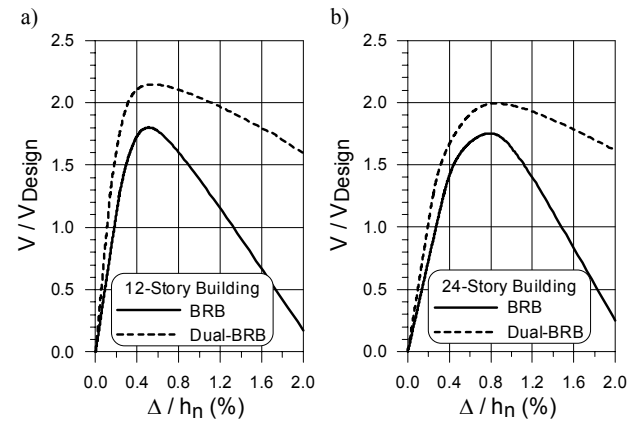


Fig. 10: Lateral load-lateral roof deformation response under incremental static loading corresponding to design story shear: a) 12-story building; and b) 24-story building.

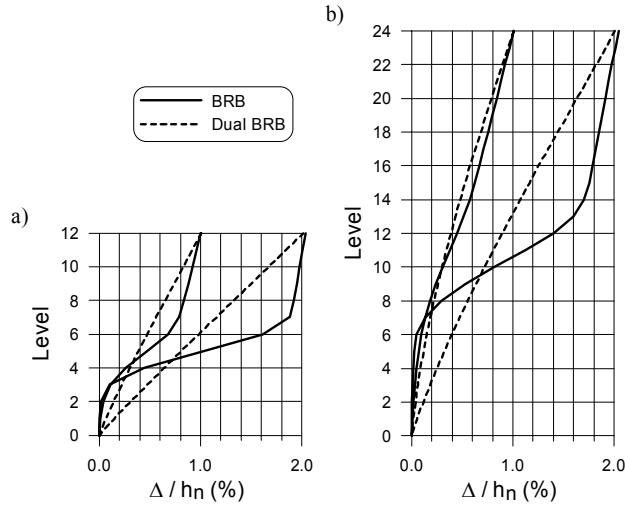


Fig. 11: Lateral deformation profile under incremental static loading at 1% and 2% roof drift: a) 12-story building; and b) 24-story building.

In both figures, the two structures of each braced frame system behaved very similarly, regardless of the building height. Similar responses were also obtained for the 8-, 16- and 20-story buildings. The BRB structures exhibit a stable response until peak lateral load is reached. After that point, the lateral resistance decreases very steeply due to gravity loads acting on the intermediate floors undergoing large lateral deformations due to yielding of the braces at these levels. For the Dual-BRB frames, the elastic trusses forced yielding to develop in the braces at every floor, resulting in a more uniform deformed shape and less dominant P-delta effects.

Table 1 gives the ratios V_{max}/V obtained in the push-over analyses, where V_{max} is the maximum lateral capacity reached in the analysis and V is the design base shear. As shown, these ratios vary between 1.75 and 2.12 for the BRB system and between 1.99 and 2.40 for the Dual-BRB system. In both cases, higher V_{max}/V values are found for the lower buildings, most likely because the back-up moment frame response from beams and columns acting in flexure is relatively more significant for these structures. On average, the dual BRB frames have 13% higher capacity, essentially due to the fact that the response is constrained to follow the first mode shape response and to the higher column flexural capacities.

4.2 Overall Behavior under Design Level Earthquakes

In this section, the global response of the structures as observed in the nonlinear dynamic analyses at the design ground motion level is presented and discussed. Detailed results of the structure responses under these time histories are presented in the next section.

In several analyses, very large lateral displacements

developed in the structures and collapse by dynamic instability occurred due to gravity loads acting on the deformed structure. In this paper, collapse by dynamic instability is said to have occurred when the story drift in any given floor exceeded 5% of the story height, h_s . Table 2 summarizes the occurrences of collapse by dynamic instability for each building height, frame system and ground motion ensemble. Instability is more frequent for the lower buildings (8-16 stories), the BRB frames, and for the long duration Cascadia subduction earthquakes.

Table 3: Occurrences of dynamic instability.

n floors	8	12	16	20	24
<i>BRB</i>					
Crustal (/10)	3	5	3	0	0
Subduction (/4)	4	4	4	3	4
<i>Dual BRB</i>					
Crustal (/10)	0	0	0	0	0
Subduction (/4)	1	2	0	0	0

The influence of the height can be explained by the fact that the 12-story and taller structures were designed for the same V/W ratio, due to the minimum floor value for V prescribed in the NBCC when T_a exceeds 2.0 s (see Table 1). Since the force demand from ground motions typically decreases with the period, the demand-to-supply resistance ratio decreased with the building height, resulting in better performance for the taller buildings.

In the BRB frames, higher mode response could easily develop in the inelastic range, which led to significant concentration of inelastic demand over the height, as illustrated in Figs. 12 to 14 for the 12- and 24-story structures subjected to one typical record from each ensemble. Figures 12 and 13 show the computed time history of the lateral deformation at the roof level for the two building heights and the two braced frame systems. Figure 14 illustrates snap shots of the lateral building displacements near collapse or at peak roof deformation. As shown, the 12-story BRB frame collapsed under both ground motions due to progressive accumulation of plastic deformations towards one direction, also referred to as ratcheting effect (Gupta and Krawinkler 2000). For that structure, the deformation mainly developed at the intermediate floors, very similarly to what was observed under the static incremental analysis (see Figs. 11 and 14).

The 24-story BRB frame remained stable under intra-plate ground motion S10 but experienced nearly the same collapse response as the 12-story building under the long duration Cascadia earthquake B. In this case, inelastic deformations concentrated more in the lower floors, as illustrated in Fig.

14. It is clear from Table 3 that inter-plate earthquakes were more critical in terms of dynamic instability, as all BRB frames collapsed under all motions except in one case. For short period structures, the force demand from these ground motions is lower than that imposed by crustal and sub-crustal earthquakes (see Fig. 8). As the period is increased, the difference diminishes and subduction earthquakes eventually generate higher building accelerations than inter-plate events (40% more, on average, at $T = 4.0$ s). That force demand remains smaller, however, than the minimum design elastic force level determined at $T = 2.0$ s (0.18 g). The much longer duration of the inter-plate shaking certainly has a major effect on the building stability response since the progressive build-up of plastic deformations is more likely to develop and reach a critical condition if strong ground motion is maintained over a longer period of time, as was the case here.

More stable response was observed for the Dual BRB structures. Three occurrences of collapse were observed: one for the 8-story frame and two for the 12-story building. All failures developed under the simulated Cascadia ground motions. These trends are identical to those observed for the BRB frames and the same reasons can also apply for these frames. As noticed under the push-over analysis, the inelastic response of all frames under the ground motions mimicked that of the first mode, thanks to the elastic truss systems which played well their role of a back bone preventing any marked deviations in the inelastic story drift demand. This response was also observed for the structures which collapsed, as can be seen for the Dual BRB 12-story frame subjected to the Cascadia ground motion (Figs. 13 and 14a). For these structures, the deformation pattern remained nearly perfectly straight from the foundation to the roof level.

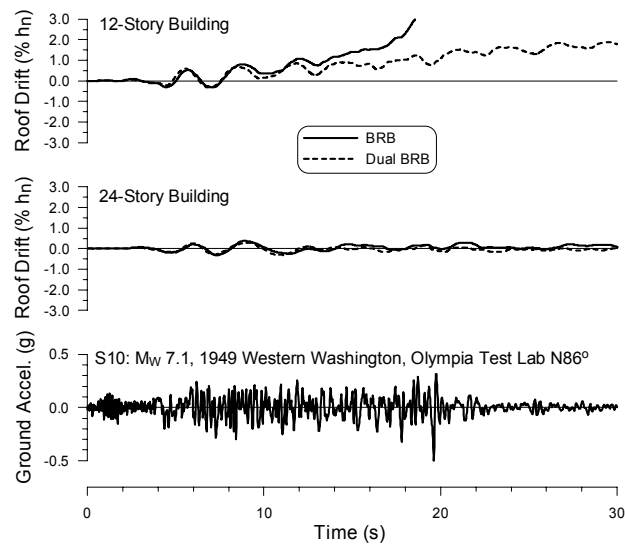


Fig. 12: Time history of the lateral roof deformation of the 12-story and 24-story buildings under crustal earthquake ground motion S10.

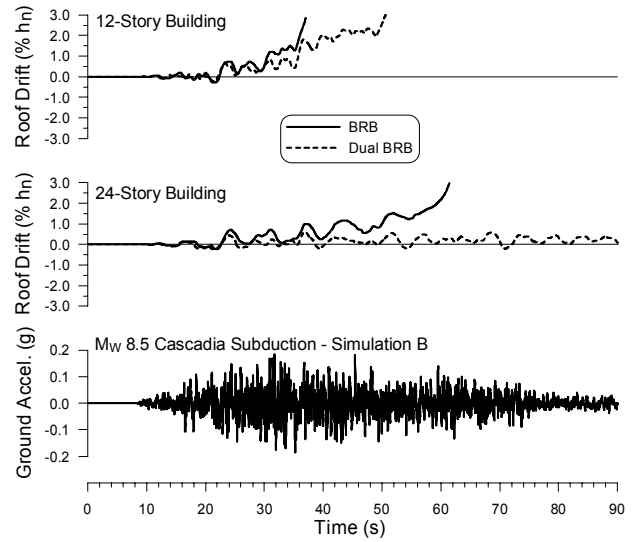


Fig. 13: Time history of the lateral roof deformation of the 12-story and 24-story buildings under simulated M_w 8.5 Cascadia earthquake ground motion B.

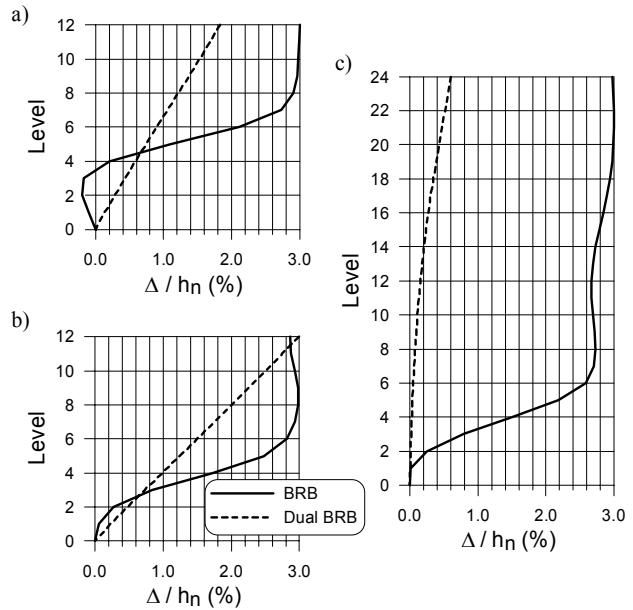


Fig. 14: Laterally deformed shape near collapse: a) 12-story frames under crustal ground motion S10; b) 12-story frames under subduction ground motion simulation B; and c) 24-story building under subduction ground motion simulation B. (In Figs a and c, the Dual BRB frame did not collapse and the deformation is plotted at maximum roof deformation.)

4.3 Response under Design Level Earthquakes

The first two parameters that are used to evaluate the seismic performance of the structural systems are the peak story drift and the peak horizontal floor acceleration. The

story drift is expressed as a percentage of the story height, h_s , whereas the floor acceleration is given as a fraction of the peak ground acceleration (PGA) of the applied ground motion. Story drift values can be directly compared to code limits ($2.5\% h_s$ in this case) and represent a good measure of the damage to the framing system, building façades and windows, partitions, elevators, and mechanical equipment. Floor accelerations create damage to ceiling and light systems, the electrical and mechanical equipment, and the building contents.

The variation of the peak story drift along the building height is a good indicator of the extent of concentration of the inelastic demand and was therefore also examined in this study. For each ground motion, the coefficient of variation (COV = standard deviation divided by the mean) of the peak story drift values over the height of the structure was used to assess this behavior.

The fourth response parameter considered herein is the peak strain demand in the core of the buckling restrained braced elements. This parameter reflects the extent of plasticity experienced by the brace core material and can be used to determine whether the braces need to be replaced after the earthquakes. Using this parameter to compare the response of two structural systems also provides information on the likelihood of damage concentration as well as on the dependency of the system on hysteretic energy dissipation to withstand the ground motions. The peak strain demand on the BRB cores, ε_c , is obtained from the computed peak total brace elongation, δ_e , and peak brace force P :

$$\varepsilon_c = \left[\delta_e - P \left(\frac{1}{K_e} - \frac{1}{K_c} \right) \right] \frac{1}{L_c} \quad (2)$$

, where $K_e = EA_e/L_w$ and $K_c = EA_c/L_c$ (with $A_e/A_c = 1.465$). The second term in brackets corresponds to the brace elastic deformation outside of the core segment.

For all four parameters, the maximum of the peak values obtained over the building height are kept for each ground motion. The 50th and 84th fractiles of these maximum values are determined for each ground motion ensemble. The results are presented in Figs. 15 to 18 for all four response parameters. Values for the collapsed buildings were omitted in these calculations. Cases for which values are missing are indicated by a “*” in the figures. Statistical results are not presented for the BRB frames and the 12-story Dual BRB frames subjected to subduction earthquakes.

In Fig. 15, the use of a Dual BRB system resulted in generally lower and more consistent peak story drift values under the intra-plate earthquakes. For this ground motion ensemble, the 84th fractile results meet the 2.5% NBCC limit for all building heights constructed with the Dual BRB system, which is not the case for the BRB frames. For all Dual BRB structures, the subduction earthquakes induced

larger story drift values, in excess of the code limit for the 16-story building. As anticipated from previous discussions, the BRB frames experienced more pronounced variations of the story deformation along their height. Figure 16 also shows that crustal and sub-crustal earthquakes are more critical for soft-story formation compared to subduction earthquakes, likely because they contain large acceleration pulses. It is also clear from the figure that the likelihood of damage concentration increases with the building height, even if the Dual BRB system is adopted. Peak acceleration ratios are generally higher for the Dual BRB frames, which is attributed to the fact that these structures have shorter periods of vibrations (see Table 1) and to the presence of the elastic truss that likely resulted in larger horizontal forces.

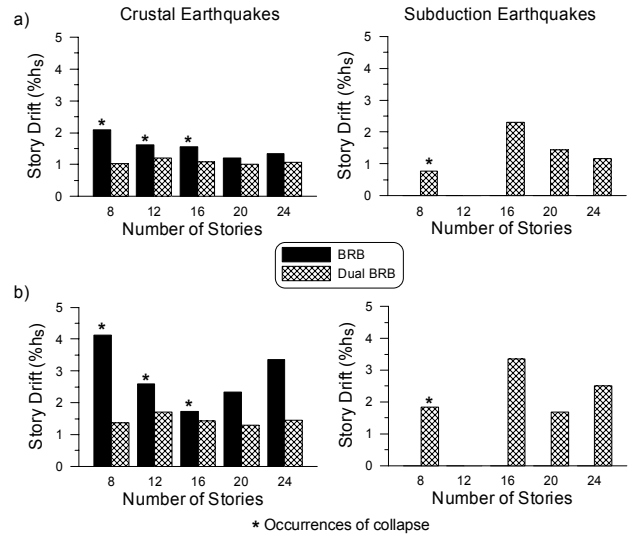


Fig. 15: Statistics of the maximum peak story drifts: a) 50th fractile; b) 84th fractile.

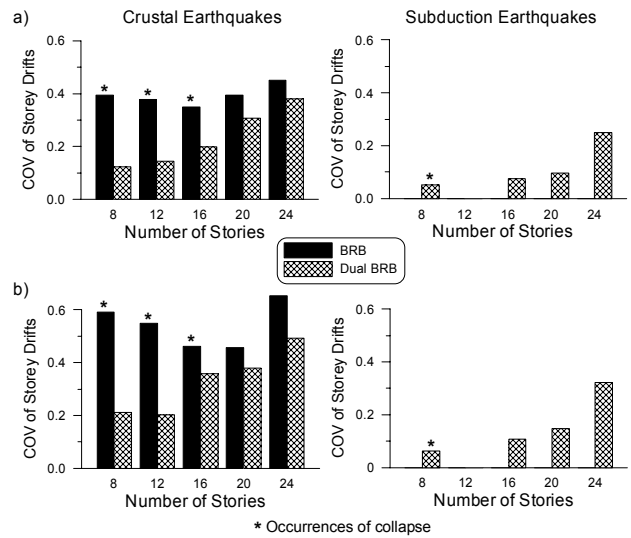


Fig. 16: Statistics of the COV of peak story drifts: a) 50th fractile; b) 84th fractile.

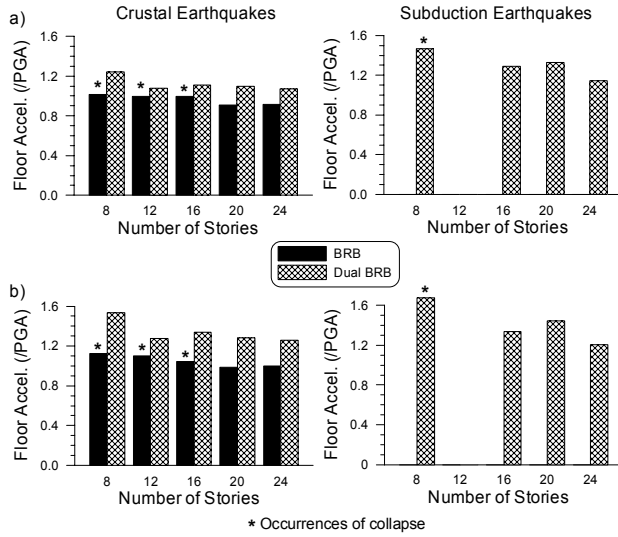


Fig. 17: Statistics of the maximum peak horizontal floor accelerations: a) 50th fractile; b) 84th fractile.

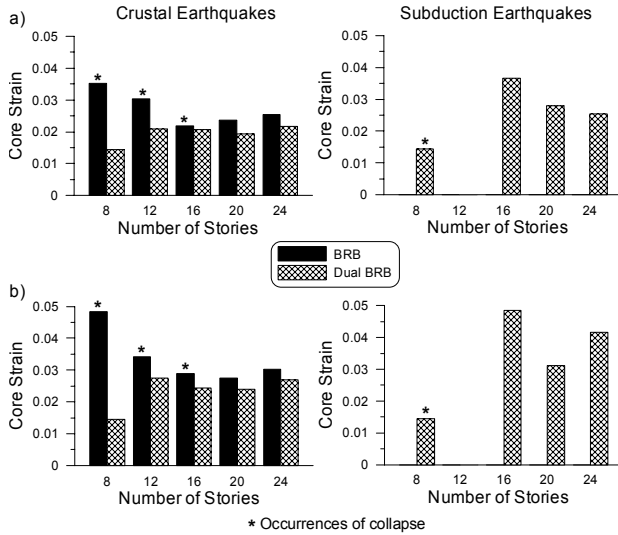


Fig. 18: Statistics of the maximum brace core strain demand: a) 50th fractile; b) 84th fractile.

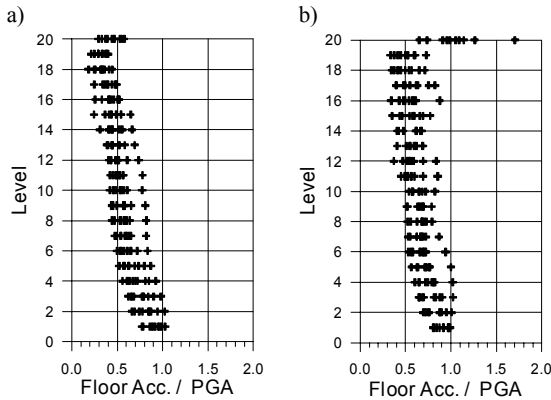


Fig. 19: Peak floor accelerations computed in the 20-story building under intra-plate ground motions: a) BRB frame; b) Dual BRB frame.

Subduction earthquakes also induced relatively larger floor accelerations in the structures. Interestingly, however, the acceleration levels computed in all cases do not vary much with the building height and are remarkably close to the PGA values. Inertia forces and, thereby, floor accelerations, were constrained by the limited lateral resistance of the structures, as reduced by applying the force modification factor R_d . Figure 19 shows peak floor acceleration ratios for each of the 10 intra-plate earthquakes for the 20-story structures. Floor accelerations generally reduce along the building height of the BRB frames, due to yielding of the BRB elements, which is contrary to acceleration profiles typically prescribed in codes for the design of building components. The same trend exists for the Dual BRB frame except that higher accelerations developed at the roof level due to the elastic truss response.

The values of the brace core demand presented in Fig. 18 are consistent with the peak story drift results discussed earlier. The 84th fractile strain values for the BRB system are typically higher than the 0.01-0.02 range anticipated for typical BRB applications. For the Dual BRB system, the 84th fractile results are closer to that range for the intra-plate earthquakes and could probably be reduced by adjusting the length of the brace core segment. However, the demand imposed by the subduction earthquakes on the braces of the dual system would likely require an increase in the design load to control the inelastic deformations.

The comparison of the peak strain values computed for the two systems under the intra-plate earthquake illustrates well the superior inelastic behavior of the Dual BRB system. The strain demand imposed on the braces in that system is lower than that obtained in the BRB frames, despite the fact that the Dual BRB frames include only half the number of BRB elements, indicating a more efficient use of the energy dissipation capacity available.

Figure 20 presents the 84th fractile values of the peak axial load computed in the braces of the elastic trusses of the Dual BRB frames. These results are normalized with respect to the value that was used in the design. This permits to validate the assumptions made regarding the maximum forces that would develop in the elastic truss assemblies. As shown, the predictions were conservative except at the first level of the 8- and 12-story structures. For these structures, the brace forces at the first floor should be increased by approximately 20% to ensure full elastic brace response. At upper level, the forces assumed in the design appear to be appropriate. For the 16-story and taller buildings, a reduction of the design forces could be justified for the intermediate and upper floors. It is anticipated that such a relaxation will have limited impact on the global response of these buildings, provided that the members of the elastic truss possess sufficient strength to respond in the elastic range.

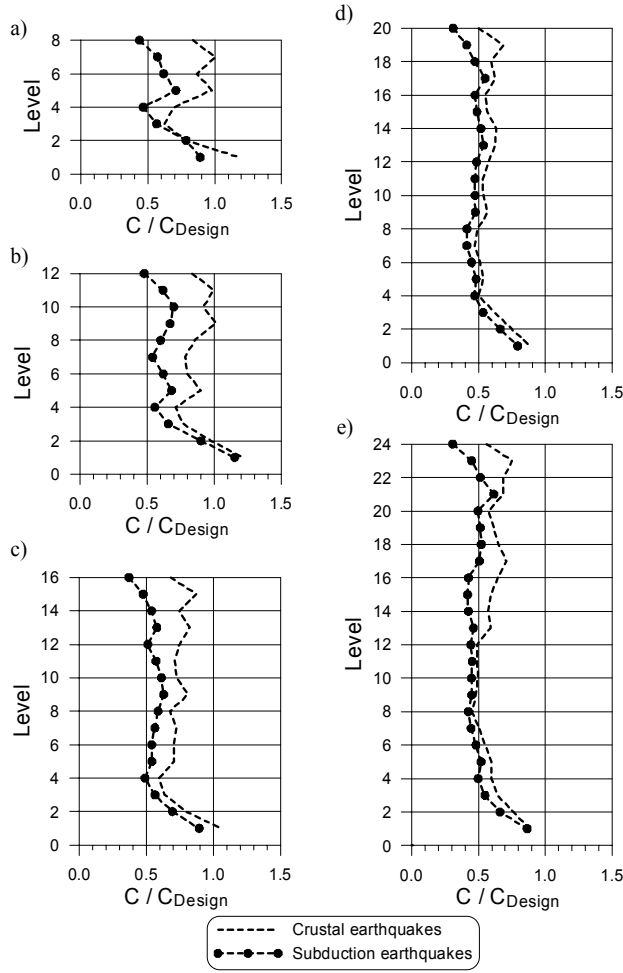


Fig. 20: Ratio of the computed brace forces (84th fractile) to the design brace forces for the Dual BRB elastic braces: a) 8-story building; b) 12-story building; c) 16-story building; d) 20-story building; and e) 24-story building.

4.4 Incremental Dynamic Analysis Results

The 84th fractile IDA curves obtained for all structures and ground motions are plotted in Fig. 21. The plots demonstrate the benefits of using a Dual BRB system and the relatively higher demand imposed by the long duration subduction ground motions. Clearly, the BRB frames do not possess enough capacity to sustain either type of ground motions. For the Dual BRB system, the reduction in capacity is very significant when subduction earthquakes are applied in lieu of the intra-plate earthquakes. Such behavior could not be predicted from the static push-over results presented in Fig. 10.

Using the IDA curves, the confidence level that the different structures will not collapse under a given ground motion ensemble can be estimated following the procedure that was developed for steel moment resisting frames in the FEMA 350 document (FEMA 2000, Yun et al. 2002). According to that method, a confidence index parameter, λ , is first determined from the factored demand-to-capacity

ratio given by:

$$\lambda = \frac{\gamma \gamma_a D}{\phi_R \phi_U C} \quad (3)$$

In that equation, γ is the demand variability factor, γ_a is the analysis uncertainty factor, D is the median estimate of the demand as obtained from structural analysis, ϕ_R is a resistance factor that accounts for randomness in capacity, ϕ_U is a resistance factor that accounts for uncertainty in relationships between tests and actual behavior in real buildings, and C is the median capacity estimate.

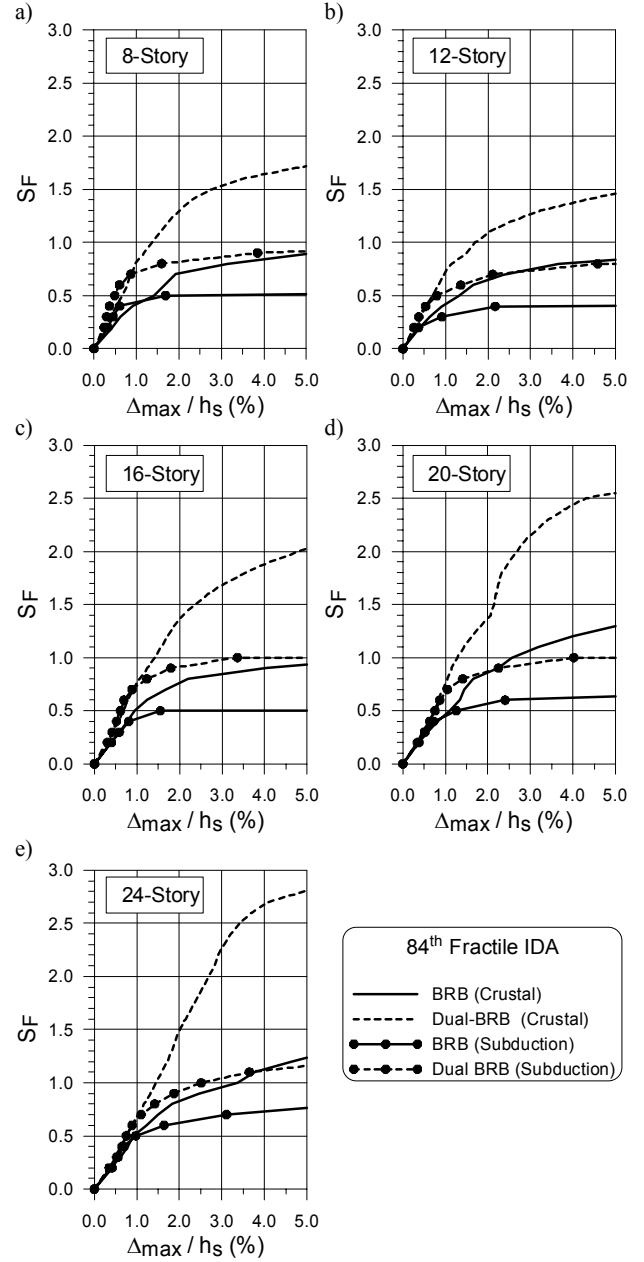


Fig. 21: Ground motion amplitude-maximum peak story drift (84th fractile) relationship for the: a) 8-story building; b) 12-story building; c) 16-story building; d) 20-story building; and e) 24-story building.

Once λ is known, the confidence level is back-calculated from:

$$\lambda = e^{-\beta_{UT} \left(K_x - \frac{k}{2b} \beta_{UT} \right)} \quad (4)$$

, where β_{UT} is an uncertainty factor related to the assessment of demands and capacities, K_x is the Gaussian variate associated with the probability x of not being exceeded as a function of number of standard deviations above (positive) or below (negative) the mean, k is the logarithm slope of the hazard curve, and b is the ratio between incrementing demand and incrementing ground motion amplitude. The general expressions for the γ and ϕ factors of Equation (3) are respectively:

$$\gamma = e^{\frac{k}{2b} \beta^2} \quad (5)$$

$$\phi = e^{-\frac{k}{2b} \beta^2} \quad (6)$$

, where β accounts for uncertainty in demand and capacity estimation.

For the buildings studied, the parameter b was taken equal to 1.0 and a value of $k = 2.39$ was determined between 10% and 2% in 50 years hazard levels, using spectral ordinates at a period of 2.0 s for the site. The uncertainty coefficient β used in Equation (5) to determine γ is equal to the standard deviation of the logarithm of the story drifts at $S_F = 1.0$. A value of 1.10 was adopted for γ_a , as recommended for 12-story and taller moment resisting frames in the FEMA document. The value of D in Equation (3) corresponds to the median estimate of the maximum peak story drifts in any story for $S_F = 1.0$. Using $\beta = 0.25$ as proposed in FEMA for tall moment-resisting frames, the parameter ϕ_i in Equation (3) were set equal to 0.93, respectively. The capacity parameter C is the median value of the story drifts at collapse. For a given ground motion, the story drift at collapse was taken as the point along the IDA curve where the slope of the curve becomes less than 20% of the initial slope. Story drifts at collapse are indicated by a solid dot along the individual IDA curves in Fig. 9. In Equation (6), β is taken equal to the standard deviation of the logarithm of the story drift values at collapse to determine ϕ_R . In absence of values for braced steel frames, β_{UT} in Equation (4) was set equal to 0.50, as recommended in the FEMA procedure for the global interstory drift evaluation for special steel moment-resisting frames taller than 12 stories.

Table 4 presents the computed values of the confidence levels (in %). Note that the calculations were not performed for the BRB frames and the 12-story Dual BRB frames under Cascadia ground motions. For these cases, dynamic instability occurred under more than 50% of the records at the design ground motion level, clearly indicating unsatisfactory performance. In the FEMA procedure, a minimum level of confidence of 90 % is suggested for the

global collapse limit state. This level was not achieved for none of the BRB frames subjected to the intra-plate earthquakes. The use of a Dual BRB system for this seismic hazard was found to provide satisfactory performance level for all building heights. The higher confidence levels exhibited by the taller buildings are a consequence of the conservatism resulting from applying a minimum seismic design load for the design of long period structures. As shown, the performance of the Dual BRB system under the subduction earthquakes was not sufficient for the intermediate building heights.

Table 4: Confidence level (%) for collapse prevention.

n floors	8	12	16	20	24
<i>BRB frames</i>					
Crustal	< 1	< 1	< 1	63.7	70.5
Subduction	-	-	-	-	-
<i>Dual BRB frames</i>					
Crustal	92.8	93.4	99.2	99.9	99.9
Subduction	97.7	-	17.1	75.8	97.4

4.5 Influence of the Design Seismic Loads Level

Inadequate performance was obtained for BRB frames under both types of earthquakes and for some of the Dual BRB frames under the subduction earthquakes. All these structures were designed using a ductility-related factor R_d of 4.0 and it is anticipated that their response could be improved by increasing their lateral resistance to reduce the inelastic demand on the bracing members. The sensitivity of the response to the design lateral resistance is evaluated by examining the response of the 12-story frames designed with two higher levels of seismic loads. That building height was selected because of the higher demand imposed on the frames of this height in the previous series of analysis. For the second design, the increment of resistance was obtained by scaling the results of the dynamic analysis in the design process such that the dynamic base shear was equal to the force V determined with Equation (1). The 20% reduction in design loads, as permitted in the NBCC when a dynamic analysis method is used, was therefore neglected in that second design, resulting in an 25% increase in design seismic loads. In the third design, the lateral resistance was increased further by using an R_d factor of 3.0 instead of 4.0 in the calculation of V . For consistency, the elastic truss members in the Dual BRB system were designed using $\alpha = 3.0$ in that third design.

Table 5 summarizes the seismic loading parameters as well as the design base shear ratios V/W . In the table, Design No. 1 refers to the initial design examined earlier whereas Design Nos. 2 and 3 refer to the structures with increased

lateral loads. The periods in the first two vibration modes are given for the different structures. In each case, the number of occurrences of dynamic instability and the 84th fractile values of the maximum peak story drifts and floor accelerations are given for comparison.

As expected, the periods of all structures decreased when increasing the design forces. For the BRB frames, the additional lateral resistance was not sufficient to ensure stable inelastic performance. Large story drift values and occurrences of dynamic instability were observed in all cases, even under the crustal and sub-crustal ground motions when $R_d = 3.0$ was used in design. Conversely, the Dual BRB frame designed with $R_d = 3.0$ exhibited adequate performance under the subduction ground motions. It is noted that the increase in lateral resistance resulted in higher acceleration levels, which may impact on the performance of building content.

Table 5: Influence of the design seismic loads on the response of the 12-story structures (Note: response parameters correspond to the 84th fractile).

Design No.	1	2	3
$V_{Dyn.} / V_{Eq. (I)}$	0.80	1.0	1.0
R_d	4.0	4.0	3.0
V/W (%)	2.77	3.46	4.62
<i>BRB frames</i>			
T_1 (s)	3.48	3.08	2.74
T_2 (s)	1.24	1.09	0.97
<i>Crustal earthquakes</i>			
Occurrences of collapse /10	5	2	1
Peak story drift (%)	2.60	2.36	2.28
COV of peak story drifts	0.55	0.43	0.46
Peak floor accelerations	1.10	1.10	1.11
<i>Subduction earthquakes</i>			
Occurrences of collapse /4	4	4	3
Peak story drift	-	-	-
COV of peak story drifts	-	-	-
Peak floor accelerations	-	-	-
<i>Dual BRB frames</i>			
T_1 (s)	2.70	2.43	2.24
T_2 (s)	0.90	0.81	0.76
<i>Crustal earthquakes</i>			
Occurrences of collapse /10	0	0	0
Peak story drift (%)	1.70	1.62	1.34
COV of peak story drifts	0.20	0.31	0.33
Peak floor accelerations	1.28	1.46	1.56
<i>Subduction earthquakes</i>			
Occurrences of collapse /4	2	0	0
Peak story drift	-	4.74	1.87
COV of peak story drifts	-	0.12	0.16
Peak floor accelerations	-	1.43	1.48

5 CONCLUSIONS

The analytical study presented in this paper showed that multi-story concentrically braced steel frames built with buckling restrained braces and designed with $R_d = 4.0$ do not exhibit satisfactory seismic performance for the site studied. The structures experienced large story drifts and several occurrences of dynamic instability were observed. Increasing the lateral resistance by as much as 1.67 for one of the structures was not sufficient to achieve proper seismic response. Multi-story structures of this type do not seem to possess enough capacity to redistribute the inelastic demand over their height. Therefore, they remain prone to the formation of story mechanisms, even if buckling restrained braces with superior hysteretic response are used in lieu of conventional steel bracing members.

The study also demonstrated that satisfactory performance can be achieved under intra-plate earthquake ground motions if an elastic truss system capable of mitigating damage concentration along the building height is added to form a dual buckling restrained braced frame system. For the site studied, this dual system was also found to behave properly under long duration subduction earthquake ground motions, provided that the lateral resistance is increased to that corresponding to $R_d = 3.0$ and the 20% load reduction permitted when dynamic analysis is performed in design is neglected. For this dual system, further studies are needed to develop final design recommendations for the subduction hazard and to optimize the design of the elastic truss system.

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