# Seismic Performance Improvement of A Torsional Irregular Concrete Shear Wall Building Using Toggle-Brace-Dampers

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## Abstract

This paper describes a preliminary-assessment case study for seismic performance improvement of a stiffness and strength asymmetric concrete shear wall building using toggle-brace-dampers (TBD).

The structure is a 16-story major regional phone switch building located in downtown Seattle. Its bottom 7-story including the basement was constructed in the mid 1950's. The additional 9-story vertical expansion was completed in the early 1970's. It is a rectangular concrete shear wall building with a complete steel beam-column building gravity system encased in the concrete. The building has solid exterior concrete shear walls at its South and East sides and exterior concrete shear walls with regular punched window/louver openings at its North and West sides. This creates a stiffness and strength asymmetrical or torsional irregular building lateral system. TBD were used since they amplify damper strokes under relatively small story drift of a concrete shear wall building and reduce damper sizes while providing efficient effective damping. These dampers were arranged at the opposite sides of the solid concrete walls to balance and reduce the building stiffness and strength eccentricity.

Three-dimensional computer models were constructed. Nonlinear-static-procedure of FEMA356/ATC40 was utilized to determine the building performance-points under site specific 50%50-year, 10%50-year, and 2%50-year seismic events. A series of site-specific nonlinear time history analyses were performed to evaluate the effectiveness of TBD. The analyses indicate that TBD assist the building existing lateral system to achieve the Enhanced Rehabilitation Objective of FEMA356 and provide a cost-effective solution for building seismic performance improvement in comparison with conventional rehabilitation methods.

## Introduction

The introduction of various damping devices to dissipate energy and reduce building seismic responses has become an acceptable design approach for existing building seismic retrofit or rehabilitation. However, it is generally recognized that stiff structural systems, such as reinforced concrete shear wall system, are characterized by small drifts and small relative velocities such that the implementation of seismic energy dissipation devices is likely not feasible. Recent research and testing on TBD system (Constantinou et. al., 1997 and 2001) and practical application for wind and seismic control of steel high-rise building (McNamara et. al., 2000) led authors to believe that such system can be an excellent application for building seismic performance improvement of a stiff non-ductile concrete shear wall building. This paper presents a preliminary assessment case study of such application.

Nonlinear-static-procedure of FEMA356/ATC40 was utilized to determine the building performance-points under site specific 50%50-year, 10%50-year, and 2%50-year seismic events. A series of site-specific nonlinear time history analyses were performed to evaluate the effectiveness of TBD. The building seismic performance objective is Enhanced Rehabilitation Objective defined in FEMA 356.

## Building Description

The structure under study is a 16-story major regional phone switch building located in downtown Seattle. Its bottom 7-story including the basement was constructed in the mid 1950's with typical floor height of 14'-7". The additional 9-story vertical expansion was completed in the early 1970's with floor height varies from 14'-7" to 17'-0" and to 21'-4" at top floor. The total building height above the basement is approximately 240 feet. It is a rectangular building with a building footprint of 129'-10 ½" (North-South direction) by 110'-11 ¾" (East-West direction) (Figure 1). The building floors are constructed with 4" and 4 ½" one-way cast-in-place concrete slab supported by wide flange steel beams and girders encased in concrete. The floor beams and girders are supported by W14 steel columns at each gridline intersections. These steel columns are also encased in concrete. Column bays in the North-South direction are typically 18'-8" with an irregular South end bay of 13'-10 ½". Column bays in the East-West direction are typically 18'-0" with irregular East and West end bays of 14'-11 ¾" and 20'-0" respectively (Figure 1).

The building lateral system consists of solid exterior concrete shear walls at its South and East sides and exterior concrete shear walls with regular punched window/louver openings at its North and West sides (Figure 2). Typical window/louver openings are 6'-6" wide. Opening heights are varies from 11'-7" to 14'-0" to create 3'-0" deep spandrels at typical floors with the exception of top floor where full story height spandrels are provided. These walls are 8" thick from top of the basement to Level-11 and 6" thick from Level-11 to roof. All 8" and 6" walls are reinforced with horizontal and vertical reinforcement at wall center with #5 @ 15 (h&v) and #4 @ 12 (h&v) respectively. Additional 2#5 trim bars are provided at all edges of wall openings. 6" thick full-building-height interior concrete walls are also provided at stairways, elevators, and mechanical shafts. These interior shear walls are located near the solid exterior shear walls (Figure 1). This creates a stiffness and strength asymmetrical or torsional irregular building lateral system.

The building foundation consists of 12'-0"x12'-0"x3'-3"deep spread footings for middle interior columns, 24'-0"x13'-0"x 4'-0"deep combined footings with two columns per footing for columns in the exterior walls and columns adjacent to the exterior walls, and 24'-0"x24'-0"x4'-0" deep combined footings with four columns per footing at four building corners. Footings are supported on native till like material with specified allowable soil bearing capacity of 10,000 psf per the original 1955 construction documents. Limited soil boring information is also provided in these documents.

The following material strengths were specified in the original 1955 (1 to 3) and 1970 (4 to 5) construction documents and these material properties were used for this case study:

1. Concrete fc = 3,000 psi
2. Rebar allowable stress fs = 20,000 psi, intermediate grade
3. Structural steel allowable design stress = 20,000 psi

4. Concrete fc = 3000 psi
5. Rebar fy = 60,000 psi
6. Structural steel fy = 36,000 psi

Building seismic mass, besides building structure, includes permanently attached phone equipments inside the building, interior 8" CMU partition walls, and 2" stone cladding on all four sides of the building exterior.

## Seismic Demand and Ground Motions

The site is located in a seismically active area of the Pacific Northwest currently zoned as seismic zone 3 per the 1997 UBC. Site-specific seismic hazard information is obtained via a comprehensive study performed for the site 5-blocks South of the building site with similar soil profile (C.B. Crouse, 2001). Earthquakes to the site can originate from three types of sources: (1) interplate earthquakes on the Cascadia Subduction Zone associated with eastward movement of the Juan de Fuca tectonic plate beneath the North American plate, (2) intraplate earthquakes within the subducting Juan de Fuca plate as it sinks and breaks up below the North American plate, and (3) shallow crustal earthquakes on faults within the North American plate. One known shallow crustal fault at the site is the Seattle fault that runs in the East West direction right underneath downtown Seattle. Figure 3 shows the site-specific response spectra for the 50%50-year, 10%50-year, and 2%50-year seismic events. The 1997 UBC zone 3 response spectrum is also shown in the Figure for comparison. Three sets of matching time histories are provided that are developed based on the site-specific response spectrum for the 10%50-year event using recorded time histories from past seismic events. These selected recorded time histories are listed in Table 1. The Hachinohe, Olympia, and Port Island events are representative of the interplate, intraplate, and shallow crustal Seattle fault earthquakes respectively. These time histories are further modified to match 2%50-year site-specific response spectrum.

## Building Seismic Performance Objective

Because the structure is a major regional phone switch building serving a very large population of the region, it is most desirable to achieve the building seismic performance objective beyond Life Safety (LS). Thus, Enhanced Rehabilitation Objective defined in FEMA 356 is set as the targeted building performance objective.

## Structural Computer Modeling

Two linear three-dimensional computer models of the structure were created using ETABS 7.24. One uses shell elements for all shear wall spandrels and shear wall panels and one uses beam and column elements for shear wall spandrels and shear wall panels respectively. A careful corroboration between two models was made to adjust beam-column joint stiffness and consequently to best represent the building overall stiffness. The following cracked section properties were used for the models: (1) 0.5 Ig for all spandrels, (2) 0.7 Ig for all wall panels and (3) 0.8 Ig for all basement walls for the latter model. These equivalent cracked section properties are recommended by FEMA 356. Figure 2 shows the model using beam column elements. Because limited lateral resistant capacities are provided by these 3'-0" deep spandrels on the North and West exterior walls, models without these spandrels were also created.

The building periods of vibration were determined with model analysis. Building fundamental periods and mode mass participation ratio are presented in Table 2 and Table 3.

The three-dimensional model built with beam and column elements that includes 3'-0" spandrels was then converted to a nonlinear model with (1) providing moment hinges at each end and shear hinge at the middle of all spandrels, (2) providing P-M-M hinges at top and bottom and shear hinge at the middle height of all column/wall-panels at each floor.

## Existing Building Evaluation: Nonlinear Static (Push Over) Analyses

Nonlinear static analyses were performed to determine the ductility and strength characteristics of the structure. Force distribution patterns equivalent to the results of the 10%50-year dynamic response spectrum were used in both orthogonal directions. After the gravity load pushover case with load combination of (1.1 dead load + 0.275 live load), the pushovers for the two orthogonal directions were performed independently.

Figures 4 and 5 are demand-capacity curves for the pushover in the North-South and East-West directions respectively. The hinge formation sequencing for the East-West direction pushover is as follows: (1) shear hinges are formed at 3'-0" deep spandrels on the North wall and quickly lose their lateral load carrying capacities, (2) meanwhile, shear hinges are also formed and quickly lose their lateral load carrying capacities at 3'-0" deep spandrels on the West wall as well due to building's asymmetrical stiffness and strength characteristics, (3) wall pier moment hinges start to form on both North and West walls at the top of the basement walls and at the bottom of the roof top full story height spandrels and these hinges are below Immediate Occupancy (IO) level, (4) meanwhile, shear hinges are also formed at the roof top full story height spandrels on the North and West walls and gradually lose their lateral load carrying capacities while the wall pier moment hinges mentioned in item 3 slowly move to positions where these hinges are in-between Life Safety (LS) and Collapse Prevention (CP) and in some cases beyond. The North-South direction pushover has similar hinge formation sequencing. 50%50-year, 10%50-year, and 2%50-year demand spectra are also plotted on Figures 4 and 5. It can be seen that the performance point for the 50%50-year event is at the end of above-mentioned item 2 and early stage of item 3. The performance point for the 10%50-year event is at a point between above-mentioned item 3 and the early stage of item 4. Performance point for the 2%50-year event is at the point between above-mentioned item 3 and middle stage of item 4.

Since the structure has a complete steel beam-column gravity system encased in the concrete, losing 3'-0" deep spandrels on the North and West walls will not affect its gravity load carrying capacity and its function as a building that can be immediately occupied. Thus, the existing building achieves IO performance level for the 50%50-year event. The same concept can also be applied to the roof top full-story height spandrels and wall piers on the North and West walls. Thus, the building achieves (1) LS performance level but slightly below IO performance level for the 10%50-year event and (2) CP performance level but slightly below LS performance level for the 2%50-year event.

The nonlinear static (pushover) analyses indicate that the existing building achieves the Basic Safety Objective (BSO) based on (1) LS performance under 10%50-year event and (2) CP performance under 2%50-year event. This building also achieves the Enhanced Rehabilitation Objective if IO performance under 50%50-year event is justified. The building performance level can also be further improved to achieve the Enhanced Rehabilitation Objective by achieving (1) IO performance under 10%50-year event and (2) LS performance under 2%50-year event. Because the structure is a major regional phone switch building serving a very large population of the region, it is most desirable that this higher level of building seismic performance level can be achieved. Thus, the final building retrofit performance objective target is (1) IO performance under 10%50-year event and (2) LS performance under 2%50-year event. This retrofit target translates approximately 10% to 20% required reductions for the roof top displacements.

## Building Retrofit Scheme: Toggle-Brace-Dampers (TBD)

After explorations on conventional retrofit methods, two conventional retrofit schemes were proposed and briefly studied. One is to remove stone claddings on the North and West exterior walls and thicken these walls with applying reinforced shotcrete from outside. The other scheme is to bolt steel plates to the North and West exterior walls from inside. The developed plans and details of proposed schemes were sent to the contractor for pricing. The preliminary costs for the proposed schemes are $4.5 million for the shotcrete scheme and $7.1 million for the steel plate scheme.

Along with aforementioned retrofit schemes, the third scheme is to use toggle-brace-dampers (TBD). TBD was first studied and tested at State University of New York at Buffalo (Constantinou et. al., 1997 and 2001). It utilizes toggle braces to magnify damper displacement and reduce the required damper force, while still producing the desired damping effect. Figure 6 presents two different toggle-brace-damper configurations namely lower damper and upper damper. The study indicates that a magnification factor of 2 to 3 can be obtained that is insensitive to small variations in the inclination angles $\theta_1$ and $\theta_2$ shown in Figure 6. This finding makes TBD design practical for its building seismic application. TBD are recently used on structural steel high-rise building for wind and seismic vibration controls (McNamara et. al., 2000). Figure 8 shows an example of a typical TBD installation.

It could be a challenge to make TBD application feasible for a building cluttered with wires and phone equipments. Thus, a site survey was performed to identify best locations suitable for toggle-brace-dampers with minimum disruptions to the building function. Also, to effectively activate TBD, dissipate energy, and consequently improve building seismic performance, it was decided that TBD will be arranged along Gridlines 2 and B (Figure 1) that are column lines adjacent and parallel to the North and West exterior shear walls with punched window openings. This plan arrangement increases existing building dynamic strength and reduces existing building dynamic stiffness eccentricity. The survey indicates that there will be no impact to the building function if TBD are installed on floors from Level 5 to Level 11 with one TBD on Grid 2 and one TBD on Grid B at each floor. A preliminary cost analysis from the contractor indicated that the cost of TBD retrofit scheme would be only \$1.7 million. In order to prevent possible formation of soft stories, TBD were extended down to Level 3 for this study that is the top of the solid basement walls.

## Evaluation Procedure for Retrofitted Building

To evaluate the effectiveness of TBD, dampers were installed in the three-dimensional model at locations based on the site survey (Figure 9) and nonlinear time history analyses were performed for 10%50-year events and 2%50-year events. Because the building performance points are beyond the point where all aforementioned 3'-0" spandrels are failed in shear, the model without these spandrels were used for the TBD evaluation. The building dynamic properties are presented in Table 3. Site-specific time history data were used for the analyses. For comparison, building roof top displacements at its Northwest corner were monitored for models with and without TBD retrofit. The results are presented in Table 4 and corresponding demand-capacity curves for this monitored corner are presented in Figures 4 and 5. The maximum damper output force under all 10%50-year and 2%50-year events are approximately 250 kips and 450 kips respectively. Using TBD magnification factor of 3, a 150 kip damper with a half stroke of 1 ½" was chosen for all proposed TBD. It can be seen that TBD effectiveness is earthquake sensitive. The overall reductions on the roof top displacement at the building Northwest corner are in the range of 10% in the North-South direction and 20% in the East-West direction. These reductions achieve previously defined reductions for the building retrofit performance target. The limited effectiveness is due to limited number of dampers were provided on limited floors. The seismic movements of the South and East solid walls also reduce the effectiveness of the overall reduction to the roof top seismic displacements of the building Northwest corner in the North-South and East-West directions. However, the reductions to the rotational component that contributes to the roof top displacement at the building Northwest corner in both directions under all 10%50-year and 2%50-year events are almost doubled above-mentioned amounts that are in the range of 18% and 35% respectively.

## Discussion

Nonstructural elements of the building that affects the building performance objective were study concurrently with aforementioned site survey. A list of equipments that needs to be anchored was made to incorporate into the design. It is very interesting to note that none of the phone equipments were made on the list and they are all well anchored and secured to the building structure.

Building exterior cladding anchors were also reviewed. Because the re-cladding was done when the 9-story addition was added, positive anchorages were provided at that time that allow independent lateral movement of the cladding. Stone claddings that are attached to 3'-0" spandrels remain to be a possible falling hazard when shear failure of these spandrels occurs during a major earthquake. Nevertheless, with TBD retrofit the building will be functional for its intended purpose and can be put back to service immediately after a major seismic event.

The connection of TBD to building steel beams and columns were evaluated. It was found that due to the layout of these dampers, all existing columns with TBD attached do not need any reinforcement. Moreover, because TBD did not extend all the way to the roof, dead load on the columns prevented any net uplift at these columns. Consequently, no foundation work would be required.

## Conclusion

This preliminary-assessment case study demonstrates that TBD assist the building existing rigid concrete shear wall lateral system to achieve the Enhanced Rehabilitation Objective of FEMA356 and provide a cost-effective solution for building seismic performance improvement in comparison with conventional rehabilitation methods. TBD were chosen is because they amplify damper strokes under relatively small story drift of this concrete shear wall building and reduce damper sizes while providing efficient effective damping. Its plan arrangement increases the existing building dynamic strength and reduces the existing building dynamic stiffness eccentricity. Its arrangement along the floor height utilizes the existing steel column strength and loads on the columns to avoid foundation work.

## Acknowledgements

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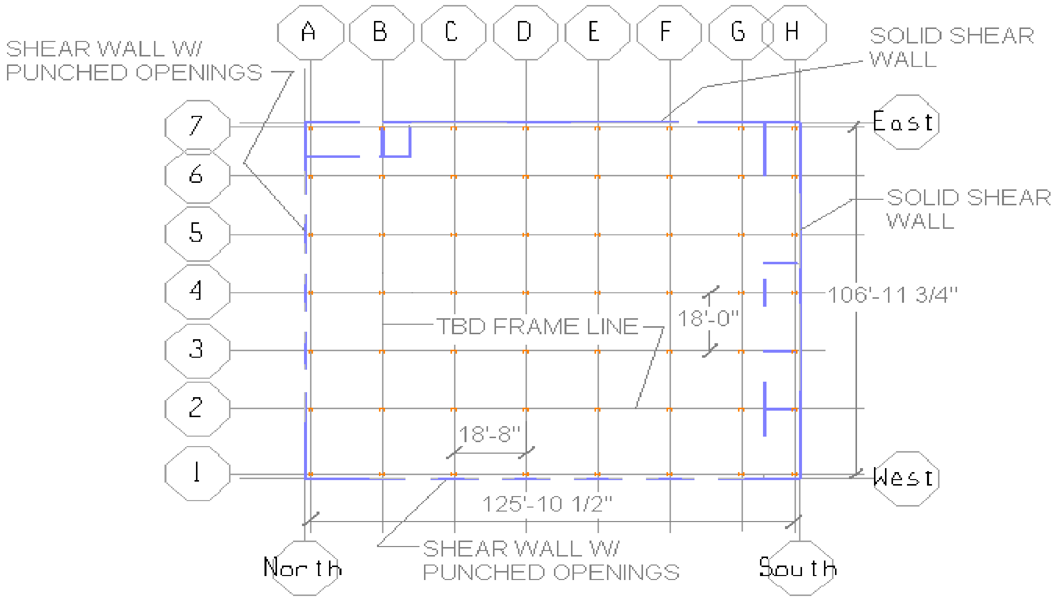
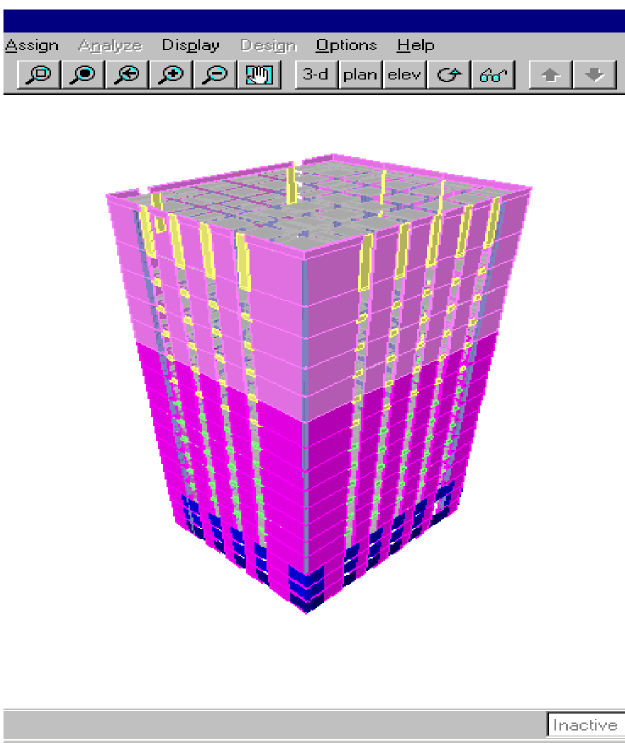



**Figure 1. Building Plan (North to Left)**



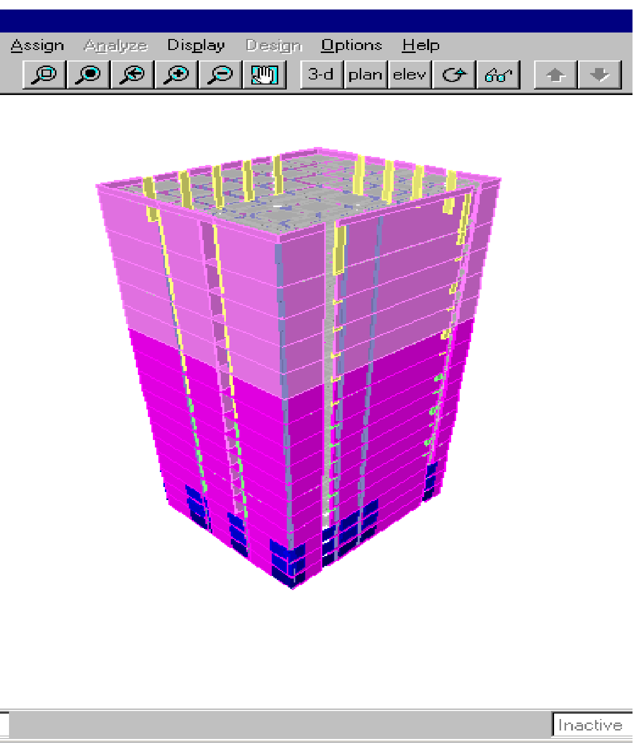




**Figure 2a. North (left) & West Walls** **Figure 2b. South (left) & East Walls**

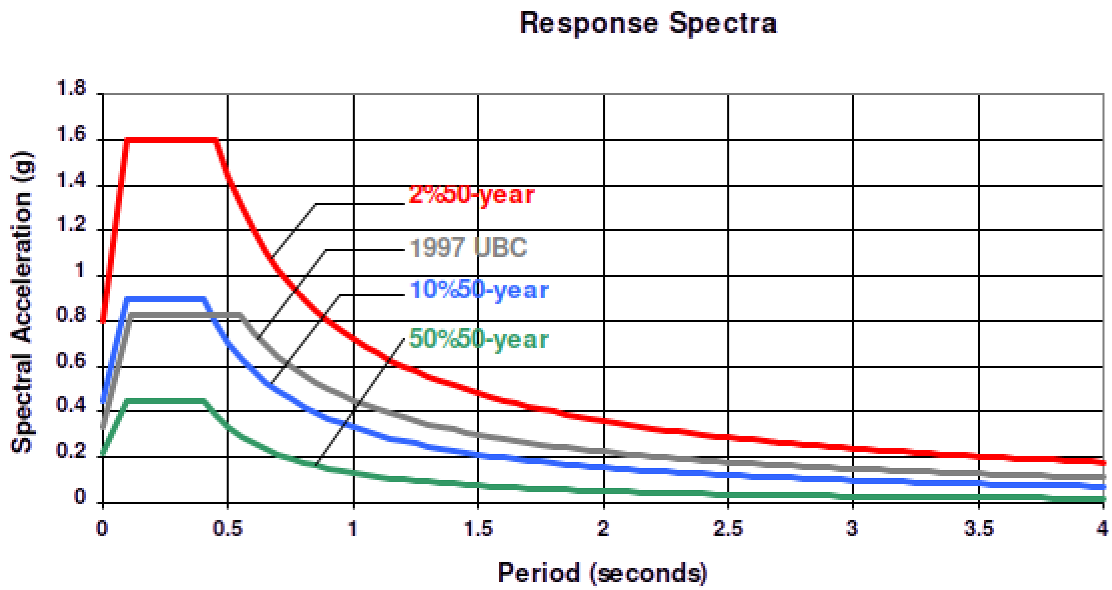



**Figure 3. Response Spectra**

### Table 1. Recorded Time Histories Selected for Modification

| Earthquake | | | Record Station | | Distance km |
|---|---|---|---|---|---|
| Name | M | Fault Type | Name | Soil Type | |
| 1968 Japan | 8.2 | Thrust | Hachinohe | Sd | 71 |
| 1949 W. Washington | 7.1 | Normal | Olympia | Sd | 60 |
| 1995 Kobe, Japan | 6.8 | Oblique | Port Island | Sc or Sd | 1 |

(Note: Table reproduced based on report by C.B. Crouse, URS, 2001)

### Table 2. Building Dynamic Properties with 3'-0" Deep Spandrels

| Mode No. | Mode Period (s) | Mode Mass Participation Ratio | | |
|---|---|---|---|---|
| | | North-South | East-West | Torsion |
| 1 | 1.68 | 8.8 % | 29.0 % | 23.1 % |
| 2 | 1.47 | 42.0 % | 28.6 % | 0.8 % |
| 3 | 1.10 | 11.6 % | 12.6 % | 37.1 % |

### Table 3. Building Dynamic Properties without 3'-0" Deep Spandrels

| Mode No. | Mode Period (s) | Mode Mass Participation Ratio | | |
|---|---|---|---|---|
| | | North-South | East-West | Torsion |
| 1 | 4.32 | 13.1 % | 18.0 % | 27.3 % |
| 2 | 2.17 | 46.9 % | 11.0 % | 4.2 % |

| | | | | |
|---|---|---|---|---|
| 3 | 1.38 | 1.7 % | 30.2 % | 28.4 % |

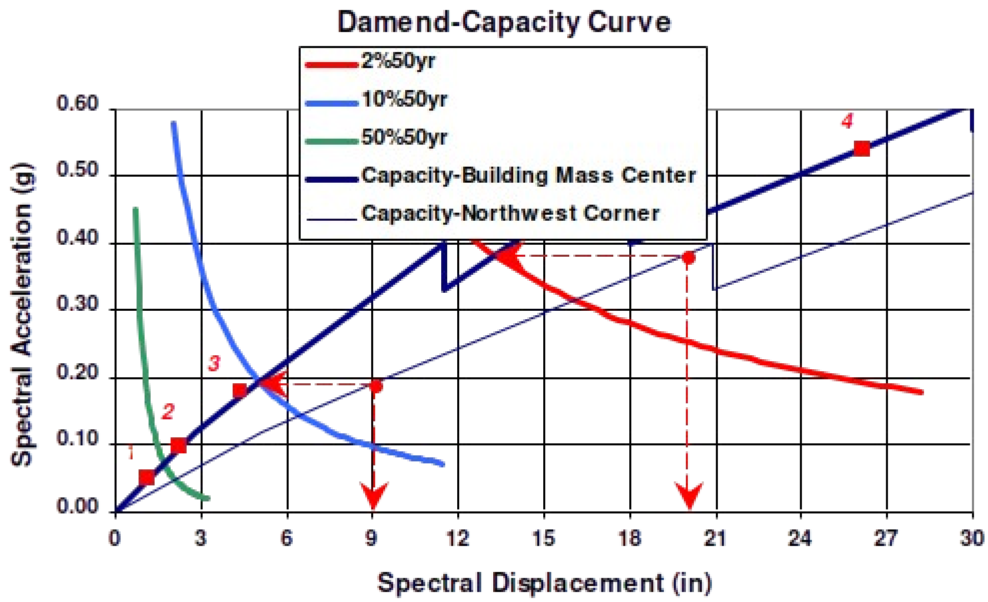



**Figure 4. North-South Demand-Capacity Curve**

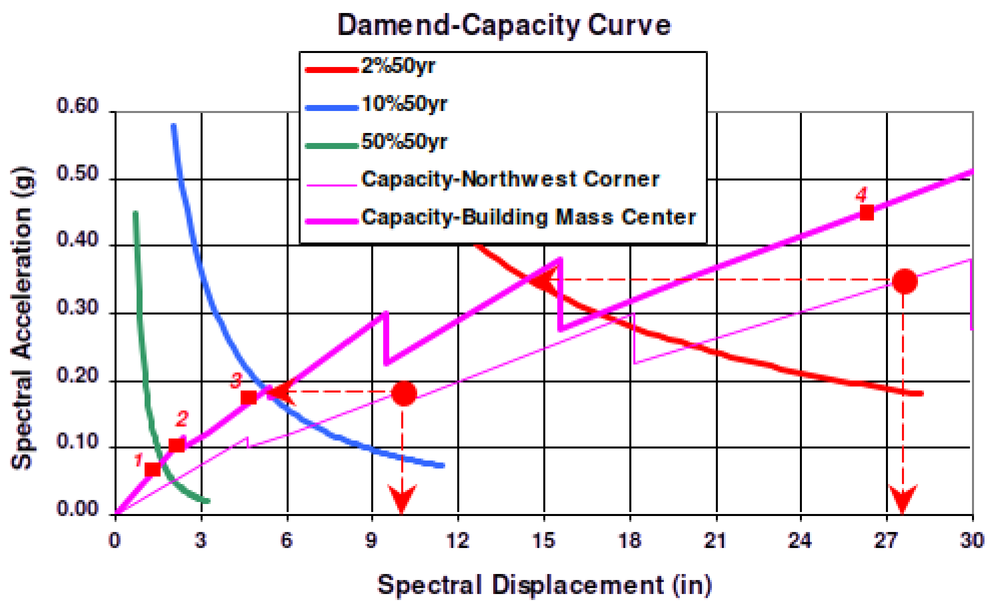



**Figure 5. East-West Demand-Capacity Curve**

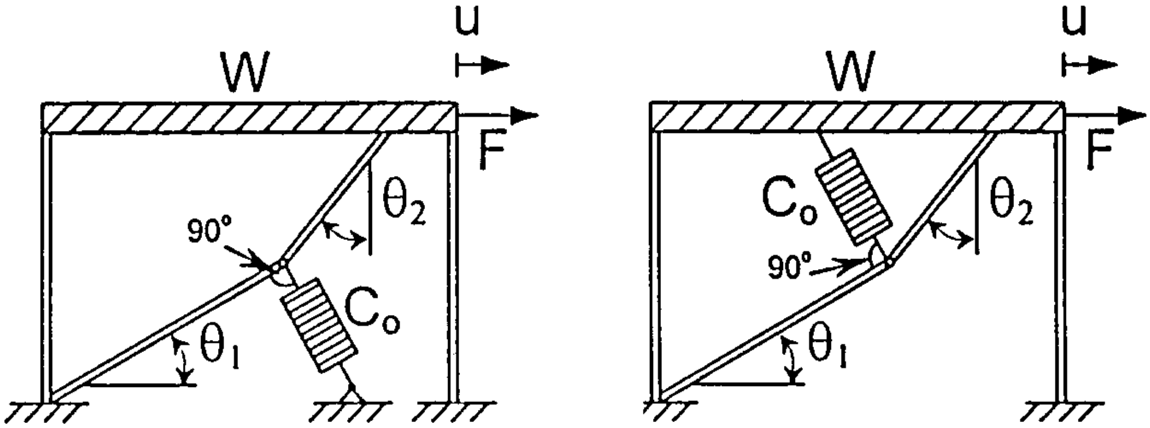



**Figure 6. Toggle-Brace-Damper Configurations**

(Note: Graphics reproduced from report by M.C. Constantinou et. al., 1997)

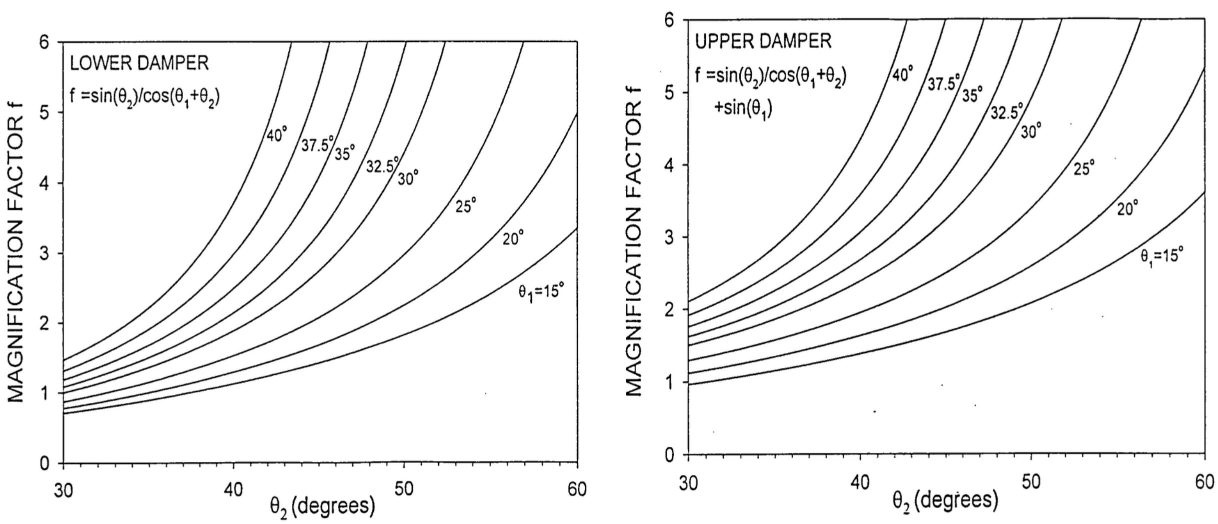



**Figure 7. Toggle-Brace-Damper Magnification Factors**

(Note: Graphics reproduced from report by M.C. Constantinou et. al., 1997)

## Table 4. Roof Top Displacement Comparison

**Building Northwest Corner Maximum Displacement (in)**

| Earthquake Modified From | 10%50-year event | | | | 2%50-year event | | | |
|---|---|---|---|---|---|---|---|---|
| | Without TBD | | With TBD | | Without TBD | | With TBD | |
| | N-S | E-W | N-S | E-W | N-S | E-W | N-S | E-W |
| Hachinohe | 13.92 | 17.99 | 11.41 | 14.30 | 34.63 | 44.93 | 28.01 | 35.68 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Olympia | 15.12 | 13.30 | 13.77 | 11.30 | 35.61 | 33.18 | 32.46 | 27.47 |
| Kobe | 13.64 | 15.14 | 11.06 | 12.43 | 32.81 | 36.29 | 26.71 | 29.94 |

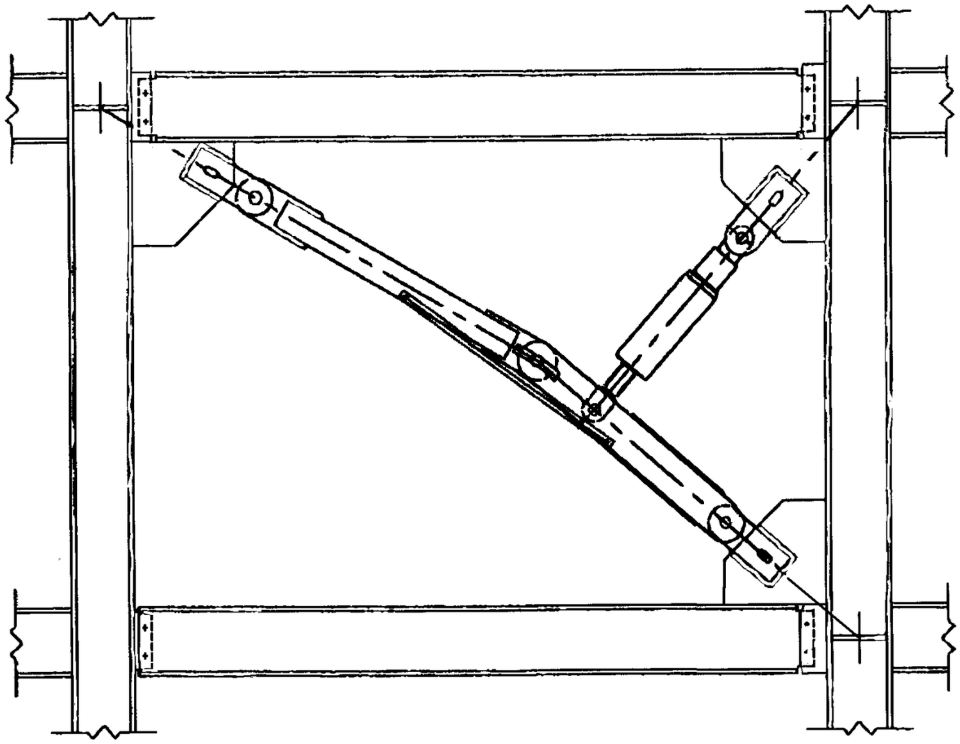
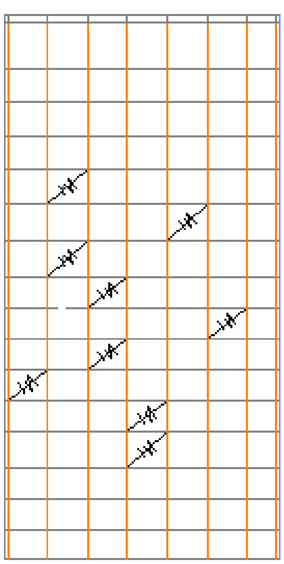


**Figure 8. Example of A Toggle Brace Damper Installation Detail - Elevation**

(Note: Representative Detail Provided by Taylor Devices, Inc.)



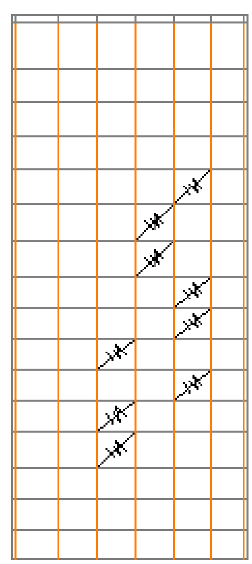


**Figure 9a. TBD Frame Elevation on Grid 2**

**Figure 9b. TBD Frame Elevation on Grid B**