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Structural Design of High-Rise Steel Structure Installed with Suspension Type Base Isolation System

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Abstract
The author has prior research paper published about Suspension Type Base Isolation (STBI); a seismic isolator intended to mitigate earthquake's devastating impact on structures. Based on the prior research performed, it was found out that the said isolator could significantly reduce the earthquake intensities "felt" by the structure installed with the said isolation system. The prior study found out that if the actual ground shaking, for instance, was around intensity 9, the structure installed with STBI would experience around intensity 5 only. Consequently, the prior study showed that the reduction in earthquake intensity could be related to reduced lateral force or base shear acting on the structures. This present paper is a sequel of the previous one; and it is intended to show how STBI could be integrated in the design of high-rise steel structures. Also, a parallel design of the same structure was performed following the standard procedure for fixed-base structures. The purpose of conducting this study was to compare how the two design methods differ; and the subsequent results in cost-effectiveness between STBI-isolated and the non-isolated or fixed-base structures. Both design schemes followed the same structural codes, material properties, loading conditions, and base shear calculation procedure. At the end of the present study, it was found out that the calculated base shear for STBI-isolated was 0.068W, while for fixed-base, it was 0.137W. This indicated that there is 50% reduction in base shear between the two structural configurations. This reduction in base shear became evident in the reduced sizes of structural members in the STBI-isolated structures. Moreover, the STBI-isolated structure requires no structural walls. On the other hand, the fixed-base structure requires larger structural members; and that structural walls were needed to prevent the said structure from failure. In terms of maximum lateral deflections, fixed-base structure incurred 318.5 mm, whereas STBI-isolated structure incurred 226.0 mm only.

Keywords: seismic isolators, base-isolation, friction pendulum bearing, lead-rubber bearing, steel moment frame building, base shear, high-rise building, high aspect ratio

1. Introduction

Destructive effects of earthquakes are consistently observed around the world. The impacts are not only observed on various types of structures, high and low, but also on living organisms. Preventing the occurrences of earthquakes is an impossible mission to accomplish. But stakeholders are not stopping to do their most, at least, to lessen the damages inflicted by earthquakes on lives and properties. Large amount of resources has been invested to developing and further upgrading methods in the design and construction of structures. For the fixed-base structures, the choices for lateral resistance against lateral forces caused by earthquakes are shear walls, braced frames, and moment-resistant frames [11]. And for structures with heights ranging from 40 to 100 stories, earthquake resistance could be achieved by implementing tube-framed buildings or employing parallel shear wall concepts [6]. Another researcher featured various structural configurations, with aspect ratios of 10:1 to 13:1, that are capable of resisting earthquake forces [7]. Apart from structural systems mentioned above, there is an alternative method of protecting structures against seismic forces. These are known as seismic isolators.

In the United States, the most popular devices for seismic isolation of buildings are the lead rubber bearings, high-damping rubber bearings and the friction pendulum system [11]. And to better implement integration of seismic isolators, the American Society of Civil Engineers [1] published detailed procedures in the design and analysis of seismically isolated structures. Similarly, New Zealand published a guideline for the design of seismic isolation for buildings, in which the emphasis is on lead-rubber bearing and friction pendulum isolators [5].

2. Justifications in the Implementation of Seismic Isolators

There is no argument about the advantages offered by seismic isolators. Firstly, the protection they offer in mitigating earthquake impact on structures; and secondly, the insight that the system would become cost-effective, eventually.

2.1. For Essential Buildings

The necessity of implementing seismic isolators among structures can be justified by their importance. Many essential buildings house extremely sensitive and costly equipment. Hospitals are keeping expensive laboratory apparatuses used for diagnoses and treatments costing more than the buildings themselves [10]. National Museums and World Heritage sites require protection against damages inflicted by seismic forces [17].

2.2. Perceived Reduction in Project Cost

Stefano and Gloria [12] reported that friction pendulum devices resulted to 15% reduction cost in reinforced-concrete building which was attributed to smaller members sizes. Conversely, Oguzcan [10] reported that building with seismic isolators resulted to the project cost increase by 29.12%. But Baker [3] clarified that achieving minimum amount of material in structural frames, proper proportioning of members should be adapted. This concept is called energy-based design [3]. It is believed that this concept could be implemented for both seismically isolated and fixed-base structures.

3. Limitations of Present Isolators

While the American Society of Civil Engineers allows implementation of seismic isolators, such as lead-rubber bearing (LRB) and friction pendulum (FB), their applications are limited only to four stories (19.8 m high) using the equivalent lateral-force procedure. Added limitation is that the structures must be located at a site with ground acceleration of less than 0.60g [1].

Japan is more liberal in the adaption of seismic isolators. A 20 - storey general research building in Tokyo, 90.4 m high, and with aspect ratio of 5.4 aspect ratio was designed and constructed using LRB isolators. Though, in the said project, the design team rendered some modification; the lower steel plate of the LRB was fixed on the pedestal using vertical loose anchors which allow the corner column a vertical deformation of up to 20 mm [15].

4. Structural Design Practices in the Philippines

In the Philippines, where this study was conducted, a media organization [8] reported that "when the Big One hits the Philippines, at least 25.6 percent or 339,800 of residential buildings will be damaged". The Big One referred to by the media outlet is the occurrence of strong earthquake (around magnitude 7.2) that could be triggered by the movement of West Valley Fault system [14]. It is believed that this scenario has to be addressed seriously.

The preparation opted in the engineering and construction industry is the implementation of the National Structural Codes of the Philippines (NSCP) which is an adaptation of USA structural codes for fixed-base structures. The NSCP allows seismic isolation, energy dissipation and damping system as an alternative lateral-force procedure in the analysis and design of structures [2]. However, the 2015 edition of the said NSCP has no detailed guidelines for the implementations of seismic isolators.

High-rise fixed-base buildings in the Philippines are designed based on Section 208 of the NSCP. The design team have the freedom to choose from among structural configurations listed in the code. Whereas, when implementing seismic isolators is an option, the contractor would take care of the "guidelines" [16].

5. Structural Design Proposal for High-Rise Buildings (with High Aspect Ratio) Utilizing Suspension Type Base Isolation (STBI) System

Basing on the preceding discussions, it could be deduced that the current trends and practices in protecting structures against earthquakes are: (a) implementing fixed-base (non-isolated) structures and selecting structural configurations applicable for the specific height of the proposed project; and (b) adapting an alternative option, specifically seismic isolation devices preferably LRB and FB, for relatively low-rise structures, and classified as essential establishments. STBI intends to address the height limitations of bearing type isolators such as LRB and FP; and that this system could be implemented to high-rise buildings with high aspect ratios.

5.1. Design Concept of STBI

The concept of STBI was introduced by the author in a published work entitled "Suspension Type Base Isolation: For Seismic Impact Mitigation in High-Rise Buildings" [4]. It was found out that an STBI-isolated prototype model experienced intensity 5 only while the shaker is accelerating at intensity 9.

It is opted to review here in brief the concept of STBI system[1]. Consider the graph shown in Figure 1 which was recorded during an investigation made where a prototype of a moment-resisting frame was placed on a shaker especially fabricated for the research purpose. Electronic sensors, accelerators, and gauges where installed in selected points in the prototype and the shaker. The electronics devices where connected to microcontroller unit, and hence, to a dedicated laptop.

Figure 1 basically presents two graphs, ground seismogram (more accurately, shaker seismogram) and base seismogram. The graph shows the recorded displacements of the ground and the base with respect to time. Data from the graph where used to calculate base accelerations (BA) and peak ground accelerations (PGA) which are all expressed in *g* or gravitational acceleration. Interestingly, also in the same figure, another waveform appears in the base seismogram. It is believed that the said waveform is caused by the swaying of the prototype's superstructure during the shaking episode.

[1] For detailed discussion about STBI system, one may refer the published paper [4].

Figure 1: Seismograms Recorded During a Single Test Episode

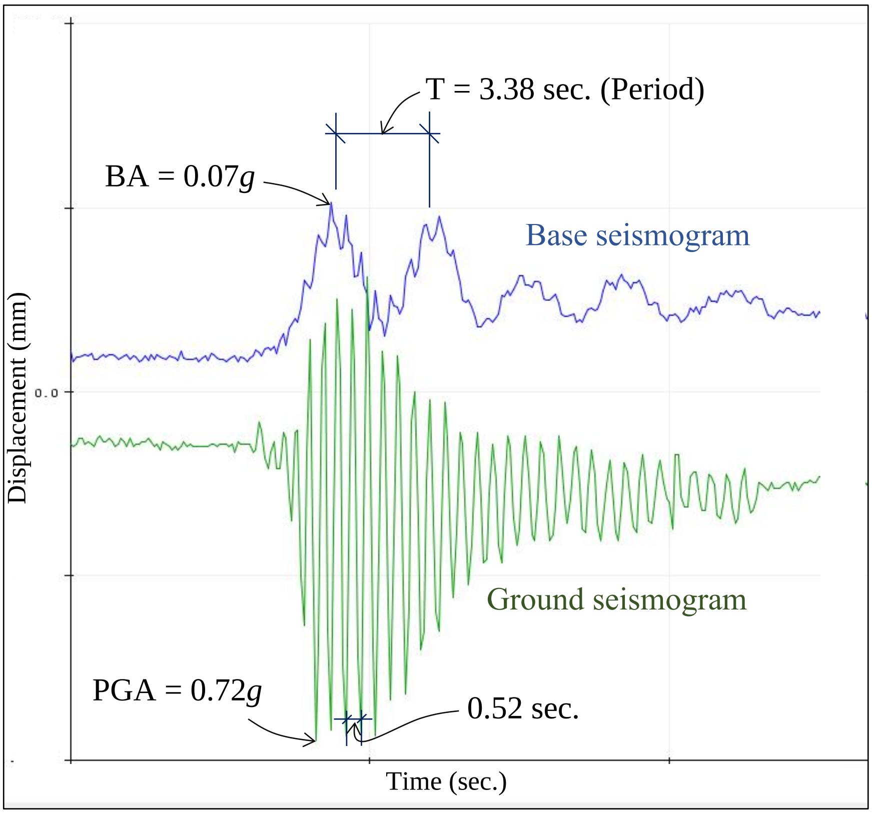
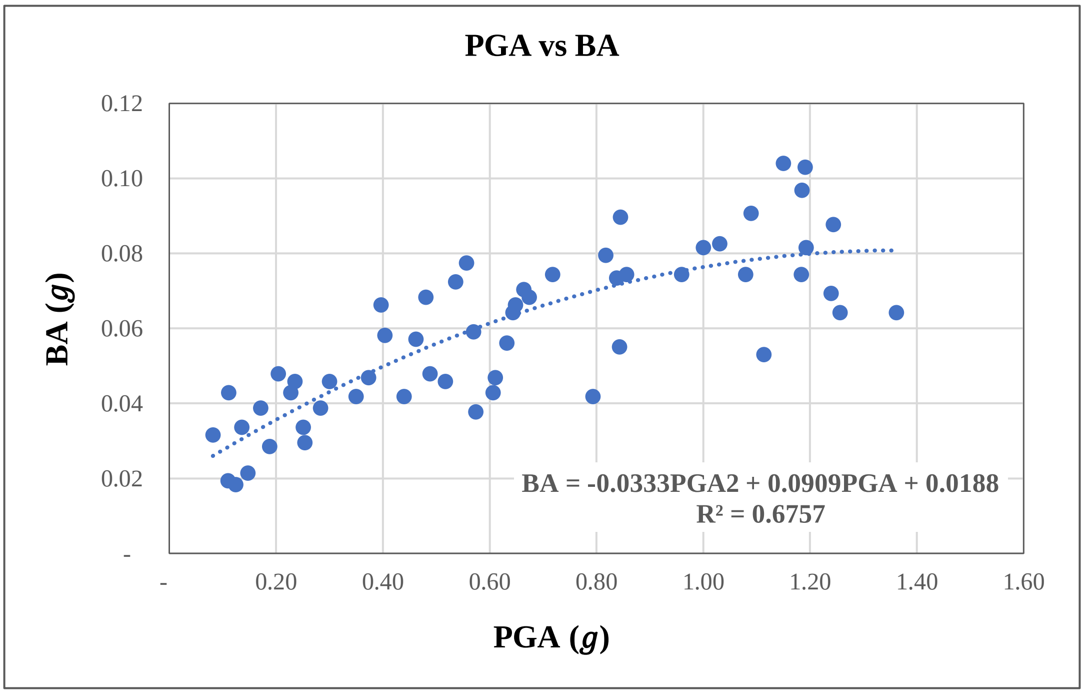


Figure 2: PGA vs BA (in *g*)



It can be noted that PGA and BA waveforms are synchronous or in-phase with each other, whereas the waveform exhibited by prototype's swaying is remarkably out of phase. The period of ground and base's shaking is 0.52 second, while the period of the superstructure is 3.38 seconds. In designing structures against seismic impact, it is strongly advisable that the structures period should be longer as possible.
Also, in Figure 1, PGA is 0.72g, which is intensity 9 in MMI scale [18], while BA is 0.07g, which intensity 5 in the same scale. Figure 2 shows a graph based on data collected from the research conducted. The graph shows the relationship between PGA and BA, both expressed in g (gravitational acceleration); thereby, an equation (1) was derived.

$$\mathrm{BA} = -0.0333\mathrm{PGA}^2 + 0.0909\mathrm{PGA} + 0.0188 \qquad (1)$$

Equation (1) seems to reduce even earthquake intensities above 9 down to around intensity 5 only. This finding would be the basis of the proposed structural design procedure for high-rise buildings with high aspect ratios.

5.2. Proposed Design Procedure

For the most part, the procedure followed the NSCP (hereinafter referred to as "the code") provisions, particularly Section 208, where the provisions for earthquake loads are elaborated in details [2].

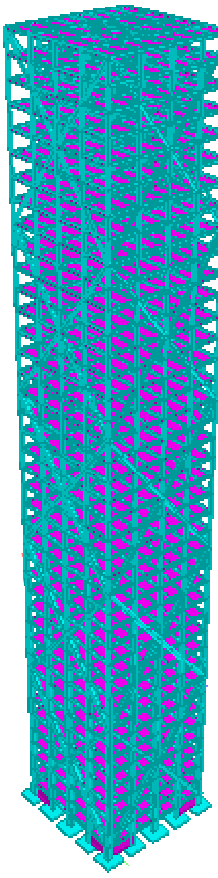
5.3. Two structural configurations chosen

For the purpose of comparison, two structural configurations were used. The first one was fixed-base or non-isolated structure, and the second one was STBI-isolated structure. They were designated as Case A for fixed-base structure, and Case B for STBI-isolated structure. Configurations on the superstructures above the isolation level for both cases were the same.

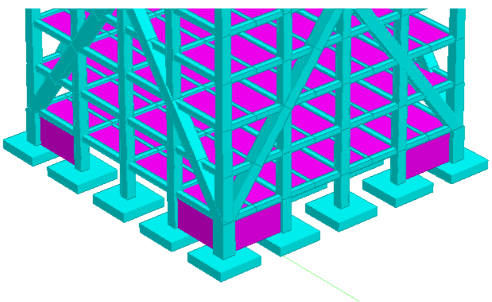
From Table 208–11B of the code, "Special steel concentrically braced frames" was selected. This is under "Dual Systems with Special Moment Frames". The code allows no height limit for this configuration [2]. A bird's eye view perspective of fixed-base structural frame is presented in Figure 3a. The structure is of structural steels laterally reinforced with Mega X-Bracings. It can be observed that the columns at the corners are laterally reinforced with structural walls just above the foundation (Figure 3b). On the other hand, a bird's eye view of STBI-isolated is shown in Figure 4a. It can be seen that its superstructure's framing system is identical to the one in Case A. And in Figure 4b, an array of STBI isolators can be closely seen.

Both structures are 40-stories and each has a total height of 164 meters, and minimum base dimension of 24 meters. Hence, both have the same aspect ratio of 6.83: classified as high-rise buildings. In Figure 5 and Figure 6, skeletal frameworks of the proposed steel structures are shown, where the details of Mega X-Bracings appear. The similarity of both steel frames just above ground level could be noted. And just below the ground level, the dissimilarity could also be noted.

Figure 3: Bird's Eye View Perspective of Fixed-Base Building (Case A)

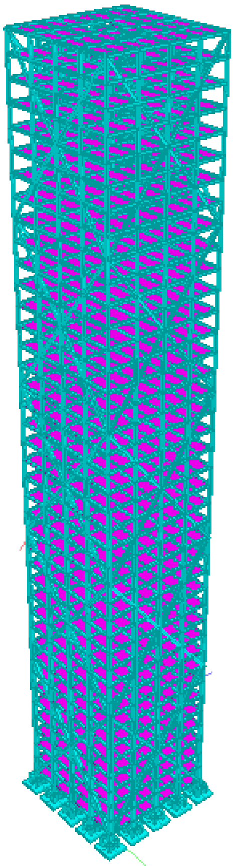


(a)

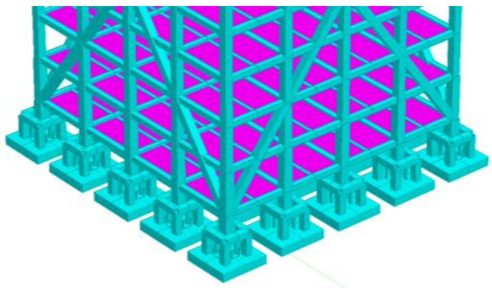


(b)

Figure 4: Bird's Eye View Perspective of STBI-isolated Building (Case B)



(a)



(b)

Figure 5: Elevation View of Steel-Framed High-Rise Building without STBI (Case A)

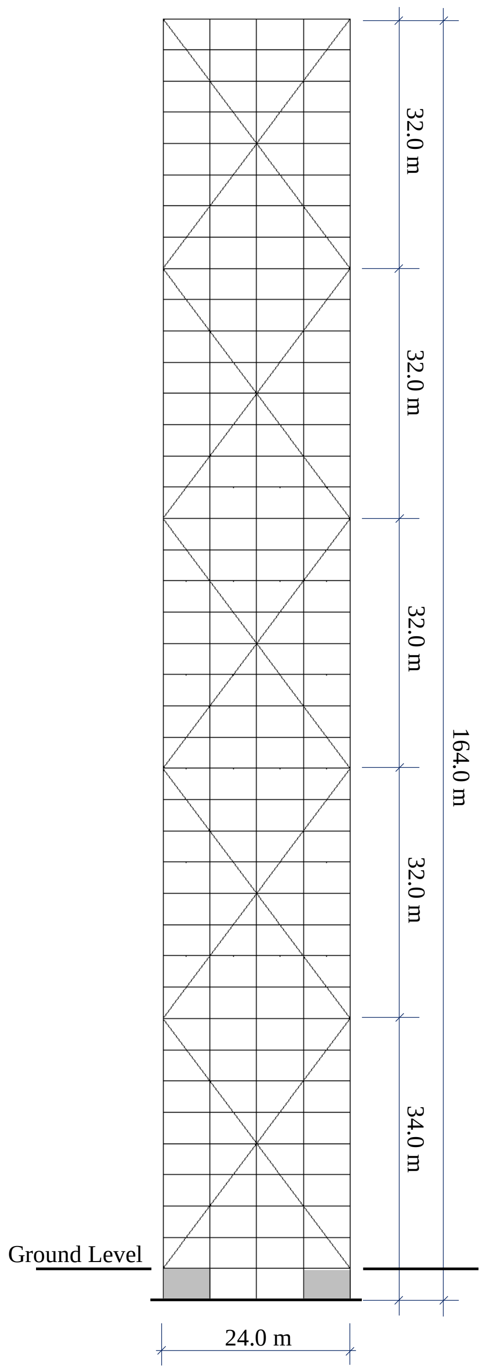
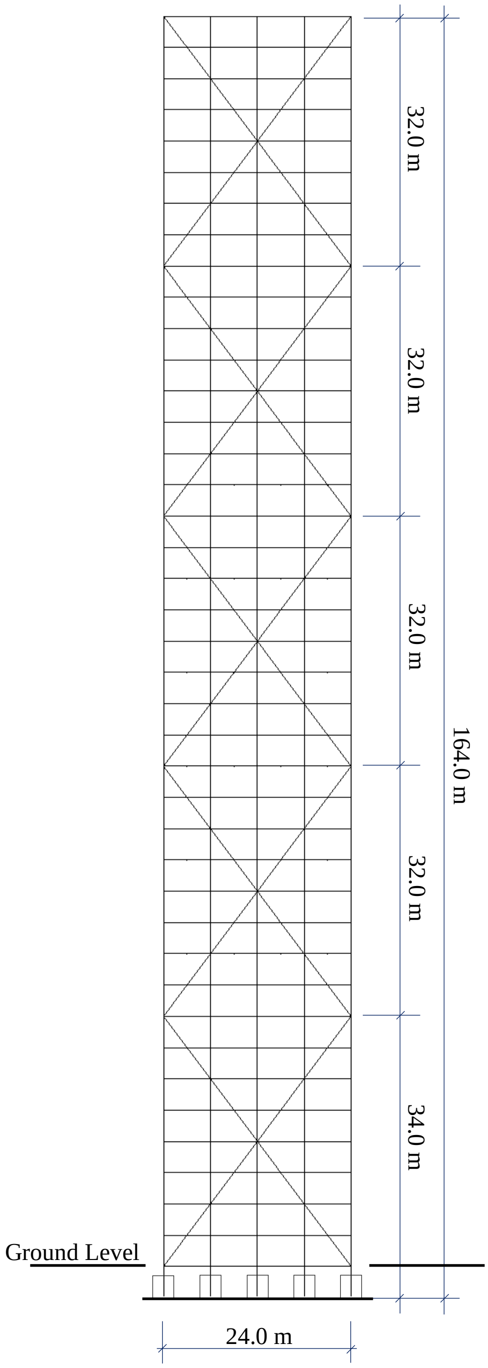


Figure 6: Elevation View of Steel-Framed High-Rise Building with STBI (Case B)



5.4. Considerations in Design and Analysis

It was mentioned above that the design and analysis would be based on the code. The only aspect of this procedure that was not adapted from the code is the implementation of the STBI system because the said code has no detailed guidelines about seismic isolations.

5.4.1. Scope of the Study

The study covered only structural members above foundations. For fixed-base structure, study included lowest columns, structural walls, and superstructure. Whereas, for the STBI-isolated structure, study included the lowest column, isolation system, and the superstructure.

5.4.2. Definition of loads

Loads applied on the proposed structures were self-weight (D) of the structures, superimposed dead loads (D), live loads (L), and seismic load (E). Wind load was not included in the study because the ground profile was unknown; and that in the Philippines' experience, earthquakes were more devastating on structures than the winds. For this study, superimposed dead load (D) was 2 kPa, while live load (L) was 3 kPa. Seismic load was calculated based on Section 208 of the code. The load combination applied was 1.20D + 1.0E + 1.0L.

5.4.2.1. Seismic load Analysis

In this study, it was assumed that the location of the proposed structures was in Metro Manila, just within 2 km from the seismic source, the West Valley Fault system. It is believed that the fault system could generate earthquake energy at magnitude of around 7.2 [14]. Table 1 summarizes the different factors considered in the calculation of seismic force, base shear (V).

Table 1: Summary of Factors Considered in the Calculation of Minimum Base Shears

Descriptions	Case A: Fixed-base	Case B: STBI-isolated
Importance, I	1.50	1.50 [1]
Soil Profile	Sc	Sc
Seismic Zone, Z	0.40	0.40
Seismic Source Type	A ($7.0 \leq M \leq 8.4$)	C ($M < 6.5$) [5]
Na [3]	1.1 [2]	1.0
Nv [3]	2.0	1.0
Ca	0.44	0.40
Cv	1.12	0.56
R [4]	7.0	7.0
Minimum Base Shear, V [6]	0.137W	0.068W

[1] ASCE 7-10 recommended that I = 1.0 for seismic isolated structures [1].
[2] For soil profile of S_c, NSCP (2015) provides that maximum $N_a = 1.1$ [2].
[3] Considered distance to Seismic Source ≤ 2 km
[4] R = 7 for the chosen structural configuration.
[5] Seismic source Type C was selected for Case B based on study that STBI-isolated structure would experience intensity 5 only even at seismic intensities of 9 and above [4].
[6] See detailed solutions for V.

Table 208-1 – Seismic Importance Factors*

Occupancy Category	Seismic Importance Factor, I
I. Essential Facilities	1.50
II. Hazardous Facilities	1.25
III. Special Occupancy Structures	1.00
IV. Standard Occupancy Structures	1.00
V. Miscellaneous Structures	1.00

* Adapted from the National Structural Codes of the Philippines, 7th edition, ASEP.

Reference was made from Table 208 – 1 through Table 208 – 8 in the calculation of minimum seismic force or base shear, V.

Table 208-2 – Soil Profile Types*

Soil Profile Types	Soil Profile Name/ Generic Descriptions	Average Soil Properties for Top 30 m of Soil Profile		
		Shear Wave Velocities, V_s (m/s)	SPT, N (blows/300 mm)	Undrained Shear Strength, S_u (kPa)
S_A	Hard Rock	> 1500		
S_B	Rock	760 to 1500		
S_C	Very Dense Soil and Soft Rock	360 to 760	> 50	> 100
S_D	Stiff Soil Profile	180 to 360	15 to 50	50 to 100
S_E^1	Soft Soil Profile	< 180	< 15	< 50
S_F	Soil Requiring Site-specific Evaluation See Section 208.4.3.1			

* Adapted from the National Structural Codes of the Philippines, 7th edition, ASEP.

Table 208-3 – Seismic Zone Factor Z*

ZONE	2	4
Z	0.20	0.40

* Adapted from the National Structural Codes of the Philippines, 7th edition, ASEP.

Table 208-4 – Seismic Source Types[1] *

Seismic Source Type	Seismic Source Description	Seismic Source Definition
		Maximum Moment Magnitude, *M*
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity	$7.0 \leq M \leq 8.4$
B	All faults other than Types A and C	$6.5 \leq M < 7.0$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity	$M < 6.5$

[1] *Subduction sources shall be evaluated on a site-specific basis*

* Adapted from the National Structural Codes of the Philippines, 7th edition, ASEP.

Table 208-5 – Near Source Factor, N_a [1] *

Seismic Source Type	Closest Distance to Known Seismic Source[2]		
	≤ 2 km	≤ 5 km	≥ 10 km
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

* Adapted from the National Structural Codes of the Philippines, 7th edition, ASEP.

Table 208-6 – Near Source Factor, N_v [1] *

Seismic Source Type	Closest Distance to Known Seismic Source[2]			
	≤ 2 km	5 km	10 km	≥ 15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

* Adapted from the National Structural Codes of the Philippines, 7th edition, ASEP.

Table 208 -7 – Seismic Coefficient, C_a *

Soil Profile Type	Seismic Zone Z	
	Z = 0.2	**Z = 0.4**
S_A	0.16	$0.32N_a$
S_B	0.20	$0.40N_a$
S_C	0.24	$0.40N_a$
S_D	0.28	$0.44N_a$
S_E	0.34	$0.44N_a$
S_F	*See Footnote I of Table 208 – 8*	

* Adapted from the National Structural Codes of the Philippines, 7th edition, ASEP.

Table 208 -8 – Seismic Coefficient, C_v *

Soil Profile Type	Seismic Zone Z	
	Z = 0.2	**Z = 0.4**
S_A	0.16	$0.32N_v$
S_B	0.20	$0.40N_v$
S_C	0.32	$0.56N_v$
S_D	0.40	$0.64N_v$
S_E	0.64	$0.96N_v$
S_F	*See Footnote I of Table 208 – 8*	

* Adapted from the National Structural Codes of the Philippines, 7th edition, ASEP.

Table 208 -7 – Seismic Coefficient, C_a *

Soil Profile Type	Seismic Zone Z	
	Z = 0.2	**Z = 0.4**
S_A	0.16	$0.32N_a$
S_B	0.20	$0.40N_a$
S_C	0.24	$0.40N_a$
S_D	0.28	$0.44N_a$
S_E	0.34	$0.44N_a$
S_F	*See Footnote I of Table 208 – 8*	

* Adapted from the National Structural Codes of the Philippines, 7th edition, ASEP.

5.4.2.1.1. Static Force Procedure for Case A

The total design base shear in a given direction shall be determine from the following equation:

$$V = \frac{C_v I}{RT} W \qquad (208-8)$$

$$V_A = \frac{1.12(1.5)}{7(2.236)} W = 0.107W$$

The total design base shear need not exceed the following:

$$V = \frac{2.5 C_a I}{R} W \qquad (208-9)$$

$$V_A = \frac{2.5(0.44)(1.5)}{7} W = 0.236W$$

The total design base shear shall not be less than the following:

$$V = 0.11 C_a W \qquad (208-10)$$

$$V_A = 0.11(0.44)W = 0.0484W$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{0.8 Z N_v I}{R} W \qquad (208-11)$$

$$V_A = \frac{0.8(0.4)(2)(1.5)}{7} W = 0.137W \qquad \text{(Adapt this value)}$$

5.4.2.1.2. Static Force Procedure for Case B

The total design base shear in a given direction shall be determine from the following equation:

$$V = \frac{C_v I}{RT} W \qquad (208-8)$$

$$V_B = \frac{0.56(1.5)}{7(3.38)} W = 0.0.0355W$$

The total design base shear need not exceed the following:

$$V = \frac{2.5 C_a I}{R} W \qquad (208-9)$$

$$V_B = \frac{2.5(0.4)(1.5)}{7} W = 0.214W$$

The total design base shear shall not be less than the following:

$$V = 0.11 C_a W \qquad (208-10)$$

$$V_B = 0.11(0.4)W = 0.044W$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{0.8ZN_vI}{R}W \qquad (208-11)$$

$$V_B = \frac{0.8(0.4)(1)(1.5)}{7}W = 0.068W \qquad \text{(Adapt this value)}$$

5.5. Structural Design and Analysis using Software

Commercially available design software was used in both Case A and Case B. Application of self-weight, dead load and live load were made as provided by the software. For seismic load, base shear equations, $V_A = 0.137W$ and $V_B = 0.068W$, that were derived as described above, they were manually included in the software.

5.5.1. Design and Analysis Scheme

The steel frames were drawn in the software. HSS (hollow structural steel) was chosen for beams, girders, columns and bracings for both cases. For slabs and structural walls, reinforced concrete was implemented. Initial sizes were assumed for all structural members. Material strengths were defined, F_y = 415 MPa for all structural steel members and rebars; F_c = 27 MPa for floor slabs and structural walls. Loads and load combinations were defined. Then, the software was instructed to run analysis. Results were evaluated. When the assumed sizes were overstressed or otherwise, the sizes were adjusted until all the members were safe.

5.5.1.1. Column, Girder, and Beam Sizes

Floor framing plan for Case A is shown in Figure 7, while floor framing plan for Case B is presented in Figure 8. Connections of beams, girders and columns are presented in detail. Members' final sizes are also listed. Figures for other floor levels are not presented in this study; but their weights are included in the summary report.

Figure 7: First Floor Framing Plan (Case A)

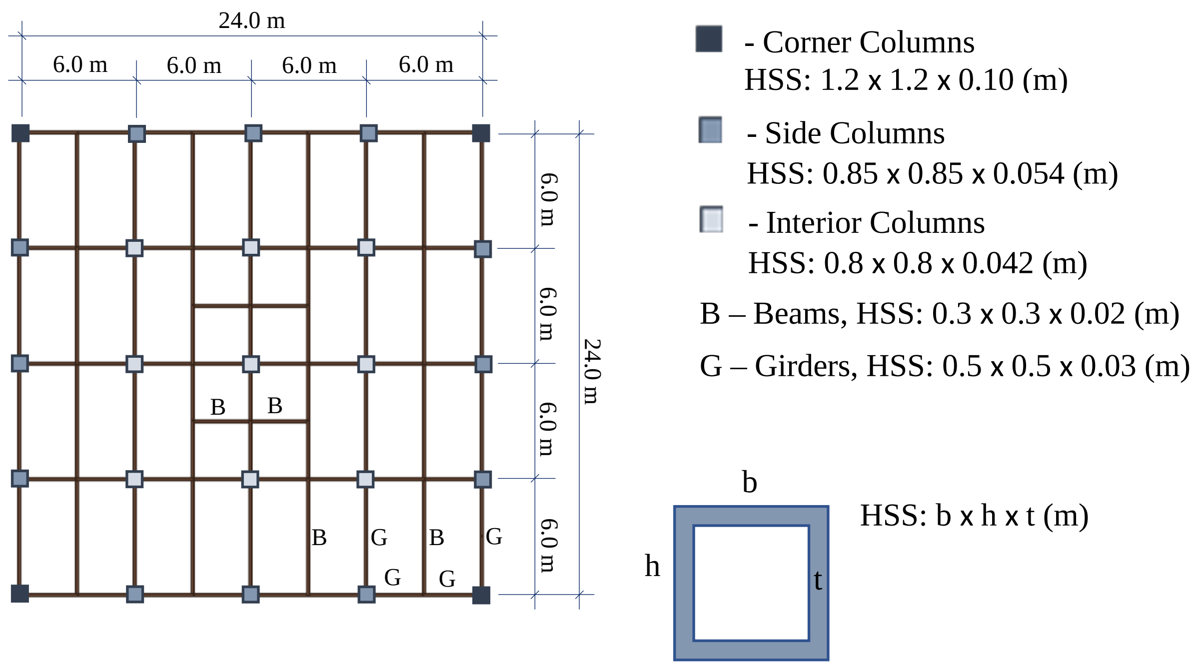
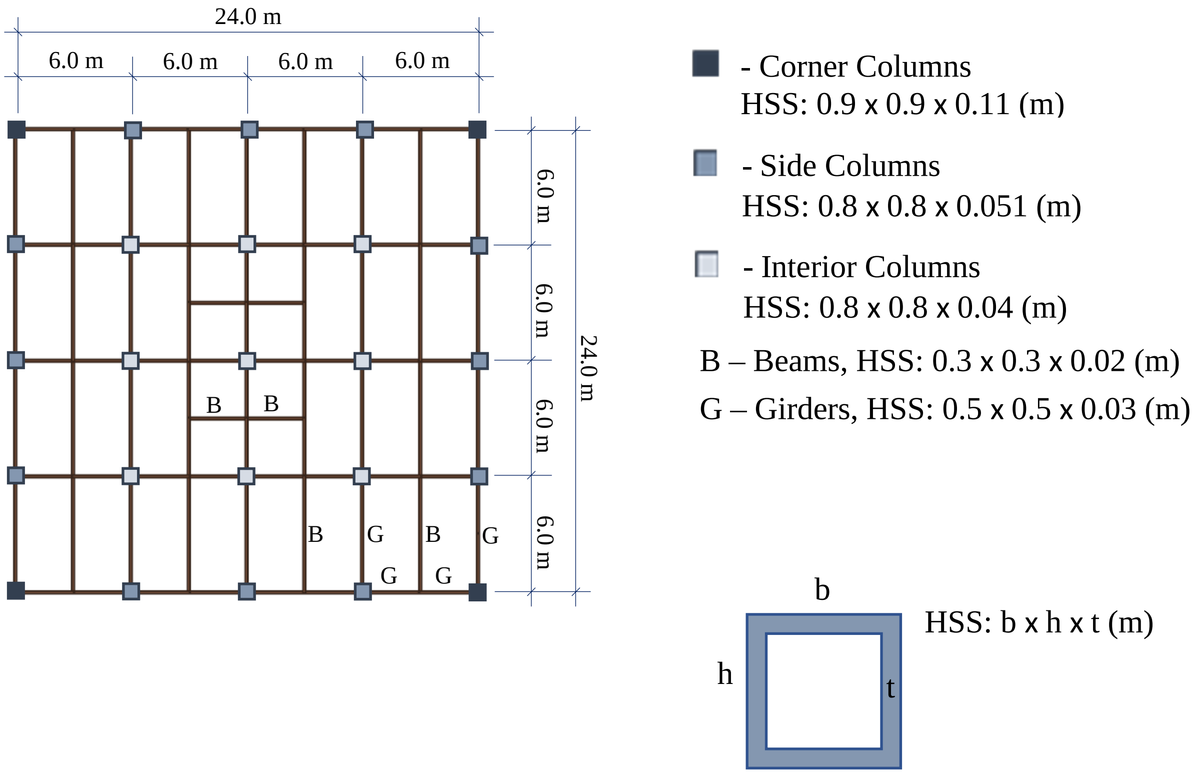


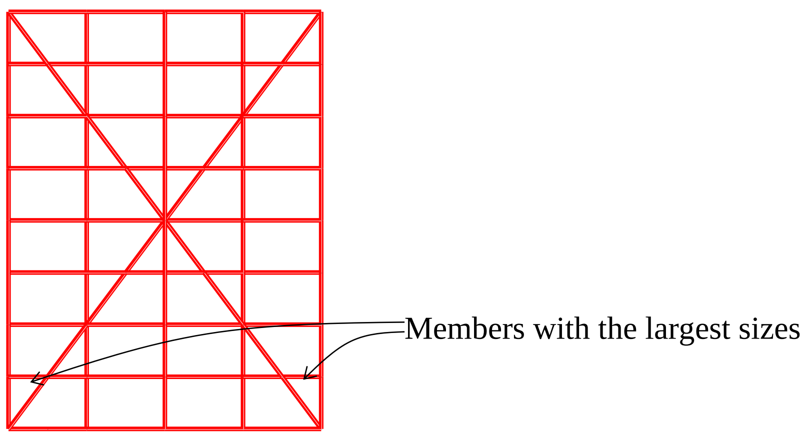
Figure 8: First Floor Framing Plan (Case B)



5.5.1.2. Mega X-Bracings

The largest size of the bracing for Case A is HSS: 1.0 x 1.0 x 0.07 (m). Whereas, for Case B, the largest size of the bracing is HSS: 0.7 x 0.7 x 0.05 (m). They were installed at the lowest end of the bracing system (Figure 9).

Figure 9: Mega X-Bracing System



5.5.1.3. Design of STBI Steel Frames and Suspenders

This procedure was applied only to Case B, STBI-isolated structure. Case A structural framing system is non-isolated. Figure 10 shows the foundation plan, while Figure 11 shows the typical STBI frame.

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Figure 10: Foundation Plan for Case B

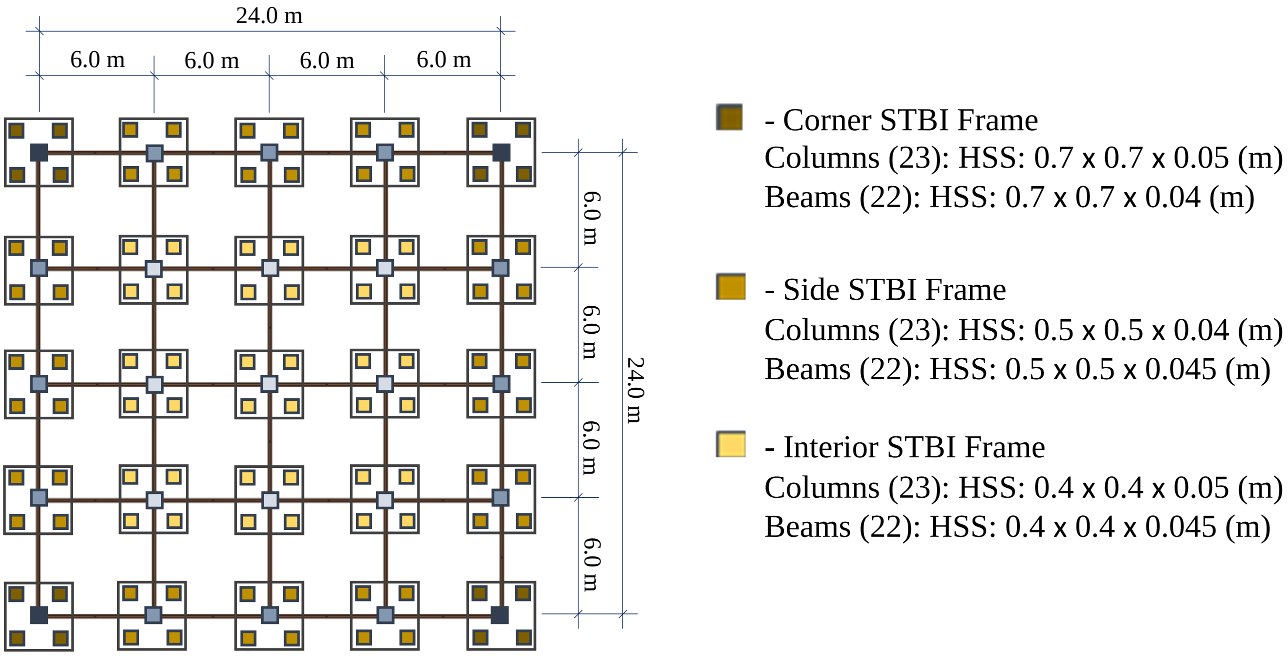

Figure 11: Typical STBI Frame with RTS
[Not Drawn to Scale]

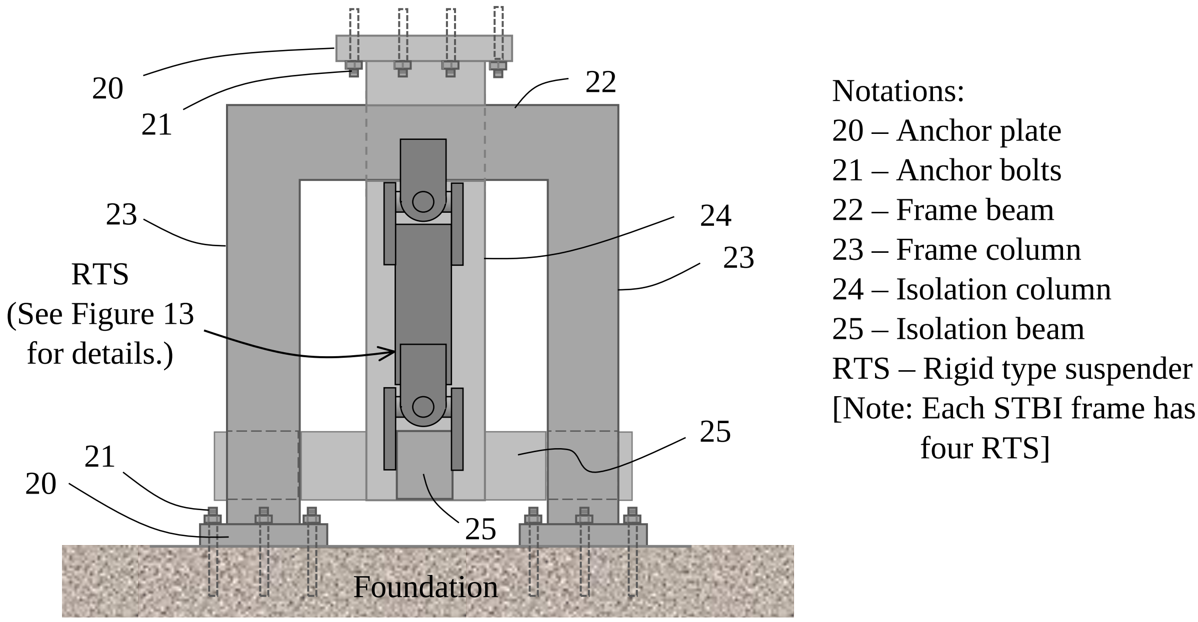

Figure 12: Plan View of STBI Frame
[Not Drawn to Scale]

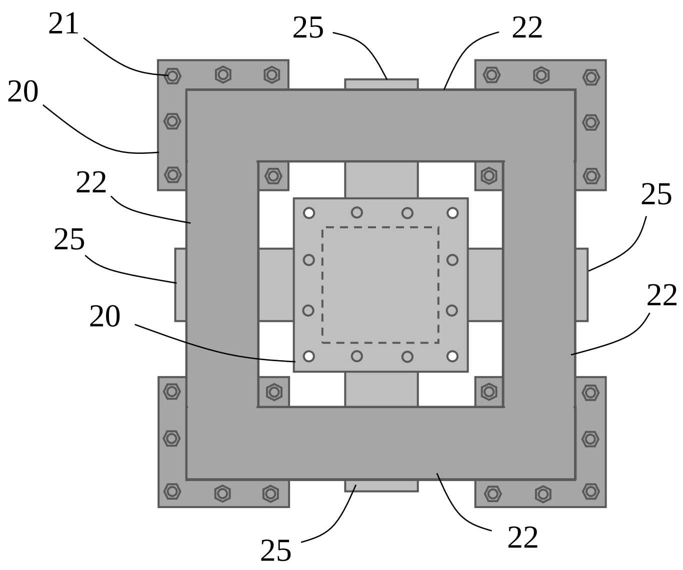

Figure 13: Details of RTS
[Not Drawn to Scale]

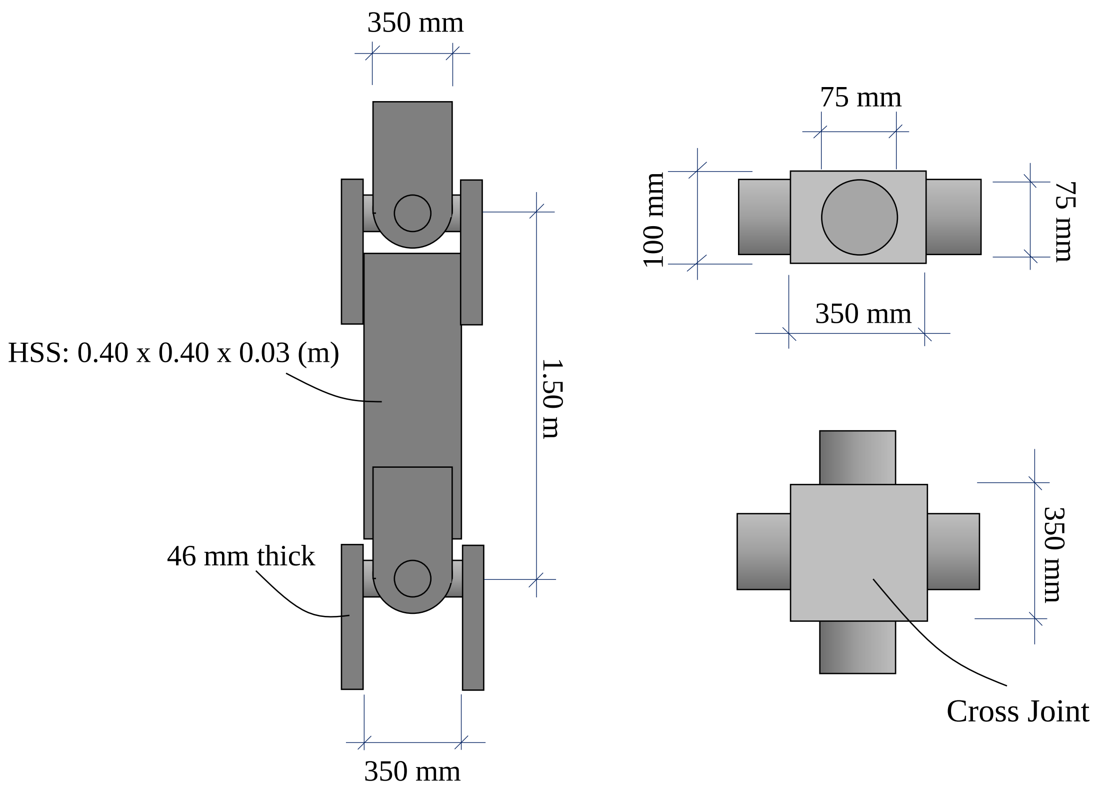

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Figure 14: Typical STBI Frame with FTS
[Not Drawn to Scale]

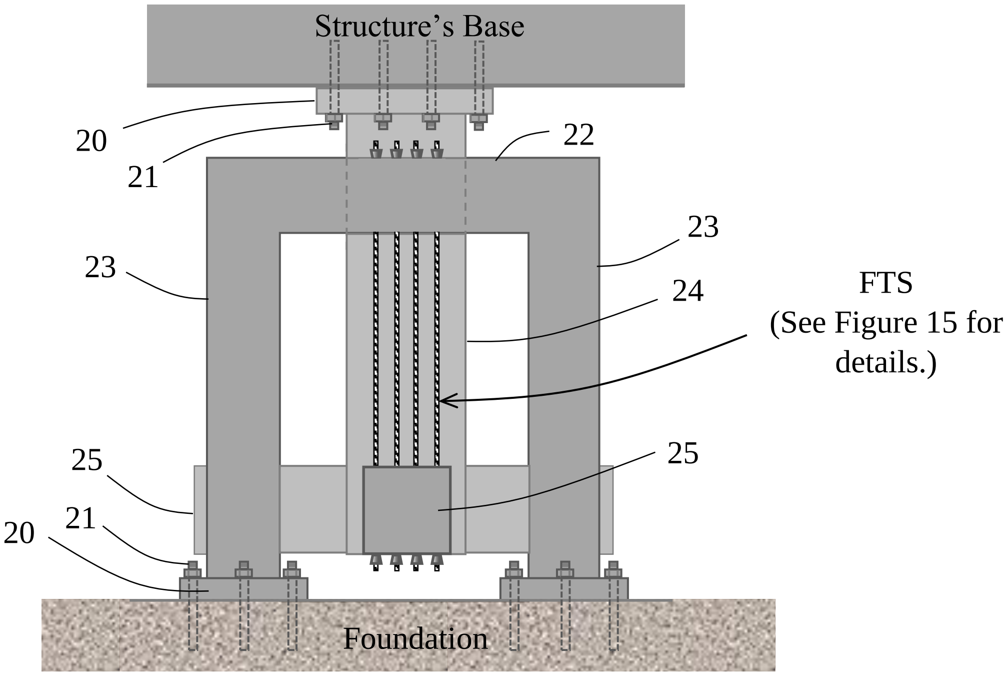
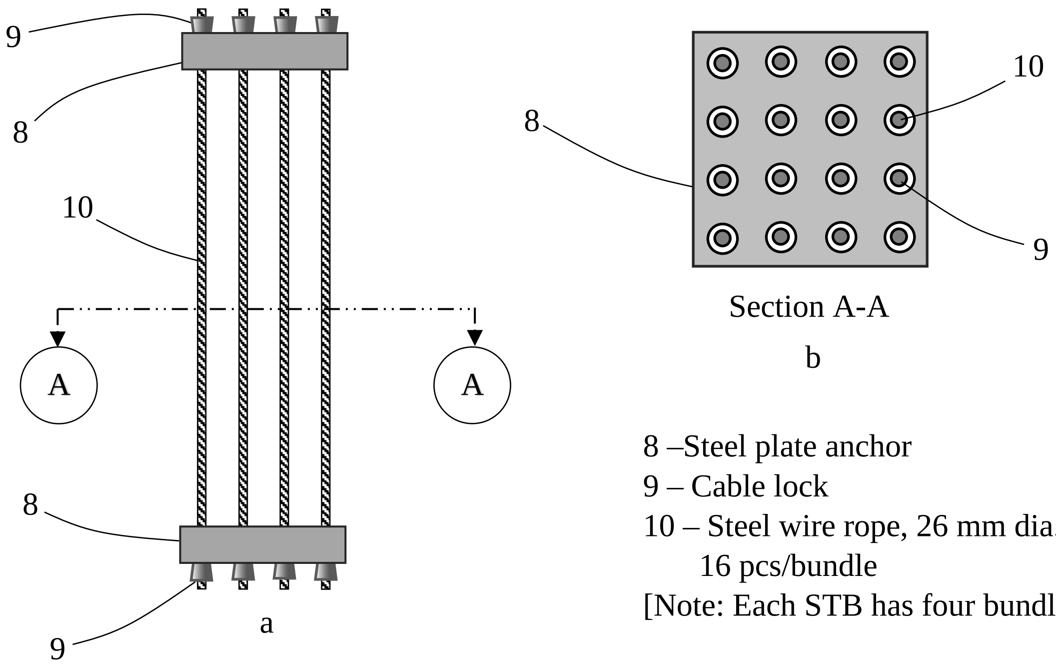


Figure 15: Details of FTS
[Not Drawn to Scale]



8 –Steel plate anchor
9 – Cable lock
10 – Steel wire rope, 26 mm dia.
16 pcs/bundle
[Note: Each STB has four bundles]

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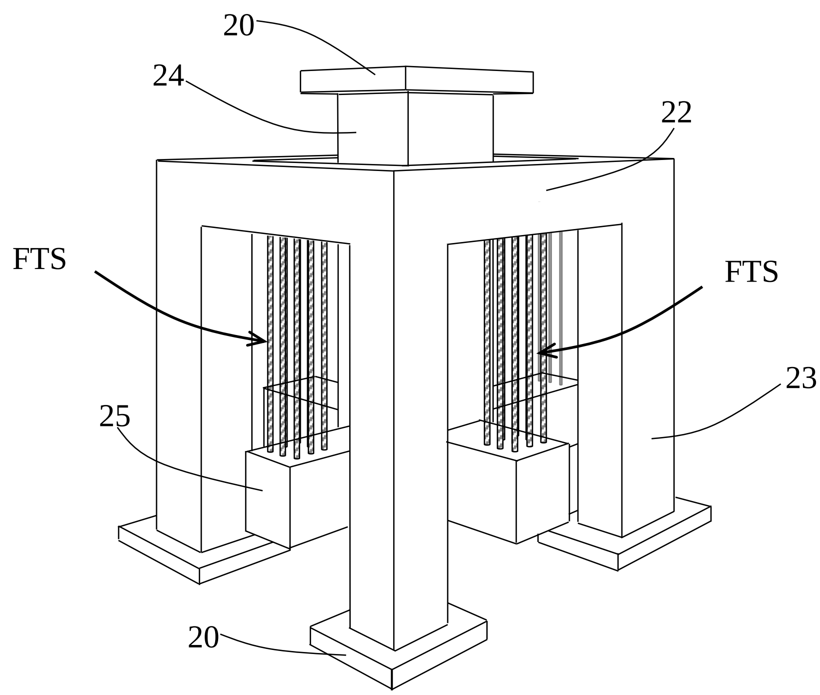
Figure 16: Isometric View of Typical STBI Frame with FTS
[Not Drawn to Scale]



5.6. Summary of Materials used in Case A and Case B.

The materials listed in Table 2 are used for structural purposes only; non-structural components are not included. Structural joints, bolts, anchor plates are not included also. And since design of foundations were not included in the study, their materials are not included, too.

Table 2: Summary of Materials Used in Case A and Case B

Materials	Case A: Fixed-Base	Case B: STBI - Isolated
Largest Column Size	HSS: 1.2 x 1.2 x 0.10 (m)	HSS: 0.9 x 0.9 x 0.11 (m)
Largest Beam Size	HSS: 0.3 x 0.3 x 0.02 (m)	HSS: 0.3 x 0.3 x 0.02 (m)
Largest Girder Size	HSS: 0.5 x 0.5 x 0.03 (m)	HSS: 0.5 x 0.5 x 0.03 (m)
Largest Bracing Size	HSS: 1.0 x 1.0 x 0.07 (m)	HSS: 0.7 x 0.7 x 0.05 (m)
Concrete (m^3)	2,692.0	2,656.0
Rebars (kg)	74,098.9	73,095.0
Structural Steel (kg)	7,178,598.0	6,878,189.8
Steel Wire Rope (m)	-	1,1097.0
Rigid Type Suspender (units)	-	16
Structural Walls (units)	8	-

6. Conclusion

Study shows that STBI-isolated structures could be implemented to high-rise structures with high aspect ratios. STBI-isolated structures subjected to high seismic intensities seem to perform better than non-isolated ones. Comparing the two modalities, non-isolated structure could be subjected to greater lateral force by around 50% over its STBI-isolated counterpart. Where structural walls are usually required for high-rise structures, STBI-isolated high-rise structures seem to fare well even without structural walls. Moreover, the evident reduction in structural members sizes could be an insight to a more cost-effective high-rise structure.

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