Adaptable shear wall layout in low-rise and light framed structures

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Abstract

As designers move towards more sustainable building materials, timber is often selected because it many advantages when compared to concrete and steel construction materials. Timber is a renewable resource that can be harvested and replenished. As long as this is done in a responsible manner, timber construction will always be around. Timber construction also has high carbon storage potential. About half the weight of dry timber comes from carbon and 1 ton of that carbon stores 3.67 tons of carbon dioxide (NSW, 2005). In a case study out of Britain, a nine-story timber residential building was examined. Along with using laminated waste wood, the building would provide 210 years of carbon storage (Timber & Sustainable Building, 2007).

For these and many more reasons, designers and engineers would prefer to use timber in construction projects. However, with the current knowledge of load paths of timber structures and concerns about fire, designs are limited to low-rise construction. This is defined as 60 to 65 feet in many municipalities. The purpose of this article is to examine the load path when using only interior shear walls given the current height limitation.

This article will address the following as it pertains to timber construction:

- adaptable shear wall layout
- rigid floor system in light-framed structure
- deconstruction and material reuse

The load path for an adaptable shear wall layout will be limited to certain material restrictions (strength and stiffness). The minimum amount of resistance given no torsional forces must be determined. Once this has been established, the amount the walls can move will be determined given specified amounts of eccentricity from the wall placement. This will be based on strength of the material as well as code requirements of drift and deflection.

Once load paths of interior shear walls can be determined given the height constraints of current building codes, taller structures can be explored using timber construction. By allowing taller timber structures designers and engineers will have the ability to make more responsible material selection.

Contents

1. Introduction

One of the most critical components of structural design is determining the load path for the forces that are applied to the structure. Mapping how the forces will travel from the top of the structure, down to the foundation will help ensure the life of the structure and the life safety of occupants in the structure.

In timber design, one of the current design methodologies for determining the lateral forces is to have a flexible diaphragm with long shear walls acting as the supports for the diaphragm. The force that is applied to each shear wall in a seismic event is proportional to the mass each wall supports and to the stiffness of the wall element relative to other lateral elements. By allowing the diaphragm to be idealized as flexible, the designer can assume no rotation of the diaphragm and not introduce additional forces caused from torsion (ASCE, 2006).

It is common to place the shear walls on the exterior of the building because walls will already exist at this location for insulation as well as gravity support for the structure. Making use of these existing walls for lateral support allows for fewer interior shear walls (if any), which can open up the floor plan and allow changes of wall placement without risking the structural integrity of the building.

Although the exterior shear walls allow for flexibility in configuring the interior, they do not give as much flexibility to the exterior skin. The building envelope traditionally is made of sheathing, weather proofing material, insulation, and an exterior facade. There is not much flexibility when it comes to the shear wall design of timber construction. However, is this really a bad thing? The design works very well at transferring the forces and creates a clear load path for the engineer.

As the building becomes larger and taller, the forces increase and determining the load path for timber structures becomes more difficult. In order to compensate for the increase of loads, the amount of wall space needed for lateral resistance increases. This is when conversations over the wall use between architects and structural engineers will begin. Designers would traditional prefer more glazing to allow for natural lighting, passive heat storage in thermal masses, sun shading, urban agriculture, etc. The engineer would like to ensure the design will not fail and perhaps be more conservative with the design of the structural system. Both sides present valid arguments for use of the wall system. No one wants to live or work in a box with no windows and it is important that the building stand up. The key is to find the balance between structural efficiency and desired use of the building.

A solution to this problem which many have to be dealt with is to move the shear walls to the interior of the building. This concept has been adopted with many high-rise concrete structures. The core of the building is where all the strength and stiffness is located and the exterior of the structure is allowed the flexibility to be what the designer would like. It appears both sides have reached an agreement and are satisfied with these designs. However, this may be difficult to do with timber construction because the material does not have the same material properties as concrete that would be required for this type of design.

Timber structures are an essential component of sustainable design. Timber is renewable and has high carbon storage potential. Along with these and many other reasons, timber construction is often preferred over concrete and steel. Except for when the structure is over height limitations for timber given the building code of a city. In this case either the building design must be changed, the building material must change, or a variance to the building code must be obtained. Though it is not likely that a skyscraper will ever have a timber shear wall and diaphragm configuration, it would be interesting to see how tall timber structures can actually become someday.

To determine if a timber building can resist lateral forces with only interior shear walls, a sample building was analyzed assuming ideal conditions. The structure was assumed to have a footprint of 50 x 100 feet (15.24 x 30.48 meters) and is located in Seattle, Washington. The 50 and 100 foot dimensions will be referred to as the transverse direction (x-direction) and the longitudinal direction (y-direction), respectively. In this region of the country, seismic may tend to govern over wind design. Therefore, seismic loads were the only lateral forces applied to the structure. However, the way wind and seismic loads are applied to the structure are similar. A distinction needed to be made in order to determine the allowable design values.

The lateral resistance in the structure was selected to be iLevel shear braces rather than traditional shear walls made of sheathing and wood framing. The braces are prefabricated in a factory and come ready to place in the structure. They do not require any modifications thus construction time is reduced; which can lower labor costs. The braces are constructed of iLevel laminated strand lumber (LSL) and act similar to a deep beam in the transfer of shear with a large moment resisting connection at the bottom. The braces currently come in widths up to 24 inches and up to 12 feet in height (Weyerhaeuser, 2010).

In Seattle, the height limit for timber construction is 65 feet with a maximum number of five stories constructed of timber and allows one story of concrete (ICC, 2006). The brace that was selected for this design example was the shear brace iSB 24x9 which will allow for five timber stories at about 9 feet tall and an additional concrete story standing about 20 feet, which is within the height limit set by the code.

In order to simplify the design of the building and to determine if this component of the lateral resisting system will work, the following assumptions were made:

- ideal rigid diaphragm with no deformation
- shear braces are not restricted to story limits or stacked applications
- connection of wall panels to diaphragm will not govern design
- building meets building fire code
- independent gravity and lateral resisting systems
- 1st story constructed of concrete is designed to resist the loads

Due to the random placement of the shear braces, a clear load path needs to be determined. If the diaphragm was modeled as flexible, it may be more difficult to trace the path of the lateral forces down the structure. Idealizing the diaphragm as rigid allows torsion to be developed upon loading, which will transfer the moments from story to story, rather than having a flexible diaphragm and adding collectors and stiffening up sections of the diaphragm in order to transfer forces from support to support.

The diaphragm selected for this design was a timber-concrete composite floor system. These floor systems are popular in Europe for many reasons. They are very efficient at transferring loads on a per weight basis compared to an all concrete floor system (Ceccotti, 2002). They also reduce the amount of noise transmitted through the floor which can be a problem in apartments and office buildings.

Even though diaphragms will deflect under loading, this model assumed no deformation of the diaphragm in order to simplify the story drift calculations. In reality, the deflection of the diaphragm would be included in the drift calculations. One particular example of timber-concrete composite floor deflection is from a study presented at the 2009 New Zealand Society for Earthquake Engineering (NZSEE) conference. The study showed the ratio of the deflection of the diaphragm to the deflection of the connections to be on average less than 5%. However, the intent of this project is not the design of the diaphragm, but rather to show the load path of the shear walls.

The composite floor can also act as a thermal mass, storing heat during the day and releasing it during the night. This will help reduce the energy demand for the structure both in the summer and winter months. By moving the lateral supports towards the center of the structure, more solar radiation can be absorbed thus increasing the efficiency of heat storage.

In the iLevel literature, only two types of walls are specified, stand-alone and stacked wall applications. The use in the design example is really neither of these. It may happen that the walls are placed above another, but they would not be directly connected together but rather to the diaphragm. It is up to the rigid floor system to allow each of the wall segments to act as independent stand-alone wall segments and to transfer the loads down to the foundation.

It is the purpose of this example to use the concept of the shear brace and the design values associated with a particular model for this structure. Therefore, the limitations on height and connections are not applied to this example. It is assumed that the braces can be used in all the stories of the structure and are not required to be directly connected to one another.

It was also assumed that the design of the building was governed by the failure of the shear braces, not any other system; all other failure mechanisms are assumed to be stronger.

Since it is hard to get full composite action between the wood and the concrete, it is often modeled as having partial composite action (Ceccotti, 2002). In order to increase the amount of composite action between the two materials, shear studs can be introduced to the system; similar to steel-concrete composite decks. The shear studs will allow the two different materials to act as more as one material (depending on the amount of composite action achieved).

Figure 1 is an example of the connection one might use for the shear wall to the diaphragm above; it would be the same for the connection to the diaphragm below. The shear stud can serve two functions, increase composite action between the two materials as well as serving as a deconstructible connection for the shear wall to the diaphragm. This connection would create an unsmooth surface for the ceiling and floor that also allows for air to travel through. Therefore, a false floor and ceiling would be needed for sound and thermal insulation as well as fire concerns. Another solution could be using plugs for the holes to stop air from traveling from story to story. Having the deconstructible connection allows for the shear brace to be reused rather than being disposed of in a landfill. The figure shown illustrates more of a concrete-steel composite floor rather than a traditional wood-concrete composite floor system. This idea shows a different option for the type of wood sheathing used in the system resembling more of a steel deck.

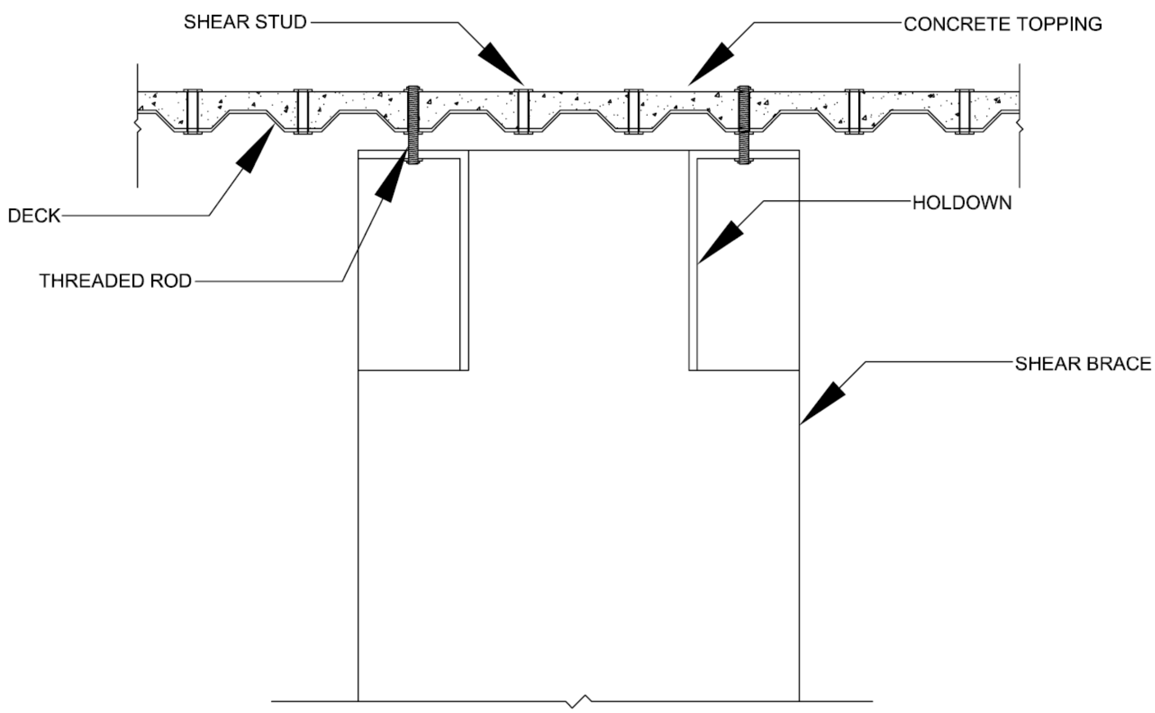


Figure 1: Connection of diaphragm to shear brace

According to the EPA, 160 million tons of waste (one-third of the nation's non-hazardous solid waste) from the construction industry will end up in a landfill each year. This waste is comprised of the material that is wasted during the construction and also the waste from the demolition of the structure. The shear braces help reduce the amount of both. The braces are made of small strands of lumber that are pressed together with adhesives. They are made off site so what little waste that comes from the assembly can be reused for another material at the production facility.

The shear brace material has already has been tested to determine the rate at which it burns. The LSL has a char-rate of 1.8 inches/hour which is similar to other wood construction material such as Douglas-Fir, which has a char rate of 1.4 inches/hour (Weyerhaeuser, 2008). The floor system is assumed to meet fire code requirements for minimum thickness of insulating materials.

If the shear braces were designed to take both the lateral and gravity loads, they would have increased capacities in terms of uplift forces. They would however have additional forces that would need to be included when designing the compression connection to the diaphragm. This is why it will be very difficult to completely exclude the gravity force in the overturning design. Also, the proximity of the holdowns of a given brace will be a point of concern in design and would need to be examined. The diaphragm would have to be designed to accommodate the large uplift and compression force only two feet apart.

Since it was never stated if this structure was a residential or office building, it would be hard to determine how open the floor plan would be. If it was an office building, a larger workspace may have been utilized and only partitions might be used. Compared to a residential building where bedrooms and bathrooms would provide intermediate supports for the gravity forces, the loads might not be transferred to the braces.

If the gravity loads were calculated to be applied to the braces, but were not present during a seismic event, actual overturning forces could be greater than what were designed for and failure could occur. The idea of having the ability to move the braces around is the main reason for different systems and the reason for the approach of not including the gravity in the lateral design of the braces.

As stated in the beginning of the article, the first floor would be of concrete design and the next five stories would be timber. The assumption was made that this portion of the structure was designed according to the code and was capable of resisting the forces from the above stories. The concrete design is not of concern in this example.

2. Objective

In an effort to develop guidelines for placement of small, interior shear wall segments while facilitating floor plans that can be adapted for changing building use; the specific objectives of this research are to:

1. Assess the minimum symmetric layout for internal shear walls given a specific prefabricated shear wall system.
2. Develop guidelines for wall placement in both directions to accommodate a maximum torsion limit without the addition of more shear braces.

3. Procedure

The allowable design loads of the shear braces were all given in the document TJ-8620 by iLevel. Given the selected iSB 24x9 section the allowable seismic shear design is 3,905 lbs., allowable drift is 0.42 inches and allowable uplift is 21,280 lbs. (Weyerhaeuser, 2010). The seismic design coefficients for the walls were obtained from the document. The seismic design coefficients are identical to those of timber construction. Using the 2006 IBC values for design the response modification coefficient is 6.5, the system overstrength factor is 3 and the deflection amplification factor is 4 (Weyerhaeuser, 2010). All of these values were taken from the stand-alone brace on concrete foundation application.

The seismic base shear was determined using the equivalent lateral force procedure from section 12.8 of the ASCE Standard 7-05. The soil properties of the site were unknown, therefore site D soil was assumed for the design.

The timber-concrete composite floor system yielded a dead load of approximately 35 psf. The weight of the partition walls was estimated to be 5 psf above and 5 psf below each floor in addition to the floor system. Also, a 5 psf load was added to the roof to account for roofing materials and mechanical ventilation units. In summary, each floor had an estimated weight of 45 psf. The total base shear was calculated to be 116 kips using allowable design, the distribution is shown below in Table 1.

Table 1: Vertical distribution of base shear

	Weight (W_x)	Story Height (h_x)	$W_x h_x^k$	C_{vx}	Force (F_x)
Roof Diaphragm	225000 lbs.	65.00 ft.	14625000 ft.-lbs.	0.2565	29.71 kips
6th Story Diaphragm	225000 lbs.	55.89 ft.	12576563 ft.-lbs.	0.2206	25.54 kips
5th Story Diaphragm	225000 lbs.	46.79 ft.	10528125 ft.-lbs.	0.1846	21.38 kips
4th Story Diaphragm	225000 lbs.	37.68 ft.	8479688 ft.-lbs.	0.1487	17.22 kips
3rd Story Diaphragm	225000 lbs.	28.58 ft.	6431250 ft.-lbs.	0.1128	13.06 kips
2nd Story Diaphragm	225000 lbs.	19.47 ft.	4382813 ft.-lbs.	0.0769	8.90 kips
$\Sigma =$	1125000 lbs.		57023438 ft.-lbs.	1.0000	115.82 kips

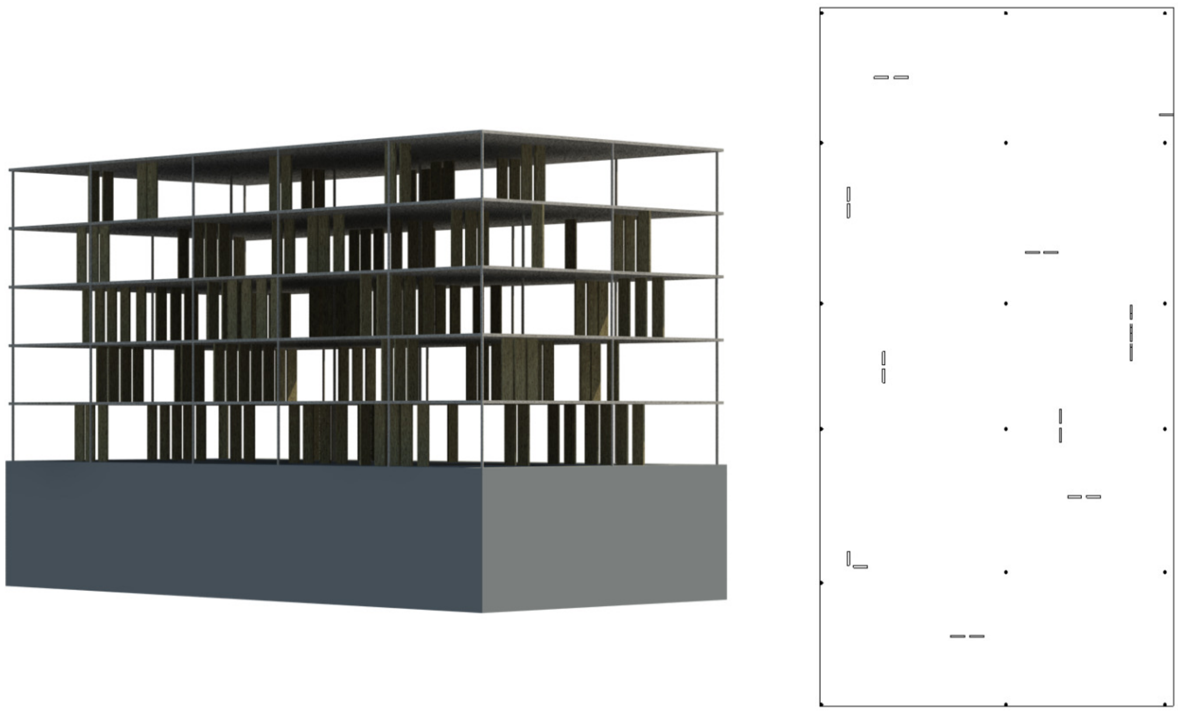
The structure was analyzed in the lateral direction with the walls placed so there was no torsion that developed due to the orientation of the walls. However, they were designed to resist small torsion caused from accidental shifting of mass in the system as defined in Section 12.8.4.2 in ASCE 7-05.

To determine the amount the walls can move without causing failure to the structure will be done by analyzing several layouts of the shear walls. The original layout from the pure lateral resistance will be the base for this analysis. From here, multiple walls will be moved to create a specified eccentricity of loading for the diaphragm. The eccentricity will essentially be the distance that the center of rigidity moves from the center of mass.

The model will be analyzed using two directions of resistance; perpendicular and parallel to the direction of loading. The direction parallel to the loading will resist only the direct lateral forces while the walls perpendicular to loading will resist the torsion. This allows the system to retain the same number of walls without causing failure to existing walls. Failure can occur from the walls resisting direct shear forces as well as forces from torsion, and the effects would need to be superimposed during design. The point of this example is to determine how much the walls can move rather than the point at which more resistance is required. As the walls move farther from the canter of mass, the amount of torsional forces increases. This extra force applied to the walls resisting the lateral forces could potentially cause failure.

A spreadsheet will be used in determining the distribution of the lateral forces. The spreadsheet will determine the shifting center of rigidity by taking a weighted average of the stiffness and the distance from the center of mass of each wall to the summation of all the walls and their stiffness. Using the same weighted average technique, the amount of force each wall will resist from torsion can be determined.

Several iterations of moving the supports will be analyzed. The wall will be moved to create a specified eccentricity. The amount of rotation from torsional forces will be plotted against the amount of eccentricity to determine what type of trend exists in the system. From this trend, guidelines on the allowable amount walls can be moved in this type of system without failure to the structure can be created.



(a) (b)

Figure 2: (a) 3D rendering of structure. (b) 6th story placement of shear braces

The spreadsheet was set up to immediately see how the forces increase and decrease when a wall is moved. The idea was to see how moving one wall affects the other walls and to find a good balance between symmetry and layout. Once all of the braces were placed and their capacities checked, the total force was transferred to the story below (including direct shear forces and torsion caused from eccentricity of loading).

4. Results

The spreadsheet was set up to determine the total shear force that the wall must resist given direct shear, accidental torsion and unsymmetrical placement of shear walls. The direct shear force is the ratio of the stiffness of a given wall over the total stiffness of the system. The shear due to torsion was determined by the ratio of distance from the center of mass times the stiffness (yR_x) over the summation of the distance from the center of mass squared times the stiffness (Σ

$y^2 R_x$). This ratio accounts for the torsion created from shifting of the center of rigidity from the center of mass.

For each story and direction of loading, a similar table was used. However, as the load was traced towards the foundation, the number of shear braces increased. The placement of the braces had an effect on the total torsion applied to each story. The total number of braces required for each story can be seen below in Table 2.

Table 2: Number of shear braces per story

Story	Transverse	Longitudinal
6	08	08
5	15	15
4	20	20
3	25	25
2	28	28
Σ	96	96

The allowable story drift was determined and checked against the actual deflections. According to Table 12.12-1 from ASCE 7-05, the allowable story drift is 2.0% of the story height (9 feet) or 2.2 inches. Since the allowable drift of the shear braces was 0.42 inches and the assumption of no deformation of the diaphragm, the allowable story drift set by ASCE 7-05 section 12.8.6 was never exceeded. The deflection of the story at the center of mass is determined by multiplying the deflection from elastic analysis by the deflection amplification factor. In this case, the deflection is 1.68 inches. The angle of rotation was then the arctangent of the displacement over the distance of the wall to the center of rigidity.

The figures below show the relationship between the rotations of the stories and amount of eccentricity for the applied load. Four different cases were evaluated and the variables that changed were the directions of applied force and the walls that moved. There are many possible configurations the walls could move, resulting in different relationships of rotation and eccentricity. This example examines one particular building geometry and prescribed wall movements.

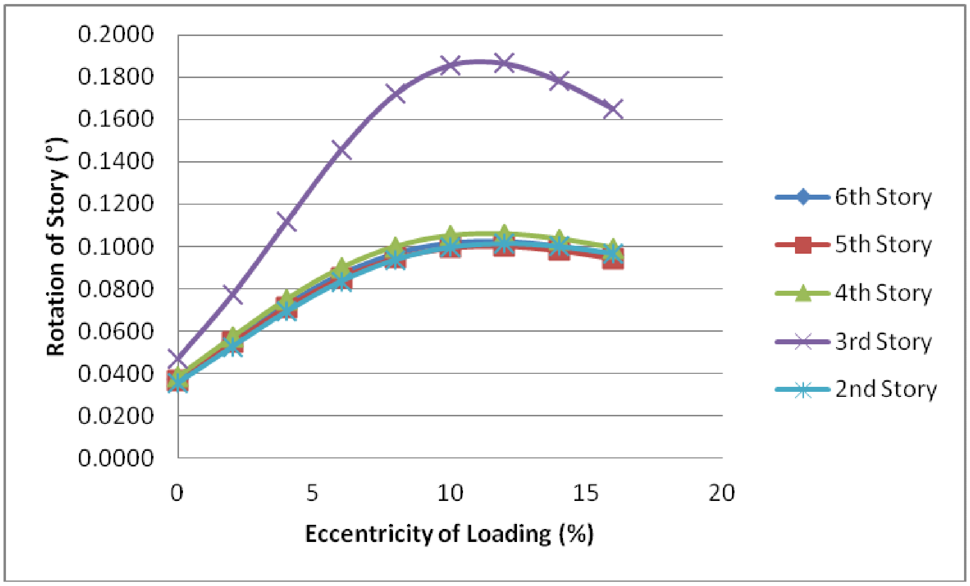


Figure 3: Rotation of story, transverse loading, and 2 directions of shifting eccentricity

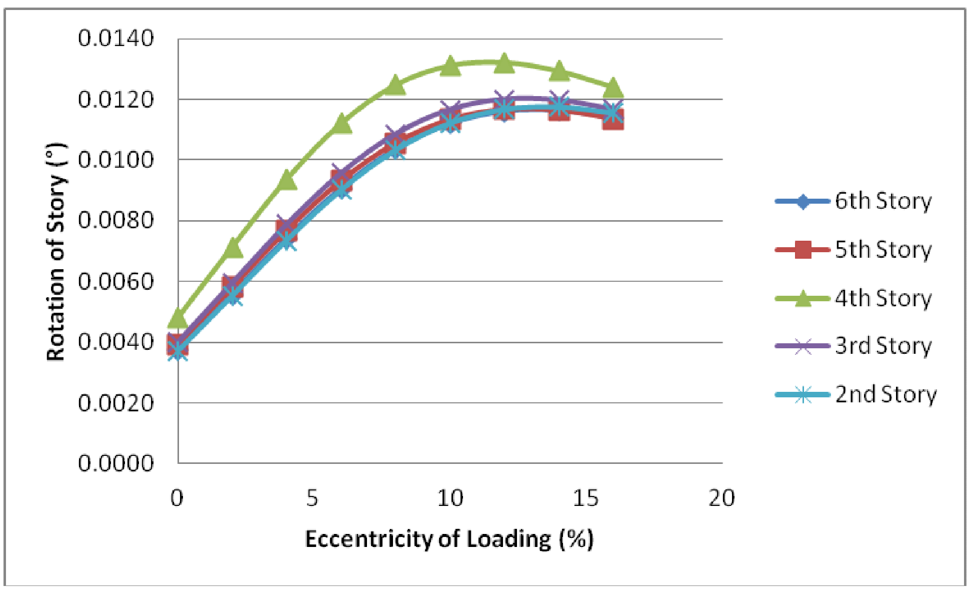


Figure 4: Rotation of story, longitudinal loading, and 2 directions of shifting eccentricity

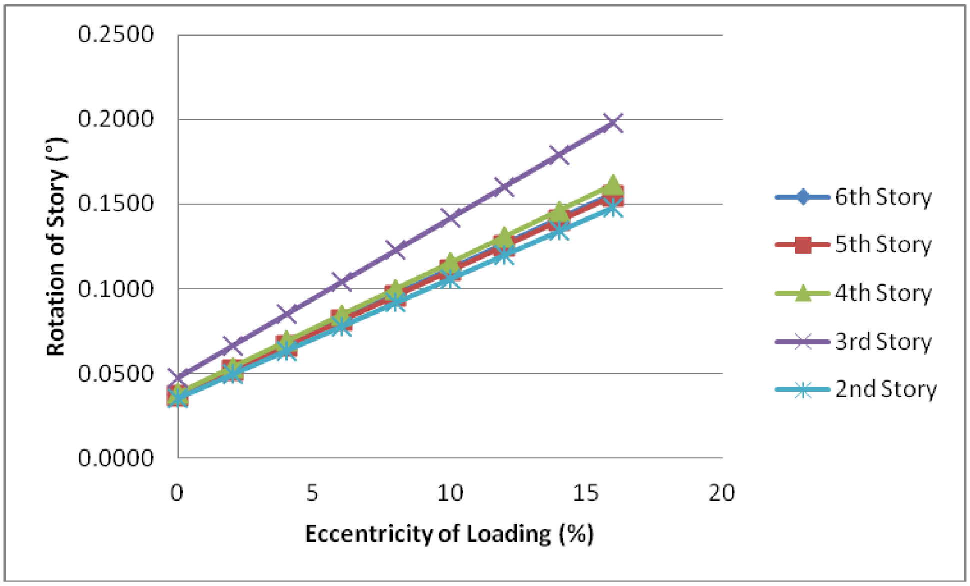


Figure 5: Rotation of story, transverse loading, and 1 directions of shifting eccentricity

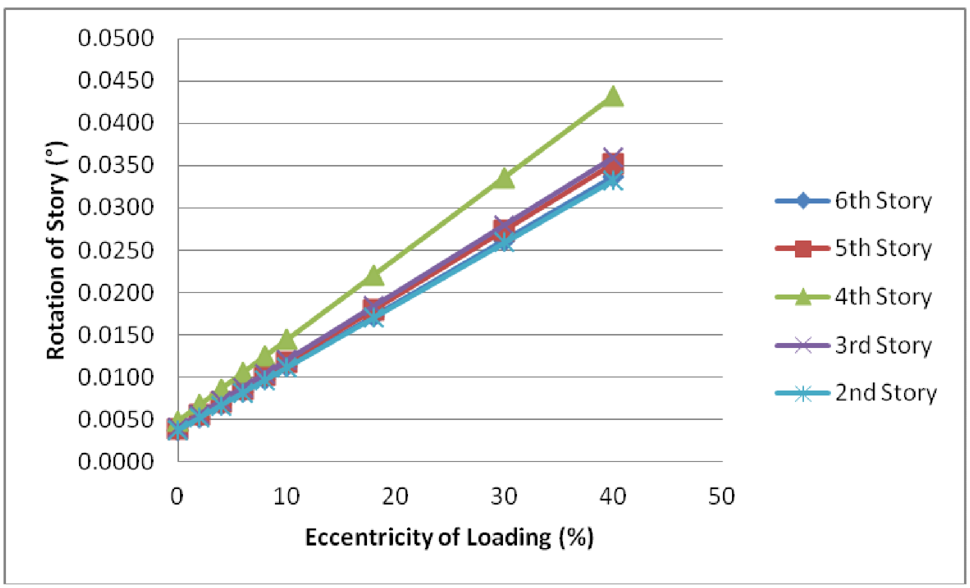


Figure 6: Rotation of story, longitudinal loading, and 1 directions of shifting eccentricity

The number of shear braces required was the same in each direction. The reasoning for this is that the seismic force is the same for each direction because it is related to the mass of the structure rather than the geometry. If the structure was analyzed with wind forces, the number of braces in each direction would be different and possibly less than when designed for seismic. Also, while there is no torsion from placement of the walls, the walls perpendicular to the loading have to resist only minimal loads.

Once torsion was introduced into the system, the way the forces were distributed changed. When the seismic force was applied in the longitudinal direction, the torsional forces that needed to be resisted were smaller when compared to the forces from transverse loading. This distribution differs because of the distance from the support to the center of rigidity is much larger in one direction (longitudinal) when compared to the other.

Two different types of wall movement were analyzed (1-directional and 2-directional movement). When the eccentricity only moved in one direction the relationship between story rotation and eccentricity of applied force was linear. Also, when the seismic force was applied in the transverse direction, the material limit was reached much quicker than when it was applied in the longitudinal. At around 5% eccentricity, the shear braces perpendicular to loading would experience failure when loaded in the transverse direction. This is because the walls resisting the twisting of the story by forming a set of couple forces are much closer together than those resisting the moment when the seismic force is applied in the longitudinal direction.

When the loads were applied in the longitudinal direction the material limit was not reached when the eccentricity of loading was 40% of that dimension. However, given the linear relationship it would make sense that the amount of eccentricity that would cause material failure could be estimated. Also, the distance at which the building system may see failure might be greater than 50% which would mean the eccentricity of loading would be off the building footprint. At 40%, the shear braces were approaching their material property limit.

In the case of both directions of walls moving, the relationship between eccentricity of loading and story rotation started out to be linear. After about 5% eccentricity of loading, the relationship became more parabolic. It was interesting that at 5% eccentricity the relationship changed in both loading cases (longitudinal and transverse) because looking back at the 1-directional wall movement, this was the point of failure for the transverse loading.

5. Conclusions

From the computer analysis and given the assumptions stated above, the interior wall system along with a rigid diaphragm has the capacity to transfer the loads from the roof down to the foundation. The building does not exhibit any extreme deflections or torsional forces that raise concern.

The conclusions drawn from this model are specific to this building geometry and the idea that no more shear braces would be added to the system. The amount the walls along the y-axis can move is limited to the amount that would create 5% eccentricity when loading in the transverse direction. The amount the walls along the x-axis can move is limitless, given this specific building.

6. Future Research

More analysis is needed on this methodology in order for it to become an accepted design. The computer model successfully transferred the loads given all of the assumptions stated above. For this design to proceed in the development of the analysis, more computer modeling is needed along with actual testing to determine if the load path idealized in the model would be demonstrated upon actual loading.

The assumptions stated above are possible areas for future work which include:

- Light weight rigid diaphragm
- Stronger wall segments
- Connection of wall panels to diaphragm

In order for this system of interior walls with no direct load path to work, a rigid diaphragm is needed. The timber-concrete composite floor system can be idealized as rigid and transfer the loads from story to story, but it would be good to see improvements in sustainability. Going back to deconstruction, it would be interesting to see if a system made of floor segments could be used rather than slabs of concrete. An alternative could be a system of post tension floor segments, which have the capacity to transfer loads while being small enough to be considered modular. This would help reduce the amount of waste if in the future the building is renovated or removed.

The walls that were used in this example were selected because of their size and their strength. Also, they were selected as an example of a prefabricated wall system that could be adopted in a future building. They are small enough to be moved without machinery, and still have the design values required for this example. However, it may be beneficial to have wall segments that have increased capacities as far as stiffness and strength as long as the increased stiffness does not cause the failure mechanism to shift from the walls to the diaphragm. With the stronger wall sections, the number of supports could also be reduced. This would open up the floor plan and allow for more flexibility within a story by increasing the limit the walls can move. Also, more natural light and solar radiance could enter the building.

With the modular connection, the segments could easily be replaced if in the future stronger segments are developed. The connections would need to be examined so that the transfer of the forces could be accomplished without failure when the walls segments are moved or replaced. This example utilizes bolted connections that can be used over and over with minimal loss in efficiency in transferring the loads. Similar to the limits created in this example, the amount a wall can move could be based on the connection and the additional forces in the connections that are a result from the walls segments moving.

This design example addressed many topics of sustainable design related to the structural system of a building. The lateral resisting system utilized timber materials which and reduce and store carbon emissions. Also, by allowing the structural system to move the occupants have the potential of changing a floor plan without demolishing and rebuilding the existing walls. The system that allows the walls to move around the structure is the use mechanical connection of the walls to the diaphragm rather than bolts embedded into concrete. There are many more ways to make a structural system more sustainable. The information presented here is designed to help provide better and more sustainable shear wall systems.

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8. Appendix

This section contains a visual representation of the adaptable shear wall system as it pertains to the shifting center of rigidity, as well as tables containing numerical output from spreadsheets that were used for the calculations of the structural system.

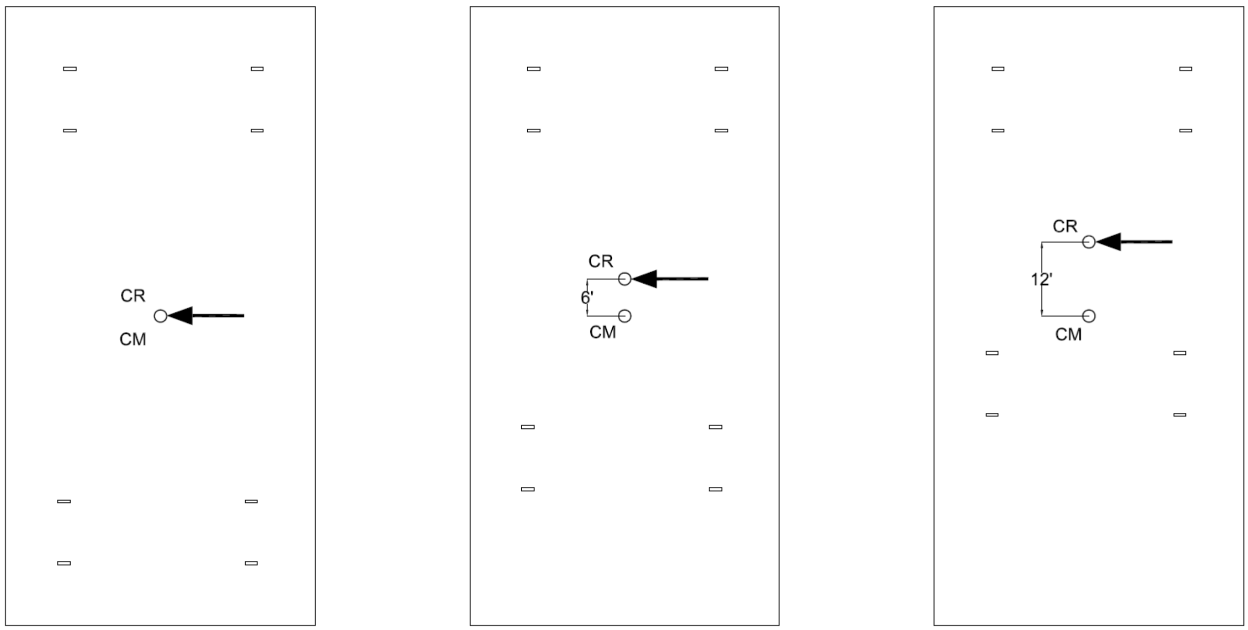

Figure 7: Shifting center of rigidity (a) 0% (b) 6% (c) 12% as a result of walls moving

Table 3: Rotation of story, transverse loading, and 2 directions of shifting eccentricity

6th Story		5th Story		4th Story		3rd Story		2nd Story	
e (%)	Rotation (°)	e (%)	Rotation (°)	e (%)	Rotation (°)	e (%)	Rotation (°)	e (%)	Rotation (°)
0	0.0372	0	0.0369	0	0.0384	0	0.0472	0	0.0353
2	0.0553	2	0.0546	2	0.0571	2	0.0772	2	0.0525
4	0.0726	4	0.0714	4	0.0750	4	0.1118	4	0.0693
6	0.0869	6	0.0851	6	0.0897	6	0.1457	6	0.0835
8	0.0967	8	0.0946	8	0.0998	8	0.1720	8	0.0937
10	0.1017	10	0.0994	10	0.1049	10	0.1856	10	0.0995
12	0.1025	12	0.1003	12	0.1058	12	0.1865	12	0.1013
14	0.1003	14	0.0983	14	0.1035	14	0.1782	14	0.1000
16	0.0962	16	0.0945	16	0.0993	16	0.1651	16	0.0966

Table 4: Rotation of story, longitudinal loading, and 2 directions of shifting eccentricity

6th Story		5th Story		4th Story		3rd Story		2nd Story	
e (%)	Rotation (°)	e (%)	Rotation (°)	e (%)	Rotation (°)	e (%)	Rotation (°)	e (%)	Rotation (°)
0	0.0037	0	0.0039	0	0.0048	0	0.0040	0	0.0037
2	0.0056	2	0.0058	2	0.0071	2	0.0059	2	0.0055
4	0.0074	4	0.0077	4	0.0094	4	0.0079	4	0.0074
6	0.0091	6	0.0093	6	0.0112	6	0.0096	6	0.0090
8	0.0104	8	0.0106	8	0.0125	8	0.0108	8	0.0103
10	0.0112	10	0.0113	10	0.0131	10	0.0117	10	0.0112
12	0.0116	12	0.0117	12	0.0132	12	0.0120	12	0.0117
14	0.0116	14	0.0116	14	0.0129	14	0.0120	14	0.0118
16	0.0114	16	0.0114	16	0.0124	16	0.0117	16	0.0116

Table 5: Rotation of story, transverse loading, and 1 directions of shifting eccentricity

6th Story		5th Story		4th Story		3rd Story		2nd Story	
e (%)	Rotation (°)	e (%)	Rotation (°)	e (%)	Rotation (°)	e (%)	Rotation (°)	e (%)	Rotation (°)
0	0.0372	0	0.0369	0	0.0384	0	0.0472	0	0.0353
2	0.0521	2	0.0517	2	0.0538	2	0.0661	2	0.0494
4	0.0670	4	0.0664	4	0.0691	4	0.0850	4	0.0636
6	0.0819	6	0.0812	6	0.0845	6	0.1038	6	0.0777
8	0.0967	8	0.0960	8	0.0998	8	0.1227	8	0.0918
10	0.1116	10	0.1107	10	0.1152	10	0.1416	10	0.1059
12	0.1265	12	0.1255	12	0.1305	12	0.1605	12	0.1201
14	0.1414	14	0.1402	14	0.1459	14	0.1793	14	0.1342
16	0.1563	16	0.1550	16	0.1613	16	0.1982	16	0.1483

Table 6: Rotation of story, longitudinal loading, and 1 directions of shifting eccentricity

6th Story		5th Story		4th Story		3rd Story		2nd Story	
e (%)	Rotation (°)	e (%)	Rotation (°)	e (%)	Rotation (°)	e (%)	Rotation (°)	e (%)	Rotation (°)
0	0.0037	0	0.0039	0	0.0048	0	0.0040	0	0.0037
2	0.0052	2	0.0055	2	0.0067	2	0.0056	2	0.0052
4	0.0067	4	0.0070	4	0.0086	4	0.0072	4	0.0067
6	0.0082	6	0.0086	6	0.0106	6	0.0088	6	0.0081
8	0.0097	8	0.0102	8	0.0125	8	0.0104	8	0.0096
10	0.0112	10	0.0117	10	0.0144	10	0.0120	10	0.0111
18	0.0172	18	0.0180	18	0.0221	18	0.0184	18	0.0170
30	0.0262	30	0.0274	30	0.0336	30	0.0280	30	0.0259
40	0.0336	40	0.0352	40	0.0432	40	0.0359	40	0.0333

Table 7: 6th story force distribution, symmetric layout

Wall Number	Stiffness of Wall, R_x (k/in)	Distance from CL, y (ft)	Distance from CL, x (ft)	Distance from CR, y (ft)	Distance from CR, x (ft)	yR_x -or- xR_y	y^2R_x -or- x^2R_y	Shear due to direct Shear, F_V	Shear due to Torsion, F_T	F_{total} (kips)
1	9.298	40		45				3.71		3.71
2	9.298	40		45				3.71		3.71
3	9.298	30		35				3.71		3.71
4	9.298	30		35				3.71		3.71
5	9.298	-30		-25				3.71		3.71
6	9.298	-30		-25				3.71		3.71
7	9.298	-40		-35				3.71		3.71
8	9.298	-40		-35				3.71		3.71
9	9.298		20		22.5	209	4707		1.63	1.63
10	9.298		20		22.5	209	4707		1.63	1.63
11	9.298		10		12.5	116	1453		0.91	0.91
12	9.298		10		12.5	116	1453		0.91	0.91
13	9.298		-10		-7.5	-70	523		-0.54	-0.54
14	9.298		-10		-7.5	-70	523		-0.54	-0.54
15	9.298		-20		-17.5	-163	2848		-1.27	-1.27
16	9.298		-20		-17.5	-163	2848		-1.27	-1.27
Σ	148.768					186	19061	29.71	1.45	

Table 8: 6th story force distribution, 2% eccentricity

Wall Number	Stiffness of Wall, R_x (k/in)	Distance from CL, y (ft)	Distance from CL, x (ft)	Distance from CR, y (ft)	Distance from CR, x (ft)	yR_x -or- xR_y	y^2R_x -or- x^2R_y	Shear due to direct Shear, F_V	Shear due to Torsion, F_T	F_{total} (kips)
1	9.298	40		47				3.71		3.71
2	9.298	40		47				3.71		3.71
3	9.298	30		37				3.71		3.71
4	9.298	30		37				3.71		3.71
5	9.298	-26		-19				3.71		3.71
6	9.298	-26		-19				3.71		3.71
7	9.298	-36		-29				3.71		3.71
8	9.298	-36		-29				3.71		3.71
9	9.298		20		23.5	219	5135		2.53	2.53
10	9.298		20		23.5	219	5135		2.53	2.53
11	9.298		10		13.5	126	1695		1.45	1.45
12	9.298		10		13.5	126	1695		1.45	1.45
13	9.298		-8		-4.5	-42	188		-0.48	-0.48
14	9.298		-8		-4.5	-42	188		-0.48	-0.48
15	9.298		-18		-14.5	-135	1955		-1.56	-1.56
16	9.298		-18		-14.5	-135	1955		-1.56	-1.56
Σ	148.768					335	17945	29.71	3.88	

Table 9: 6^{th} story force distribution, 4% eccentricity

Wall Number	Stiffness of Wall, R_x (k/in)	Distance from CL, y (ft)	Distance from CL, x (ft)	Distance from CR, y (ft)	Distance from CR, x (ft)	yR_x -or- xR_y	y^2R_x -or- x^2R_y	Shear due to direct Shear, F_V	Shear due to Torsion, F_T	F_{total} (kips)
1	9.298	40		49				3.71		3.71
2	9.298	40		49				3.71		3.71
3	9.298	30		39				3.71		3.71
4	9.298	30		39				3.71		3.71
5	9.298	-22		-13				3.71		3.71
6	9.298	-22		-13				3.71		3.71
7	9.298	-32		-23				3.71		3.71
8	9.298	-32		-23				3.71		3.71
9	9.298		20		24.5	228	5581		3.47	3.47
10	9.298		20		24.5	228	5581		3.47	3.47
11	9.298		10		14.5	135	1955		2.05	2.05
12	9.298		10		14.5	135	1955		2.05	2.05
13	9.298		-6		-1.5	-14	21		-0.21	-0.21
14	9.298		-6		-1.5	-14	21		-0.21	-0.21
15	9.298		-16		-11.5	-107	1230		-1.63	-1.63
16	9.298		-16		-11.5	-107	1230		-1.63	-1.63
Σ	148.768					483	17573	29.71	7.36	

Table 10: 6^{th} story force distribution, 6% eccentricity

Wall Number	Stiffness of Wall, R_x (k/in)	Distance from CL, y (ft)	Distance from CL, x (ft)	Distance from CR, y (ft)	Distance from CR, x (ft)	yR_x -or- xR_y	y^2R_x -or- x^2R_y	Shear due to direct Shear, F_V	Shear due to Torsion, F_T	F_{total} (kips)
1	9.298	40		51				3.71		3.71
2	9.298	40		51				3.71		3.71
3	9.298	30		41				3.71		3.71
4	9.298	30		41				3.71		3.71
5	9.298	-18		-7				3.71		3.71
6	9.298	-18		-7				3.71		3.71
7	9.298	-28		-17				3.71		3.71
8	9.298	-28		-17				3.71		3.71
9	9.298		20		25.5	237	6046		4.32	4.32
10	9.298		20		25.5	237	6046		4.32	4.32
11	9.298		10		15.5	144	2234		2.62	2.62
12	9.298		10		15.5	144	2234		2.62	2.62
13	9.298		-4		1.5	14	21		0.25	0.25
14	9.298		-4		1.5	14	21		0.25	0.25
15	9.298		-14		-8.5	-79	672		-1.44	-1.44
16	9.298		-14		-8.5	-79	672		-1.44	-1.44
Σ	148.768					632	17945	29.71	11.51	

Table 11: 6th story force distribution, 8% eccentricity

Wall Number	Stiffness of Wall, R_x (k/in)	Distance from CL, y (ft)	Distance from CL, x (ft)	Distance from CR, y (ft)	Distance from CR, x (ft)	yR_x -or- xR_y	y^2R_x -or- x^2R_y	Shear due to direct Shear, F_V	Shear due to Torsion, F_T	F_{total} (kips)
1	9.298	40		53				3.71		3.71
2	9.298	40		53				3.71		3.71
3	9.298	30		43				3.71		3.71
4	9.298	30		43				3.71		3.71
5	9.298	-14		-1				3.71		3.71
6	9.298	-14		-1				3.71		3.71
7	9.298	-24		-11				3.71		3.71
8	9.298	-24		-11				3.71		3.71
9	9.298		20		26.5	246	6530		4.99	4.99
10	9.298		20		26.5	246	6530		4.99	4.99
11	9.298		10		16.5	153	2531		3.11	3.11
12	9.298		10		16.5	153	2531		3.11	3.11
13	9.298		-2		4.5	42	188		0.85	0.85
14	9.298		-2		4.5	42	188		0.85	0.85
15	9.298		-12		-5.5	-51	281		-1.04	-1.04
16	9.298		-12		-5.5	-51	281		-1.04	-1.04
Σ	148.768					781	19061	29.71	15.82	

Table 12: 6th story force distribution, 10% eccentricity

Wall Number	Stiffness of Wall, R_x (k/in)	Distance from CL, y (ft)	Distance from CL, x (ft)	Distance from CR, y (ft)	Distance from CR, x (ft)	yR_x -or- xR_y	y^2R_x -or- x^2R_y	Shear due to direct Shear, F_V	Shear due to Torsion, F_T	F_{total} (kips)
1	9.298	40		55				3.71		3.71
2	9.298	40		55				3.71		3.71
3	9.298	30		45				3.71		3.71
4	9.298	30		45				3.71		3.71
5	9.298	-10		5				3.71		3.71
6	9.298	-10		5				3.71		3.71
7	9.298	-20		-5				3.71		3.71
8	9.298	-20		-5				3.71		3.71
9	9.298		20		27.5	256	7032		5.45	5.45
10	9.298		20		27.5	256	7032		5.45	5.45
11	9.298		10		17.5	163	2848		3.47	3.47
12	9.298		10		17.5	163	2848		3.47	3.47
13	9.298		0		7.5	70	523		1.49	1.49
14	9.298		0		7.5	70	523		1.49	1.49
15	9.298		-10		-2.5	-23	58		-0.50	-0.50
16	9.298		-10		-2.5	-23	58		-0.50	-0.50
Σ	148.768					930	20921	29.71	19.80	

Table 13: 6^{th} story force distribution, 12% eccentricity

Wall Number	Stiffness of Wall, R_x (k/in)	Distance from CL, y (ft)	Distance from CL, x (ft)	Distance from CR, y (ft)	Distance from CR, x (ft)	yR_x -or- xR_y	y^2R_x -or- x^2R_y	Shear due to direct Shear, F_V	Shear due to Torsion, F_T	F_{total} (kips)
1	9.298	40		57				3.71		3.71
2	9.298	40		57				3.71		3.71
3	9.298	30		47				3.71		3.71
4	9.298	30		47				3.71		3.71
5	9.298	-6		11				3.71		3.71
6	9.298	-6		11				3.71		3.71
7	9.298	-16		1				3.71		3.71
8	9.298	-16		1				3.71		3.71
9	9.298		20		28.5	265	7552		5.69	5.69
10	9.298		20		28.5	265	7552		5.69	5.69
11	9.298		10		18.5	172	3182		3.69	3.69
12	9.298		10		18.5	172	3182		3.69	3.69
13	9.298		2		10.5	98	1025		2.10	2.10
14	9.298		2		10.5	98	1025		2.10	2.10
15	9.298		-8		0.5	5	2		0.10	0.10
16	9.298		-8		0.5	5	2		0.10	0.10
Σ	148.768					1079	23524	29.71	23.15	

Table 14: 6^{th} story force distribution, 14% eccentricity

Wall Number	Stiffness of Wall, R_x (k/in)	Distance from CL, y (ft)	Distance from CL, x (ft)	Distance from CR, y (ft)	Distance from CR, x (ft)	yR_x -or- xR_y	y^2R_x -or- x^2R_y	Shear due to direct Shear, F_V	Shear due to Torsion, F_T	F_{total} (kips)
1	9.298	40		59				3.71		3.71
2	9.298	40		59				3.71		3.71
3	9.298	30		49				3.71		3.71
4	9.298	30		49				3.71		3.71
5	9.298	-2		17				3.71		3.71
6	9.298	-2		17				3.71		3.71
7	9.298	-12		7				3.71		3.71
8	9.298	-12		7				3.71		3.71
9	9.298		20		29.5	274	8092		5.76	5.76
10	9.298		20		29.5	274	8092		5.76	5.76
11	9.298		10		19.5	181	3536		3.81	3.81
12	9.298		10		19.5	181	3536		3.81	3.81
13	9.298		4		13.5	126	1695		2.64	2.64
14	9.298		4		13.5	126	1695		2.64	2.64
15	9.298		-6		3.5	33	114		0.68	0.68
16	9.298		-6		3.5	33	114		0.68	0.68
Σ	148.768					1227	26871	29.71	25.78	

Table 15: 6th story force distribution, 16% eccentricity

Wall Number	Stiffness of Wall, R_x (k/in)	Distance from CL, y (ft)	Distance from CL, x (ft)	Distance from CR, y (ft)	Distance from CR, x (ft)	yR_x -or- xR_y	y^2R_x -or- x^2R_y	Shear due to direct Shear, F_V	Shear due to Torsion, F_T	F_{total} (kips)
1	9.298	40		61				3.71		3.71
2	9.298	40		61				3.71		3.71
3	9.298	30		51				3.71		3.71
4	9.298	30		51				3.71		3.71
5	9.298	2		23				3.71		3.71
6	9.298	2		23				3.71		3.71
7	9.298	-8		13				3.71		3.71
8	9.298	-8		13				3.71		3.71
9	9.298		20		30.5	284	8649		5.71	5.71
10	9.298		20		30.5	284	8649		5.71	5.71
11	9.298		10		20.5	191	3907		3.84	3.84
12	9.298		10		20.5	191	3907		3.84	3.84
13	9.298		6		16.5	153	2531		3.09	3.09
14	9.298		6		16.5	153	2531		3.09	3.09
15	9.298		-4		6.5	60	393		1.22	1.22
16	9.298		-4		6.5	60	393		1.22	1.22
Σ	148.768					1376	30962	29.71	27.73	