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Seismic retrofit of flexible steel frames using thin infill panels

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Abstract

Non-linear inelastic analyses are conducted to investigate how structural behavior is affected when thin infills of steel, low yield steel, or Shearfill fabric are used to seismically retrofit steel frames located in regions of low and high seismicity, namely New York City and Memphis. A typical three-bay frame extracted from an actual 20-story hospital building in New York City is considered for this purpose. Fully rigid and perfectly flexible frame connection rigidities are considered to capture the extremes of frame behavior. It is found that the use of even very thin steel infill panels can significantly reduce story drifts without significant increases in floor accelerations, and that low yield steel behaves slightly better than standard constructional grade steel under extreme seismic conditions but at the cost of some extra material. It is also concluded that Shearfill membranes may not have the necessary strength and stiffness to be an effective retrofit solution, unless a thick membrane having multiple layers can be constructed.

1. Introduction

Numerous buildings in eastern North America have been built with steel frames having semi-rigid or flexible connections. Their resistance to lateral loads was sometimes assumed, by the original designers, to be provided by heavy cladding. This assumption evidently worked to some degree, as satisfactory resistance to wind loads was provided. However, it is not known whether such systems can provide the necessary strength and ductility to resist the rare but significant earthquakes likely to occur during the life of the structure.

This is particularly significant considering that some buildings are critical facilities (such as hospitals) that must remain operational following earthquakes. As such, the concern is not only structural damage but also the control of seismic drifts and floor accelerations. One important aspect, especially for buildings of moderate height (greater than 15 stories), is the development of P-Δ effects that may lead to large residual displacements following earthquakes.

One retrofit approach that has been commonly used by engineers to mitigate the problem of excessive drifts in steel frames during earthquakes is to add braces or shear walls to the existing structure. More recently, structural steel plate shear walls have been considered for this purpose as these have been demonstrated to have good energy dissipation capability. From a seismic retrofit perspective, steel shear walls may be advantageous over reinforced concrete or masonry walls because of their simplified detailing when the existing structure is a steel frame. Use of infill panels may also be compatible with the architectural configuration of floors in hospitals. However, for flexible buildings which may be sensitive to P-Δ effects but exposed to lower magnitude earthquakes, the additional strength provided by steel shear walls may not be necessary and other materials may be adequate to mitigate large deflections.

The research reported here investigates analytically how behavior changes when infills of a few different materials (steel, low yield steel, shearfill fabric) are used to seismically retrofit steel frames located in regions of low and high seismicity, namely New York City and Memphis for the purpose of this study. A typical three-bay frame extracted from an actual 20-story hospital building in New York City is considered for this purpose. The results obtained are compared to determine the relative benefits, if any, of using the various materials for retrofit. Fully rigid and perfectly flexible frame con-

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nection rigidities are considered to capture the extremes of frame behavior. Frames with semi-rigid connections would have a behavior between these limits as a function of the specific rigidity of the connections.

2. Literature review

Welding infill steel plates to the beams and columns of a frame in a building (Fig. 1) has the potential (if the infill plate is sufficiently thick) to modify structural behavior, converting a moment frame system into a steel plate shear wall (SPSW). The behavior of the shear wall formed in this way is analogous to that of a vertical plate girder, with the building columns corresponding to the flanges of the girder and the beams acting as its horizontal stiffeners. Because the web plates of the shear walls are relatively thin, the lateral shears are carried by the tension field action that develops in the web plates parallel to the directions of the principal tensile stresses.

A number of case studies in Japan and the United States have been conducted on steel plate shear walls that were designed with stiffeners to delay buckling under extreme lateral loading (e.g. [1,2]). In recent years, however, reliance on the post-buckling strength of the infill steel plate shear panels has been demonstrated to be an attractive and more economical alternative to resist lateral wind and seismic loads. The results of various static and quasi-static cyclic tests performed on models of varying scales since 1983 [3–9] and the analytical studies on the ultimate behavior of such steel shear walls without stiffeners [10] have demonstrated the stable energy absorption capacity of the steel panels and their adequacy as a primary lateral load-resisting system for buildings. As a result of these studies, the Canadian Limit State Design Standard CAN/CSA-S16 has included this concept for consideration by designers [11].

To capture the ultimate behavior due to the development of diagonal tension, such a shear panel is typically modeled as a series of inclined strip members capable of transmitting tension forces only, and oriented in the same direction as the principal tensile stresses in the panel. Each strip is assigned an area equal to the product of the strip width and the plate thickness (Fig. 2). The following equation for the angle of inclination of the strips has been determined based on the principle of least work and verified using experimental results [3,4]:

$$\tan^4\alpha=\frac{1+Lw\left(\frac{1}{2A_c}+\frac{L^3}{120I_bh}\right)}{1+hw\left(\frac{1}{2A_b}+\frac{h^3}{320I_cL}\right)} \tag{1}$$

where α is the angle between columns and tension strips, w is the thickness of the infill, L is the width of the panel, h is the height of the panel, A_c and I_c are the cross-sectional area and moment of inertia of the column, respectively, and A_b and I_b are the cross-sectional area and moment of inertia of the beam, respectively. Case studies show that α is usually close to 45° and that reasonably accurate results can be obtained using this value for expediency.

Note that beam and column stiffness can have an important impact on the behavior of steel shear walls. For example, two extreme cases of infinitely rigid boundary members and very flexible boundary members were considered for a wide shear wall panel. As shown in Fig. 3 the beam-to-column connections in both cases are pinned, and 12 strips are used, each having an area, A, of 5.762 in^2, a Young's modulus, E, of 30 000 ksi, and a yield strength, σ_y, of 50 ksi.

For the first case (infinitely rigid beams and columns), all braces yielded simultaneously. In the second case, arbitrarily using column and beam moments of inertia of 83.3 and 24.1 mm^4 respectively, only truss members 10, 11 and 12 yielded in tension, while truss members 1 to 8 buckled in compression. Truss member 9 was in tension but did not yield. Fig. 4 shows the deflected shape of the model illustrating how the beam and columns deflect considerably as they are pulled by the truss members in tension. The first eight truss members are therefore in compression as a result of beam and column deflections induced by the other strips in tension, and the entire tension field is taken by the last four truss members. This behavior is even worse when the bottom beam is also free to bend. This deflected shape is basically attributed to the chosen L/h ratio and the low stiffness of the boundary elements. Effective tension field distribution cannot develop effectively unless a minimum beam and column stiffness is provided. Past research [12] indicates that satisfactory behavior requires that:

$$\omega_h=0.7h\left(\frac{w}{2LI_c}\right)^{0.25} \tag{2}$$

where ω_h is a flexibility factor, I_c is the column inertia,

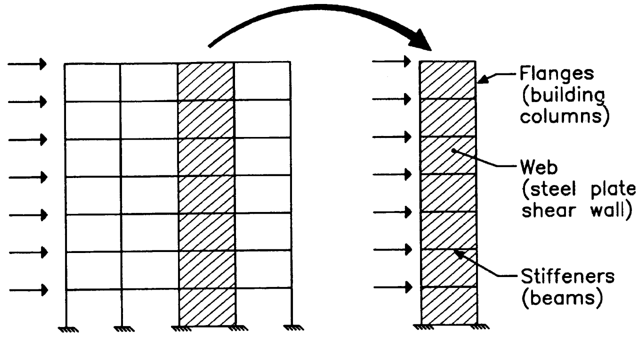

Fig. 1. Steel plate shear wall and plate girder analogy [11].

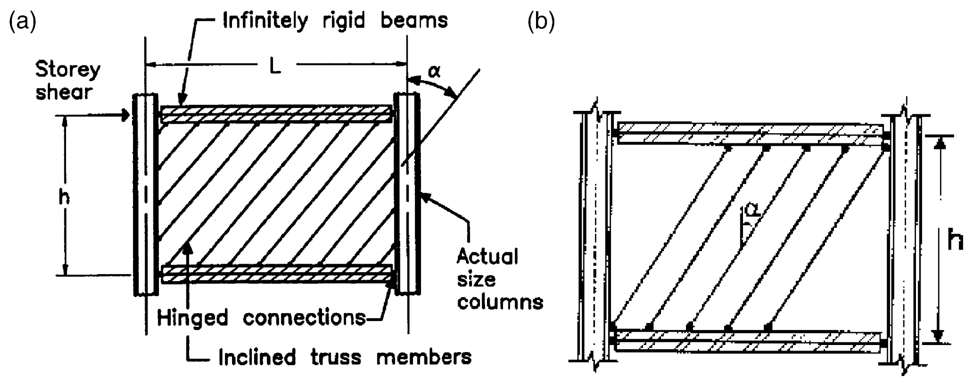

Fig. 2. Strip model representation developed by Thorburn et al. [3] for: (a) complete tension field; (b) partial tension field.

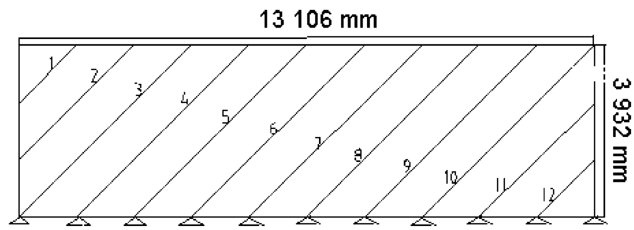

Fig. 3. Twelve-strip model, beams pinned to columns, undeformed configuration.

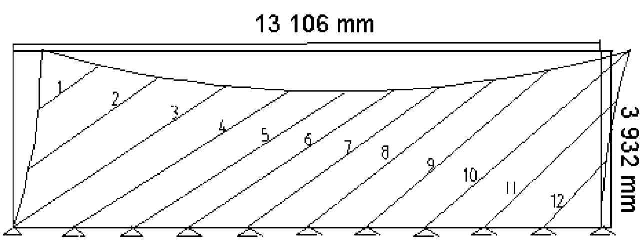

Fig. 4. Twelve-strip model, beams pinned to columns, deformed configuration.

and h, L and w are the panel height, width, and thickness, respectively.

Note that in the example of Fig. 4, ω_h equals 3.66, which violates the necessary condition expressed by Eq. (2). Furthermore, contrary to what has been assumed, the provided beam would not remain elastic. This nonetheless illustrates that a well-proportioned beam and low L/h panel aspect ratio are essential to ensure development of the desired energy dissipation mechanism (i.e. yielding of the plate) and ensure effectiveness of the system as a lateral load-resisting device.

3. Infill materials

While steel plate shear walls have been the subject of much research to date, the possibility of using other materials for infills is worthy of consideration. In this study, two alternative prospective materials were considered, in addition to the standard A572 Grade 50 steel used here as a reference point for comparison.

3.1. Low yield point steel (LYP)

Given that steel plates are the energy dissipation mechanisms in steel plate shear walls, it may be effective to use a special low yield point (LYP) steel recently developed in Japan to enhance this energy dissipation (as compared to normal steel plates used in previous research on steel plate walls).

As reported by Yamaguchi et al. [13], two grades are currently available in Japan, LYP100 and LYP235, with yield strengths of 100 and 235 MPa, respectively. Other material properties favorable for seismic control are: enhanced hysteresis characteristics, lower strain rate dependency, longer low-cycle fatigue life, and improved weldability. These LYP steels exhibit a fairly flat and long yield plateau with elongation of 72% for LYP100 and 60% for LYP235 before failure. Yamaguchi et al. [13] also demonstrated the adequacy of these steels in special applications such as unbounded braces and special ductile shear links in seismic control applications.

3.2. Shearfill

Shearfill is a brand name fabric manufactured by Chemfab. In this anisotropic material, the fibers in one direction are called fill fibers, and the orthogonal fibers threaded between the fill fibers and are called wrap fibers. Simple tension coupon tests were conducted at the University at Buffalo considering four different orientations of fabric to determine properties as a function of directionality, namely axial tension along the fill and wrap fibers, tension at 45° to the fill fibers, and tension at 30° to the fill fibers. Samples in each of the four orientations were tested at fast, medium and slow strain rate. Note that this material is available in five different thicknesses and strengths, but only the thickest and the thinnest fabrics (Shearfill 1 and 5, respectively) were tested. The fabric failures were sudden, with negligible ductility

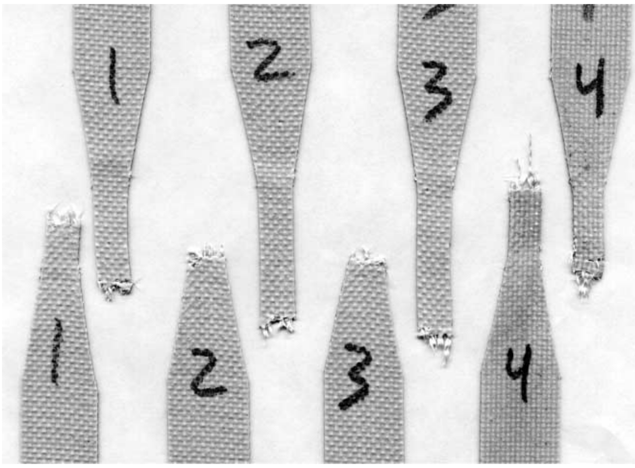

Fig. 5. Typical failed coupons of ShearFill #1: warp fiber at (1) fast, (2) moderate, and (3) slow loading rate; (4) fill fiber at fast loading rate.

in most cases; typical ruptured coupons are shown in Fig. 5, which illustrates the material texture and the absence of elongation and ruptured fibers at failure. The tension test results along the warp and fill fibers for Shearfill 1 were used in the following analyses with Young's modulus, E, of 131 ksi, and ultimate strength, σ_u, of 22 ksi.

4. Modeling of a multistory building

4.1. Description of the building structure

As a case study of an existing hospital building, nonlinear inelastic analyses were conducted to investigate various strategies for seismic retrofit using thin infill panels of steel or shearfill fabric. This hospital building, which shall remain nameless due to confidentiality agreements, was previously analyzed using SAP-2000 (linear analysis) by structural engineering consultants. A 3-D view of the building's entire steel frame system along with a side view of the fundamental vibration mode shape (displacing in the N–S direction) is shown in Fig. 6. The building consists of three sections: a plaza, a middle tower (core) and a main tower. The plaza section is seven stories high. The two towers are 20 stories high. Most floors in the building are made of two-way concrete slabs, but some floors have 6-inch-thick concrete on corrugated steel decks. The connections throughout the building are riveted. The columns are fire protected by cast-in-place concrete or masonry block encasement.

The girder beam connections are semi-rigid in the E–W direction and fully rigid in the N–S directions.

4.2. Dynamic nonlinear analysis of the model

For the purposes of nonlinear analysis, a single frame was extracted from the taller section of this building. This frame was chosen because elastic analyses indicated that it was engaged in first-mode response in the N–S direction. The selected frame is three bays wide. The middle bay is narrower as it delineates the corridors. The outer bays are wider, and because these would already have non-structural divider walls, the infill walls' retrofit devices were introduced there. Analyses indicated the beams (framing in N–S directions) would behave as rigid connections for both strength and stiffness. However, because all beam-to-column connections in the E–W direction for the chosen building were semi-rigid, to assess the significance of connection types on observed behavior, both rigid and semi-rigid beam-to-column connections were considered in the following analyses. Columns and beams were modeled using the Plastic Hinge Beam-Column Element (Type 02) in Drain-2DX with strain hardening ratio of 0.001 to approximate elastic-perfect plastic behavior [14]. The strips used to represent the infill walls were modeled using the inelastic Truss Bar Element (type 01) of DRAIN-2DX. Here, the desired element behavior is achieved by configuring the truss bar model to yield in tension but elastically buckle in compression. A strain-hardening ratio of 0.002 was chosen to approximate elastic-perfect plastic behavior in tension. The damping parameters were calculated in accordance with Rayleigh Damping using the first and the fifth periods in each

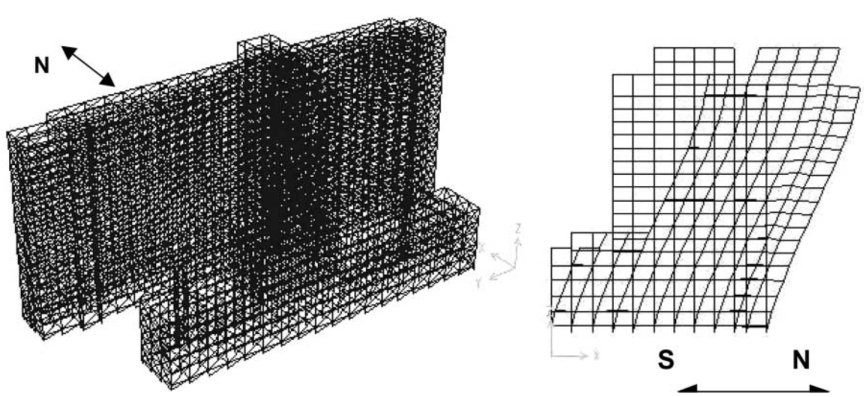

Fig. 6. 3-D Structural model of hospital building: (a) global view; (b) first mode shape.

analysis to set the damping factor at 5%, and P-Δ effects were considered in the analysis.

For the inelastic analyses, some simplifications to the model were made. The floor heights and panel widths were modified slightly to allow for the diagonal strips used to model the infills to be oriented at 45° uniformly along the entire height of the structure. This was also done to simplify the modeling and to avoid artificial bending in beams due to differential pulling of strips that would connect on non-identical nodes. Tension-only strips were also introduced in both directions to properly model behavior as the structure cycles in both directions during dynamic nonlinear analysis. In other words, the chosen strip model was duplicated in a mirror image so that the tension could be resisted in both sway directions. The complete model elevation is shown in Fig. 7.

Synthetic seismic ground excitation time histories were generated for New York City and Memphis using a program based on the specific barrier model [15]. The New York seismic exposure was generated based on the assumption of a moment magnitude 6.3 earthquake at an epicentral distance of 36 km. Memphis time histories were driven by the nearby New Madrid fault area, with a moment magnitude of 8.0 and epicentral distance of 31.7 km. The response spectra of these synthetic earthquakes are shown in Fig. 8.

4.3. Design of infill retrofit

The equivalent strip braces are designed to resist the applied seismic forces. Although many different codes provide slightly different seismic design procedures and alternative expressions for base shear calculations, most of these equations have been cross-calibrated to provide sensibly the same design forces. For expediency, the procedure used in the 1985 edition of the Uniform Building Code was followed to obtain the base shear forces to design the infills for seismic retrofit. In this process, an occupancy importance factor of 1.5 was used, given that a hospital building (essential facility) is considered here. A horizontal force factor, K, of 0.8 was selected as it was judged to be the value practicing engineers would have likely used when using that edition of the Uniform Building Code for buildings having steel shear walls or braced frames. A soil profile coefficient (S) of 1.5 was considered, corresponding to sandy or soft-to-medium clay conditions at the site. The story shears were first assumed to be resisted entirely by an equivalent truss member at each story. The resulting area of the equivalent brace at each story was then divided by a predetermined number of strips to get the area of each strip. Because story height varied slightly from base to top, the ground and top floor infill areas were divided into 11 and 13 strips, respectively, in each sway direction, the first to fourth floors into 15 strips, and the fifth to eighteenth floors into 14 strips in each sway direction. The resulting strip areas and corresponding thicknesses of infill panels for different materials are presented in Table 1 for New York City and Memphis. As an exception to this procedure, since the largest available Shearfill material thickness is 1 mm, uniform thickness of this material has been provided along all the stories.

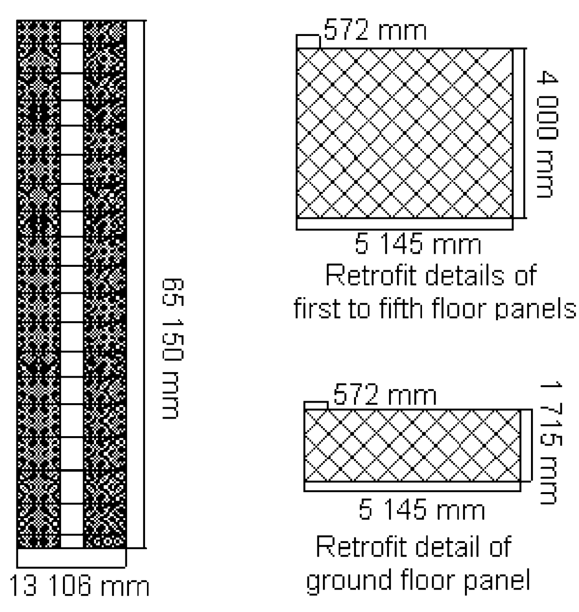

Fig. 7. Elevation of infilled frame model.

It should be noted that although the tabulated values were used in the analysis, the resulting thicknesses for the steel and LYS100 infill plates are impractical. These small values were obtained because the shear wall infills were assumed to be introduced into every frame of this structure. Usually, for steel infills, it would be more appropriate to retrofit only a few frames of the entire building. In case of Shearfill infills, however, retrofitting would most likely be required in every frame because of the weaker material properties and thin sections commercially available. Thus, to ensure a uniform comparison basis for all the different systems considered here, every frame was assumed retrofitted. However, to also provide a different perspective, a sample case was considered where only one frame was retrofitted with steel infills for every 10 frames. This case was subjected to the Memphis earthquake only. The masses and the area of infills used in the earlier analysis were thus multiplied by 10 to simulate the case mentioned above.

5. Analyses, results and observations

5.1. Analyses

Considering the existing as well as the retrofitted frames, a total of 43 time history analyses were conducted. Each retrofitted structure model consisted of 2299

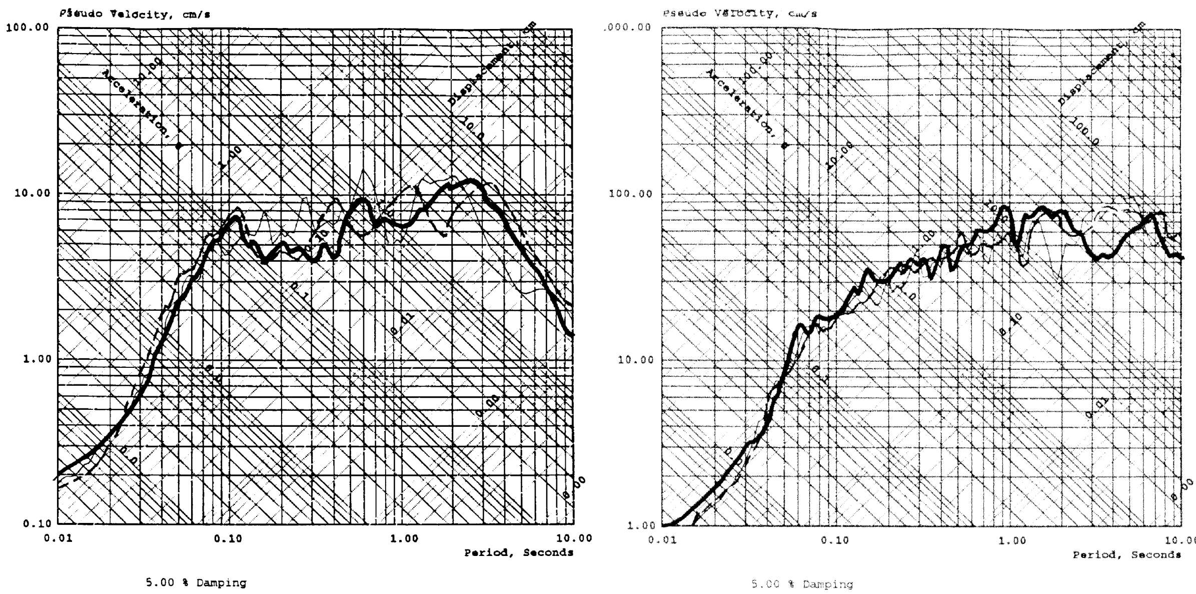

Fig. 8. Tripartite response spectra of earthquakes considered: Memphis (*left*) and New York (*right*).

Table 1
Strip areas [mm^2 (in^2)] and panel thicknesses [mm (in)] for New York City and Memphis

	New York City			Memphis		
Floor	Steel	LYS 100	Shearfill	Steel	LYS 100	Shearfill
(a) Strip areas						
Ground	96.8 (0.15)	333.7 (0.5172)	416.2 (0.645)	387.1 (0.60)	1334.0 (2.068)	416.2 (0.645)
1–4	85.8 (0.133)	295.9 (0.4586)	227.7 (0.353)	343.2 (0.532)	1183.0 (1.8344)	227.7 (0.353)
5–8	81.3 (0.126)	280.3 (0.4345)	243.2 (0.377)	325.2 (0.504)	1121.0 (1.738)	243.2 (0.377)
9–12	69.0 (0.107)	238.1 (0.369)	243.2 (0.377)	276.1 (0.428)	952.2 (1.476)	243.2 (0.377)
13–17	51.0 (0.079)	175.7 (0.2724)	243.2 (0.377)	203.9 (0.316)	703.0 (1.0896)	243.2 (0.377)
18	19.4 (0.030)	66.7 (0.1034)	271.0 (0.420)	77.4 (0.120)	266.8 (0.4136)	271.0 (0.420)
(b) Panel thicknesses						
Ground	0.237 (0.00934)	0.817 (0.0322)	1.016 (0.04)	0.949 (0.0374)	3.27 (0.1289)	1.016 (0.04)
1–4	0.38 (0.015)	1.32 (0.052)	1.016 (0.04)	1.52 (0.06)	5.28 (0.208)	1.016 (0.04)
5–8	0.34 (0.0134)	1.17 (0.0462)	1.016 (0.04)	1.36 (0.0536)	4.70 (0.185)	1.016 (0.04)
9–12	0.28 (0.011)	0.965 (0.038)	1.016 (0.04)	1.12 (0.044)	3.86 (0.152)	1.016 (0.04)
13–17	0.203 (0.008)	0.701 (0.0276)	1.016 (0.04)	0.813 (0.032)	2.79 (0.11)	1.016 (0.04)
18	0.076 (0.003)	0.254 (0.01)	1.016 (0.04)	0.305 (0.012)	1.016 (0.04)	1.016 (0.04)

degrees-of-freedom, 833 beam and column elements, and 1064 truss elements. The analyses were typically run on computers having a Pentium II 400 MHz processor with 64 MB RAM and each run required an average of four and seven hours for the New York and Memphis earthquake cases, respectively. Table 2 shows a summary of all the cases analysed and Fig. 9 shows a schematic of the relationships that can be established for comparisons between the cases considered.

The basis for comparison was the non-retrofitted existing frame (labeled as bare frame in Table 2). Two chosen site locations were considered for this building, namely New York City and Memphis, the former because it is where the actual hospital building is located, and the latter because existing buildings in Memphis show the same structural characteristics as those elsewhere in the eastern United States but are located near the New Madrid seismic zone, one of the most seismically active regions in the United States. As shown in Fig. 9 and indicated previously, cases of frames with rigid and pinned connections have been considered, as they constitute extreme cases of behavior. For each case considered, three synthetic time histories were used. For each analysis, maximum story drift, story ductility and story acceleration were recorded and used to compare behaviors between respective systems. The average results from the three time histories are used here for comparison.

Table 2
Summary of analyses conducted (three synthetic time histories per case)

Model type	New York City		Memphis	
	Rigid connections	Pinned connections	Rigid connections	Pinned connections
Steel infills	X	X	X	X
LYS100 infills	X	X	X	X
Shearfill infills	X	X	X	X
Bare frame	X		X	
Steel infills every 10 frames (sample case)			X	

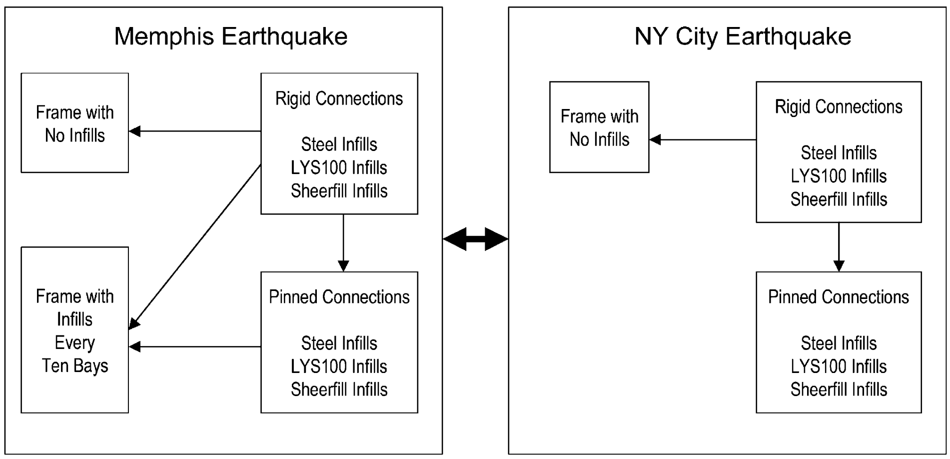

Fig. 9. Considered cases and comparison strategies.

5.2. Observations

No yielding was observed in any of the beams or columns in most of the cases. This is understandable because the approach adopted, namely to add infills in every frame, is a theoretical approach that provides comparison between infills of various materials. Yielding was observed in the story beams only for the Shearfill and bare frame cases subjected to the Memphis earthquake.

The fundamental periods of vibration of various frames considered are presented in Table 3. Comparing the Shearfill rigid case with the bare frame case, it is seen that the addition of Shearfill does not have a significant impact on the period of the structure, thereby suggesting that it provides no effective contribution to the rigidity of the structure. Steel plate infills, however, considerably reduce the period, to less than half that of the bare frame in most cases. Furthermore, since LYS100 infills require more area of material to provide the same resistance compared to standard Grade 50 steel, frames retrofitted with LYS100 infills are stiffer structures and have a lower period.

Maximum displacement, ductility, and acceleration at

Table 3
Fundamental vibration periods (seconds)

Model types	New York City		Memphis	
	Rigid connections	Pinned connections	Rigid connections	Pinned connections
Steel infills	1.9	2.53	1.488	1.791
LYS100 infills	1.52	1.84	1.32	1.539
Shearfill infills	3.08	16.36	3.08	16.36
Bare frame	3.143	∞	3.143	∞
Steel infill every 10 frames (sample case)			3.99	

each story are presented in Fig. 10a–f, respectively, for frames having rigid and pinned beam-to-column connections and located in Memphis, and in Fig. 11a–f for the same located in New York City. The structures with LYS100 infills exhibit the least story drifts, while those with steel infills show slightly larger drifts. This can be explained by the fact that LYS100 infills result in a stiffer structure compared to standard steel for the reasons mentioned above. The case with Shearfill infills and the case with bare frame show the largest story drifts (up to approximately 1% of story height for the Memphis earthquake). It should be noted that the results observed for Shearfill infills are for the weaker orientation of fibers (45°). The strongest orientation of the fiber (fill direction) gave maximum story drifts approximately 10% less than the presented results. This is consistent with the general trend demonstrated by the natural periods. Frames having pinned beam-to-column connections exhibited maximum story drifts 100% larger than for the frames with rigid connections.

An assessment of ductility demand on the infill at each story was made by recording the largest ductility demand observed on any strips at that story. For the rigid connection case, maximum ductility occurred on the second to fourth stories and gradually tapered off toward the upper stories. The large ductility observed there is due to the fact that the first four floors have the tallest story height and underwent greater drifts. The pinned case exhibited

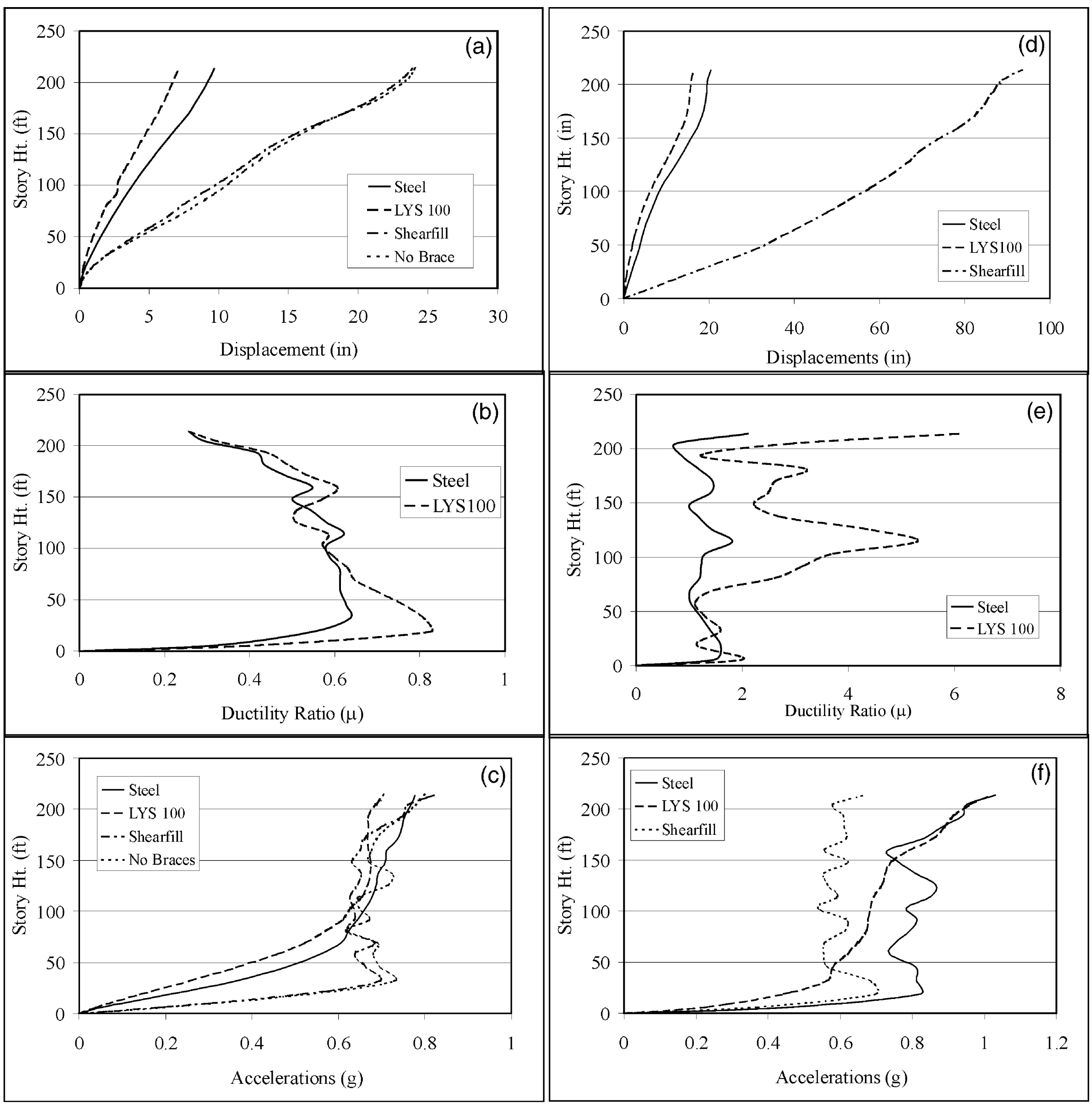

Fig. 10. Results for Memphis. Maximum story drifts, story ductility, and story accelerations for frames with rigid beam connections (a–c) and pinned beam connections (d–f), respectively. Heights of 50, 100, 150, 200 and 250 ft correspond to 15.24, 30.48, 45.72, 60.96 and 76.2 m, respectively.

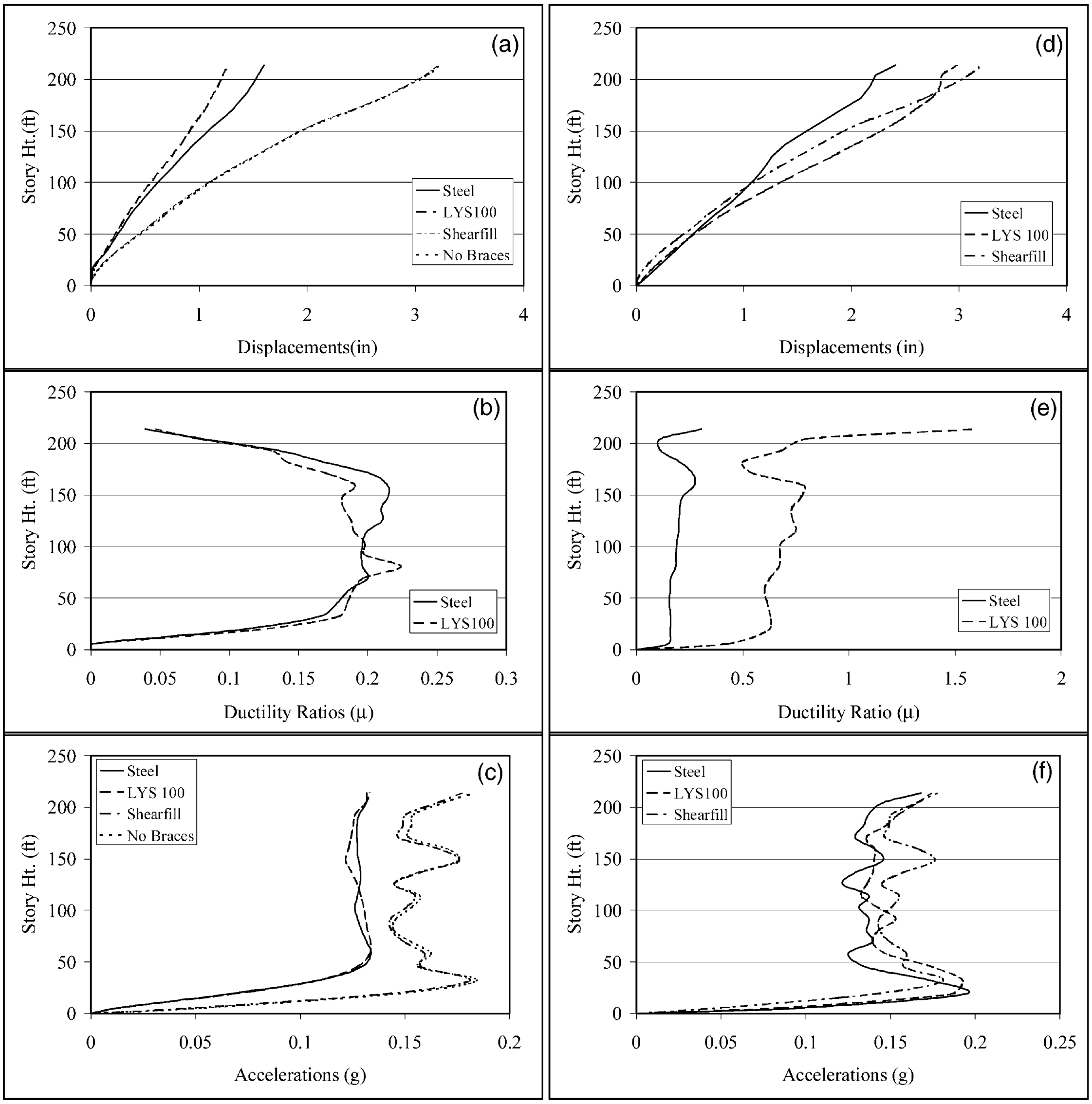

Fig. 11. Results for New York. Maximum story drifts, story ductility, and story accelerations for frames with rigid beam connections (a–c) and pinned beam connections (d–f), respectively. Heights of 50, 100, 150, 200 and 250 ft correspond to 15.24, 30.48, 45.72, 60.96 and 76.2 m, respectively.

high ductility at the eighth to tenth story levels reducing near the twelfth to sixteenth story levels and reaching a maximum at the eighteenth story. In all cases, the largest ductility was observed in the middle strip, which is one of the longer strips.

Note that frames with LYS100 infills reached a maximum ductility ratio of six which is about three times the ductility exhibited by frames having ordinary steel infills for almost the same story drifts. This is logical as it corresponds approximately to the ratio of yield stresses for these two steels. For the rigid connection cases, the difference in ductility between the two materials was not significant. In this latter case, drifts are much smaller and the infill has a lesser influence on behavior, as rigid frames contributed more significantly to the overall response. A typical range of story ductility for all strips of a panel, say for the Memphis pinned case for LYS100, at the tenth story level, is between 4 and 5.

Floor accelerations follow the general trend that they increase with increasing periods for the lower stories. At the higher floors, the acceleration values are more or less similar for all cases. It is observed that by adding infills the floor accelerations do not increase significantly and hence would not impact unfavorably the non-structural equipment, sensitive to accelerations. However, the reader should keep in mind that these are very flexible structures to begin with, and these long-period structures are usually near the constant displacement region of the response spectra used in this study. Hence, the above observation may not necessarily be true for shorter period low-rise buildings. The only exception to the above observation is the Shearfill pinned case, which exhibits a lower acceleration response. This anomaly can be attributed to the fact that the case under consideration has an unrealistic fundamental period of vibration of 16 seconds. Furthermore, the synthetic time histories generated here are not suitable for such long-period structures.

To provide some information on how the maximum values reported above are affected by permanent deformations, and how seismic response varies as a function of severity of seismic excitations, two sample time histories are presented in Fig. 12. These are for the case of frames with steel infills for Memphis and New York City earthquakes, respectively. The typical time history roof displacement results indicate that the structure essentially vibrates in a first mode (at a 1.5- to 2-second period). The longer duration and greater magnitude of response for buildings located in Memphis and subjected to an earthquake on the New Madrid fault is noticeable from this figure.

For the sample case in which a 10 times thicker steel infill plate was used to model the situation where infills are applied every 10 frames, the calculated fundamental period was 3.99 seconds. Intuitively, this is not correct because the addition of an infill should make the frame stiffer and so its period should be less than the case with bare frame (no infills). This discrepancy occurs because of the absence of the remaining nine frames in the model. As flexural behavior begins to dominate the shear behavior in such a retrofitted wall, increasing the thickness of infills alone does not effectively modify the flexural stiffness of the retrofitted frame, even though it may provide shear-type energy dissipation. This behavior can be explained using the analogy of an I-beam with columns as flanges and infills as webs, where increasing the web thickness by a factor of 10 would not give a behavior equivalent to 10 beams when behavior is dictated by shear deformations alone. The correct model for comparison purposes would therefore require linking all the 10 frames together to ensure compatibility of deformations at all floors. Then, logically, the period should reduce to a value less than that of the bare frame case. Unfortunately, modeling of all 10 frames with DRAIN-2DX is computationally prohibitive (it would involve a total of 3444 nonlinear structural elements) and was not accomplished here.

The case of a single frame having a 10 times thicker steel infill plate than previously considered (and 10 times more mass) was nonetheless analyzed as it allowed us to check how the yield behavior of the system was affected by the thickness of the infill plates alone. It was observed in this case that ductility demands were 67% less than the corresponding case with the steel infills and rigid connections. The accelerations were about 20% less.

6. Conclusions

Based on the limited study carried out, it can be concluded that:

- The use of steel or any other ductile material as infill panels can significantly reduce story drifts (by as much as 200% in some of the cases considered). Furthermore, this was achieved without any significant increase in floor accelerations. Low yield steel does behave slightly better than standard constructional grade steel under extreme seismic conditions but at the cost of some extra material.
- Analyses using Shearfill as infill indicated that, because of the low strength and stiffness of this material, the behavior of the frame was not affected favorably. Unless a thick membrane having multiple

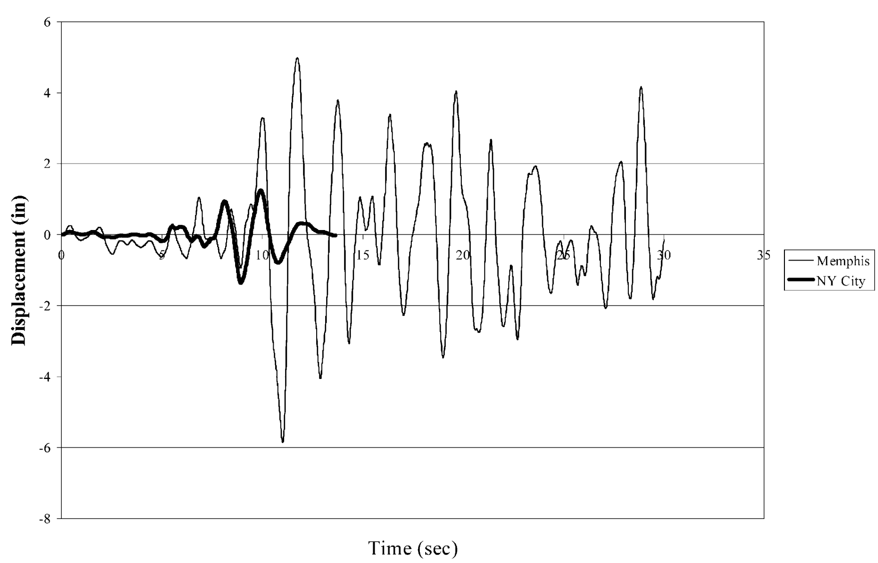

Fig. 12. Comparison of displacement time histories (top story) for Memphis and New York earthquakes. (Displacement of 1 equals 25.4 mm.)

layers can be constructed, the membrane may not have the necessary strength to be an effective retrofit solution. Furthermore, because of a low Young's modulus of elasticity, pre-tensioning or layering with other materials may be required to increase the stiffness of this infill.

- Limited parametric studies on frames having large aspect ratios ($L/h>3$) indicate that steel plate shear walls may not be as effective as their applications in frames with a smaller aspect ratio. This loss in effectiveness could be attributed to larger beam deformations that would develop with diagonal tension yielding of infill plates. This observation would apply mainly to beams having infill plates on one side only, such as at the top story of a building. However, similar behavior could be expected on the columns of steel plate shear walls having L/h significantly less than 1. This deserves further investigation.
- The limited study presented here has not addressed the whole range of available infill materials that could potentially be used as a retrofit strategy. It has nonetheless investigated the relative significance of materials with various levels of stiffness and yield strength. Furthermore, the study was limited to one very flexible structure having high initial period (above 3 seconds). Different conclusions may be obtained for low or mid-rise buildings and hence further research is recommended to provide a better understanding of the role of infills as a potential seismic retrofit strategy for existing structures.
- A sample case in which a 10 times thicker steel infill plate was used to model the situation where infills are applied every 10 frames did not provide conclusive results. This is because proper comparison would require that the 10 frames (retrofitted and non-retrofitted) be linked together to ensure compatibility of deformations. This proved to be computationally prohibitive but is recommended for future research.

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