INCREASING COLLAPSE RESISTANCE IN TALL STEEL MOMENT FRAME BUILDINGS

PEER Tall Building Initiative

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Absract

How would tall steel moment frame buildings collapse under seismic loading? The distribution of moments in a steel moment frame subjected to lateral loads is such that it produces double curvature in all the columns and beams resulting in shear-racking of the frame. Thus, shear deformation and not flexural deformation dominates moment-frame response. This allows the drawing of an analogy between steel moment frames excited by earthquake ground motion and a shear wave traveling through a shear beam. However, moment frame buildings differ from uniform shear beams in three important ways- first, the buildings are not uniform, there is typically stiffness gradation and mass variation over the height of the structure; second, gravity is not present in the shear beam wave propagation problem, whereas, it plays a dominant role in the response of moment frame buildings. Not only do building columns carry axial loads, but gravity also causes second-order overturning moments associated with the self-weight of the structure acting through its deformed configuration under lateral loading, the so-called P-Δ effect; third, steel-frame buildings do exhibit low levels of damping. Damping has the effect of attenuating the response (which impacts the response to multi-pulse excitation more than single-pulse excitation) and lengthening the apparent period. But the low level of damping inherent in steel structures means that it plays a relatively minor role in the damage localization phenomenon. Still, despite of these difference in nature we find that a 18-story moment frame building responds very similar to a linear shear beam when subjected to a moderate pulse of 1s. More specifically, strain doubling occurs due to constructive interference of the reverse phase of the incident wave with the forward phase that is reflected off the free end similar to the behavior of an ideal beam. Such strain doubling can lead to damage localization which can result in the formation of a shear-compliant block collapse mechanism, consisting of column yielding at floors corresponding to the top and bottom of the shear-compliant block, with significant yielding of the beams or columns or panel zones at each joint in each of the intermediate floors. For moderate loading (motions that are not strong enough to cause structural collapse), the **C**onfines **o**f the **M**ost **P**ronounced **L**ocalization of **Y**ielding (CoMPLY) is controlled by the period of the input ground motion relative to the fundamental period of the structures, with behavior that is very similar to that of an idealized shear beam. However, the response to "collapsogenic" input motions shows a "convergence" of the CoMPLY to the same few stories, rendering it invariant with respect to various measures of the ground motion. This implies that there is a characteristic mechanism of collapse for a given building regardless of the frequency-duration characteristics of the incident ground motion. This characteristic collapse mechanism happens to be a function of the structural system alone and can be predicted using its basic properties. The ability of being able to predict these characteristics encourages exploration of possibilities of local retrofitting measures. We give a simple example of such a measure which significantly improves the collapse resistance of a model of an existing moment frame building.

Moment frame building
(97UBC, T_1=4.06s, T_2=3.85s, T_3=2.60s)

Waveforms of the idealized pulse used as input ground motions
(T=1s, amplitude is small enough to keep the building elastic)

Schematic of a discretized shear beam

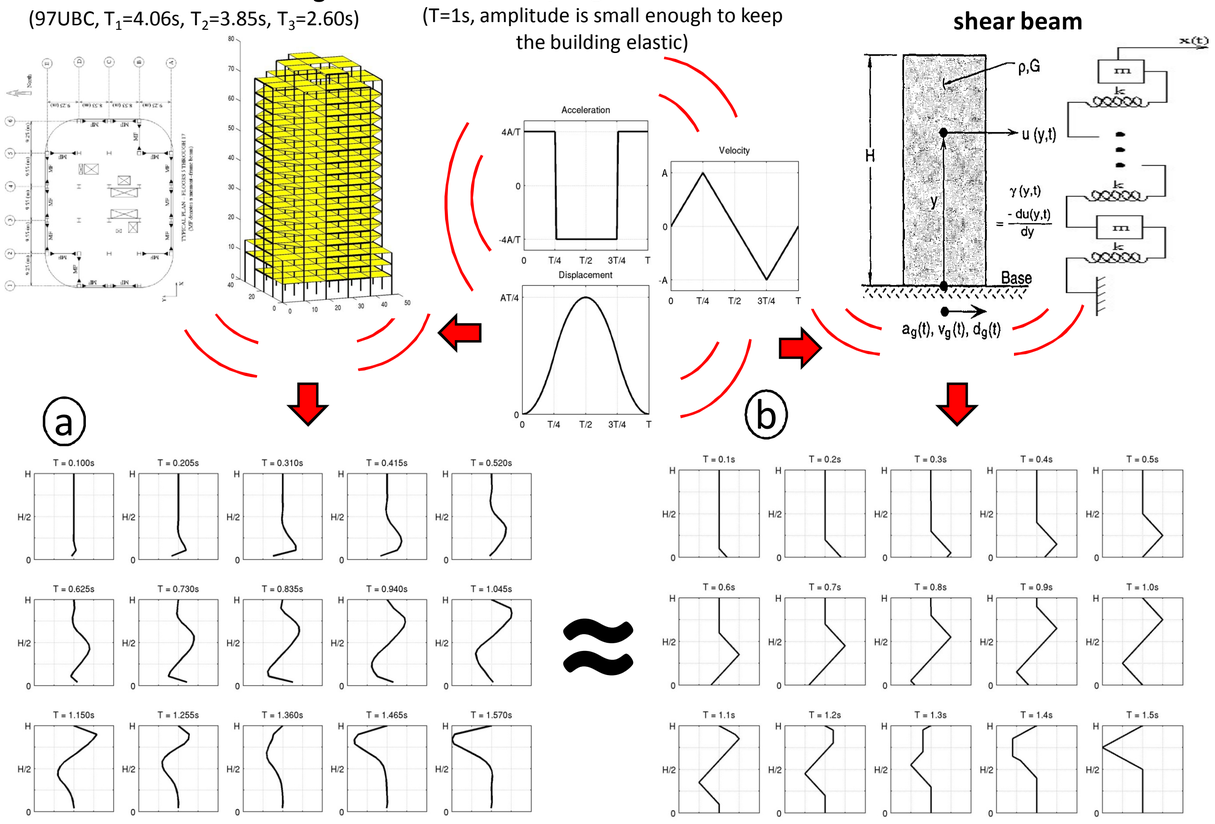
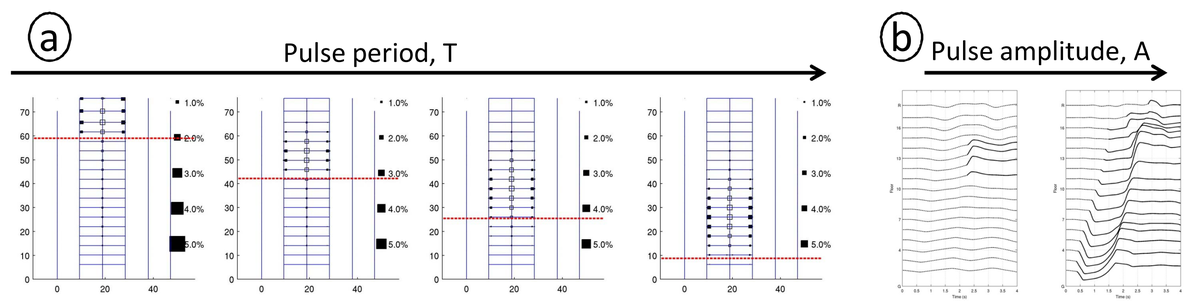


Figure 1: (a) When subjected to the idealized pulse the traveling wave is imaged through snapshots in time of the inter-story drift ratio (IDR) response of the moment frame building. The IDR is the difference in the displacements of the roof and floor of a given story normalized by its height and is approximately analogous to the shear strain observed in the uniform shear beam, shown in (b). While the IDR distribution in the former qualitatively agrees quite well with the strain distribution in the latter, non-uniformity in the building structure results in a slight, but consistent, mismatch in the location of strain-doubling.

Figure 2: (a) Plastic hinge rotations for Building 1 (perfect connections) resulting from a moderate single-pulse X-direction excitation with periods of 1s, 2s, 3s, and 4s, respectively. The dashed lines indicate the predicted location for the peak shearing force in a linear shear beam with the same fundamental period as the building model. (b) Strain in a lower-flange fiber for an elastofiber beam element at each floor of a MF building (perfect connections) subjected to an input wave with a period of 2.25s and an amplitude of 0.5 m.s^{-1}. Yielding occurs in the upper floors, after the reflection of the incident wave. However, by increasing the amplitude to 1.5 m.s^{-1}, keeping the same period, we see that yielding occurs as the incident wave travels up the building.



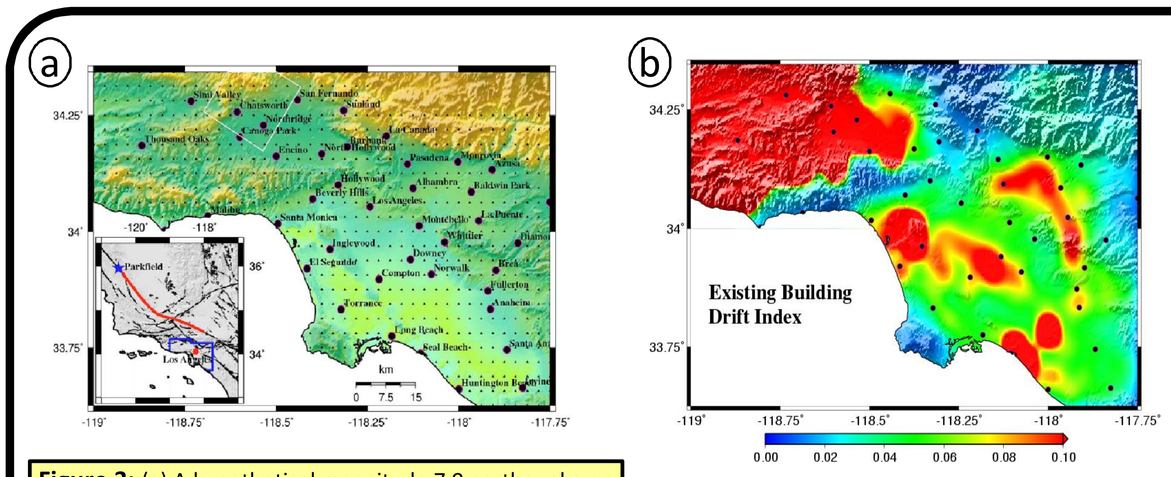
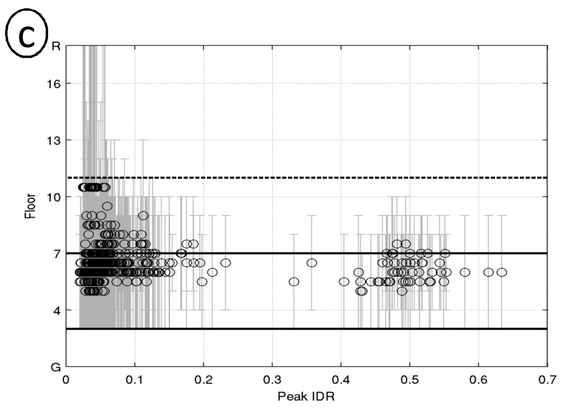


Figure 3: (a) A hypothetical magnitude 7.9 earthquake on the San Andreas fault. Initiating at Parkfield and rupturing in a southeasterly direction with a peak displacement of 2 m and peak velocity of 2 ms-1. The Los Angeles and the San Fernando basins, have been divided into a spatial grid of 636 sites (marked with black dots) where the ground motions are calculated. (b) Resulting IDRs when an existing building, described below, is subjected to the calculated ground motions at each site. The color bar refers to the peak IDR at each site. (c) The unique CoMPLY observed beyond IDR =0.05 that is reasonably well predicted by the simple plastic analysis (black dashed lines) and moment equilibrium (black solid lines) approaches.



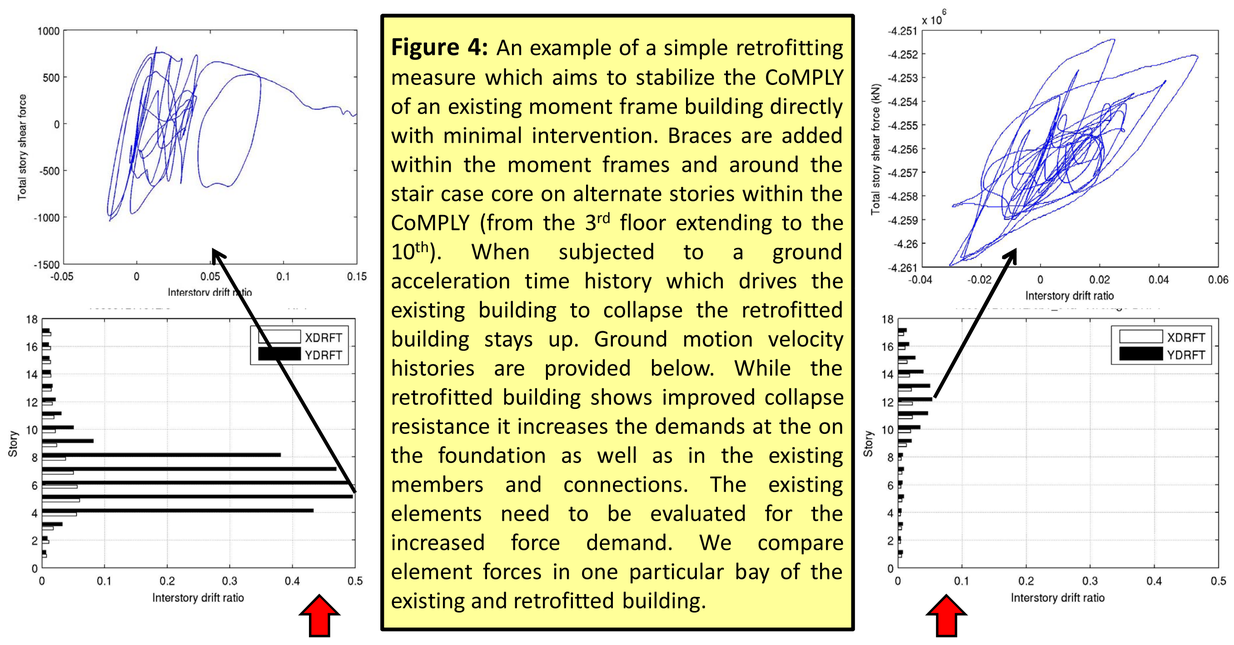
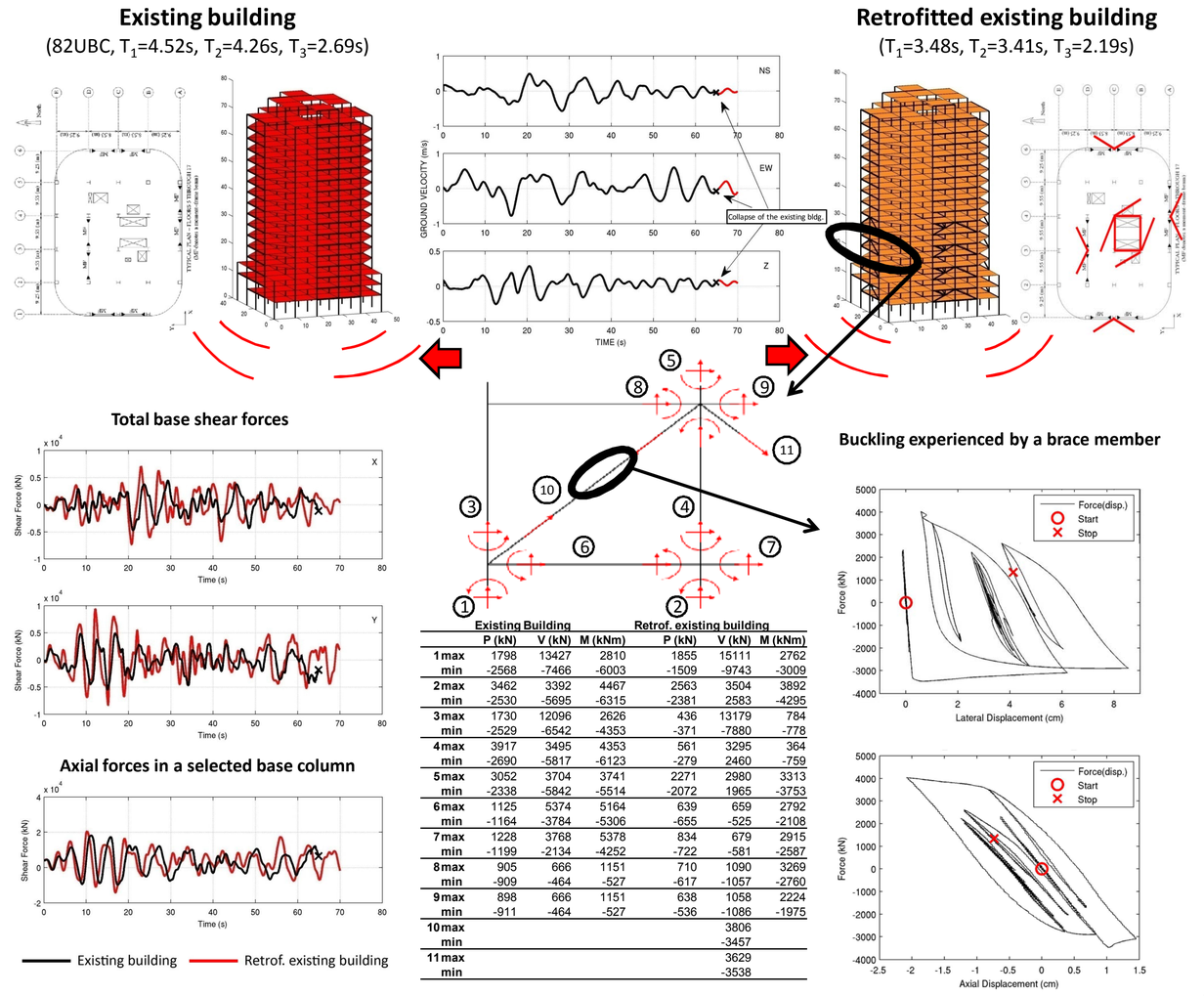


Figure 4: An example of a simple retrofitting measure which aims to stabilize the CoMPLY of an existing moment frame building directly with minimal intervention. Braces are added within the moment frames and around the stair case core on alternate stories within the CoMPLY (from the 3rd floor extending to the 10th). When subjected to a ground acceleration time history which drives the existing building to collapse the retrofitted building stays up. Ground motion velocity histories are provided below. While the retrofitted building shows improved collapse resistance it increases the demands at the on the foundation as well as in the existing members and connections. The existing elements need to be evaluated for the increased force demand. We compare element forces in one particular bay of the existing and retrofitted building.

Existing building
(82UBC, T_1=4.52s, T_2=4.26s, T_3=2.69s)

Retrofitted existing building
(T_1=3.48s, T_2=3.41s, T_3=2.19s)



	Existing Building			Retrof. existing building		
	P (kN)	V (kN)	M (kNm)	P (kN)	V (kN)	M (kNm)
1 max	1798	13427	2810	1855	15111	2762
min	-2568	-7466	-6003	-1509	-9743	-3009
2 max	3462	3392	4467	2563	3504	3892
min	-2530	-5695	-6315	-2381	2583	-4295
3 max	1730	12096	2626	436	13179	784
min	-2529	-6542	-4353	-371	-7880	-778
4 max	3917	3495	4353	561	3295	364
min	-2690	-5817	-6123	-279	2460	-759
5 max	3052	3704	3741	2271	2980	3313
min	-2338	-5842	-5514	-2072	1965	-3753
6 max	1125	5374	5164	639	659	2792
min	-1164	-3784	-5306	-655	-525	-2108
7 max	1228	3768	5378	834	679	2915
min	-1199	-2134	-4252	-722	-581	-2587
8 max	905	666	1151	710	1090	3269
min	-909	-464	-527	-617	-1057	-2760
9 max	898	666	1151	638	1058	2224
min	-911	-464	-527	-536	-1086	-1975
10 max					3806	
min					-3457	
11 max					3629	
min					-3538	

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