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PERFORMANCE OF ROCKING CORE WALLS IN TALL BUILDINGS UNDER SEVERE SEISMIC MOTIONS

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ABSTRACT

This paper presents detailed numerical simulation results of the behavior of rocking core walls in the earthquake resistance of tall buildings, particularly those potentially affected by near-fault pulse-type ground motions due to forward directivity or fling. Fully nonlinear LS-DYNA models of a 50-storey 200 meter tall reinforced concrete core-only building with both fixed- and rocking-base core wall conditions were subjected to a ground motion suite approximating the predicted 84th percentile motion of a M8.0 event 10 km away. For these large motions, the rocking base structure exhibited elastic re-centering behavior and a 30% reduction in base moment demand over a fixed base structure of similar wall cross section. For large near-fault motions, such as that recorded during the 1992 Landers earthquake, the rocking base structure continued to exhibit elastic re-centering behavior and large displacement capacity while a fixed-base structure with the same wall cross section collapsed. Further, the seismic design of the rocking base structure appears to be governed primarily by the gravity load, making it an ideal solution for tall buildings sited in regions where the variability of the seismic load is not well constrained.

Introduction

The potential benefits of uplifting structures under seismic ground motions were first recognized by Housner based on observations of elevated water tanks that survived the 1960 Chilean earthquake, primarily due to weak foundations that inadvertently allowed a rocking mechanism to form (Housner 1963). While much research has followed into the dynamic behavior of rocking systems, most studies have focused on overturning of short rigid structures (Makris and Konstantinidis 2003) or behavior of footings and foundations (Harden et al. 2005), though recent research has led to structural rocking systems suitable for buildings of short to moderate height

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(Priestly et al. 1999). The research presented here is an effort to characterize the global behavior of a tall building with rocking core walls allowed to uplift on their foundations and compare the performance of this novel system with that of a typical core-only tall building designed to form a plastic hinge at the base of the wall. Due to the significant restoring force provided by the self-weight of a tall building, the proposed rocking system does not require any post-tensioned tendons or other mechanisms to re-center the core following an event.

Limitations of Fixed-base Tall Buildings

This research proposes an alternative solution for withstanding severe ground motions through the use of rocking walls and is not intended to be a thorough analysis of the performance of existing tall buildings. However, the following observations regarding the fundamental deficiencies of fixed-base tall buildings under severe ground motions should be noted in light of the possible benefits of rocking structures. Current fixed-base tall buildings are designed with the same capacity design methodology as is prescribed by building codes for shorter buildings in that the core wall is designed to form a plastic hinge during severe shaking, which then limits the amount of force transmitted to the rest of the building. While the intent of a capacity design approach is consistent with current earthquake engineering understanding, the formation of a plastic hinge is a destructive process which, depending on the severity of shaking and the extent of hinging, can lead to significant damage and residual drift. These same hazards exist for shorter buildings but the risks are higher for tall buildings due to the considerably larger investments required in their construction and the difficulties involved in repairing them following a severe event. The economic losses associated with damage are straightforward, though not typically considered, and can be significant given the size of a tall building and the number of components, both structural and non-structural, that could potentially be damaged. It is also unclear how to repair severe damage at the base of the wall due to the formation of a plastic hinge given that it is typically also a part of the gravity supporting system of a tall building.

While some residual drift is taken to be acceptable in buildings of more moderate height, the ramifications of residual drift in tall buildings are poorly understood. Complications range from the inability to correct residual drift in tall buildings using presently available methods to municipal anxiety over any observable drift that might indicate structural instability, whether actual or perceived, and could result in a total loss of investment following a severe event even if the structure is otherwise sound. Large near-fault displacements can also lead to very large local ductility demands at the base of the wall, particularly for long period structures, and could pose a collapse risk. Further issues arise when considering ground motions with very long duration, characteristic of large magnitude events (i.e. M8.0 and greater), which can result in cyclic degradation of the plastic hinge and lead to failure under gravity load. Tall buildings have not been tested by an actual severe ground motion and thus these issues, and the practicalities involved in mitigating them, remain unresolved.

In essence, the proposed rocking mechanism limits the seismic loads induced into the superstructure by providing a localized base hinge without the detriments of material yielding and residual displacements associated with typical plastic hinge zones. Such a mechanism is ultimately a more sustainable practice as little post-event repair is required and the probability of a total loss of investment following a severe event is significantly less than typical fixed-base designs.

Case Study Building and Modeling

The case study building considered is a 200 meter tall 50 storey core-only building with typical plan shown in Fig. 1 and typical floor load of 6 kPa. For simplicity, the core is taken as a solid wall section rather than a series of coupled walls and the perimeter frame does not form part of the lateral resisting system. The case study building was modeled and analyzed using the explicit finite element code LS-DYNA. The wall elements were modeled as 9 layer composite shells with separate nonlinear hysteretic materials for unconfined concrete, confined concrete, shear reinforcement, and flexural reinforcement. The slab elements were modeled similarly with nonlinear hysteretic concrete and reinforcement materials.

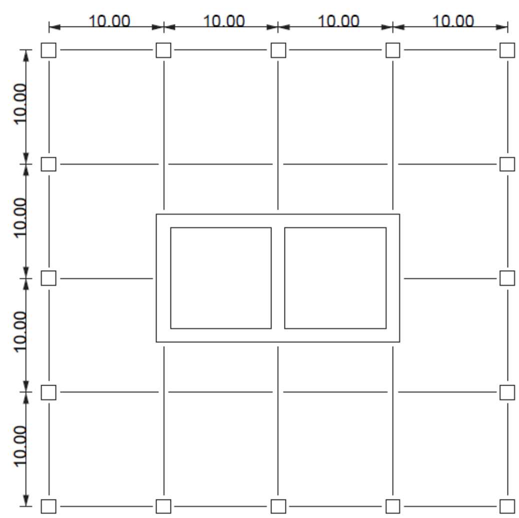

Figure 1. Case study core-only building plan and dimensions.

The building was sited 10 km from the San Andreas fault with the seismic hazard taken as the 84th percentile mean ground motion predicted by the NGA relations (Abrahamson et al. 2008) resulting from a M8.0 deterministic event, shown in Fig. 2. This hazard led to a code-based design of a 1 meter thick reinforced concrete wall with 1% flexural reinforcement and the wall section was kept the same over the building height both for simplicity and suppression of higher mode effects causing wall yielding midway up the building. 55 MPa concrete and 415 MPa reinforcing steel was used in the wall and slab sections. The first three natural periods of vibration of the fixed-base building are at 6.6, 5.0, and 2.4 seconds.

The rocking-base building used the same wall section as the fixed-base building for comparison purposes, with the addition of 1.25 meter square confined concrete sections at the corners of the core wall extending two storeys above the base. These sections were added to ensure elastic behavior as the core rocks up on a single corner during bi-directional ground motions. The rocking mechanism was modeled using surface-to-surface contact between the base wall surface and a 4 meter thick mat slab modeled with elastic solid elements. The behavior of the contact surface was validated against the theoretical rigid body rocking response derived by Housner (Housner 1963) for a 200 m by 10 m wall section subjected to a single direction ground motion component of the 1995 Kobe JMA record, as shown in Fig. 3. While some period lengthening exists for the LS-DYNA simulation in the free vibration phase, the peak response and behavior during the forced vibration phase are well matched to the theoretical response. Potential rocking due to foundation uplift and soil yielding is not modeled in either building.

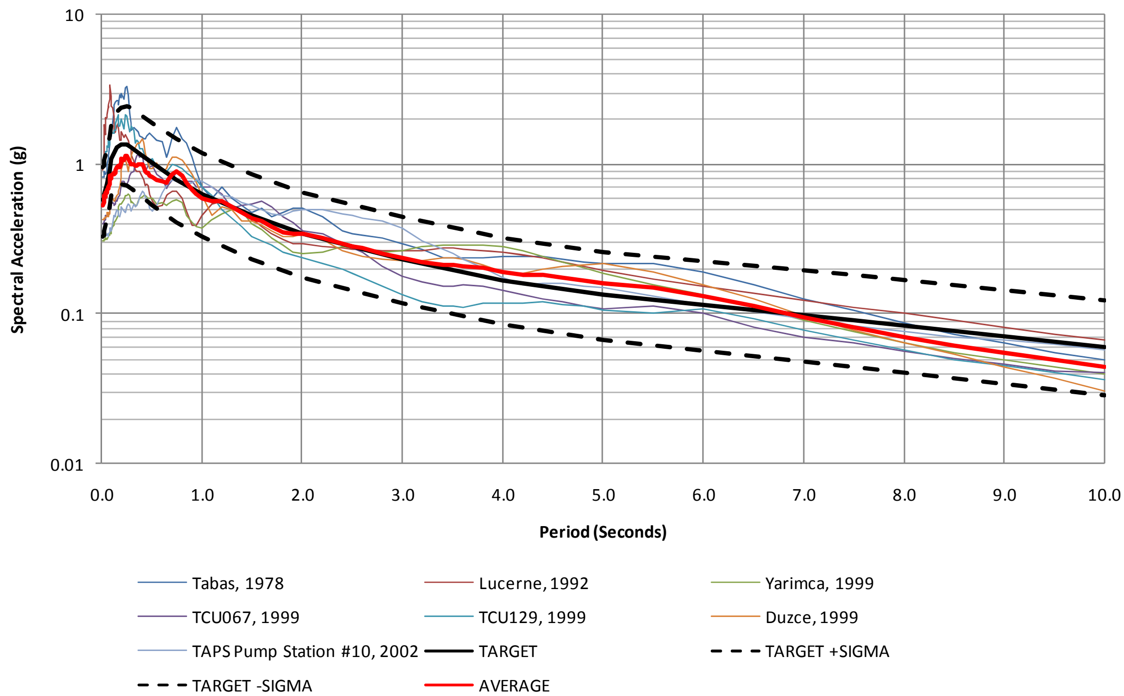

Figure 2. Average horizontal acceleration response spectra of ground motion suite compared to deterministic M8.0, R=10km, 84th percentile spectrum from mean of NGA relations (Abrahamson et al. 2008).

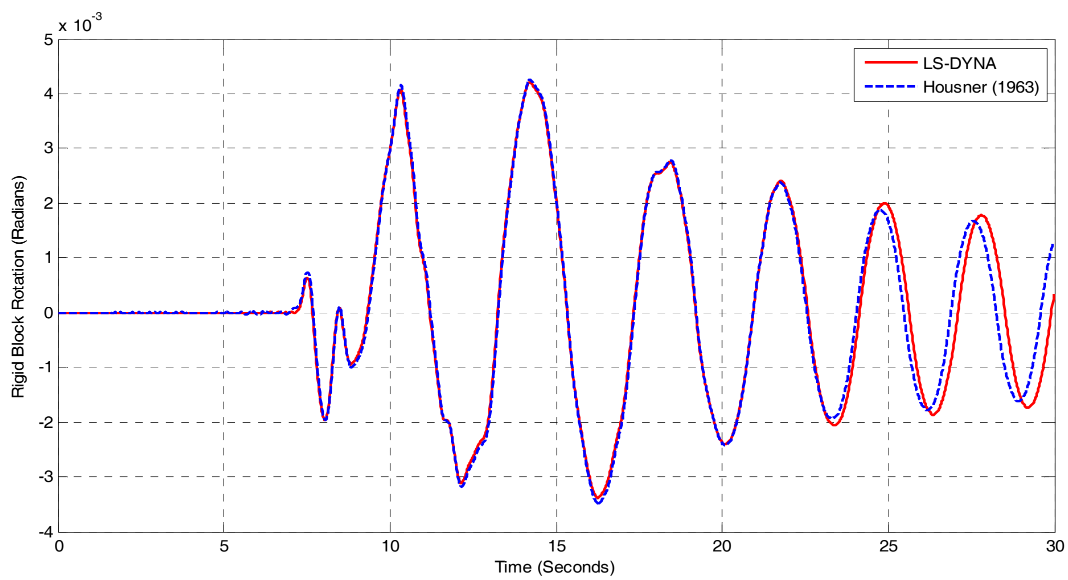

Figure 3. LS-DYNA contact surface validation against theoretical rigid body rocking response due to 100% 1995 Kobe JMA ground motion.

Analysis Results

Both fixed-base and rocking-base buildings were subjected to a suite of seven tri-directional ground motion recordings. The ground motion suite was chosen based on magnitude and distance with minimal amplitude scaling used to achieve a median spectrum near the target 84^{th} percentile deterministic spectrum and includes the following recordings: 100% 1978 Tabas, Iran; 125% 1992 Landers, Lucerne; 100% 1999 Kocaeli, Yarimca; 100% 1999 Chi Chi TCU067; 125% 1999 Chi Chi, TCU129; 100% 1999 Duzce, Turkey; 100% 2002 Denali, TAPS Pump Station #10. Of the records considered, only the Lucerne record exhibits a near-fault forward directivity pulse with a maximum fault-normal displacement of 3.3 meters after scaling to 125%. While this is not representative of ground motion expected 10 km from a fault rupture, it does provide insight into the behavior of fixed-base versus rocking-base buildings under severe fault displacements.

The response of both structures subjected to the 125% Landers, Lucerne motion is shown in Fig. 4. Isosurface plots of wall plastic strain indicate that the core of the fixed-base building forms a failure mechanism through a large distributed plastic hinge at the base, ultimately leading to crushing failure of the concrete and collapse of the structure. The top story drift response of the structure show that incipient collapse occurs at 2% top story drift and indicates that the typical wall has more limited ductility than is typically assumed. The response of the rocking-base structure shows not only survival of the structure under the same severe ground motion but nearly elastic wall response with wall plastic strains of less than 0.4%. The corresponding top story drift response shows a harmonic response dominated by the rocking mode of the structure with damping provided by a combination of the restitution effect inherent to rocking systems and local yielding of the slabs at the wall connection (the system is artificially damped beginning 5 seconds from the end of the record to assess residual drifts).

A more typical response is shown in Fig. 5, with both structures subjected to the 100% Denali TAPS Pump Station #10 motion. The fixed-base structure exhibits typical base wall plastic hinge formation while the rocking-base structure continues to exhibit nearly elastic wall response with wall plastic strains of less than 0.4%. Further, the fixed-base structure has 0.3% residual top story drift following the earthquake while the rocking-base structure re-centers elastically.

The lack of flexural yielding at the base of the rocking-base structure indicates a significant reduction in base moment relieved by the lack of a tension chord between the wall and foundation. The strong-axis moment distribution over the building height, presented in Fig. 6, shows that the base moment is reduced to below the yield moment of the wall section for the rocking-base structure. Fig. 7 shows that the mean response of the rocking system leads to a 30% reduction in both strong- and weak-axis base moments while not significantly changing the moment distribution over the rest of the building. Interstory drift response is shown in Fig. 8 and indicates that peak interstory drift is not significantly increased by allowing the core to uplift and the rocking system sees no residual interstory or top story drift. These results demonstrate the significant advantage of a rocking core wall system in reducing peak wall demands over similar fixed-base tall buildings while also providing elastic re-centering behavior.

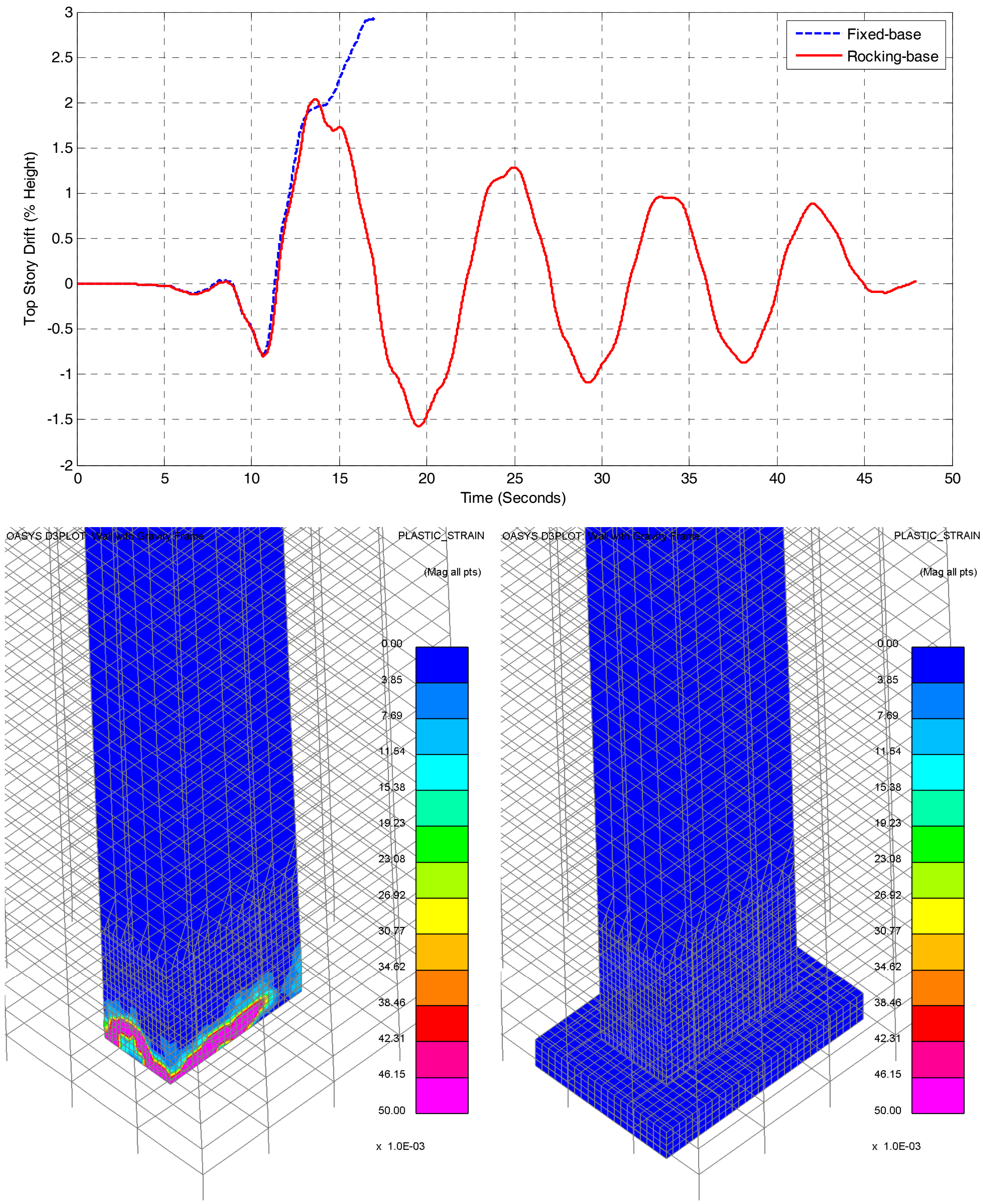

Figure 4. Top story drift response (top) and wall plastic strains for fixed-base (bottom left) and rocking-base (bottom right) buildings subjected to 125% 1992 Landers, Lucerne ground motion at incipient collapse of fixed-base core wall.

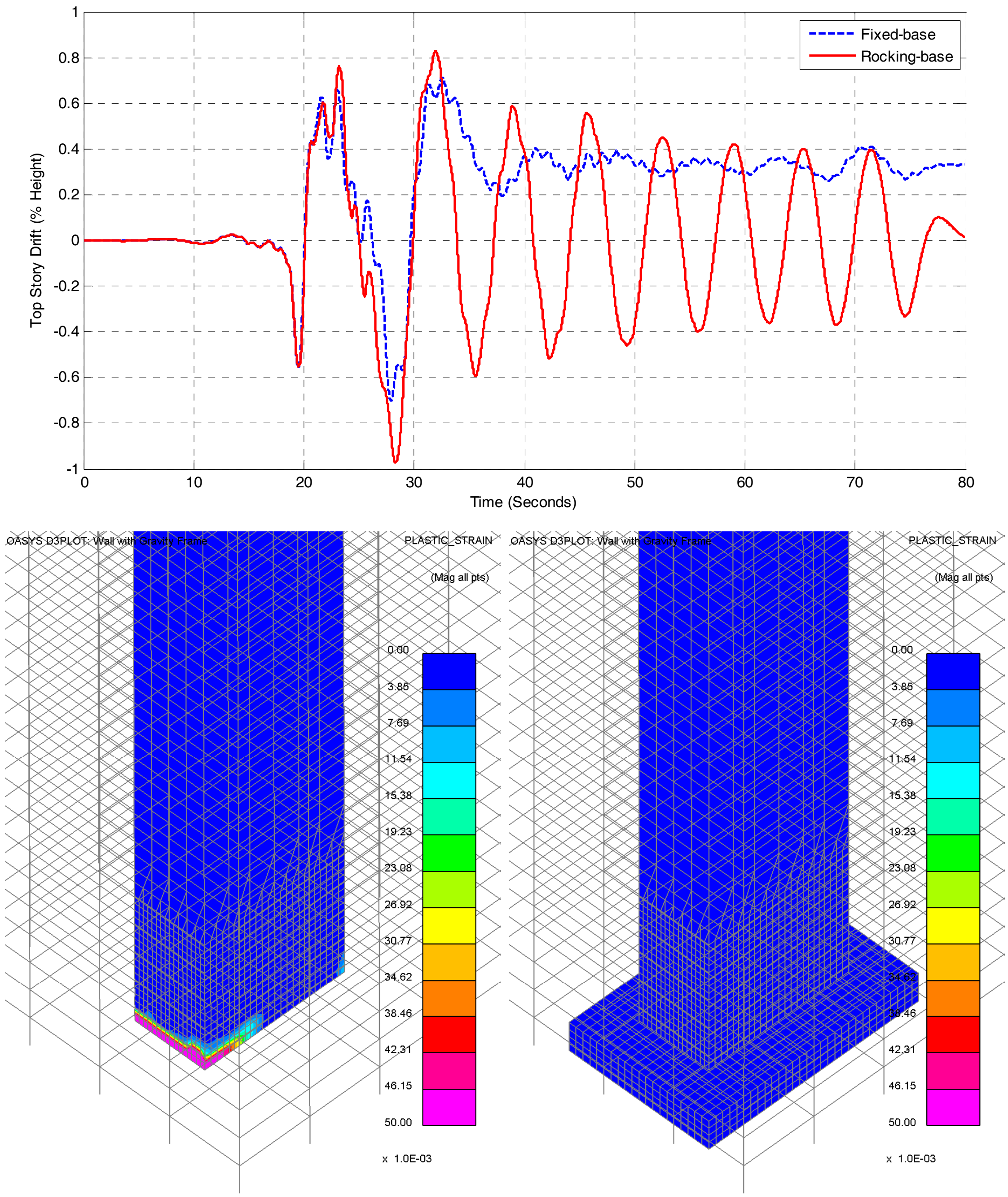

Figure 5. Top story drift response (top) and wall plastic strains for fixed-base (bottom left) and rocking-base (bottom right) buildings subjected to 100% 2002 Denali TAPS Pump Station #10 ground motion at end of transient response.

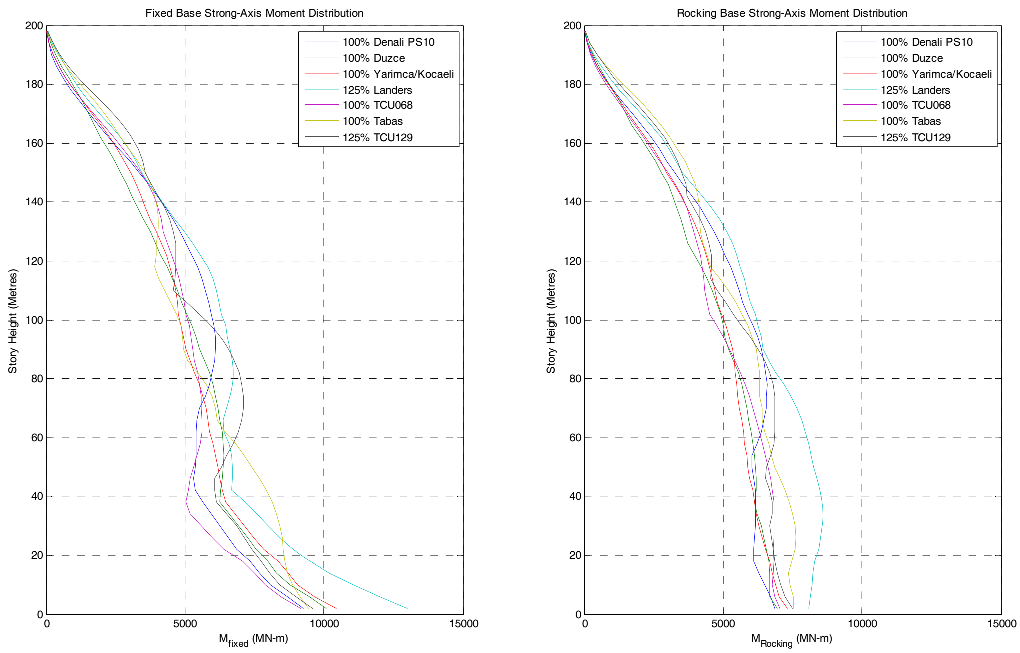

Figure 6. Strong-axis core bending moments over building height for fixed-base (left) and rocking-base (right) buildings subjected to ground motion suite.

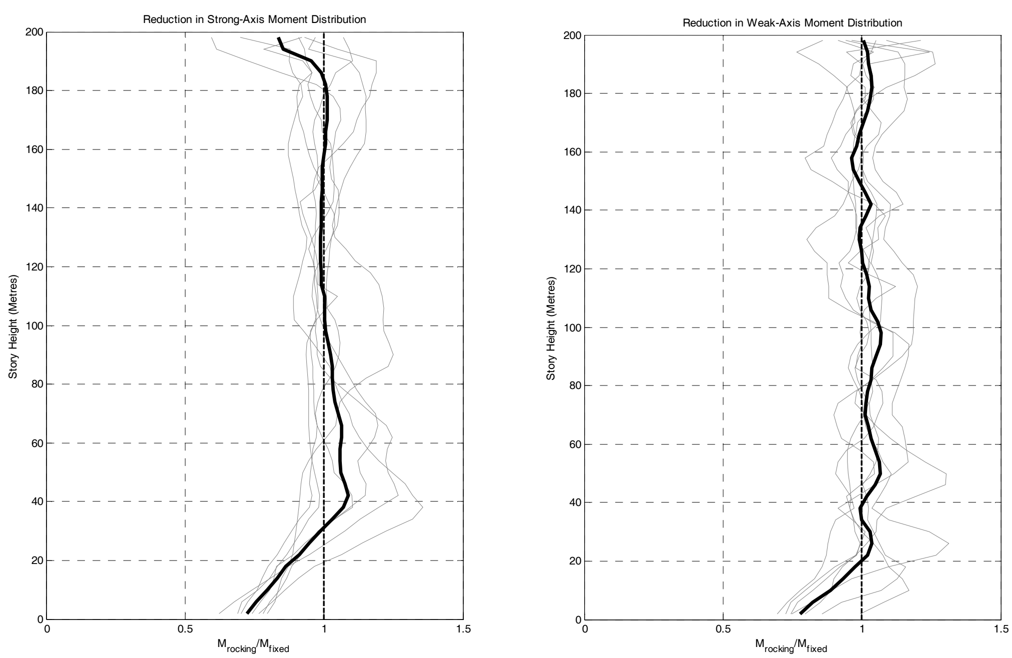

Figure 7. Ratio of rocking-base core strong-axis (left) and weak-axis (right) bending moments to fixed-base core bending moments showing a 30% reduction in base moment demand for the rocking-base building.

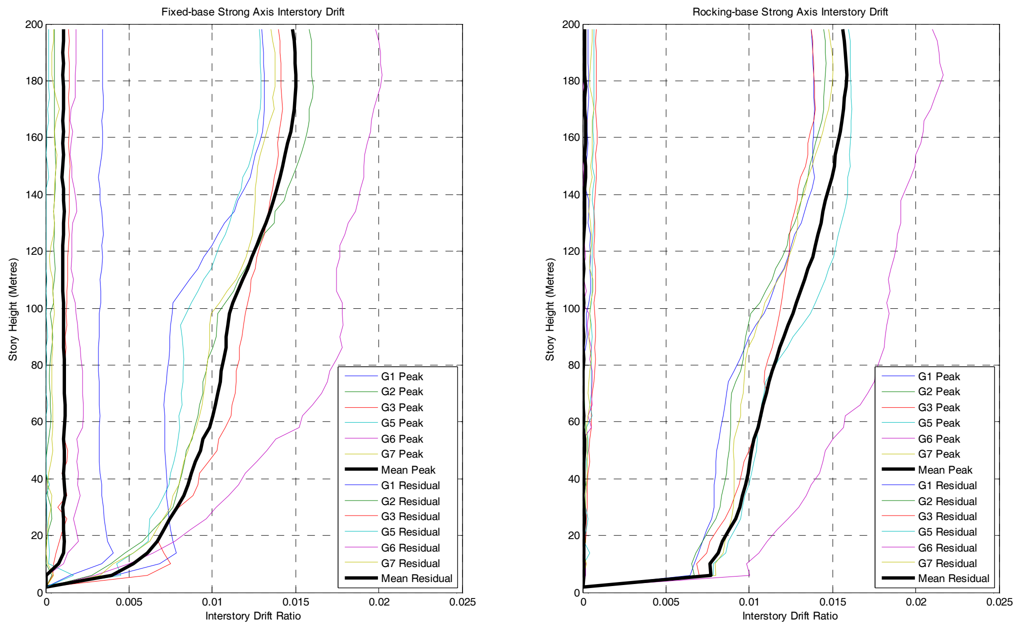

Figure 8. Strong-axis interstory drift ratio over building height for fixed-base (left) and rocking-base (right) buildings subjected to ground motion suite.

Table 1. Peak uplift, peak slip, and residual slip results for rocking-base structure subjected to ground motion suite.

	Denali	**Duzce**	**Kocaeli**	**Landers**	**TCU067**	**Tabas**	**TCU129**
Peak Uplift	11.9 cm	12.3 cm	12.7 cm	29.0 cm	14.6 cm	17.2 cm	13.8 cm
Peak X Slip	0.64 cm	1.86 cm	1.35 cm	1.25 cm	0.88 cm	1.13 cm	0.71 cm
Peak Y Slip	1.23 cm	1.41 cm	1.45 cm	1.02 cm	1.32 cm	1.52 cm	1.13 cm
Residual X Slip	0.01 cm	0.67 cm	0.04 cm	0.03 cm	0.10 cm	0.59 cm	0.16 cm
Residual Y Slip	0.06 cm	0.32 cm	0.85 cm	0.19 cm	0.04 cm	0.35 cm	0.12 cm

The proposed system does not include explicit base translation restraint other than friction between the base of the wall and the foundation mat (e.g. $\mu_{static} = 0.5$, $\mu_{dynamic} = 0.3$) and the restraint provided by the perimeter gravity frame in conjunction with flexible diaphragms. Table 1 provides a list of peak slip and residual slip for each motion in the considered ground motion suite, demonstrating that adequate restraint is provided by the aforementioned mechanisms and no further restraint is required. The peak uplift of the base of the rocking wall is also tabulated in Table 1 and, while a significant amount, the peak uplift is both expected and manageable considering the scale of the rocking system.

Conclusions

The preceding research demonstrates the potential of rocking core wall systems in tall buildings to reduce base moment demands up to 30% over similar fixed-base tall buildings without leading to significant increases in other response quantities. The proposed rocking system, due to its scale and significant gravity load, requires no translational base restraint or additional re-centering mechanisms such as post-tensioned tendons. The response of the rocking system is relatively insensitive to ground motion severity and provides significantly more ductility than a typical fixed-base plastic hinging core without requiring extensive post-event repair. This system points to a sustainable alternative to the earthquake-resistant design of new tall buildings or a possible retrofit solution for existing tall buildings with insufficient moment capacity or displacement ductility.

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