SEISMIC ANALYSIS OF PRECAST CONCRETE BUILDING WITH FRICTION-DAMPING ANCHORAGE SYSTEM

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Abstract

Structural isolation systems that are designed to moderate seismic response, in recent application, have typically been applied at the base of a building; however, applications of inter-story isolation systems have been proposed to satisfy other design considerations. In this research, the behavior of a four-story precast concrete building is investigated with implementation of an isolation system involving energy dissipation through friction damping and lead rubber bearing devices that are placed at inter-story locations between floors and the lateral force resisting system. Methods for preliminary analysis are covered, including computer analysis of a five-story reinforced concrete moment frame and preliminary modeling of the proposed structure system. Issues with MCE analysis data obtained from the moment frame study are discussed as well as design approach and finite element modeling models for initial analysis of the prototype building structure with isolation system.
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Introduction

Overview

Buildings designed only to meet lateral force requirements do not necessarily behave as expected under seismic loading and can be improved through implementation of inertial force-limiting anchorage systems. These moderate the transfer of seismic energy between ground motion and structural components to protect the structural and non-structural integrity of buildings. The floor isolation system limits inertial forces in the floor diaphragm using a ductile floor anchorage system with predetermined cut-off strength, such that seismic loads acting on the lateral force-resisting system (LFRS) are reduced. This then reduces the seismic demands for the structure (Force-Limiting Anchorage Systems, 2013).

The proposed force-limiting anchorage system will dissipate energy through relative motion between the floor diaphragms and the structural walls and frames of the lateral system as well as from resistance from the columns of the gravity load resisting system (GLRS). Efficient energy dissipation will mitigate the forces and accelerations that are transferred to the building and reduce overall potential for structural failures during an earthquake (Force-Limiting Anchorage Systems, 2013). The system presents the self-centering properties of post-tensioning tendons in combination with hysteretic energy dissipating devices placed at the LFRS to floor diaphragm connection at multi-level locations. The damping devices allow the structural elements of the building to decouple during earthquakes and seismic energy transferred to the building structure is dampened significantly (Elgamal and Fraser, 2004). This innovative anchorage system poses as an alternative to other isolation systems that have been studied. Further research and experimentation will advance knowledge of the behavior of these systems in order to improve design of precast building structures in seismic zones.

Motivation

Design codes for building structures in seismic zones have long been considered, although practical application is minimal in terms of segregated earthquake design, specifically. Structural design criteria for buildings under general lateral loading are nearly similar to that for buildings designed to resist cyclic loading but the transmission of forces and accelerations through buildings during an earthquake are considerably different from those imparted by other lateral loads, such as wind load, lateral hydrostatic and soil pressure, or blast.

The development of precast concrete structures has been limited in seismic zones because of the absence of these seismic design provisions in major model building codes. Poor performance of several precast concrete structures in previous years has further contributed to restrictions on precast usage in seismic zones. Certain isolation systems have been
shown to be effective, but these can be costly or impractical for application to existing buildings (Pampanin, 2003). Because of the advantages of precast reinforced concrete (RC) structures, there is a need to develop dynamic systems for application to structural design against earthquakes that is both economic and better structural performance. In turn, seismic design codes can be modified to improve overall performance of RC structures during earthquakes as well as improve general knowledge of their structural behavior with use of energy dissipation systems.

Research Objective

The proposed anchorage system offers an alternative to base isolation and floor isolation systems in building structures that are designed to deflect earthquake energy as smoothly as possible during seismic activity. Implementation of the system presents the opportunity to advance knowledge in the area of building structure design for seismic loading. The objective is to further knowledge of dynamic systems that reduce inertial forces in structural components of buildings during earthquakes in order to regain a centered floor after seismic activity. The long-term goal is to generate research through a combination of dynamic analysis and experimentation to determine optimum design parameters that will produce desired seismic performance for a variety of building geometries and properties. Design code for earthquake loads will be improved and continued research will allow for further refinement of design codes and availability of anchorage or isolative technologies for building structures that are expected to experience earthquake forces. Acquired knowledge of the behavior and benefits of these isolation systems will go toward application of seismic design codes to a wide array of construction types and building systems. The potential benefits include reduced seismic forces that are transmitted through the building, reduced lateral drift, reduced structural damage, reduced nonstructural contents damage, and higher factor of safety against collapse. This approach has the potential for producing more economic and safer building design under earthquake loads, which are unpredictable and threaten structural resistance against failure or collapse.

The prototype design will be later tested at the large scale structural and hybrid testing laboratory at Lehigh University and the shake table at the University of California, San Diego. The following sections of this paper include research for preliminary analysis for a five-story RC moment frame to model inelastic behavior under earthquake loading and a four-story precast concrete building with the energy dissipating anchorage system. The first portion of this paper focuses on the RC moment frame, including design approach followed with 2-dimensional finite element modeling and analysis using Opensees software. This design does not implement the innovative energy dissipating system but contributes toward knowledge of concrete structures under seismic loading. The research was conducted at the University of Arizona and the University of California, San Diego. The later sections cover preliminary 3-dimensional finite element modeling/analysis of building structure with the isolation system also using Opensees. If successful, this research will transform the way that current codes allow for building design to resist lateral loads imposed by earthquakes by modifying the structural response to cyclic loading and better understanding system
behavior and the effect of energy damping in compatibility with conventional structural dynamics.

Isolation System Devices

Friction dampers, rubber bearings and plastic bumper device configurations will be modeled and tested in the following research. The friction device is the primary yielding energy dissipation device. It limits the floor inertial forces up to the predefined yielding force level and energy dissipation occurs through hysteresis. Friction devices have been used to improve the seismic design of structures beyond the conventional ductility design approach (Cherry and Filiatrault, 1993) and can be modeled with force-displacement hysteretic relationships (Hanson and Soong, 2001). In this type of system, the friction devices are inserted at connections between the LFRS and floor diaphragms, which allow slip during lateral movement once a predetermined optimal load during earthquake excitation is met. These connections are primarily designed to be stiff and strong. The slip mechanism occurs when the predetermined load is exceeded while prior to reaching this capacity, seismic energy is transferred from ground motion excitation to the structure without relative movement of the floor (Force-Limiting Anchorage Systems, 2013). Energy is dissipated mechanically through friction rather than through stressing the structural elements and causing inelastic deformations (Cherry and Filiatrault, 1993).

The rubber bearings provide high stiffness and flexibility when induced lateral motion occurs in building structures. Their purpose is to provide elastic restoring force to reduce residual relative displacement and to prevent the LFRS shear walls from buckling. This enables them to be used as seismic and vibration isolators for buildings susceptible to seismic induced lateral movement (Kelly and Konstantinidis, 2011). The plastic bumper prevents excessive relative displacement between the floor diaphragms and the LFRS, limiting large deformation in non-structural components, and protects the other devices from failure if large relative displacements occur.

Literature Review

The use and development of precast concrete structures in seismic areas have been typically limited by the lack of confidence and knowledge about their performance in seismic regions as well as by the absence of rational seismic design provisions in major model building codes. The poor performance of several precast parking structures in the 1994 Northridge Earthquake due to incorrect design detailing has contributed to further restrictions on precast usage in seismic zones (Pamparin, 2003). Numerous innovative ‘ductile’ jointed connections were characterized with analytical and experimental tests on subassemblies. Simulated tests on a five-story, 2/3 scale, precast concrete building were performed at the University of California, San Diego, using pseudodynamic, cyclic, quasi-static, and flexibility testing methods. The performance of the frame and wall systems was extremely satisfactory, with minor damage to panel zone joint regions and structural precast members. The self-centering property due to the presence of the unbonded post-
tensioning (PT) tendons, which will be used in the shear walls of the proposed system, resulted in minimal residual crack opening width and significant reduction of residual displacements (Pampanin, 2003).

Energy dissipation is recognized as an effective means for controlling vibration in structural systems under dynamic loading. Energy from ground excitation during earthquakes can transfer to structural components of a building, causing non-ductile shear failure and leading to collapse, significant damage to structural and non-structural components or injury to building occupants (Hanson and Soong, 2001). Application of similar energy dissipation systems was conducted under Shana Crane in a 2004 research study of two six-story miniature buildings at the University of California, San Diego. The models were designed and tested to compare the performance of a building that implemented passive energy dissipating devices to that of a building with essentially rigid connections. Triangular-Plate Added Damping and Stiffness (TADAS) devices were used as the passive energy-dissipating devices and were installed at the connections between the lateral system and floor diaphragms, similar to that of the friction devices in the new system. Dynamic tests were performed on both buildings with 25% and 50% scaled inputs of the 1994 Northridge, Canoga Park record input motion to produce elastic responses during the 25% excitations and inelastic response for the 50% excitations. The inclusion of the TADAS connections reduced floor accelerations and displacements for the input earthquake motions at all floor levels. Overturning moment demands were reduced and maximum inter-story drifts were either reduced or kept the same. In some cases, though, residual inter-story drifts were increased by large amounts (Crane, 2004). Conversely, large scale testing of the proposed system will be conducted at the UC San Diego shake table later this year.

Seismic isolation is a reliable and effective approach for alleviating the seismic response of a building by installing a ductile interface that absorbs seismic energy and shifts the natural frequency of the system away from the frequency range of the largest seismic demands. Isolation systems are traditionally applied at the base level but other considerations such as architectural, functional, or cost may encourage the use of isolators at other levels of a building, like in inter-story isolation systems. Past applications of inter-story isolation systems include 185 Berry Street in San Francisco (Dutta et al., 2008), Iidabashi First Building and Shiodome Sumitomo Building in Japan (Tasaka et al., 2008), and Umeda DT Tower in Japan (Yamane et al., 2003; Ryan and Earl, 2010). 185 Berry Street was built in 1989 as a 3-story special RC moment frame but was retrofitted with two additional floors and isolation system. This solution allowed floors to be added to an existing structure without increasing the base shear demand (Dutta et al., 2008). Iidabashi First and Shiodome Sumitomo are multi-story buildings with isolation systems installed at mid-building-height (Tasaka et al., 2008). Umeda DT Tower uses inter-story isolation to isolate its 30-story tower from the multi-level lobby at the base (Yasaka et al., 2003). Testing by Ryan and Earl, which studied the effects of applying the isolation system at various locations in a building, showed that single-story isolation systems were effective in reducing force demands above the isolation system but less effective in reducing forces below it (Ryan and Earl, 2010).
Damping devices have been used mainly for retrofitting and strengthening existing buildings in the U.S. Installation of base isolation systems is straightforward for new buildings, but complicated and costly for retrofit applications, while installing an isolation system at the roof level would be comparatively simple and inexpensive. Over 20 pre-existing buildings in P.R. China have been retrofitted with top-story isolation systems (Xie and Zhou, 1998; Ryan and Earl, 2010). Roof level isolation systems that utilize the tuned mass damper concept have been designed and experimentally tested. These systems were effective in reducing the seismic response of the system to cyclic ground motions by adding harmonics to match the natural frequency of the original structure. These types of systems can be used to lower accelerations or protect against potentially damaging inelastic response of structural elements and connections. They help to control inter-story drift, whereas common practices have been to increase stiffness by increasing the size of structural members (Hanson and Soong, 2001). Chinese researchers have also experimented on multi-story frames with isolation systems installed at various levels above the base (Ryan and Earl, 2010). Segmenting the superstructure of a base isolated building by adding multiple isolation levels was also explored as a technique for increasing the effectiveness of seismic isolation for taller buildings (Pan et al., 1995; Pan and Cui, 1995). Comparison of a 16-story superstructure isolated at four levels (segmented superstructure) with the same building isolated at the base showed similar acceleration responses in the superstructure, but the segmented superstructure displayed a substantial reduction in base deformation (Ryan and Earl, 2010). In the proposed system, the friction devices will be applied between at each shear wall connection on all middle floors. Energy dissipation through hysteresis will be the focus for moderating seismic response in the structure. Previous studies by Ryan and Earl (2010) explored the effectiveness of inter-story and multi-story isolation systems placed at various levels, but did not consider the non-linear force-deformation behavior of the isolation system.

The proposed system also benefits from the self-centering properties of the post-tensioning cables in the shear walls of the structure. Several self-centering systems have been studied as part of the PREcast Seismic Structural Systems research program (PRESSS) on precast concrete systems in the U.S. (Pampanin, 2003). The program tested a 60% scale five-story building that included precast prestressed concrete frames with partially debonded tendons in one direction and a coupled wall designed to provide lateral force resistance in the other loading direction (Filiatrault et al., 2004). Structural systems that possess self-centering characteristics are economically viable alternatives to current lateral force resisting systems. They incorporate the nonlinear characteristics of yielding structures and the self-centering properties allow the structure to return to its original position after an earthquake. Priestley and Tao (1993) proposed the use of self-centering precast concrete moment resisting frame systems prestressed with partially unbonded tendons as the primary lateral force resisting systems in seismically prone areas (Priestley and Tao, 1993). Stanton et al. (1993) proposed a hybrid system, in which mild steel reinforcement was combined with unbonded tendons in the critical connections to provide hysteretic energy dissipation to the system (Stanton et al., 1993). A numerical parametric study was conducted recently by Christopoulos et al. (2002) to determine the seismic response of single-degree-of-freedom (SDOF) systems incorporating self-centering, hysteretic structural behavior. The responses of the hysteretic SDOF systems were compared to the responses of similar bilinear elasto-
plastic hysteretic SDOF systems. It was shown that the displacement demands of the hysteretic SDOF system matched or bettered the response of the elasto-plastic hysteretic SDOF system. It, also, did not sustain any residual drift (Christopoulos et al., 2002).
Contributions

Moment Frame Analysis

Analytical research for the reinforced concrete moment frame included structural design for a prototype 180’x100’ residential building and finite element modeling using Opensees software. For design, RISA software was used for structural analysis and Xtract software for finite element analysis of fiber sections. The LFRS is made up of special RC moment frames located on the building perimeter in both transverse and longitudinal directions. Modeling also included design for gravity frames, shown aligned in transverse section along the interior of the building. The footprint below (Figure 1) shows the floor plan for the structure. Analysis was performed for a five-story building geometry. Only the moment frame and gravity frames in the transverse directions were designed for this case.

Calculation of equivalent lateral forces for modeling of the vertical distribution of earthquake load at each story were applied to the RISA model and used to determine drift values, moment, axial, and shear demands in the exterior columns, interior columns, and beams, and to determine member size and reinforcement based on these demands. Equivalent lateral force (ELF) values were calculated at each floor level. First story height is 16’ and typical floor-to-floor height is 10.5’ for all other stories. Vertical distribution of ELF is calculated using equation:

\[ F_x = C_{xx} V = \sum w_x h_x^k V \]

(1)

Where \( h_x \) is the height of the shear vector measured from the base of the building, \( w_x \) is the building weight at that level, and \( V \) is the seismic shear at the base, which is equal to the seismic response coefficient times the total weight of the structure.
Table 1 gives the seismic design parameters that were considered for RC moment-resisting frame design according to ASCE2005 Design code (ASCE, 2005). Explanation of each of the variables is given below it. Table 2 shows distribution of lateral forces at each floor height, which were modeled as point loads in our elastic frame model built in RISA. We then checked drift values in order to satisfy drift and stability limitations for selected member sizes.

### Table 1. Seismic Design Parameters According to ASCE 2005 Design Code

<table>
<thead>
<tr>
<th>Seismic Design Parameters</th>
<th>Design Site</th>
<th>Generic</th>
<th>$F_v$</th>
<th>1.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>SDC</td>
<td>D</td>
<td></td>
<td>$S_{ms} = F_a \times S_s$</td>
<td>1.5</td>
</tr>
<tr>
<td>$S_s$</td>
<td>1.5</td>
<td></td>
<td>$S_{m1} = F_v \times S_1$</td>
<td>0.9</td>
</tr>
<tr>
<td>$S_1$</td>
<td>0.6</td>
<td></td>
<td>$S_{DS} = 0.67 \times S_{ms}$</td>
<td>1</td>
</tr>
<tr>
<td>Soil Site Class</td>
<td>D</td>
<td></td>
<td>$S_{D1} = 0.67 \times S_{m1}$</td>
<td>0.6</td>
</tr>
<tr>
<td>$F_a$</td>
<td>1</td>
<td></td>
<td>$C_t$</td>
<td>0.016</td>
</tr>
<tr>
<td>(Special RC Moment Frame)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\Omega_0$</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$C_d$</td>
<td>5.5</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Where:
- SDC = Seismic Design Category
- $S_s$ = Mapped spectral response acceleration at short periods
- $S_1$ = Mapped spectral response acceleration at 1-s periods
- $F_a$ = Site coefficient for short-period spectral acceleration
- $F_v$ = Site coefficient for 1-s period spectral acceleration
- $S_{ms}$ = Adjusted spectral response acceleration for site class effects at short periods
- $S_{m1}$ = Adjusted spectral response acceleration for site class effects at 1-s periods
- $S_{DS}$ = Design spectral response acceleration at short periods
- $S_{D1}$ = Design spectral response acceleration at 1-s periods
- $C_t$ = Building period coefficient
- $R$ = Response modification coefficient
- $\Omega_0$ = System overstrength factor
- $C_d$ = Deflection amplification factor

The seismic response coefficient, $C_s$, is calculated using equations (2) and (3):

$$T_a = C_t H_n^{0.9} = 0.016 \times 58^{0.9} = 0.618 \text{ sec}$$  \hspace{1cm} (2)

Where $T_a$ is the fundamental building period and $H_n$ is total building height.

$$C_{S,\text{max}} = \frac{S_{D1}}{(R/I_E)T_a} = 0.121$$  \hspace{1cm} (3)
Where $S_{D1}$ is the design spectral response acceleration at 1-second periods with 5% damping, $R$ is the response modification coefficient, and $I_E$ is the importance factor.

$C_i$ is then used to calculate $V$ from Eq. (1), given by Eq. (4). The vertical ELF distribution values are shown in Table 2.

$$V = C_i W \quad (4)$$

<table>
<thead>
<tr>
<th>Story</th>
<th>$h_x$ (ft)</th>
<th>$W_x$ (kips)</th>
<th>$W_x h_x^k$</th>
<th>$C_{vx}$</th>
<th>$F_x$ (kips)</th>
<th>$F_{px}$ (kips)</th>
<th>$V$ (kips)</th>
<th>$M$ (k-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>58.0</td>
<td>3546</td>
<td>432065</td>
<td>0.334</td>
<td>514</td>
<td>709</td>
<td>514</td>
<td>5392</td>
</tr>
<tr>
<td>4</td>
<td>47.5</td>
<td>3546</td>
<td>341160</td>
<td>0.264</td>
<td>406</td>
<td>709</td>
<td>919</td>
<td>15043</td>
</tr>
<tr>
<td>3</td>
<td>37.0</td>
<td>3546</td>
<td>253882</td>
<td>0.196</td>
<td>302</td>
<td>709</td>
<td>1221</td>
<td>27862</td>
</tr>
<tr>
<td>2</td>
<td>26.5</td>
<td>3546</td>
<td>171071</td>
<td>0.132</td>
<td>203</td>
<td>709</td>
<td>1424</td>
<td>42816</td>
</tr>
<tr>
<td>1</td>
<td>16.0</td>
<td>3546</td>
<td>94187</td>
<td>0.073</td>
<td>112</td>
<td>709</td>
<td>1536</td>
<td>67395</td>
</tr>
<tr>
<td>Sum</td>
<td>17730</td>
<td>1292366</td>
<td>1</td>
<td></td>
<td>1536</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Selected member sizes to satisfy drift limitations were 36”x36” square columns and 36”x32” rectangular beams for all floors. The frame inter-story drift was calculated based on the seismic loads calculated above and analyzed in RISA. Cracked section properties were assigned to the beams (0.35Ig) and columns (0.7Ig). The obtained frame drift at each story is shown in Table 3. Based on 2% drift limitation, the selected member sizes are adequate.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>1.908</td>
<td>10.5</td>
<td>1.11</td>
<td>0.88</td>
<td>2.0</td>
<td>3996</td>
<td>514</td>
<td>126</td>
<td>0.012</td>
<td>0.09</td>
</tr>
<tr>
<td>4</td>
<td>1.706</td>
<td>9.4</td>
<td>1.69</td>
<td>1.34</td>
<td>2.0</td>
<td>7992</td>
<td>919</td>
<td>126</td>
<td>0.021</td>
<td>0.09</td>
</tr>
<tr>
<td>3</td>
<td>1.398</td>
<td>7.7</td>
<td>2.18</td>
<td>1.73</td>
<td>2.0</td>
<td>11988</td>
<td>1221</td>
<td>126</td>
<td>0.031</td>
<td>0.09</td>
</tr>
<tr>
<td>2</td>
<td>1.001</td>
<td>5.5</td>
<td>2.50</td>
<td>1.98</td>
<td>2.0</td>
<td>15984</td>
<td>1424</td>
<td>126</td>
<td>0.040</td>
<td>0.09</td>
</tr>
<tr>
<td>1</td>
<td>0.547</td>
<td>3.0</td>
<td>3.01</td>
<td>1.57</td>
<td>2.0</td>
<td>19980</td>
<td>1536</td>
<td>192</td>
<td>0.037</td>
<td>0.09</td>
</tr>
</tbody>
</table>

*P_x is calculated based on unfactored vertical load (197 psf dead+ 50 psf live)

For strength design, the selected member sizes were tested with various combinations of steel reinforcement to satisfy moment demands in the beams. The strength design is only considered for flexural strength for the purpose of the multiple degrees-of-freedom (MDOF) analytical situation. The analytical model is based on the assumption that plastic hinges only occur in the beams and there is no shear failure in any of the beams or columns. For 2-dimensional analysis, 197 psf dead load and 50 psf live load are converted to linear uniform loads, considering live load reduction and 15’ tributary width. Figure 2 shows the maximum moment demands in each of the beams under applied load combination of $(1.2 + 0.2S_{DS})D + 1.0L + 1.0E$. 

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The selection of beam reinforcement is shown in Table 4 and Figure 3. The critical moments at each floor as shown in Figure 2 are used for flexural design.

Table 4. Moment frame beam flexural design

<table>
<thead>
<tr>
<th>Story</th>
<th>$M_u$ (kip-in)</th>
<th>$M_n$ (kip-in)</th>
<th>$\Phi M_n/M_u$ ($\Phi=0.9$)</th>
<th>Steel</th>
<th>$d_{min}$ (in)</th>
<th>$A_s$ (in$^2$)</th>
<th>Spacing (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>3610</td>
<td>4063</td>
<td>1.01</td>
<td>10#4 T&amp;B</td>
<td>0.500</td>
<td>3.93</td>
<td>3.00</td>
</tr>
<tr>
<td>4</td>
<td>5887</td>
<td>6732</td>
<td>1.03</td>
<td>11#5 T&amp;B</td>
<td>0.625</td>
<td>6.75</td>
<td>3.75</td>
</tr>
<tr>
<td>3</td>
<td>8393</td>
<td>9338</td>
<td>1.00</td>
<td>8#7 T&amp;B</td>
<td>0.875</td>
<td>9.62</td>
<td>5.25</td>
</tr>
<tr>
<td>2</td>
<td>10217</td>
<td>11580</td>
<td>1.02</td>
<td>10#7 T&amp;B</td>
<td>0.875</td>
<td>12.03</td>
<td>5.25</td>
</tr>
<tr>
<td>1</td>
<td>11100</td>
<td>12698</td>
<td>1.03</td>
<td>11#7 T&amp;B</td>
<td>0.875</td>
<td>13.23</td>
<td>5.25</td>
</tr>
</tbody>
</table>

For gravity frame design, a similar process was followed. Figure 4 is the equivalent RISA model under the gravity load combination, 1.2D + 1.6L. Table 4 contains selection of beam
reinforcement calculations. Here, the same reinforcement is used in each floor. Figure 5 shows the reinforcement detailing for the gravity frame beams.

![Fig.4 Bending moments in gravity frame beams (units in k-in)](image)

**Table 5. Gravity frame beam flexural design**

<table>
<thead>
<tr>
<th>Story</th>
<th>$M_u$ [k-in]</th>
<th>$M_n$ [k-in]</th>
<th>$fM_u/M_n$</th>
<th>Tension steel</th>
<th>$A_s$ [in$^2$]</th>
<th>$r$</th>
<th>$r_{min}$</th>
<th>$r_{max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>4558</td>
<td>5084.3</td>
<td>1.00</td>
<td>16#6</td>
<td>7.07</td>
<td>1.54%</td>
<td>0.35%</td>
<td>2.12%</td>
</tr>
<tr>
<td>4</td>
<td>4223</td>
<td>5084.3</td>
<td>1.08</td>
<td>16#6</td>
<td>7.07</td>
<td>1.54%</td>
<td>0.35%</td>
<td>2.12%</td>
</tr>
<tr>
<td>3</td>
<td>4286</td>
<td>5084.3</td>
<td>1.07</td>
<td>16#6</td>
<td>7.07</td>
<td>1.54%</td>
<td>0.35%</td>
<td>2.12%</td>
</tr>
<tr>
<td>2</td>
<td>4345</td>
<td>5084.3</td>
<td>1.05</td>
<td>16#6</td>
<td>7.07</td>
<td>1.54%</td>
<td>0.35%</td>
<td>2.12%</td>
</tr>
<tr>
<td>1</td>
<td>4542</td>
<td>5084.3</td>
<td>1.01</td>
<td>16#6</td>
<td>7.07</td>
<td>1.54%</td>
<td>0.35%</td>
<td>2.12%</td>
</tr>
</tbody>
</table>

The column reinforcement for all interior and exterior columns was designed according to ACI 318-2002 (ACI, 2002). Selection of column reinforcement is shown in Figure 6.

![Beam for All Stories](image)
Fig. 6. Moment Frame Column Detailing

The fiber sections for beams on each floor, interior column, and exterior column were then modeled in Xtract to obtain moment-curvature data and check moment demands of each section design. See Appendix A for figures containing moment-curvature relationship for each designed member. Curves shown in green represent the moment capacity in each respective member for concrete compressive strength of 5ksi. In Xtract, the sections were modeled with concrete compressive strength of 7.5 ksi, such that moment capacity in inelastic behavior exceeds moment demand.

Once the member design was complete, the moment-curvature data obtained from Xtract were applied to define section properties for the elastic members in the OpenSees model. Four points were taken along the curves previously mentioned (shown in Appendix A as red curve on moment-curvature relationship) in the inelastic range to model inelastic behavior after hinges are formed at the beam ends. When writing the codes, separate files were defined for building geometry and building parameters in order to run analysis for structures with various geometries or properties. The analysis obtained data only for five-story geometry for both maximum considered earthquake (MCE) and design-based earthquake (DBE) analysis. All data shown in the results applies to MCE analysis. Another researcher working on the same project ran the DBE analysis. Appendix B contains scripts for code unique to the model, including materials, sections and beam and column element definition. For preliminary analysis, Matlab files were also written for post-processing the data.

The preliminary analysis is not yet completed due to error in output files. The graphs in Figures 7-9 demonstrate obtained results although modifications will be necessary. Identical response data was recorded in the output files for each of the 10 different earthquake tests, where values for displacements, deformations, accelerations, etc. should have varied for each earthquake. The cause of this has not yet been determined. Because of this, it is difficult to draw conclusions on data. This error most likely stems from an issue with the output directories or OpenSees analysis algorithms and needs to be updated in order to re-run the analysis to get conclusive results. Figure 7 should demonstrate 10 various curves for maximum node acceleration in the RC moment frame with respect to device strength (strength of joint connections), although clearly only one plot is appearing. Here, all 10 curves are being plotted but the ambiguity in the recorder data was recorded incorrectly, so the values overlap and appear as a single curve.
Hysteresis plots display the same issue and the same argument can be made for the source of this error. See figures 8 and 9 below for force vs. deformation plots for the exterior beam plastic hinges of the moment frame under a single cyclic loading in the first and second story beams. Plots for the third, fourth, and fifth story floors are included in Appendix C.
With continued work, Matlab scripts will be finalized and eigenvalue analysis should be processed a second time to output conclusive values. Because of the ambiguity that appeared in the output files, it is difficult to currently draw conclusions on the results of the analysis. DBE analysis obtained similar error, which confirms that there may be issue with the analysis algorithms. When the modified post-processing is complete, the data will be updated as soon as possible.

Precast Concrete Structure with Isolation System

Similar methods were used for preliminary analysis of the four-story precast reinforced concrete building structure with isolation system using friction damping energy dissipation devices installed at the connections between LFRS unbonded post-tensioned shear walls and floor diaphragms of the middle three floors. The design consists of nine exterior columns and one interior column located between the eccentric and concentric shear wall locations. All column and wall members will be precast concrete. Cases for eccentric shear wall placement and concentric shear wall placement will be tested. In addition, shear walls will be pre-stressed with two unbonded post-tensioning cables each with symmetric placement within the wall.

My part in the project involved building a 3-dimensional computer model in Opensees, which includes floor and column elements, post-tensioned shear walls using unbonded post-tensioning cables, friction dampers, and lead rubber bearing devices modeled in their respective locations. Figure 10 shows a SketchUp model for the mock up of the design structure and Figure 11 demonstrates the build up of the model geometry as defined by
nodes and elements in Opensees. This image was taken from equivalent Ansys model (being that Opensees does not have a compatible graphic user interface for displaying model geometry). The modeling of floor slabs and columns are similar, but the shear walls in the Opensees model are not modeled as 2-dimensional elements (as shown by the teal grid elements), but rather as one-dimensional vertical elements with 3-dimensional shear fiber modeling at the base. Post-tensioning cables are modeled using truss element with initial stress value, and applied as separate elements, rather than adjusting the material properties of the vertical wall elements. All columns are modeled with linear elastic elements.

![Figure 10. 3-D SketchUp model of prototype structure. Note that both longitudinal shear walls are shown but only eccentric and concentric cases will be tested.](image)

![Figure 11. Image of prototype geometry from Ansys graphics](image)

The main component of the energy dissipation system is the friction-damping device that is installed between the shear walls and floors, one at each shear wall location aligned with the long span of the shear wall. See Figure 12 for the design of these connections, along with placement for the lead rubber bearings that will also be placed at the same connection, flush with the face of the shear wall. These elements are modeled as node link elements in Opensees defined with the appropriate parameters. For the 3-dimensional model, separate material tags are defined for the rubber bearing properties acting in the vertical, horizontal, and in plane directions to account for unequal values of shear stiffness, axial stiffness in compression, and axial stiffness in tension in the respective directions. To model the different axial stiffness of the rubber bearing in tension and compression, an elastic linear material is used to define various points on the stress-strain curve for the device. To model the friction device, steel01 material is used, which models...
device with rectangular hysteretic behavior. Figure 13 shows expected hysteresis for the friction damper. Data for force and deformations in the friction devices will be a major focus for analysis and will be obtained using Matlab scripts once the Opensees model can be run.

Figure 12. Friction Device connection detailing

Figure 13. Expected hysteresis response in friction damper
The shear wall post-tensioning modeling and fibers at the base connection were the other main component of the model. Even though the shear wall itself was modeled as a one-dimensional vertical element, the fiber modeling at the base is modeled in three dimensions. Parameters for concrete and mild steel reinforcement connections are defined in material and section properties and can be seen in Appendix C. The actual connections are modeled with zerolength elements in Opensees and concrete and steel connections are modeled separately, considering heavier reinforcing in the ends of the shear wall.

The post-tensioning cable is modeled as an initial stress element with a yielding strength of 270 ksi. Whereas the shear wall elements are defined between each floor, the post-tensioning cable is a single span element from the top to bottom node of the shear wall. Definition of post-tensioning cable material can also be found in Appendix C. Figures 14 and 15 display the general approach to fiber modeling at the base. In Figure 14, the blue section represents the area of less dense reinforcement, while the black areas represent heavier steel reinforcement sections. Properties also vary for confined (black) and unconfined (blue) concrete sections. Figure 15 displays the use of multi-dimensional shear fiber modeling of the model in compatibility with a one-dimensional shear wall element. The images shown are for a two-dimensional model, but the fiber modeling in the Opensees model will extend in three directions, accounting for earthquake motion in all degrees of freedom.

![Figure 14. Shear fiber modeling at shear wall foundation](image)

![Figure 15. Geometry for shear fibers relative to vertical shear wall](image)
Currently, the Opensees model is being finalized and results have not yet been obtained. Once the analysis can be run, similar time history, parametric plots, and hysteretic response plots will be of interest and will be achieved with similar Matlab scripts as written for the moment frame analysis. Because the model is incomplete, there are no current output data to be discussed, although some modifications to the modeling approach may be later justified and updated.

Conclusion

As this was the first time I had used Opensees, it took a few weeks to become familiar with the software and the language before I could efficiently begin to build the model geometry. The design process was also new; I had not taken reinforced concrete class so much of the research work was just introduced to me and was a learning process. As I began to become familiar with the software though, it became more clear how powerful Opensees is for analytical purposes and preliminary analysis. Although we ran across errors in the output files obtained from the recorders in Opensees during the moment frame analysis, it is fulfilling to be able to see the model completely built and the process of eigenvalue analysis and algorithms to output actual data that we can process, such as node accelerations, displacements, forces in the plastic hinges, and deformations in the plastic hinges, etc. Even with the error in the output files, we were able to post-process most of the files with Matlab scripts and will be able to achieve further analysis once the errors are realized.

The moment frame analysis work provided a basis for building models in Opensees in two dimensions, so model building in three dimensions presented a further challenge. Although most of the geometry was defined with loops, the complexity of the model was much higher than that for the concrete moment frame. Also, since the project is in design phase, many of the design details were changing while the model was being built so many of the parameters needed to be updated to reflect the changes. Working to finalize the model is still ongoing as I have run into several errors within the model and it has become tedious trying to determine what is causing them and to locate them exactly within the model. Once this process is completed, I will be able to run the model using Opensees and extract output data using the Matlab files I have written. The results will be added to the end of this paper and discussed thoroughly.

Future Research

There are no current plans for large scale testing of the special reinforced concrete moment frame although testing of the four-story prototype structure as discussed in the second portion of this paper is planned for later this Fall. The testing will be conducted at 1/3 scale at the large shaketable at UC, San Diego. Currently, modifications to the Opensees and Ansys models are being made and once results are obtained, they will be added and discussed here.
Acknowledgements

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I would like to thank Dr. Robert Fleischman, my principal investigator, for the opportunity to work on this project and for introducing me to the NEES program and allowing me to participate in undergraduate research. I would also like to give thanks to Dichuan Zhang, post doctorate and head of research staff, as well as Zhi Zhang, PhD student, for all of their guidance throughout this process and knowledge on the project to help me through my research. Lastly, I would like to acknowledge the rest of the team for their many contributions to the project: Dr. Jose Restrepo and Dr. Richard Sause (Co-principal investigators), Joseph Maffei and David Mar (Design Consultants), Giorgio Tsampras, Arpit Nema, and Steve Mintz (Graduate students), Giorgio Monti, Alessando Scodeggio, and Giuseppe Marano (International Collaborators), Austin Houk and Scott Kuhlman (REUs).

Contact Information

If there are any questions or comments regarding this research or the project, you may contact me via email at mlostra@email.arizona.edu. Thank you.
References

American Concrete Institute (2002). Building Code Requirements for Structural Concrete, Earthquake-Resistant Structures (ACI 318-323).


Appendices

Appendix A

Moment-Curvature for Beams First Story

Moment-Curvature for Beams Second Story
Appendix B

The following code is taken from Opensees model for material and section tag definition, beam and column definition, applied for RC moment frame analysis.

```plaintext
# define Materials----------------------------------------
set BeamHingeMatTag1 1;
set BeamHingeMatTag2 2;
set BeamHingeMatTag3 3;
set BeamHingeMatTag4 4;
set BeamHingeMatTag5 5;
set ColHingeMatTagI 6;
set ColHingeMatTagE 7;
set ColelasMatTag 8;
set ConninelasMatTag 9;
set ConnelasMatTag 10;
set SteelMatTag 11;
set ConnelasMatTag2 12;
set BeamHingeMatTaggf 13;
set ColHingeMatTagIgf 14;
set ColHingeMatTagEgf 15;
set ColelasMatTaggf 16;

uniaxialMaterial Steel01 $SteelMatTag $connfyval $connkval 0;
uniaxialMaterial Elastic $ConnelasMatTag2 $connksecval;
uniaxialMaterial Parallel $ConninelasMatTag $SteelMatTag $ConnelasMatTag2;
uniaxialMaterial Elastic $ConnelasMatTag $connkval;
uniaxialMaterial Pinching4 $BeamHingeMatTag1 $momb11 $rotb11 $momb12 $rotb12 $momb13 $rotb13 $momb14 $rotb14 0.85 0.85 0.05 1.0 0.0 0.0 0.0 0.88 0 0 0 0 0 0 0 0 0 0 0 0 10 "cycle";
uniaxialMaterial Pinching4 $BeamHingeMatTag2 $momb21 $rotb21 $momb22 $rotb22 $momb23 $rotb23 $momb24 $rotb24 0.85 0.85 0.05 1.0 0.0 0.0 0.0 0.88 0 0 0 0 0 0 0 0 0 0 0 0 0 "cycle";
uniaxialMaterial Pinching4 $BeamHingeMatTag3 $momb31 $rotb31 $momb32 $rotb32 $momb33 $rotb33 $momb34 $rotb34
```

27
0.85 0.85 0.05 1.0 0.0 0.0 0.0 0.88 0 0 0 0 0 0 0 0 0 0 0 0 10
"cycle";

uniaxialMaterial Pinching4 $BeamHingeMatTag4 $momb4 $rotb4 $momb42 $rotb42 $momb43 $rotb43 $momb44 $rotb44
0.85 0.85 0.05 1.0 0.0 0.0 0.0 0.88 0 0 0 0 0 0 0 0 0 0 0 10
"cycle";

uniaxialMaterial Pinching4 $BeamHingeMatTag5 $momb5 $rotb5 $momb52 $rotb52 $momb53 $rotb53 $momb54 $rotb54
0.85 0.85 0.05 1.0 0.0 0.0 0.0 0.88 0 0 0 0 0 0 0 0 0 0 0 10
"cycle";

uniaxialMaterial Pinching4 $ColHingeMatTagI $momci $rotci $momci2 $rotci2 $momci3 $rotci3 $momci4 $rotci4 0.85 0.85
0.05 1 0.0 0.0 0.0 0.88 0 0 0 0 0 0 0 0 0 0 0 10 "cycle";

uniaxialMaterial Pinching4 $ColHingeMatTagE $momce $rotce $momce2 $rotce2 $momce3 $rotce3 $momce4 $rotce4 0.85 0.85
0.05 1 0.0 0.0 0.0 0.88 0 0 0 0 0 0 0 0 0 0 0 10 "cycle";

uniaxialMaterial Elastic $ColelasMatTag [expr $Ec*$coli];

uniaxialMaterial Pinching4 $BeamHingeMatTagg $momb1g $rotb1g $momb2g $rotb2g $momb3g $rotb3g $momb4g $rotb4g
0.85 0.85 0.05 1.0 0.0 0.0 0.0 0.88 0 0 0 0 0 0 0 0 0 0 0 10 "cycle";

uniaxialMaterial Pinching4 $ColHingeMatTagIg $momci1g $rotci1g $momci2g $rotci2g $momci3g $rotci3g $momci4g $rotci4g 0.85 0.85
0.05 1 0.0 0.0 0.0 0.88 0 0 0 0 0 0 0 0 0 0 0 10 "cycle";

uniaxialMaterial Pinching4 $ColHingeMatTagEg $momce1g $rotce1g $momce2g $rotce2g $momce3g $rotce3g $momce4g $rotce4g 0.85 0.85
0.05 1 0.0 0.0 0.0 0.88 0 0 0 0 0 0 0 0 0 0 0 10 "cycle";

uniaxialMaterial Elastic $ColelasMatTagg [expr
$Ec*$coligf];

# define Sections ---------------------------------------
set BeamHingesecTag1 1;
set BeamHingesecTag2 2;
set BeamHingesecTag3 3;
set BeamHingesecTag4 4;
set BeamHingesecTag5 5;
set ColHingesecTagI 6;
set ColHingesecTagE 7;
set ColelassecTag 8;
set BeamHingesecTaggf 9;
set ColHingesecTagIgf 10;
set ColHingesecTagEgf 11;
set ColelassecTaggf 12;

section Uniaxial $BeamHingesecTag1 $BeamHingeMatTag1 Mz;
section Uniaxial $BeamHingesecTag2 $BeamHingeMatTag2 Mz;
section Uniaxial $BeamHingesecTag3 $BeamHingeMatTag3 Mz;
section Uniaxial $BeamHingesecTag4 $BeamHingeMatTag4 Mz;
section Uniaxial $BeamHingesecTag5 $BeamHingeMatTag5 Mz;
section Uniaxial $ColHingesecTagI $ColHingeMatTagI Mz;
section Uniaxial $ColHingesecTagE $ColHingeMatTagE Mz;
section Uniaxial $ColelassecTag $ColelasMatTag Mz;
section Uniaxial $BeamHingesecTaggf $BeamHingeMatTaggf Mz;
section Uniaxial $ColHingesecTagIgf $ColHingeMatTagIgf Mz;
section Uniaxial $ColHingesecTagEgf $ColHingeMatTagEgf Mz;
section Uniaxial $ColelassecTaggf $ColelasMatTaggf Mz;

#define MOMENT FRAME ELEMENTS----------------------------------------
#define 1st story beams with hinges
for {set level 1} {$level <= 1} {incr level 1} {
    for {set horiz 0} {$horiz <= [expr $bnn-1]} {incr horiz 1} {
        set elemID [expr $horiz+1+($level-1)*$bnn];
        set nodeI [expr $horiz+1+$level*($bnn+1)];
        set nodeJ [expr $nodeI+1];
        element beamWithHinges $elemID $nodeI $nodeJ
        $BeamHingesecTag1 [expr $beamd/2] $BeamHingeMatTag1 [expr $beamd/2] $Ec $beama $beami $IDTransf;
    }
}

#define base exterior columns
for {set level 1} {$level <= 1} {incr level 1} {

for {set horiz 0} {$horiz <= $bnn} {incr horiz $bnn} {
    set elemID [expr $fnn*$bnn+$horiz+1+($level-1)*($bnn+1)];
    set nodeI [expr $horiz+1+($level-1)*($bnn+1)];
    set nodeJ [expr $nodeI+$bnn+1];
    element beamWithHinges $elemID $nodeI $nodeJ
    $ColHingesecTagE [expr $cold/2] $ColelassecTag [expr $cold/2] $Ec $cola $coli $IDTransfcol;
}

#define remain exterior columns
for {set level 2} {$level <= $fnn} {incr level 1} {
    for {set horiz 0} {$horiz <= $bnn} {incr horiz $bnn} {
        set elemID [expr $fnn*$bnn+$horiz+1+($level-1)*($bnn+1)];
        set nodeI [expr $horiz+1+($level-1)*($bnn+1)];
        set nodeJ [expr $nodeI+$bnn+1];
        element elasticBeamColumn $elemID $nodeI $nodeJ $cola $Ec $coli $IDTransfcol;
    }
}

#define base interior columns
for {set level 1} {$level <= 1} {incr level 1} {
    for {set horiz 1} {$horiz <= [expr $bnn-1]} {incr horiz 1} {
        set elemID [expr $fnn*$bnn+$horiz+1+($level-1)*($bnn+1)];
        set nodeI [expr $horiz+1+($level-1)*($bnn+1)];
        set nodeJ [expr $nodeI+$bnn+1];
        element beamWithHinges $elemID $nodeI $nodeJ
        $ColHingesecTagE [expr $cold/2] $ColelassecTag [expr $cold/2] $Ec $cola $coli $IDTransfcol;
    }
}

#define remain interior columns
for {set level 2} {$level <= $fnn} {incr level 1} {
    for {set horiz 1} {$horiz <= [expr $bnn-1]} {incr horiz 1} {
        set elemID [expr $fnn*$bnn+$horiz+1+($level-1)*($bnn+1)];
        set nodeI [expr $horiz+1+($level-1)*($bnn+1)];
        set nodeJ [expr $nodeI+$bnn+1];
        element elasticBeamColumn $elemID $nodeI $nodeJ $cola $Ec $coli $IDTransfcol;
    }
}
Appendix C

Moment Frame Third Story Beam Force vs. Deformation

Moment Frame Fourth Story Beam Force vs. Deformation
Appendix D

The following code is taken from Opensees model for material and section tag definition, applied for precast concrete structure with isolation system analysis.

```python
# DEFINE MATERIALS-----------------------------------------------
set IDconcPW1SEns 1;
set IDconcPW1SMns 2;
set IDconcCoverEns 3;
set IDconcCoverMns 4;
set IDSteel1EndEns 5;
set IDSteel1EndMns 6;
set IDSteel1MidEns 7;
set IDSteel1MidMns 8;
set IDconcPW1SEew 9;
set IDconcPW1SMew 10;
set IDconcCoverEew 11;
set IDconcCoverMew 12;
set IDSteel1EndEew 13;
set IDSteel1EndMew 14;
set IDSteel1MidEew 15;
set IDSteel1MidMew 16;
set ConcslabmatTag 17;
set IDccns 18;
set IDucns 19;
set IDccew 20;
set IDucew 21;
set IDnsBRBconn 22;
set IDewBRBconn 23;
set IDnsrbaxial 24;
set IDewrbaxial 25;
set IDnsrbsheax 26;
set IDewrbssheax 27;
set IDnsrbsheary 28;
set IDewrbsheary 29;
set IDPTcsteelw 30;
set IDPTcsteelns 31;
set IDPTcinitstressSwew 32;
set IDPTcinitstressSwns 33;

# define concrete in slab
uniaxialMaterial Concrete01 $ConcslabmatTag $fc1U $eps1U $fc2U $eps2U;

# define concrete in sw base connection
uniaxialMaterial Concrete01 $IDconcPW1SEns [expr $AConcreteEndEns*$fc1CPW1S] [expr $H*$eps1CPW1S]
```
uniaxialMaterial Concrete01 $IDconcPW1SMns [expr $AConcreteEndMns*$fc1CPW1S] [expr $H*$eps1CPW1S] [expr $AConcreteEndMns*$fc2CPW1S] [expr $H*$eps2CPW1S]; # Core concrete (confined)

uniaxialMaterial Concrete01 $IDconcCoverEns [expr $AConcreteMidEns*$fc1U] [expr $H*$eps1U] [expr $AConcreteMidEns*$fc2U] [expr $H*$eps2U]; # Cover concrete (unconfined)

uniaxialMaterial Concrete01 $IDconcCoverMns [expr $AConcreteMidMns*$fc1U] [expr $H*$eps1U] [expr $AConcreteMidMns*$fc2U] [expr $H*$eps2U]; # Cover concrete (unconfined)

uniaxialMaterial Concrete01 $IDconcPW1SEew [expr $AConcreteEndEew*$fc1CPW1S] [expr $H*$eps1CPW1S] [expr $AConcreteEndEew*$fc2CPW1S] [expr $H*$eps2CPW1S]; # Core concrete (confined)

uniaxialMaterial Concrete01 $IDconcPW1SMew [expr $AConcreteEndMew*$fc1CPW1S] [expr $H*$eps1CPW1S] [expr $AConcreteEndMew*$fc2CPW1S] [expr $H*$eps2CPW1S]; # Core concrete (confined)

uniaxialMaterial Concrete01 $IDconcCoverEew [expr $AConcreteMidEew*$fc1U] [expr $H*$eps1U] [expr $AConcreteMidEew*$fc2U] [expr $H*$eps2U]; # Cover concrete (unconfined)

uniaxialMaterial Concrete01 $IDconcCoverMew [expr $AConcreteMidMew*$fc1U] [expr $H*$eps1U] [expr $AConcreteMidMew*$fc2U] [expr $H*$eps2U]; # Cover concrete (unconfined)

# define steel in sw base connection and sw section
uniaxialMaterial Hysteretic $IDSteel1EndEns $s1pBEns $e1p $s2pBEns $e2p $s3pBEns $e3p $s1nBEns $e1n $s2nBEns $e2n $s3nBEns $e3n $pinchX $pinchY $damage1 $damage2 $beta;

uniaxialMaterial Hysteretic $IDSteel1EndMns $s1pBMns $e1p $s2pBMns $e2p $s3pBMns $e3p $s1nBMns $e1n $s2nBMns $e2n $s3nBMns $e3n $pinchX $pinchY $damage1 $damage2 $beta;
uniaxialMaterial Hysteretic $IDSteel1MidEns $s1pMEns $e1p $s2pMEns $e2p $s3pMEns $e3p $s1nMEns $e1n $s2nMEns $e2n $s3nMEns $e3n $pinchX $pinchY $damage1 $damage2 $beta;

uniaxialMaterial Hysteretic $IDSteel1MidMns $s1pMMns $e1p $s2pMMns $e2p $s3pMMns $e3p $s1nMMns $e1n $s2nMMns $e2n $s3nMMns $e3n $pinchX $pinchY $damage1 $damage2 $beta;

uniaxialMaterial Hysteretic $IDSteel1EndEew $s1pBEew $e1p $s2pBEew $e2p $s3pBEew $e3p $s1nBEew $e1n $s2nBEew $e2n $s3nBEew $e3n $pinchX $pinchY $damage1 $damage2 $beta;

uniaxialMaterial Hysteretic $IDSteel1EndMew $s1pBMew $e1p $s2pBMew $e2p $s3pBMew $e3p $s1nBMew $e1n $s2nBMew $e2n $s3nBMew $e3n $pinchX $pinchY $damage1 $damage2 $beta;

uniaxialMaterial Hysteretic $IDSteel1MidEew $s1pMEew $e1p $s2pMEew $e2p $s3pMEew $e3p $s1nMEew $e1n $s2nMEew $e2n $s3nMEew $e3n $pinchX $pinchY $damage1 $damage2 $beta;

# define concrete for sw section
uniaxialMaterial Concrete01 $IDccns [expr $SWLns*$SWt*$fc1CPW1S] [expr $H*$eps1CPW1S] [expr $SWLns*$SWt*$fc2CPW1S] [expr $H*$eps2CPW1S];

uniaxialMaterial Concrete01 $IDucns [expr $SWLns*$SWt*$fc1U] [expr $H*$eps1U] [expr $SWLns*$SWt*$fc2U] [expr $H*$eps2U];

uniaxialMaterial Concrete01 $IDcccew [expr $SWLew*$SWt*$fc1CPW1S] [expr $H*$eps1CPW1S] [expr $SWLew*$SWt*$fc2CPW1S] [expr $H*$eps2CPW1S];

uniaxialMaterial Concrete01 $IDucew [expr $SWLew*$SWt*$fc1U] [expr $H*$eps1U] [expr $SWLew*$SWt*$fc2U] [expr $H*$eps2U];

# define material for PT CABLE ------------
uniaxialMaterial Steel02 $IDPTcsteelEew $FyPTcSWew $EPTcSWew $bPTcSWew $R0 $cR1 $cR2;

uniaxialMaterial Steel02 $IDPTcsteelMns $FyPTcSWns $EPTcSWns $bPTcSWns $R0 $cR1 $cR2;
uniaxialMaterial InitStrainMaterial $IDPTcinitstressSWew $IDPTcsteelew $PTcinitstressSWew;

uniaxialMaterial InitStrainMaterial $IDPTcinitstressSWns $IDPTcsteelnS $PTcinitstressSWns;

# define material for FRICTION DEVICE
uniaxialMaterial Steel01 $IDnsBRBconn $Fybrbns $Kbrb $bbrb; uniaxialMaterial Steel01 $IDewBRBconn $Fybrbew $Kbrb $bbrb;

# define material for RUBBER BEARING
uniaxialMaterial ElasticMultiLinear $IDewrbaxial strain [expr -$Ubig/$ErbaxcomSWew] 0.0 [expr $Ubig/$ErbaxtenSWew] -stress [expr -$Ubig] 0.0 [expr $Ubig];

uniaxialMaterial ElasticMultiLinear $IDnsrbaxial strain [expr -$Ubig/$ErbaxcomSWns] 0.0 [expr $Ubig/$ErbaxtenSWns] -stress [expr -$Ubig] 0.0 [expr $Ubig]; # material tags for horizontal shear

uniaxialMaterial Elastic $IDewrbshearx $ErbshearxSWew $rbdampratioSWew; uniaxialMaterial Elastic $IDnsrbshearx $ErbshearxSWns $rbdampratioSWns; # material tags for vertical shear

uniaxialMaterial Elastic $IDewrbsheary $ErbshearySWew $rbdampratioSWew; uniaxialMaterial Elastic $IDnsrbsheary $ErbshearySWns $rbdampratioSWns;

# DEFINE SECTIONS----------------------------------------
set ConcslabsecTag 1;
set nsSWccsecTag 2;
set ewSWccsecTag 3;
set nsSWucsecTag 4;
set ewSWucsecTag 5;

section ElasticMembranePlateSection $ConcslabsecTag $Ec $v $slbt $slbmd; section Fiber $nsSWccsecTag { patch rect $IDccns 10 2 [expr -$SWLns/2] [expr -$SWt/2] [expr $SWLns/2] [expr $SWt/2]; patch rect $IDSteel1EndEns 1 2 [expr -$shearGridXMns/2-4*$shearGridXEns/$FibIntEnd/2] [expr -$SWt/2] [expr -$shearGridXMns/2-3*$shearGridXEns/$FibIntEnd/2] [expr $SWt/2]; # end patch rect $IDSteel1EndMns 1 2 [expr -$shearGridXMns/2-3*$shearGridXEns/$FibIntEnd/2]
patch rect $IDSteel1EndEns 1 2 [expr -$shearGridXMns/2-1*$shearGridXEns/$FibIntEnd/2] [expr -$SWt/2] [expr -$shearGridXMns/2-0*$shearGridXEns/$FibIntEnd/2] [expr $SWt/2];

patch rect $IDSteel1MidEns 1 2 [expr $shearGridXMns/2*(-1+0/$FibIntMid)] [expr -$SWt/2] [expr $shearGridXMns/2*(-1+1/$FibIntMid)] [expr $SWt/2];

patch rect $IDSteel1MidMns 1 2 [expr $shearGridXMns/2*(-1+1/$FibIntMid)] [expr -$SWt/2] [expr $shearGridXMns/2*(-1+3/$FibIntMid)] [expr $SWt/2];

patch rect $IDSteel1MidMns 1 2 [expr $shearGridXMns/2*(-1+3/$FibIntMid)] [expr -$SWt/2] [expr $shearGridXMns/2*(-1+5/$FibIntMid)] [expr $SWt/2];

patch rect $IDSteel1MidMns 1 2 [expr $shearGridXMns/2*(-1+5/$FibIntMid)] [expr -$SWt/2] [expr $shearGridXMns/2*(-1+7/$FibIntMid)] [expr $SWt/2];

patch rect $IDSteel1MidMns 1 2 [expr $shearGridXMns/2*(-1+7/$FibIntMid)] [expr -$SWt/2] [expr $shearGridXMns/2*(-1+9/$FibIntMid)] [expr $SWt/2];

patch rect $IDSteel1MidMns 1 2 [expr $shearGridXMns/2*(-1+9/$FibIntMid)] [expr -$SWt/2] [expr $shearGridXMns/2*(-1+11/$FibIntMid)] [expr $SWt/2];

patch rect $IDSteel1MidMns 1 2 [expr $shearGridXMns/2*(-1+11/$FibIntMid)] [expr -$SWt/2] [expr $shearGridXMns/2*(-1+12/$FibIntMid)] [expr $SWt/2];

patch rect $IDSteel1MidEns 1 2 [expr $shearGridXMns/2+0*$shearGridXEns/$FibIntEnd/2] [expr -$SWt/2] [expr $SWt/2];
\$shearGridXMns/2+1*shearGridXEns/$FibIntEnd/2\] [expr
\$SWt/2];

patch rect \$IDSteellEndEns 1 2 [expr
\$shearGridXMns/2+1*shearGridXEns/$FibIntEnd/2] [expr -
\$SWt/2] [expr
\$shearGridXMns/2+3*shearGridXEns/$FibIntEnd/2] [expr
\$SWt/2];

patch rect \$IDSteellEndMns 1 2 [expr
\$shearGridXMns/2+3*shearGridXEns/$FibIntEnd/2] [expr -
\$SWt/2] [expr
\$shearGridXMns/2+4*shearGridXEns/$FibIntEnd/2] [expr
\$SWt/2];
}