Tall Building Design Criteria

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Topics

❖ Some General Observations
  • There are legitimate reasons to hate tall buildings, you do not need to invent one
  • Criticize us for what we do, not what you think we do
  • Sideway collapse (toppling) of tall buildings is very rare
  • We need to adjust our strategy and objectives for seismic design of buildings in general, PBSD of tall buildings has already taken steps in that direction

❖ A comparative look at the provisions of 2017 TBI-II and 2018 LATBSDC documents
Want to hate tall buildings?

- Hate them for their impact on the environment.
- Hate them for creating traffic congestions and unbearable population density.
- Do not hate PBS D tall buildings because you think they pose more threat to life and limb than similarly situated short buildings because evidence is to the contrary.
During an NAE sponsored resiliency workshop in 2017, I was asked these questions by very distinguished and respected individuals:

1. Why do you permit larger drift in PBSD of tall buildings compared to code?

   ANSWER: We do not.

2. In PBSD of tall buildings why do you only use response spectra and therefore ignore the duration and near-fault effects? You should use nonlinear time history analysis.

   ANSWER: We do not do the first and we routinely do the second.
Seismic Performance of Tall & Short Buildings

Does this mean that tall buildings in LA, SF and elsewhere are safe?

- No. It does not. It means that we have to look at relative seismic performance objectively and based on rational scientific criteria.
- We have quite a few tall and mid-rise pre-Northridge SMRF buildings in LA and elsewhere.
- We have an abundance of soft-story buildings.
- We have a large number of non-ductile reinforced concrete buildings.
- These are the issues that we need to deal with. Assuming that PBSD of new tall buildings poses a major risk, distracts us from concentrating on real and urgent issues that we face.
We have a problem!

- Our problem is that for more than 50 years our codes have concentrated on collapse prevention and life safety as performance objectives.
- I believe that for significant buildings and large metropolitan areas in advanced countries that is no longer a reasonable objective.
- We need to move towards repairability as the primary objectives.
- As our presentations today show repairability is an implicit objective of PEER-TBI and LATBSDC guidelines.
TBI
Tall Buildings Initiative
Guidelines for Performance-Based Seismic Design of Tall Buildings
Version 2.03
May 2017

AN ALTERNATIVE PROCEDURE FOR SEISMIC ANALYSIS AND DESIGN OF TALL BUILDINGS LOCATED IN THE LOS ANGELES REGION
A CONSENSUS DOCUMENT
2017 Edition with 2018 Supplements

March 20, 2018
Development History

Two Editions:

TBI-1: 2010
TBI-II: 2016 (V 2.03 5/17)

Revised every three years, with a supplement issued the following year:

2007, 2008
2011, 2012
2014, 2015
2017, 2018

➢ TBI and LATBSDC guidelines have never been closer in intent, objective, and content as they are today.
Both Guidelines use realistic estimates of stiffness for SLE and MCE$_R$

<table>
<thead>
<tr>
<th>Component</th>
<th>Service-Level Linear Models</th>
<th>MCE$_R$-Level Nonlinear Models</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Axial</td>
<td>Flexural</td>
</tr>
<tr>
<td>Structural walls (in-plane)</td>
<td>$1.0E_c A_g$</td>
<td>$0.75E_c I_g$</td>
</tr>
<tr>
<td>Structural walls (out-of-plane)</td>
<td>--</td>
<td>$0.25E_c I_g$</td>
</tr>
<tr>
<td>Basement walls (in-plane)</td>
<td>$1.0E_A g$</td>
<td>$1.0E_c I_g$</td>
</tr>
<tr>
<td>Basement walls (out-of-plane)</td>
<td>--</td>
<td>$0.25E_c I_g$</td>
</tr>
<tr>
<td>Coupling beams without diagonal reinforcement</td>
<td>$1.0E_A g$</td>
<td>$0.07 \left( \frac{E}{k} \right) E_c I_g$</td>
</tr>
<tr>
<td>Coupling beams with diagonal reinforcement</td>
<td>$1.0E_A g$</td>
<td>$0.07 \left( \frac{E}{k} \right) E_c I_g$</td>
</tr>
<tr>
<td>Steel Coupling Beams$^3$</td>
<td>$1.0E_A g$</td>
<td>$0.07 \left( \frac{E}{k} \right) (EI)_{tr}$</td>
</tr>
<tr>
<td>Non-PT diaphragms$^3$</td>
<td>$0.5E_A g$</td>
<td>$0.5E_c I_g$</td>
</tr>
<tr>
<td>PT diaphragms$^3$</td>
<td>$0.8E_A g$</td>
<td>$0.8E_c I_g$</td>
</tr>
<tr>
<td>Beams</td>
<td>$1.0E_A g$</td>
<td>$0.5E_c I_g$</td>
</tr>
<tr>
<td>Columns</td>
<td>$1.0E_A g$</td>
<td>$0.7E_c I_g$</td>
</tr>
<tr>
<td>Mat (in-plane)</td>
<td>$0.8E_A g$</td>
<td>$0.8E_c I_g$</td>
</tr>
<tr>
<td>Mat (out-of-plane)</td>
<td>--</td>
<td>$0.8E_c I_g$</td>
</tr>
</tbody>
</table>
Coupling Beam Stiffness (Diagonal)

- Test database (56)
  - BRI 12-Story (8)
  - Naish (7)
  - Hwang (4)
  - Lequesne (3)
  - Fortney (2)
  - Canbolat (1)
  - Dogus (1)
  - Zhou (1)
  - Binney (3)
  - Barney (2)
  - Tassios (2)
  - Kwan & Zhou (1)
  - Galano & Vignoli (7)
  - Kimura (10)
  - Adebar (1)
  - Penelis (1)
  - Sanobe (2)

\[ EI_{eff} / EI_g = 0.07 \left( l_n / h \right) \]

\[ GA = 0.4E_c A \]

Test Data
-0.04 + 0.085(ln/h)

\[ EI_{eff} = 0.07(l_n/h) \]

Slide courtesy of John Wallace
Coupling Beam Stiffness (Conventional)

- Test database (36)
  - Hwang (4)
  - Naish (1)
  - Zhu (1)
  - Zhou (2)
  - Brena (4)
  - Kwan & Zhao (9)
  - Galano & Vignoli (4)
  - Bristowe (4)
  - Barney (3)
  - Binnay (2)
  - Tassios (2)
  - Paulay (12) [outliers]

Test Data:

$$EI_{eff} / EI_g = 0.07(l_n/h) \leq 0.3$$

$$GA = 0.4E_cA$$

Slide courtesy of John Wallace
Coupling Beam Stiffness (SRC)

- Steel-Reinforced RC (Motter et al, ASCE, 2016)
  - Embedment Length
  - V_n & M_n
  - Detailing
  - Stiffness
  
  \[
  (EI)_{\text{eff}} = 0.06 \left( \frac{l_n}{h} \right) E_s I_{\text{trans}}
  \]

- PEER TBI 2.0 and LATBSDC Recommendation

  \[
  EI_{\text{eff}} = 0.07 \left( \frac{l_n}{h} \right) E_s I_{\text{trans}}
  \]

  \[
  GA = 0.4E_c A
  \]

Slide courtesy of John Wallace
Both Documents define the maximum permitted damping in analysis the same way:

\[ \zeta_{\text{critical}} = \frac{0.36}{\sqrt{H}} \leq 0.05 \]

- \( H \) = Building height in feet

2.5% floor for \( MCE_R \)

MCE\(_R\)

SLE
A minimum of 11 pairs of ground motions required by both documents

LATBSDC basically the same as in PEER TBI-II and ASCE 7-16 with the following exceptions:

◦ Vertical accelerations to be specifically incorporated in $MCE_r$ evaluations where significant.
◦ Penalties modified in the commentary:

LATBSDC Commentary: ASCE 7-16 requires that the target spectrum be increased by 110% if spectral matching of the selected seed ground motions is used instead of scaling of the ground motions. The increase of 110% need not be applied if the ground motions are spectrally matched to the target spectrum if it is based on a uniform hazard spectrum. The increase of 110% should be applied if spectral matching is used to match time histories to target scenario spectra.
Vertical Ground Motion Effects

- Both documents require explicit simulation of vertical earthquake response where there are significant discontinuities in the vertical-load-carrying elements.

- PEER TBI-II utilizes ASCE 7’s $\pm 0.2S_{MS}D$ for modeling effect of vertical accelerations. 
  - **Exception:** No need for $\pm 0.2S_{MS}D$ when explicit vertical response analysis is performed, and the effects of vertical response analysis are included.

- LATBSDC does not utilize $\pm 0.2S_{MS}D$.

- In these cases, vertical masses (based on the effective seismic weight) shall be included with sufficient model discretization to represent the primary vertical modes of vibration in the analysis model used to simulate vertical response.
Classification of Structural Actions

Contrary to PEER-TBI and ASCE 7-16, LATBSDC classifies force-controlled actions into two, not three, categories (i.e., non-critical classification is eliminated.

- **Force-controlled action** – An action that is not expected to undergo nonlinear behavior in response to earthquake shaking, and which is evaluated on the basis of its available strength.
  - **Critical action** – A force-controlled action, the failure of which is likely to lead to partial or total structural collapse.
  - **Ordinary action** – A force-controlled action, the failure of which the failure of which is unlikely to lead to structural collapse or it might lead to local collapse comprising not more than one bay in a single story.
ASCE 41-06 (ASCE 2006) clearly distinguished the capping point of the backbone curve as the point corresponding to the Primary Collapse Prevention acceptance criteria.

That distinction is not present with clarity in the later editions of ASCE-41 (ASCE 2013, 2017).

For this reason, the proper use of ASCE-41 backbones are clarified in LATBSDC.

LATBSDC uses average of maxima of M$C_{IR}$ responses for these evaluations.
LATBSDC Exception: Larger values may be used only if substantiated by appropriate laboratory tests or approved by the peer review process.

If larger values are used, and response values for any ground motion indicate that the larger values used are exceeded, then: (a) strength degradation, stiffness degradation and hysteretic energy dissipation appropriate for these larger values shall be considered, and (b) base shear capacity of the structure that considers element strength degradation shall not fall below 90% of the base shear capacity prior to the initiation of strength degradation in any element. Coupling beams in special reinforced concrete shear walls provide an example of where this exception may be applied.”
TBI-II evaluates results obtained from each MCER analysis individually (TBI-II, Sec. 6.8.2)

6.8.2 Deformation-Controlled Actions

If the ultimate deformation capacity ($\delta_u$) associated with any mode of deformation in a component is exceeded in any of the response history analyses, it is permitted either to:

1. Assume the strength associated with this mode of deformation is negligible for the remainder of that analysis and evaluate the stability of the structure and the effects on related strength quantities, or,

2. Consider the analysis to have unacceptable response.

For this purpose, $\delta_u$ shall be taken as the valid range of modeling as demonstrated by comparison of the hysteretic model with suitable laboratory test data or as described in Chapter 4.

We will examine the ramification of “unacceptable response” classification later in this presentation.
LATBSDC $MCE_R$ Evaluations

Force-Controlled Actions:

(a) Critical Actions

Force-controlled critical actions shall satisfy either Evaluate adequacy in accordance with either Equation (5a) or Equation (5b):

$$1.0I_e Q_{ns} + 1.3I_e (Q_T - Q_{ns}) \leq \phi_n BR_n$$  \hspace{1cm} (5a)

$$1.0I_e Q_{ns} + 1.5I_e (Q_T - Q_{ns}) \leq \phi_{nem} BR_{nem}$$  \hspace{1cm} (5b)

(b) Ordinary Actions

Force-controlled critical actions shall satisfy either Evaluate adequacy in accordance with either Equation (6a) or Equation (6b):

$$1.0I_e Q_{ns} + 0.9I_e (Q_T - Q_{ns}) \leq \phi_n BR_n$$  \hspace{1cm} (6a)

$$1.0I_e Q_{ns} + I_e (Q_T - Q_{ns}) \leq \phi_{nem} BR_{nem}$$  \hspace{1cm} (6b)

EXCEPTION: For buildings located in the Los Angeles region if the serviceability acceptance criteria are satisfied per requirements of Section 3.5.5.1, then $I_e$ may be taken as 1.0.
LATBSDC $MCE_R$ Evaluations

Force-Controlled Actions:

**C.3.6.3.2.1.** For force-controlled actions are limited by a well-defined yield mechanism, the adequacy may be evaluated using the following equations:

\[ 1.2D + 1.0L \pm 0.2S_{MS}D + E_M \leq \phi_s R_n \quad (C-1) \]
\[ 0.9D + 0.2S_{MS}D + E_M \leq \phi_s R_n \quad (C-2) \]

In regions of high seismicity such as Los Angeles the serviceability criterion generally controls the design of deformation-controlled actions. Therefore, if for the serviceability evaluation, higher design forces are used for deformation-controlled actions as required per Section 3.5.5.1, this would automatically result in higher strength for force-controlled actions via the capacity design requirements enforced at the collapse prevention evaluation and therefore the use of risk reduction factor for force-controlled actions will not be necessary.
LATBSDC provides the values of $B$, $\phi_s$ and rationale for values in Appendices.

<table>
<thead>
<tr>
<th>Component</th>
<th>Seismic Action</th>
<th>Classification</th>
<th>$A$</th>
<th>$B$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Below Grade Perimeter Walls</td>
<td>Flexure</td>
<td>Ordinary</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Ordinary</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td>Below Grade Non Perimeter / Non-Core Walls</td>
<td>Shear</td>
<td>Ordinary</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td>Core Walls Above and Below Grade and Above Grade Walls</td>
<td>Shear</td>
<td>Critical</td>
<td>0.75</td>
<td>1.35</td>
</tr>
<tr>
<td>Diaphragm with Major Shear Transfer</td>
<td>Flexure</td>
<td>Ordinary</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Critical</td>
<td>0.75</td>
<td>1.35</td>
</tr>
<tr>
<td>Typical (non-transfer slab) Diaphragm Forces (excludes columns and shear transfer to vertical element)</td>
<td>Axial (includes chord forces)</td>
<td>Ordinary</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Flexure</td>
<td>Ordinary</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Ordinary</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td>Drag (Collector) Members</td>
<td>Compression</td>
<td>Critical</td>
<td>0.65</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Tension</td>
<td>Critical</td>
<td>0.65</td>
<td>1.0</td>
</tr>
<tr>
<td>Vertical Element-to-Diaphragm Connection</td>
<td>Bearing</td>
<td>Critical</td>
<td>0.65</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Shear Transfer (Shear Fraction)</td>
<td>Critical</td>
<td>0.75</td>
<td>1.0</td>
</tr>
<tr>
<td>Gravity Columns and Special Moment Frames (Beams, Columns, Beam-Column joints) excluding, Intentional Outrigger Columns, &amp; Columns Supporting Discontinuous Vertical Elements</td>
<td>Axial</td>
<td>Critical</td>
<td>0.65</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Critical</td>
<td>0.75</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Flexure (in Axial Flexure Combinations)</td>
<td>Ordinary</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td>Intentional Outrigger Columns &amp; Columns Supporting Discontinuous Vertical Elements*</td>
<td>Axial</td>
<td>Critical</td>
<td>0.65</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Critical</td>
<td>0.75</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Flexure (in Axial Flexure Combinations)</td>
<td>Ordinary</td>
<td>0.9</td>
<td>1.0</td>
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<tr>
<td>Transfer Girders*</td>
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<td>Critical</td>
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<td>1.0</td>
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<td></td>
<td>Shear</td>
<td>Critical</td>
<td>0.75</td>
<td>1.0</td>
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<tr>
<td>Foundations</td>
<td>Flexure</td>
<td>Ordinary</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Shear (with $A_v$)</td>
<td>Critical</td>
<td>0.75</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td>Shear (w/o $A_v$)</td>
<td>Critical</td>
<td>0.75</td>
<td>1.00</td>
</tr>
<tr>
<td>Foundation Piles (Structural Capacity)</td>
<td>Compression</td>
<td>Critical</td>
<td>0.65</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Tension</td>
<td>Ordinary</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Flexure</td>
<td>Ordinary</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Critical</td>
<td>0.75</td>
<td>1.0</td>
</tr>
</tbody>
</table>

* Effects of vertical acceleration shall be considered.
** $A_v$ - Shear reinforcement

ACI 318-14 Simplified ACI 318-14 Detailed

<table>
<thead>
<tr>
<th>RC no Av</th>
<th>RC with Av</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>COV</td>
</tr>
<tr>
<td>1.51</td>
<td>0.38</td>
</tr>
<tr>
<td>1.25</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Data and images courtesy of John Wallace.
Unacceptable Response

- Unacceptable response to ground motion is defined as:
  1. Analytical solution fails to converge,
  2. Predicted demands on deformation-controlled or force-controlled elements exceed the valid range of modeling, or
  3. Predicted deformation demands on elements not explicitly modeled exceed the deformation limits at which the members are no longer able to carry their gravity loads.

- PEER-TBI-II permits
  a. Not more than one unacceptable response in a suite of 11 ground motions for Risk Category II buildings
  b. Not more than one unacceptable response in a suite of 20 ground motions for Risk Category III buildings

- LATBSDC does not permit any unacceptable response
Modeling criteria, unacceptable response and acceptance criteria

- Modeling criteria in PEER-TBI-II contains more details than LATBSDC
- Incorporation of modeling criteria and unacceptable response has created some confusion among engineers
- These, in my opinion, should not be confused with acceptance criteria
- For example $15A_{cv}\sqrt{f_{ce}}'$ is a modeling limit. Acceptance criteria is as defined in material code (i.e., Chapter 18 of ACI 318-14).

For structural concrete walls and diaphragms having height-to-length ratio $h_{w}/\ell_{w} \geq 2$, with calculated concrete compressive strain $\bar{\varepsilon}_c \leq 0.005$ and calculated longitudinal reinforcement tensile strain $\bar{\varepsilon}_t \leq 0.01$ at all points in a cross section, limit the expected shear strength of the wall to:

$$V_n = A_{cv} \left( 2\lambda \frac{f_{ct}}{f_{ce}} + \rho A f_{ye} \right) \leq 10A_{cv} \sqrt{f_{ce}}$$

$$V_{ne} = 1.5A_{cv} \left( 2\lambda \frac{f_{ct}}{f_{ce}} + \rho A f_{ye} \right) \leq 15A_{cv} \sqrt{f_{ce}}$$

Where $f_{ct}$ and $f_{ye}$ are the expected material strengths and $\bar{\varepsilon}_c$ and $\bar{\varepsilon}_t$ are the mean values of the maximum strain values determined from the 11 or more ground motions considered. Therefore,

$$\frac{V_{ne}}{V_n} = 1.5$$

$$B = 0.9 \frac{V_{ne}}{V_n} = 1.35$$
Seismic Instrumentation Requirements

- LATBSDC requires that every building designed according to its provisions to be extensively instrumented.
- It provides detailed guidelines for such instrumentation.
- PEER-TBI-II adopts LATBSDC instrumentation provisions in Appendix A to be used when “required by the Authority Having Jurisdiction or desired for other reasons”
Independent Peer Review Requirements

- Both documents contain detailed seismic peer review requirements

Scope of peer review:

1) Basis of Design document, including the seismic performance objectives, the overall seismic design methodology, and acceptance criteria;

2) Proposed structural system and materials of construction;

3) Earthquake hazard determination, and selection and modification of earthquake ground motions for application to the building model;

4) Modeling approaches for structural materials and components;

5) Structural analysis model, including soil–foundation–structure interaction as applicable, and including verification that the structural analysis model adequately represents the properties of the structural system within accepted norms for tall building designs;

6) Review of structural analysis results and determination of whether calculated response meets approved acceptance criteria;

7) Design and detailing of structural components;

8) Drawings, specifications, and quality control/quality assurance and inspection provisions in the design documents; and

9) Any other considerations that are identified as being important to meeting the established performance objectives.
Thank you!
Seismic Performance of Tall & Short Buildings

SCEC Puente Hills Simulations
Seismic Performance of Tall & Short Buildings

SCEC Puente Hills Simulations

![Graph showing seismic performance of tall and short buildings with various readings and waveforms.]
**Seismic Performance of Tall & Short Buildings**

**Naeim & Graves** Typical Building Types Considered to Measure Ductility Demands Imposed by a Postulated $M_W$ 7.15 Puente Hills Earthquake

<table>
<thead>
<tr>
<th>Building Type</th>
<th>Fundamental Period (sec.)</th>
<th>Yield Seismic Base Shear Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-Story Braced Frame System</td>
<td>0.30</td>
<td>0.20</td>
</tr>
<tr>
<td>2-Story Shear Wall System</td>
<td>0.30</td>
<td>0.40</td>
</tr>
<tr>
<td>6-Story Moment Frame System</td>
<td>1.0</td>
<td>0.20</td>
</tr>
<tr>
<td>10-Story Shear Wall System</td>
<td>1.0</td>
<td>0.40</td>
</tr>
<tr>
<td>12-Story Moment Frame System</td>
<td>2.0</td>
<td>0.20</td>
</tr>
<tr>
<td>25-Story Moment Frame System</td>
<td>4.0</td>
<td>0.10</td>
</tr>
<tr>
<td>54-Story Moment Frame System</td>
<td>6.0</td>
<td>0.05</td>
</tr>
</tbody>
</table>
Seismic Performance of Tall & Short Buildings

Variation of Ductility Demand for Whittier N-S Records

Ductility Demand

2-Story Braced Frame 2-Story Shear Wall 6-Story Frame 10-Story Shear Wall 12-Story Frame 25-Story Frame 54-Story Frame
How about sideway collapse of tall buildings?

- There is no doubt that sideway collapse of a tall building poses a major hazard and it can render a many blocks and neighborhood unsafe and cause serious casualties.

- Sideway collapse of tall buildings, however, is very rare.

- Consider the case of Alto Rio building during the 2010 Chile earthquake
How about sideway collapse of tall buildings?

Figure 1. Preliminary coseismic displacement indicating a movement of 303.9 cm at Concepción

From: Alimoradi & Naeim, 2010
How about sideway collapse of tall buildings?

Figure 2. GPS Station CONZ, 27 February 2010 recording indicating a maximum displacement of about 3 m, 2.8 m of which were attained during the first 25 s of the ground motion (modified from GEO’s Chile Event Supersite Website, 2010)

Figure 4. Dynamic equilibrium of a rigid block (after Makris and Roussos, 1998)

From: Alimoradi & Naeim, 2010
How about sideway collapse of tall buildings?

From: Alimoradi & Naeim, 2010