Mitigating of Progressive Collapse in Structures

Day 2 - 7 Hour Course

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Presenters

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- Member of Disproportionate Collapse Technical Committee

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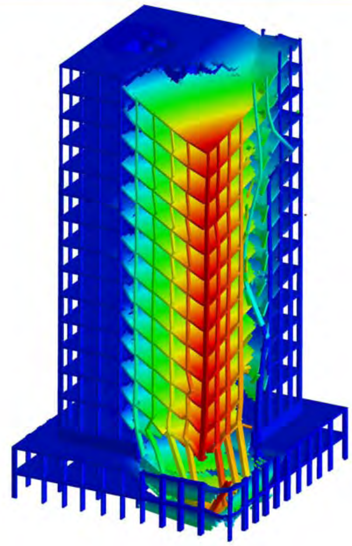
Morning Outline

1. **Definition of Progressive Collapse**
2. **Progressive Collapse Historical Events & Evolution of Code Regulations**
3. **Historic Progressive Collapse Case Studies**
4. **Design Philosophy for Progressive Collapse Mitigation in Codes and Standards**



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What is progressive collapse?

Mizan Dogan
Department of Civil Engineering, Eskişehir Osmangazi University, TURKEY

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Definition of Progressive Collapse

- **Term is debated with respect to specific historical events**
- **Was the collapse of Oklahoma City Federal Building progressive incident?**
- **Was the collapse of WTC towers progressive collapse?**
- **Other terms used to address the same phenomenon**
  - Disproportionate collapse
  - Widespread collapse

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Definition of Progressive Collapse Continued

- **British Standards, Eurocode and ASCE 7, use term "disproportionate":**
  - BS "the building shall be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause"
  - Eurocode 1 Part 1-7 General Actions: "a structure shall be designed and executed in such a way it will not be damaged by events like fire, explosion, impact, or consequences of human errors, to an extent disproportionate to the original cause"
- **ASCE 7 Sec. 1.4 "buildings and other structures shall be designed to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage"**

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Definition of Progressive Collapse Continued

- **NBC of Canada uses term "widespread ":**
  - NBC of Canada Sec. 4.1.1.3 (1) "buildings …. Shall be designed to have sufficient structural capacity and structural integrity …"
  - "structural integrity is defined as the ability of the structure to absorb local failure without widespread collapse"

Definition of Progressive Collapse Continued

"A collapse that is triggered by localized damage that can't be contained and leads to a chain of failures resulting in a partial or total structural collapse, where the final damage is disproportionate to the local damage of the triggering event"

- **Why collapse can't be contained?**
- Answer: Lack of -
  - Continuity of tension ties (even in compression members)
  - Redundancy of load path
  - Ductility

Why Are Code Provisions for PC Necessary?

- **Is it because of recent terrorist incidents?**
- **Older structures were able to resist some extreme loads due to the inherent strength, continuity, and redundancy in its structural systems.**
- **Recent structures have used optimization and refinement techniques that reduce their margin of safety**
- **Framing systems designed:**
  - for ease of construction
  - have lower level of inherent continuity
  - have lower level of redundancy

Definitions of Related Terms

- **Deformation-Controlled Action:** A deformation-controlled action provides a resistance that is proportional to the imposed deformation until the peak strength is reached, after which the resistance remains at a significant level, as the deformation increases.

Definitions of Related Terms

- **Force-Controlled Action:** A force-controlled action provides a resistance that is proportional to the imposed deformation until the peak strength is reached, after which the resistance drops to zero.

Definitions of Related Terms

- **Yield Rotation:** The yield rotation θy corresponds to the flexural rotation at which the extreme fibers of the structural elements reach their yield capacity fy.

$$\text{Beams:} \quad \theta_y = \frac{ZF_{ye}l_b}{6EI_b}$$

$$\text{Columns:} \quad \theta_y = \frac{ZF_{ye}l_c}{6EI_c}\left(1 - \frac{P}{P_{ye}}\right)$$

DoD UFC 4-023-03, 2005, “Design of Buildings to Resist Progressive Collapse”

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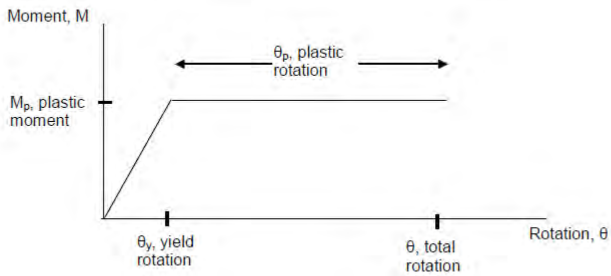
Definitions of Related Terms

- **Plastic Rotation:** The plastic rotation θp is the inelastic or non-recoverable rotation that occurs after the yield rotation is reached and the entire cross section has yielded



DoD UFC 4-023-03, 2005, "Design of Buildings to Resist Progressive Collapse"

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Historic Progressive Collapse Case Studies

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Ronan Point 1968

Figure 1–1 Ronan Point collapse: a gas explosion on the 18th floor resulted in a progressive collapse

Ronan Point: Lessons Learned

- Ronan Point lacked the connection details necessary to effectively redistribute load.
- Explosion was small and pressure less than 10 psi
- Kitchen and living room walls would fail at a pressure ~ 11.7 kPa.
- Exterior wall would fail at a pressure ~20.7 kPa (3 psi). Collapse was attributed to lack of structural integrity.
- No alternate load path for redistribution of forces.
- Strong winds and/or effects of a fire could have caused progressive collapse.
- Building had been built with very poor workmanship.

Ronan Point: References

- Pearson C., and Delatte N., 2005, The Ronan Point Apartment Tower Collapse and its Effect on Building Codes, J. Perf. Constr. Fac. Volume 19, Issue 2, pp. 172-177.
- HMSO, "Report of the inquiry in the Collapse of Flats at Ronan Point, Canning Town," 1968.
- Rouse C., and Delatte N., "Lessons from the Progressive Collapse of the Ronan Point Apartment Tower," Forensic Engineering: Proceedings of the Third Congress, pp. 190 – 200, Bosela, Paul A., Delatte, Norbert J., and Rens, Kevin L., Editors, ASCE, October 19 – 21, 2003.
- McGuire W. The Why of General Structural Integrity, "," Engineering Foundation Conference, Structural Failures II. Palm Coast, Florida, December 1987.

A.P. Murrah Federal Building 1995

Alfred P. Murrah Federal Building, Oklahoma City, Oklahoma (office facility for U.S. government).

Truck bomb detonated in front of its north side.

4000 pounds of explosives close to central supporting column

Extensive structural damage to building

Fig. 1—Murrah Building Prior to Blast.

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A.P. Murrah Building: Damage

[69] STAROSSEK U., 2006, Progressive Collapse of Bridges—Aspects of Analysis and Design, Invited Lecture, International Symposium on Sea-Crossing Long-Span Bridges, Mokpo, Korea, Feb. 15-17.

A.P. Murrah Building: Damage

- **Extent of Collapse**
  - Destroyed due to blast
    - Columns G16, G20 and G24.
  - Subsequent collapse due to failed columns
    - Third floor transfer girders between G16 and G26.
    - All floors and roof panels bounded by column lines 12, 28, F and G.
  - Total Building Floor Area: 137,800 ft 2
    - 4% (5,850 ft 2) destroyed by blast.
- **42% (58,100 ft 2) destroyed by blast + progressive collapse.**

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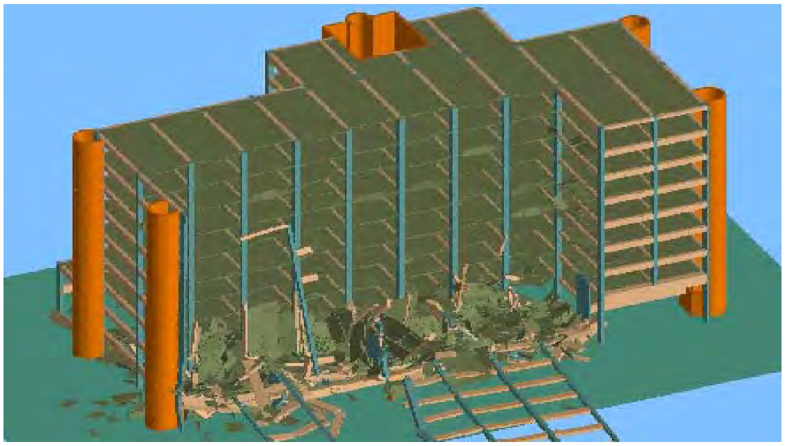
Extreme Loading® Technology



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A.P. Murrah Building: References

- Corley W. G., Mlakar P. F., Sozen M. A., and Thornton C. H. "The Oklahoma City Bombing: Summary and Recommendations for Multihazard Mitigation", Journal of Performance of Constructed Facilities, 12(3), 1998, P 100-112.
- Corley W. G., Mlakar P. F., Sozen M. A., and Thornton C. H. "The Oklahoma City Bombing: Structural Details and Possible Mechanisms for the Murrah Building", Journal of Performance of Constructed Facilities, 12(3), 1998, P 120-136.
- FEMA 277 "The Oklahoma City Bombing: Improving Building Performance Through Multi-Hazard Mitigation," Federal Emergency Management Agency and ASCE, August 1996.
- Hinman E. E. and Hammond D. J. "Lessons from the Oklahoma City Bombing: Defensive Design Techniques." 1997 ASCE Press.
- Hinman, H. (1997). Lessons from the Oklahoma City Bombing: Defensive Design Techniques, ASCE, New York.
- Corley, W. Gene; Sozen, Mete A.; Thornton, Charles H.; and Mlakar, Paul F., "The Oklahoma City Bombing: Improving Building Performance Through Multi-Hazard Mitigation," FEMA-277, Mitigation Directorate, August 1996, U.S. Government Printing Office, Washington, DC, 90 pp.
- Corley, W. G., Smith, R.G., and Colarusso, L. J., "Structural integrity and the Oklahoma City bombing," Concrete Construction, A Hanley-Wood Publication, Addison, Illinois, December 2001, Vol. 46, No. 12, pp. 29-30.

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Khobar Towers - 1996

A complex of numerous apartment buildings in Al-Khobar near Dhahran, Saudi Arabia.

One of apartment buildings was extensively damaged and others were seriously damaged.

Massive bomb was detonated in roadway that passed in front of building.

Khobar Towers : Structure Description

- The most heavily damaged building was eight stories tall, and T-shaped in plan.
- Constructed of precast concrete wall and floor components.
- Vertical and lateral loads were carried by wall systems.
- British concrete design code (CP-110) used.

Khobar Towers : Damage

Side of building facing the explosion

[59] Ellingwood B. R., Smilowitz R., Dusenberry D. O., Duthnh D., Lew H. S., Carino N. J., 2006, Best practices for reducing the potential for progressive collapse in buildings, NISTIR 7396.

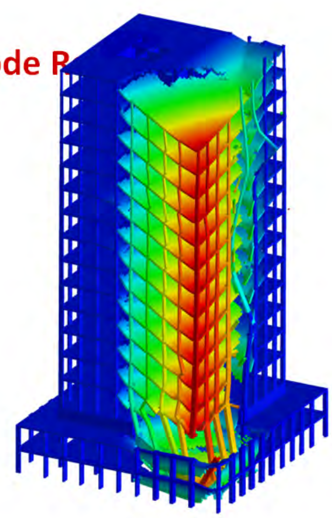
Khobar Towers : References

- **Ellingwood B. R., Smilowitz R., Dusenberry D. O., Duthnh D., Lew H. S., Carino N. J., 2006, Best practices for reducing the potential for progressive collapse in buildings, NISTIR 7396.**

Disproportionate Collapse Codes

Seminar Outline

- Progressive Collapse Historical Events & Evolution of Code R
- Design Philosophy for Progressive Collapse Mitigation
- Overview of current codes



Some Concepts in Progressive Collapse Design

- Threat-independent hazard scenarios.
- Threat-dependent hazard scenarios.

Current US Progressive Collapse Code Provisions

- **Direct Design Methods**
  - Alternate path method
    - Local element failure is permitted (removing a column) and structure must be able to bridge across the removed element
  - Specific local resistance method
    - Design all structural elements for a specific threat or abnormal load
- **Indirect Design Methods**
  - Tie force method
    - Achieve minimum level of strength, continuity and ductility
    - Activate "catenary" response of the structure

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Concept of Sacrificial Column

- **The whole structure is designed to sustain the loss of any of its columns**
- **Each column is only designed to resist an explosion at a reasonable distance equal to the column spacing.**

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Current Progressive Collapse Code Provisions

- ASCE/SEI 7-10, "Minimum Design Loads for Buildings and Other Structures", Sec. 1.4, Structural Integrity
- ACI 318-11, "Building Code Requirements for Structural Concrete and Commentary", Sec. 7.13, Structural Integrity Reinforcement Requirements
- 2009 IBC, "International Building Code", Sec. 16.14 on Structural Integrity and 16.15 on Risk Assessment
- GSA 2013, "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects"
- DoD UFC 4-023-03, 2009, change 3-2016, "Design of Buildings to Resist Progressive Collapse"
- International Building Code (IBC 2009) Section 16.14 on Structural Integrity
- Eurocode 1 – Actions on Structures, Part 1-7, Accidental actions, 2006.

ASCE/SEI 7-10

Minimum Design Loads for Buildings and Other Structures, Sec. 1.4, Structural Integrity

ASCE/SEI 7-10

1.4.1 Load Combinations of Integrity Loads

The notional loads, *N*, specified in Sections 1.4.2 through 1.4.5 shall be combined with dead and live loads in accordance with Section 1.4.1.1 for strength design and 1.4.1.2 for allowable stress design.

1.4.1.1 Strength Design Notional Load Combinations

a. $1.2D + 1.0N + L + 0.2S$
b. $0.9D + 1.0N$

1.4.2 Load Path Connections

All parts of the structure between separation joints shall be interconnected to form a continuous path to the lateral force-resisting system, and the connections shall be capable of transmitting the lateral forces induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements having strength to resist a force of not less than 5% of the portion's weight.

ASCE/SEI 7-10

indicated in Section 1.4.2. All members of the structural system shall be connected to their supporting members in accordance with Section 1.4.3.

ously. For purposes of analysis, the force at each level shall be determined using Eq. 1.4-1 as follows:

$$F_x = 0.01\ W_x \qquad (1.4\text{-}1)$$

where

F_x = the design lateral force applied at story *x* and
W_x = the portion of the total dead load of the structure, *D*, located or assigned to level *x*.

Structures explicitly designed for stability, including second-order effects, shall be deemed to comply with the requirements of this section.

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Structural walls shall be anchored to diaphragms and supports in accordance with Section 1.4.4. The effects

also be connected to the diaphragm. The connection shall have the strength to resist a force of 5 percent of the unfactored dead load plus live load reaction imposed by the supported member on the supporting member.

1.4.5 Anchorage of Structural Walls

Walls that provide vertical load bearing or lateral shear resistance for a portion of the structure shall be anchored to the roof and all floors and members that provide lateral support for the wall or that are supported by the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting a strength level horizontal force perpendicular to the plane of the wall equal to 0.2 times the weight of the wall tributary to the connection, but not less than 5 psf (0.24 kN/m^2).

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ASCE/SEI 7-10

2.5.2.1 Capacity

For checking the capacity of a structure or structural element to withstand the effect of an extraordinary event, the following gravity load combination shall be considered:

$$(0.9 \text{ or } 1.2)D + A_k + 0.5L + 0.2S \qquad (2.5\text{-}1)$$

in which A_k = the load or load effect resulting from extraordinary event A.

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2009 IBC

International Building Code, Sec. 16.14 on Structural Integrity and 16.15 on Risk Assessment

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IBC 2009

- **Added 2 new sections 16.14 on Structural Integrity and 16.15 on Risk Assessment**
- **Sec. 16.14 applies to buildings Occupancy Category II, III, and IV and more than 3 stories above grade**
-

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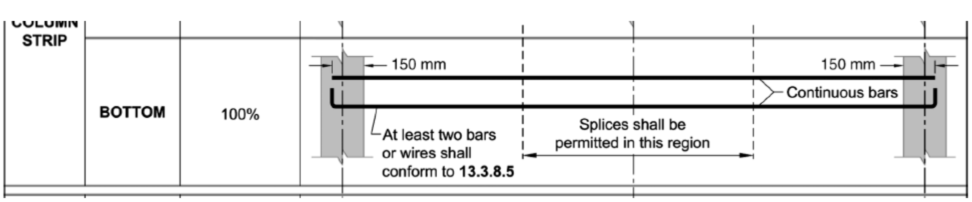
ACI 318-11

Building Code Requirements for Structural Concrete and Commentary, Sec. 7.13, Structural Integrity Reinforcement Requirements

ACI 318-11

13.3.8.5 — All bottom bars or wires within the column strip, in each direction, shall be continuous or spliced with Class B tension lap splices or with mechanical or welded splices satisfying 12.14.3. Splices shall be located as shown in Fig. 13.3.8. At least two of the column strip bottom bars or wires in each direction shall pass within the region bounded by the longitudinal reinforcement of the column and shall be anchored at exterior supports.

R13.3.8.5 — The continuous column strip bottom reinforcement provides the slab some residual ability to span to the adjacent supports should a single support be damaged. The two continuous column strip bottom bars or wires through the column may be termed "integrity steel," and are provided to give the slab some residual strength following a single punching shear failure at a single support.[13.9] In the 2002 Code, mechanical and welded splices were explicitly recognized as alternative methods of splicing reinforcement.



Horizontal Ties

ACI 318 13.3.8.5

- At least two of the column strip bottom bars in each direction shall pass within the column core and shall be anchored at the exterior supports.

New York City §28.2-1917.2.3

- At each column, provide bottom reinforcement that can develop a tension equal to the maximum of:
  - 3x the self weight of the structure entering the column at that level
  - 1.5x the load entering the column at that level using the load combination 1.2D+1.6L or 1.4D

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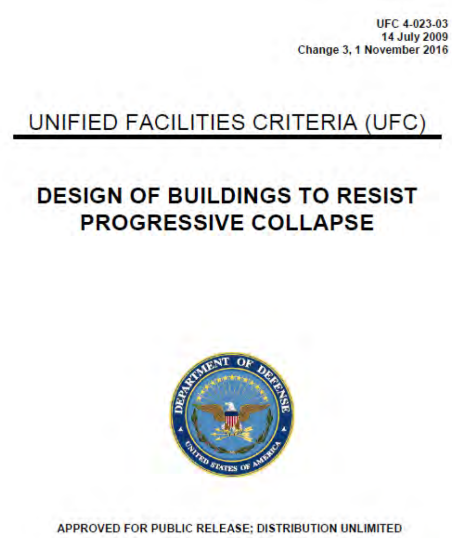
Department of Defense Requirements

- **Unified Facilities Criteria**
- **(UFC) 4-023-03**


UFC 4-023-03
14 July 2009
Change 3, 1 November 2016

UNIFIED FACILITIES CRITERIA (UFC)

DESIGN OF BUILDINGS TO RESIST PROGRESSIVE COLLAPSE

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Occupancy Categories and Design Requirements

Occupancy Category	Design Requirement
I	No specific requirements
II	Option 1: Tie Forces for the entire structure and Enhanced Local Resistance for the corner and penultimate columns or walls at the first story. **OR** Option 2: Alternate Path for specified column and wall removal locations.
III	Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first story columns or walls.
IV	Tie Forces; Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first and second story columns or walls.

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Occupancy Categories

OC	Nature of Occupancy
I	Minor storage facilities-fewer than 11 DoD personnel
II	less than 50 personnel - high occupancy family housing
III	more than 300 people school, or daycare facilities with an occupant load greater than 250 water treatment facilities Storage of toxic, flammable, or explosive substances
IV	Hospitals, Fire, rescue, and police stations, and emergency vehicle garages Power-generating stations Key national defense assets nuclear, chemical, biological, or radiological facilities

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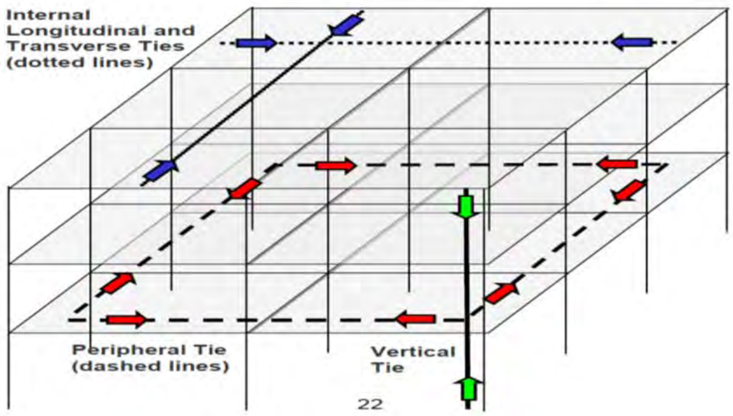
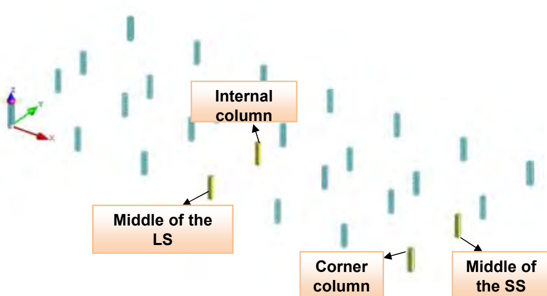
Tie Force Requirements



The design tie strength **with no other loads acting** must be greater than or equal to the required tie strength

Alternate Path Method



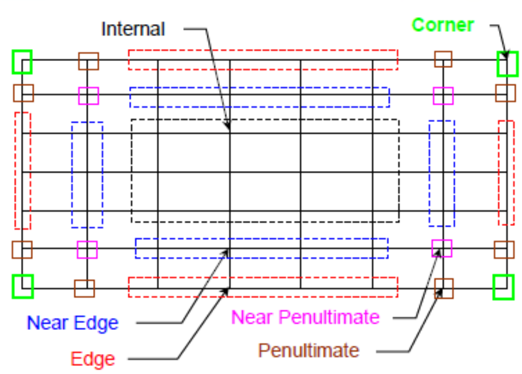
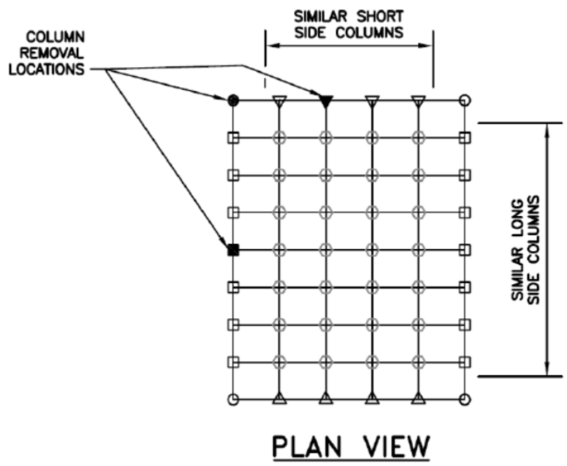


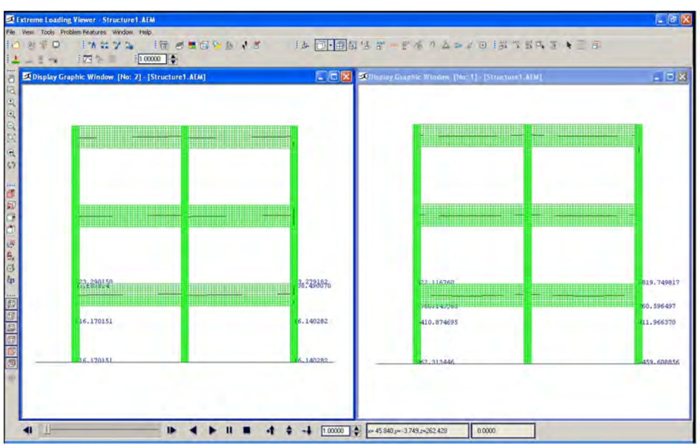
Figure C-2. Column Removal Locations

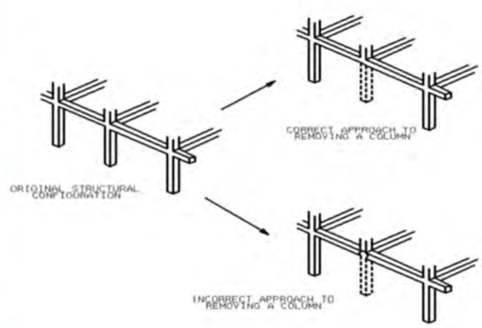
Location of Removed Load-Bearing Columns

- **Exterior only**



Alternate Path Method





Additional Column Removal

- Plan geometry of the structure changes
- Abrupt decrease in bay size
- Re-entrant corners
- Adjacent columns are lightly loaded
- Bays have different tributary sizes
- Members frame-in at different orientations or elevations.
- Use engineering judgment

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Advantage of Nonlinear when doing AP

- Nonlinear analysis: same analysis for checking both deformation controlled and force controlled
- Linear analysis: separate analysis is needed for deformation controlled and force controlled

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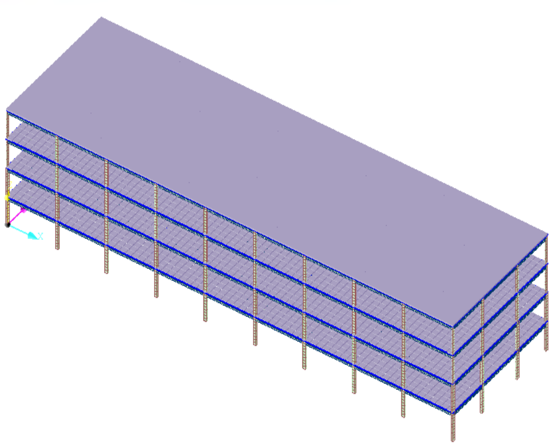
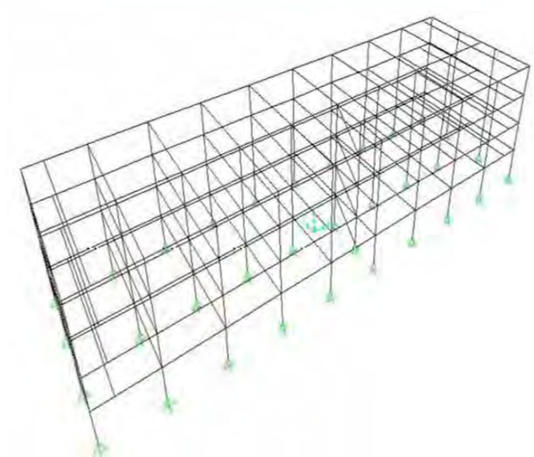
Location of Removed Load-Bearing Elements

- **If any other column is within a distance of 30% of the largest dimension of the associated bay from the column removal location, it must be removed simultaneously.**





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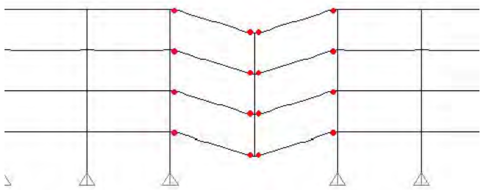

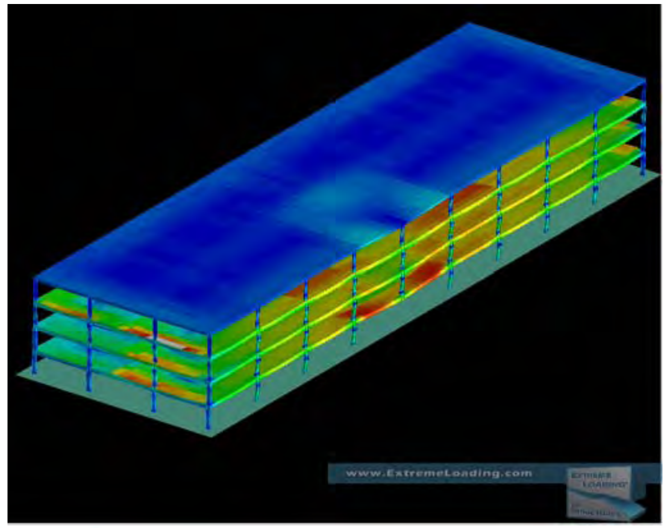

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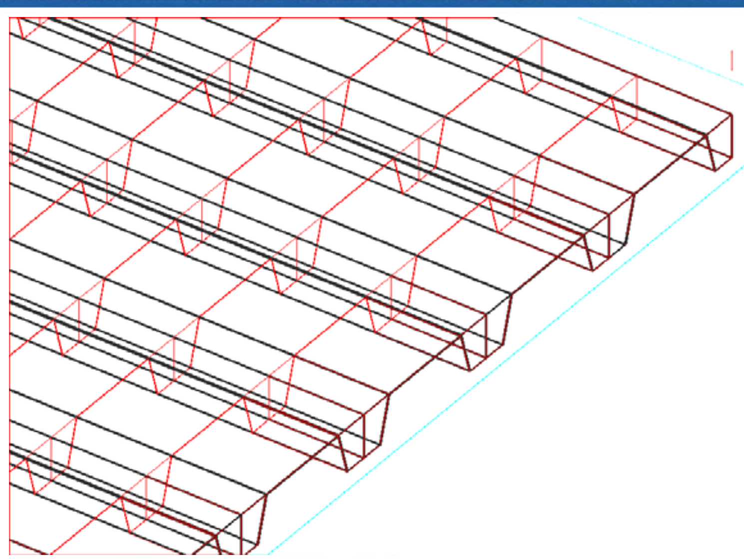
Modeling Corrugated Sheets



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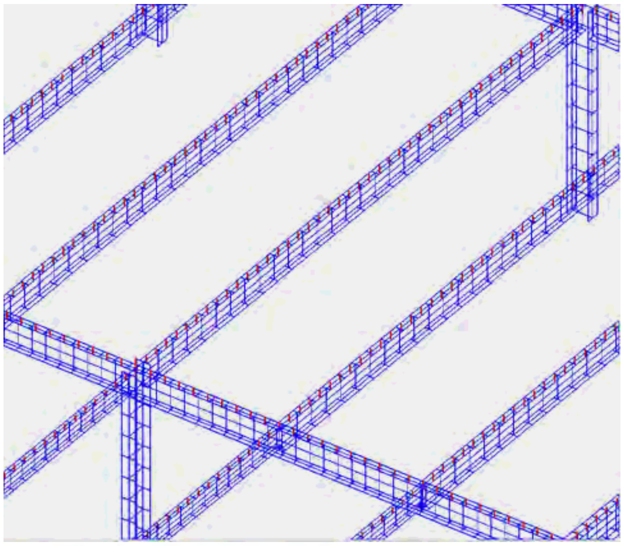
Connection between Slabs and Beams



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Analysis Guidance for AP

- 3-dimensional
- sufficient amount of structural detail in the model to allow the correct transfer of vertical loads from the floor and roof system to the primary elements
- Number of column-removal cases varies depending on the problem
- Removal at multiple levels

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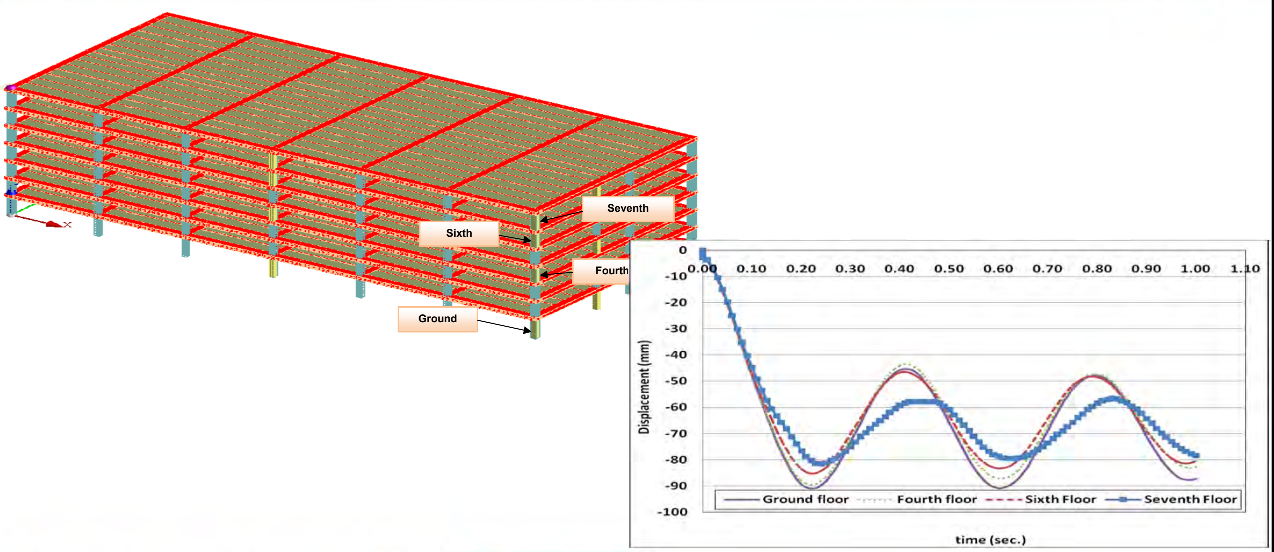
Location of Removed Load-Bearing Elements

- **For each plan location remove**
  - First story above grade
  - Story directly below roof
  - Story at mid-height
  - Story above the location of a column splice or change in column size

Effect of removed column vertical location



Considerations For Upgrading Existing Buildings

- Vierendeel action
- Moment frames intended to support lateral loads can span over a location of damage through Vierendeel action
- Beams experience severe double-curvature deformation
- Buildings designed to resist lateral loads with moment frames have the essential members and connections in place
- if beams and columns—and their connections—can be reinforced to support the applied loads, this method to add robustness can be relatively unobtrusive

WHY AP was added as an option for TF?

- **In the 2005 UFC 4-023-03, only tie forces were used for LLOP (i.e., OC II) buildings**
- **Tie Forces difficult to implement in existing buildings and load-bearing wall construction**
- **Option 2 (AP) was added. Thus, this provides some relief for existing buildings and load bearing redundant systems**

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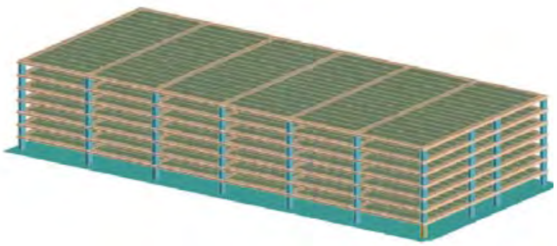

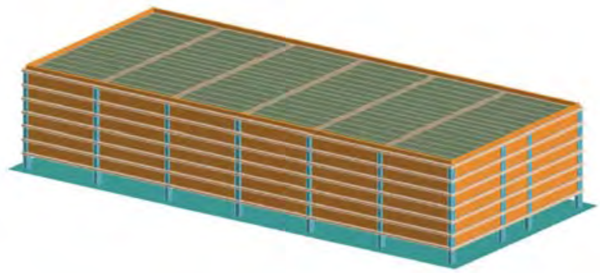

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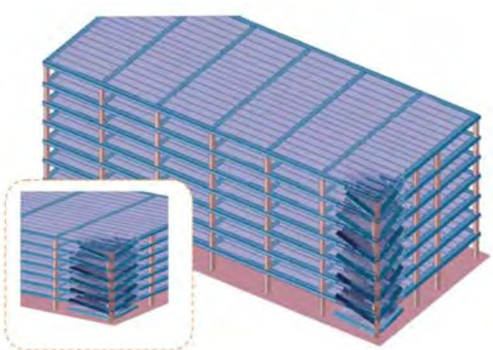
WHY AP was added as an option for TF?

- **Option 2 (AP) was added. Thus, this provides some relief for existing buildings and load bearing redundant systems**



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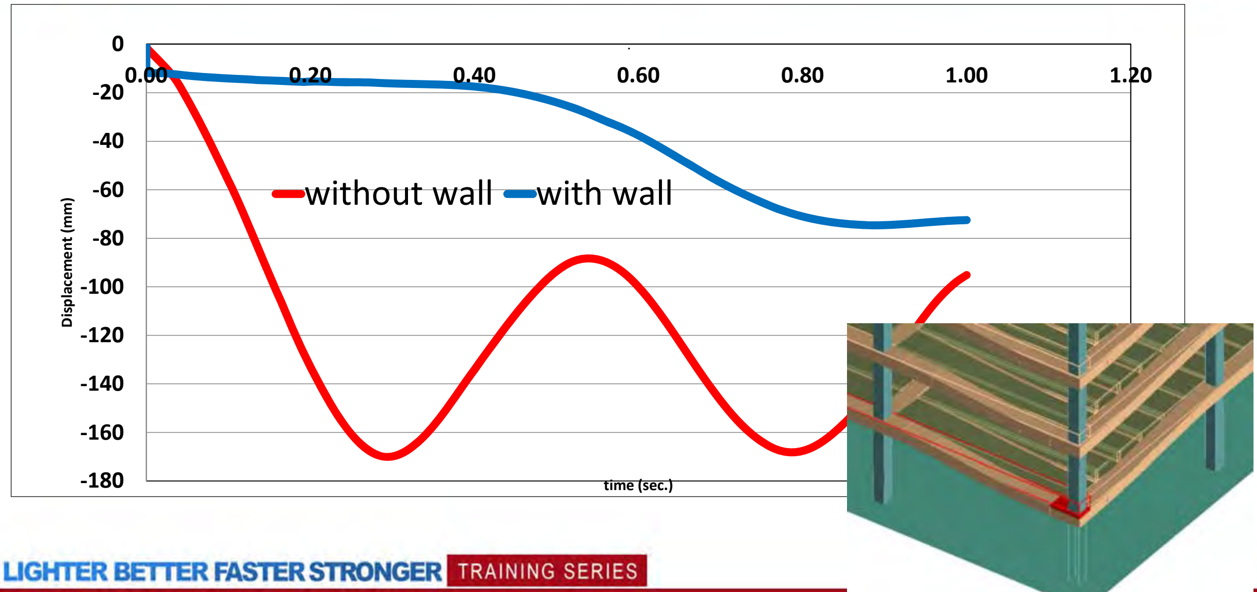
Effect of walls on deflection

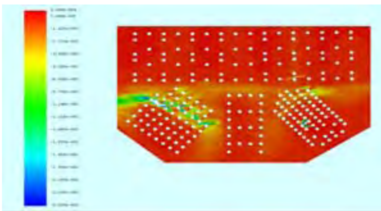


Analysis Guidance for AP

- **Performance based design based on ASCE 41**
- **Involves load factors and capacity factors**
- **Acceptance Criteria depending on the Analysis Method**
  - Stricter criteria for linear than nonlinear
  - Stricter criteria for static than dynamic

Known FEM modeling issues

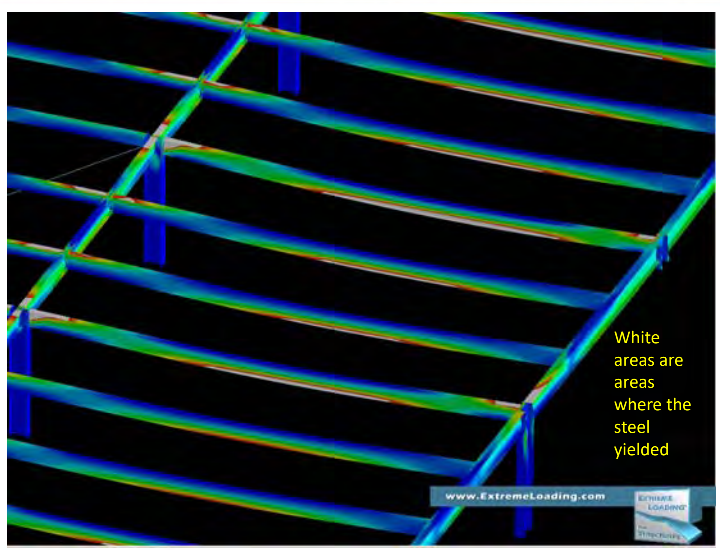
- **Connection behavior: While information is available in current guidelines to evaluate the behavior of connections, these elements are difficult to model within a typical building model that uses mostly one-dimensional elements.**



Analytical Rupture of gusset plate U10E from ELS

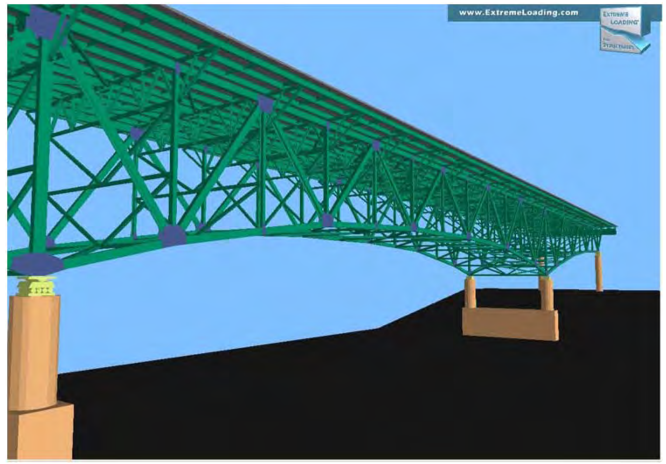
Known FEM modeling issues

- **Hinges: the use of localized plasticity through hinges allows commercial software to use a linear solution for most of the beam and column elements.**
- **Hinges are known to spread through elements and the user may be unaware that other sections of a beam exceed the yield stress if no additional hinges have been assigned to them.**



Known FEM modeling issues

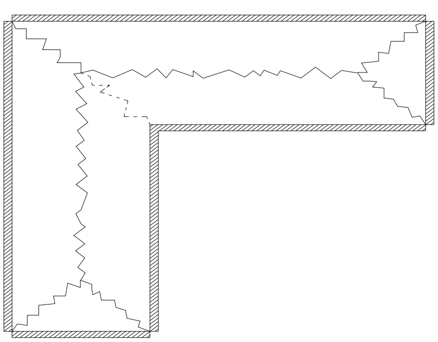
A solution involving the automatic consideration of the material strain-stress curve would result in a considerably larger computational time, offsetting the benefits of the use of simplified models.

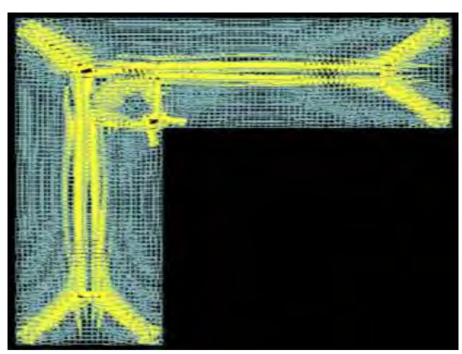


Minnesota I-35 Bridge Collapse

Known FEM modeling issues

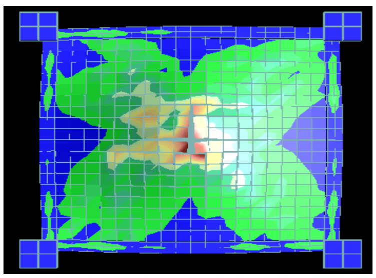
- **Slab plasticity: Commercial design software uses two-dimensional elements to model slabs and walls but their formulations do not allow yielding of the material through the development of cracks and yield lines.**

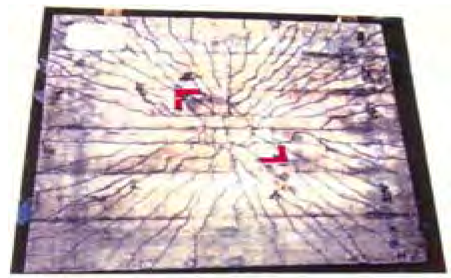




Known FEM modeling issues

- Testing of systems under column removal scenarios has demonstrated that slabs have great capacity for load redistribution but no simplified models that could be implemented in commercial design software have been developed and compared with test results. In addition, existing guidelines for the design of structures for progressive collapse do not include slab systems – like flat plate slabs.





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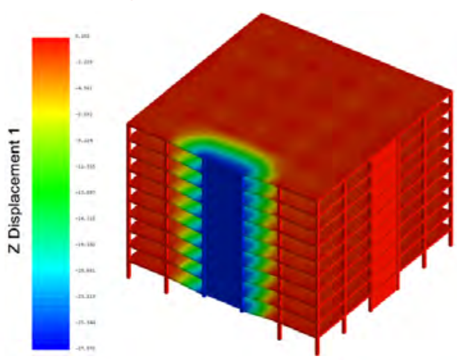
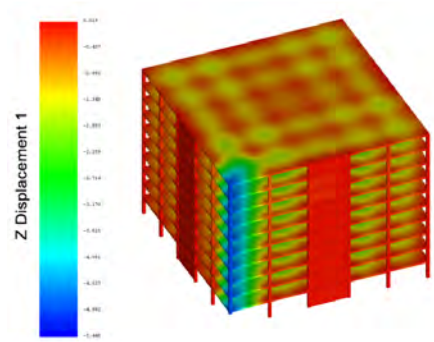
Known FEM modeling issues

- In addition, existing guidelines for the design of structures for progressive collapse do not include slab systems – like flat plate slabs.
- Therefore, it is current practice to neglect the effects of the slab and design a redundant system that will work without it.





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Progressive Collapse Analysis and Design Guidelines

General Services Administration (GSA 2013)

New Federal Office Buildings and Major Modernization Projects

GSA 2013

3 DESIGN PROCEDURES

These Guidelines employ the Alternate Path (AP) method only.

3.1 ~~Tie Forces~~

This UFC section is removed in its entirety, including the following figures:

~~Figure 3.1.~~
~~Figure 3.2.~~
~~Figure 3.3.~~
~~Figure 3.4.~~
~~Figure 3.5.~~
~~Figure 3.6.~~

TIE FORCE METHOD NOT AN OPTION

Limitation on linear procedure

- Limited to consideration of low-to-medium-rise facilities.
- Typically such facilities consist of buildings and specialty structures that are nominally 10 stories above grade or less.
- more than 10 stories above grade, and/or exhibit an atypical structural configuration, project engineers should use a “Nonlinear Procedure”.

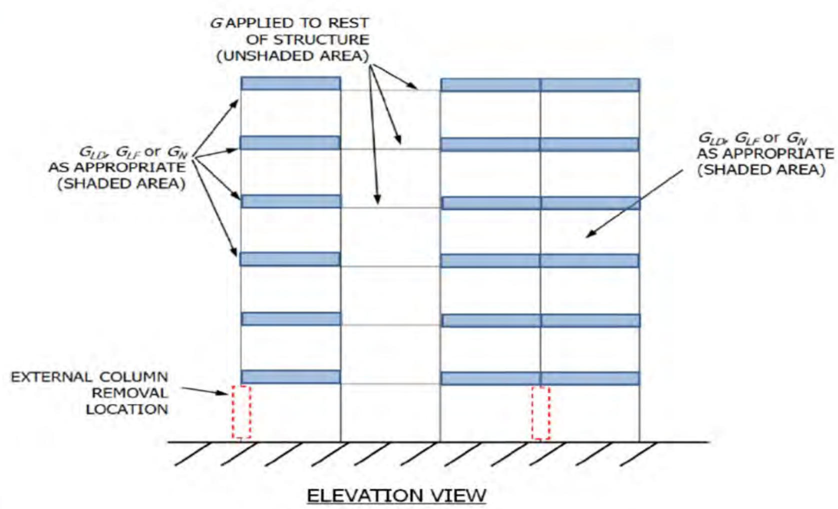
Nonlinear procedure

- Capture both material and geometric nonlinearity.
- for facilities that contain atypical structural configurations and/or high-rise buildings
- Assessment team will require experience and demonstrated expertise in structural dynamics, abnormal loading, and nonlinear structural response

Nonlinear Procedure

- Less restrictive acceptance criteria are permitted
- Potential numerical convergence problems with FEM
- Sensitivities to assumptions for boundary conditions, geometry and material models
- Complications due to the size of the structure with FEM

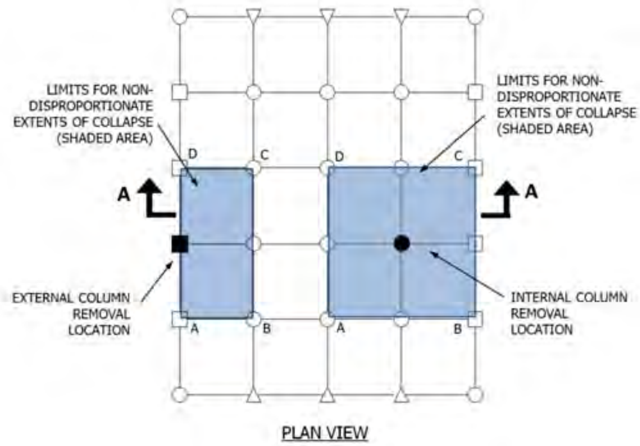
Different Load Increase Factors for Different Areas



Allowable Extents of Collapse for Interior and Exterior Column Removal in Plan

<u>The most recent version of the UFC removed any allowance of collapsed area, requiring that all elements, including those directly above the removed element, be designed to meet the defined acceptance criteria.</u>

The definition of "disproportionate" in GSA is taken similar to that utilized in the previous UFC, where an extent of collapse is allowed at structural bays on either side of and at the floor level above the removed element.



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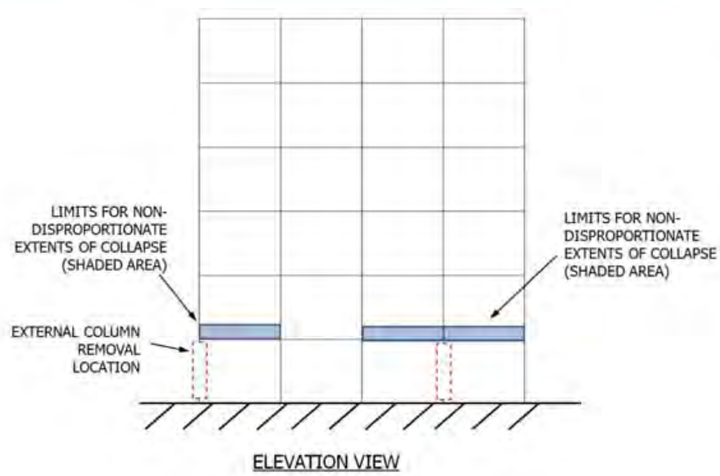
Allowable Extents of Collapse for Interior and Exterior Column Removal in Elevation

<u>The most recent version of the UFC removed any allowance of collapsed area, requiring that all elements, including those directly above the removed element, be designed to meet the defined acceptance criteria.</u>

The definition of "disproportionate" in GSA is taken similar to that utilized in the previous UFC, where an extent of collapse is allowed at structural bays on either side of and at the floor level above the removed element.



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ASCE/SEI, Standard for Mitigation of Disproportionate Collapse Potential in Building and Other Structures

(Under Development)

Hazard Independent Damage Scenarios (HIDS)

- **HIDS Level H-1**
  - component level damage
- **HIDS Level H-2**
  - damage to an individual secondary member.
- **HIDS Level H-3**
  - damage to an individual primary member.
- **HIDS Level H-4**
  - damage to two simultaneous primary members

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Debris Loading

- **Point of ongoing research**
- **If collapse of a portion of the structure is accepted, the effects of debris loading on the structure below shall be considered when evaluating the extent of collapse.**
- **Debris from the initiating damage event need not be considered when evaluating debris loading.**

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Alternative Load Path Analysis

ASCE/SEI – Guidelines (Under Development)

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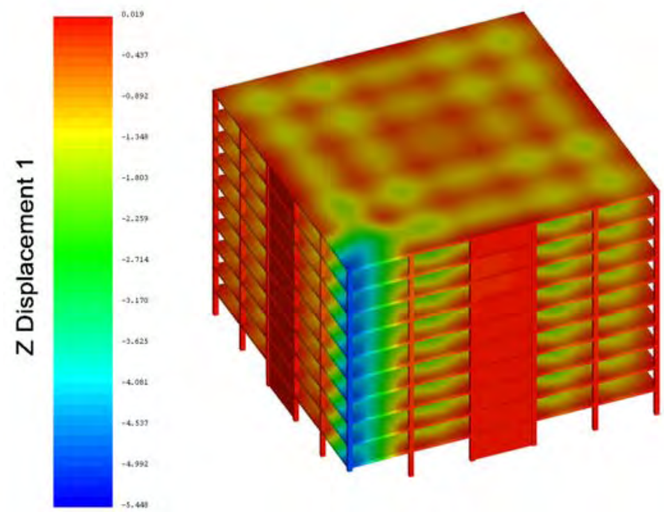
Alternative Load Path Analysis



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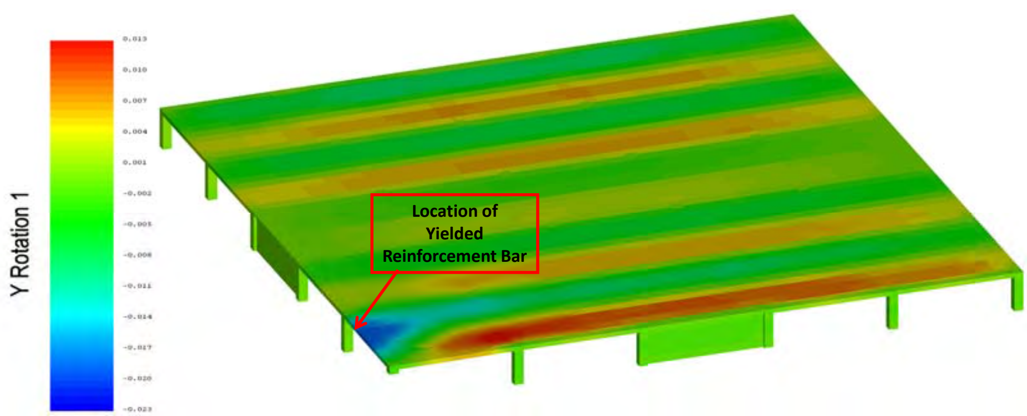
Alternative Load Path Analysis



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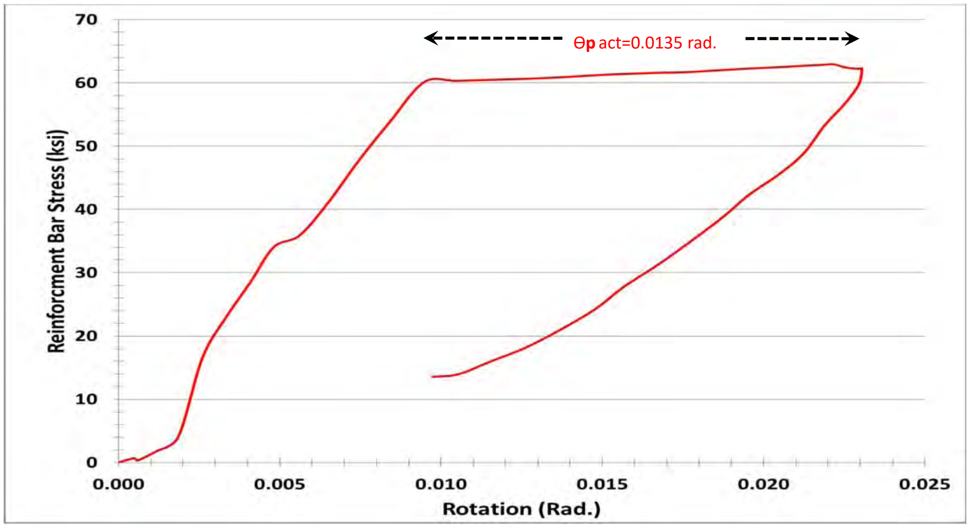
Alternative Load Path Analysis



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Alternative Load Path Analysis: Conclusion

- **Direct Design Methods**
  - Alternate path method
    - Provides a balance
    - Allows use of new technology
  - Specific local resistance method
    - Design all structural elements for a specific threat or abnormal load
- **Indirect Design Methods**
  - Tie force method and other integrity requirements will remain useful but not sufficient

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Applied Element Method (AEM)

Day 2 - 7 Hour Course

Ahmed Khalil, Ph.D., P.E. khalilaa@appliedscienceint.com

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www.appliedscienceint.com | www.extremeloading.com

Contents

- Why AEM?
- Theoretical Background
- Recent Publications using AEM
- AEM – FEM Comparison
- Verification Examples

Methods for Structural Analysis

- Finite Element Method (FEM)
- Boundary Element Method (BEM)
- Finite Difference Method
- Discrete Element Method (DEM)
- Discontinuous Deformation Analysis (DDA)
- Truss Method and Lattice Method
- Strut and Tie Method
- Spring Network Method
- Finite Section Method
- Rigid Body and Spring Method (RBSM)
- Mesh-Free Methods

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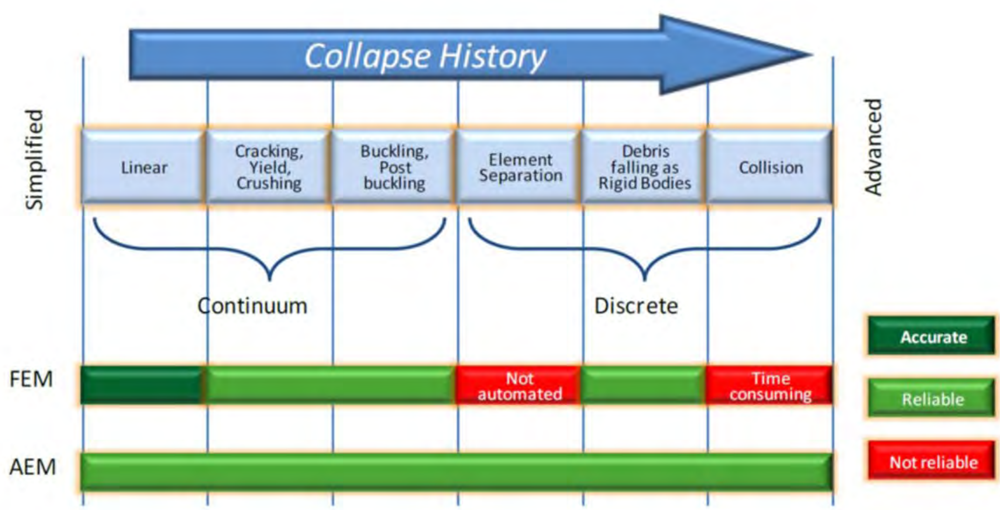
Why AEM?



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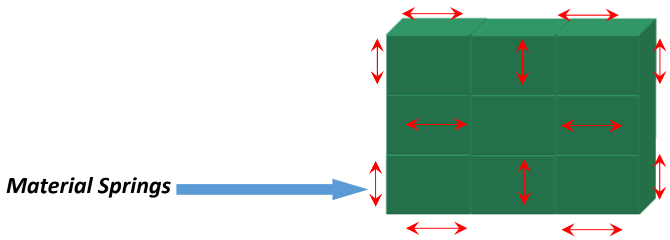
Continuum Simulation Using AEM



The continuum is discretized into elements connected together with nonlinear springs that represent the material behavior

The springs represent axial deformations as well as shear deformations

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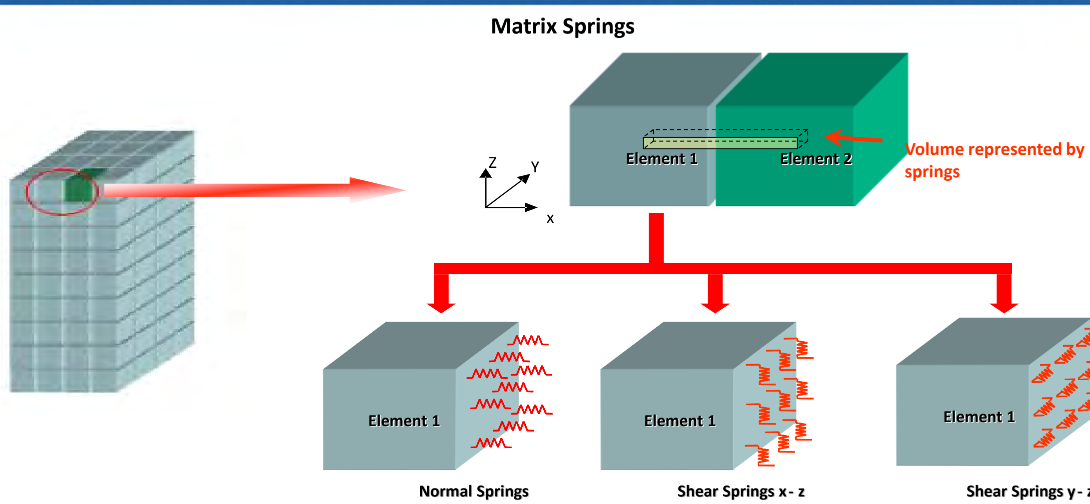
Types of Connecting Springs



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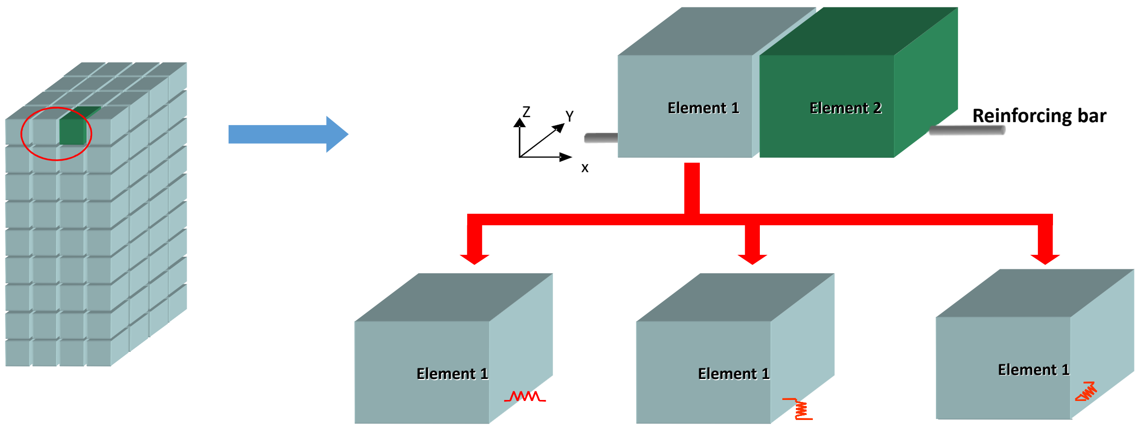
Connectivity Springs

Reinforcing bars springs



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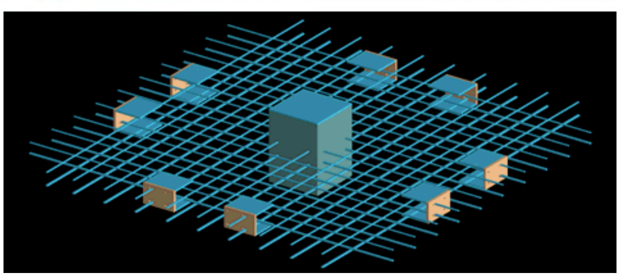
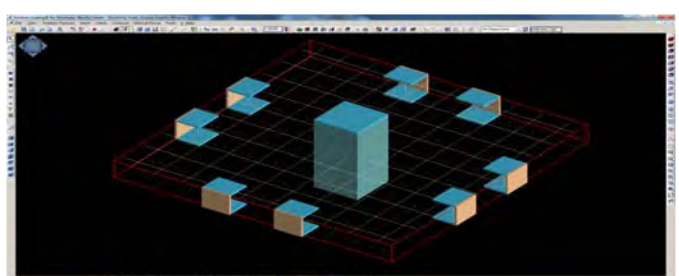
Options for modeling reinforcement using ELS



Explicit modeling in the form applied elements

Implicit modeling in form of equivalent springs at the location of every reinforcement bar



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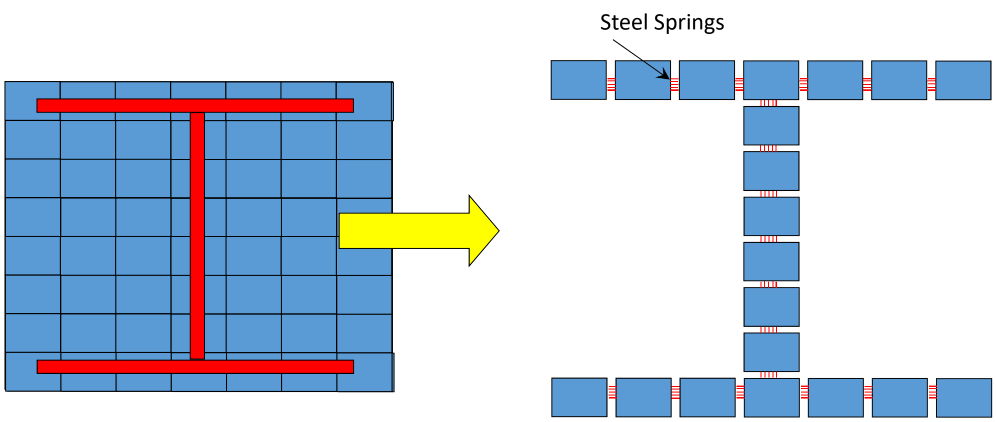
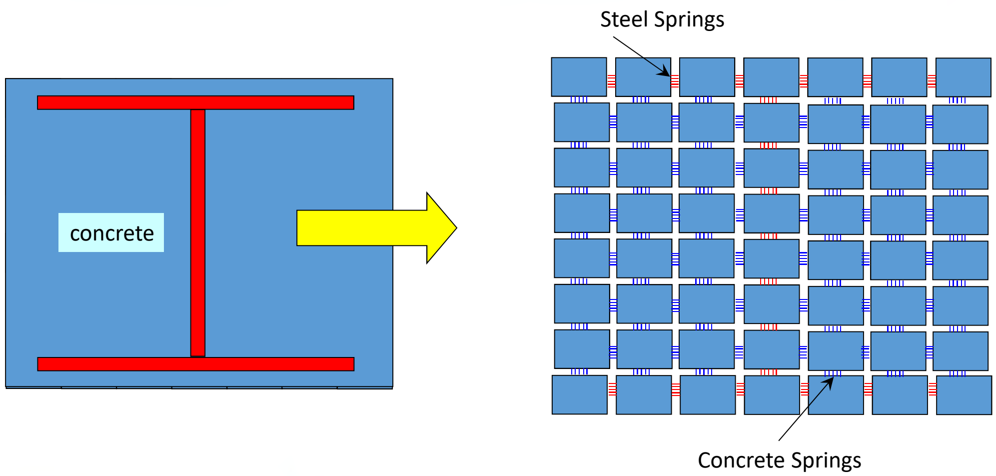
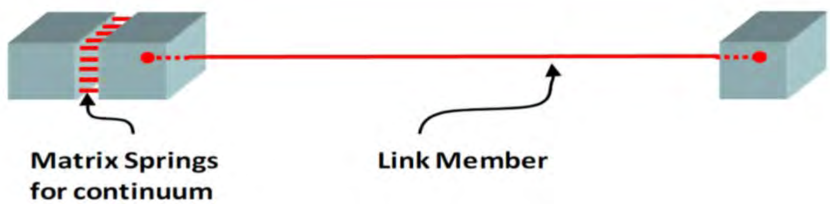
Implicit Steel Sections



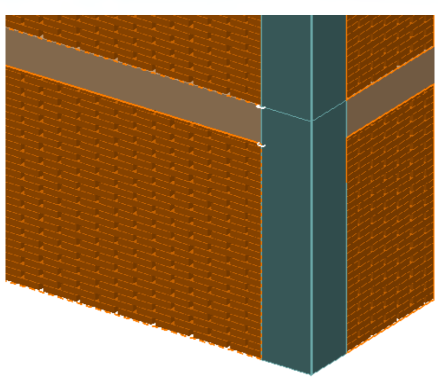
Implicit Composite Sections

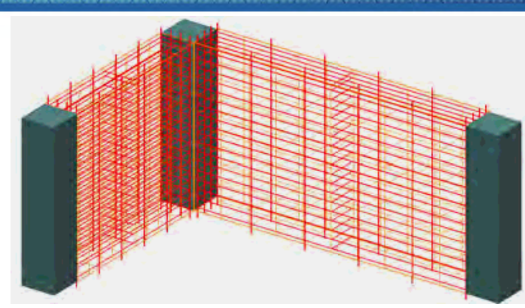


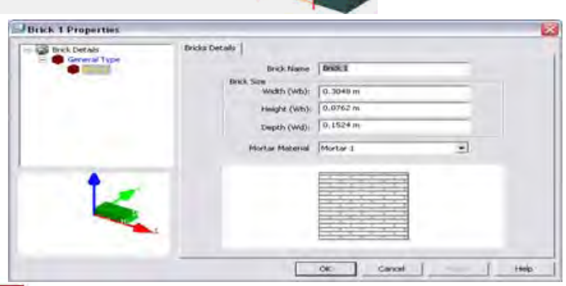
Link Members



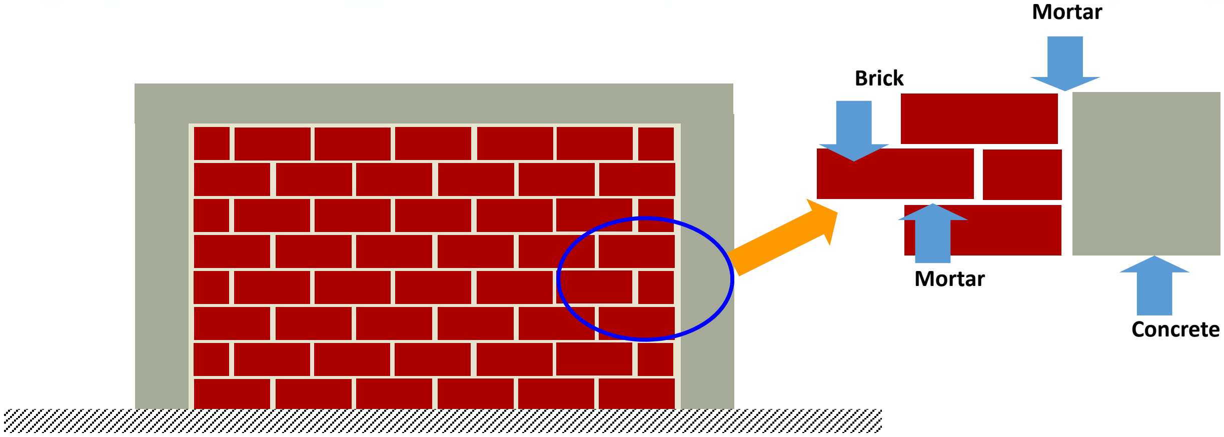
Brick Walls



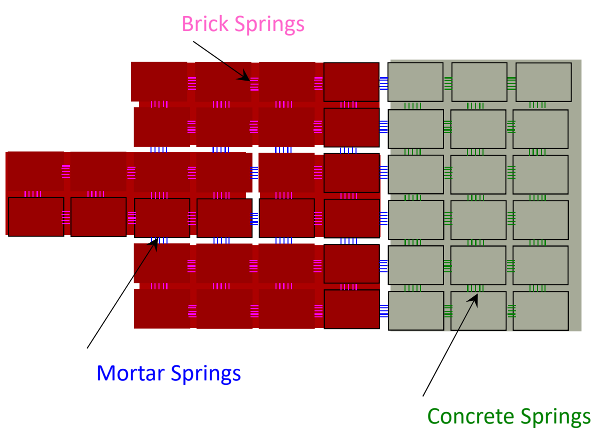




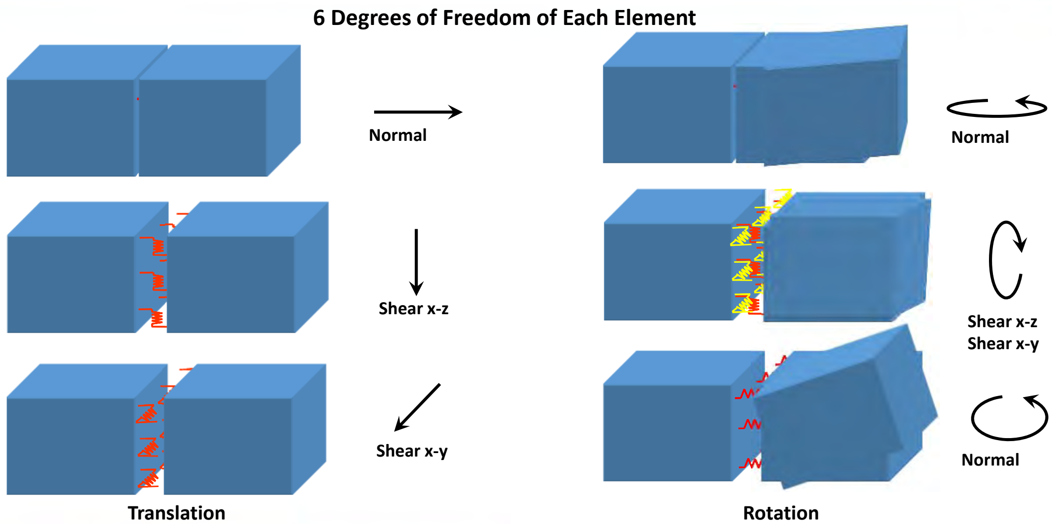
Masonry Modeling



Masonry Modeling



Formulation of Stiffness Matrix



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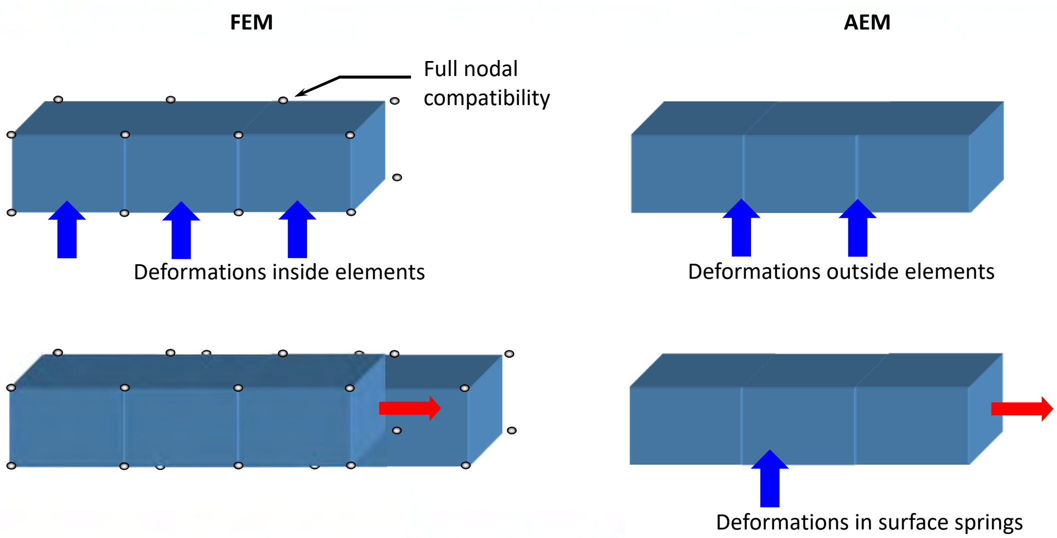
AEM – FEM Comparison



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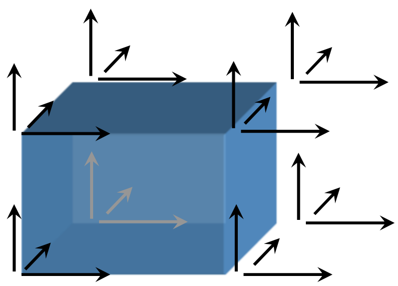
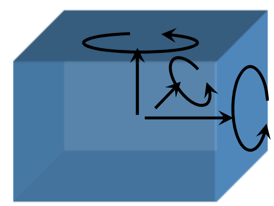
AEM vs. FEM: Degrees of Freedom



FEM

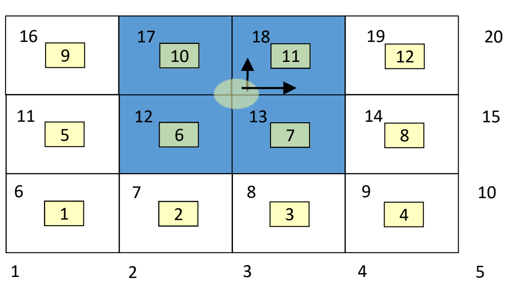


AEM

8 nodes x 3 DOF → 24 DOF/ Element

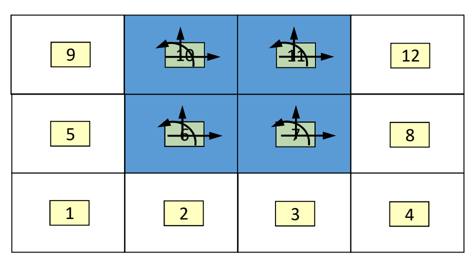
6 DOF/ Element

AEM vs. FEM: Element Connectivity



FEM

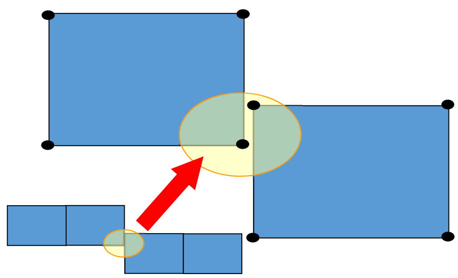
- Elements compatible at nodes (moves together there)
- For example Node 13 connects Elements 6,7,10,11
- Deformations are inside the elements



AEM

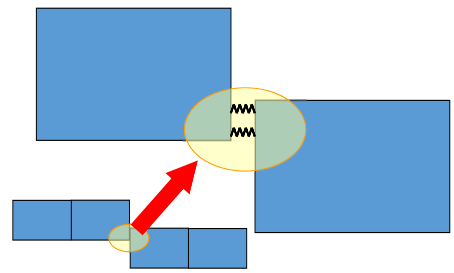
- Elements are connected through their faces
- For example elements 6,7,10,11 are not compatible in deformations
- Deformations are localized at the faces of the elements

AEM vs. FEM: Partial Element Connectivity



FEM

No Connectivity

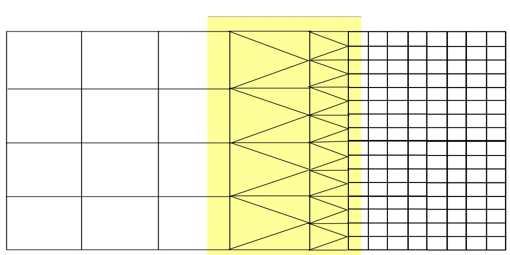


AEM

Connectivity Included

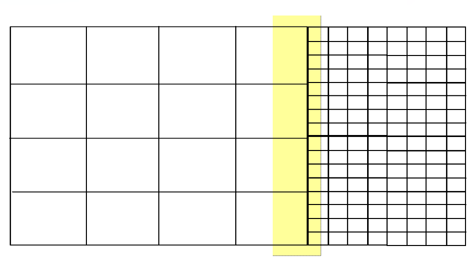
AEM vs. FEM: Modeling Advantages

Easy Element Connectivity



FEM

There should be transition elements between large elements and small elements

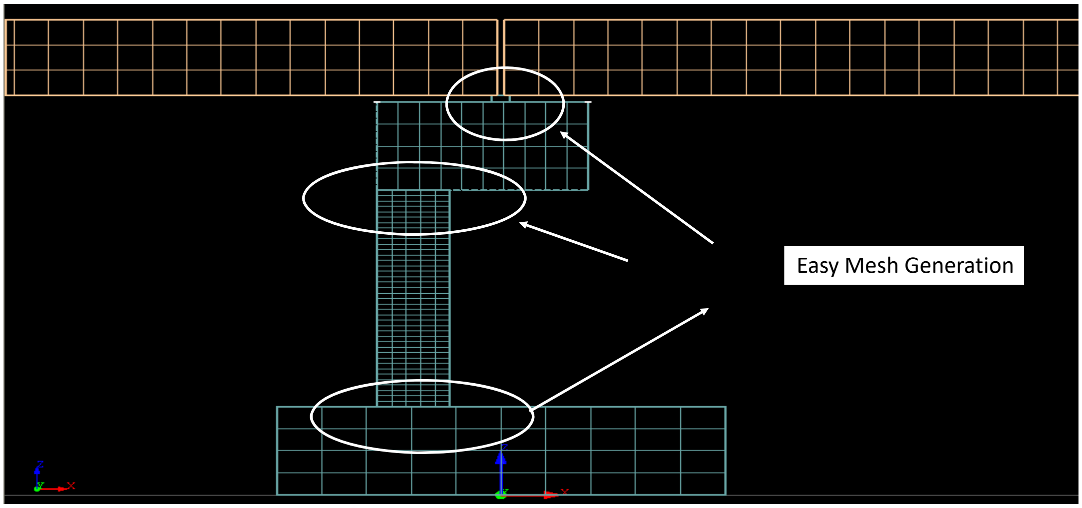


AEM

There is no need for the transition elements between large elements and small elements

AEM vs. FEM: Modeling Advantages

Easy Element Connectivity



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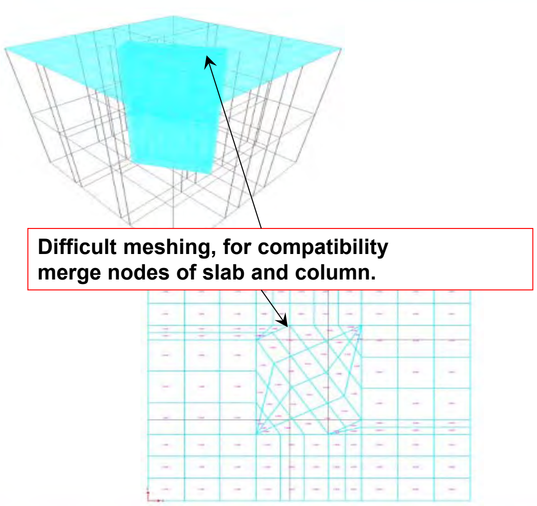
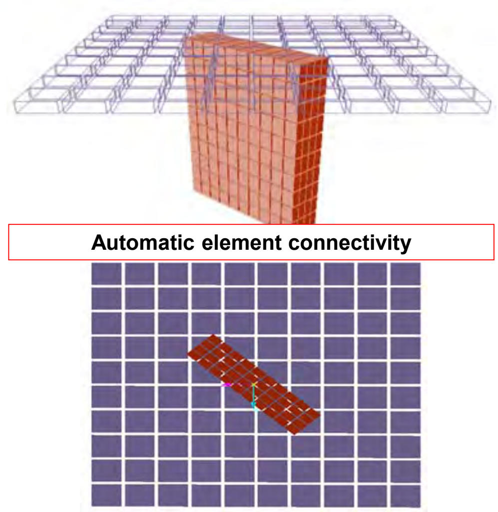
AEM vs. FEM: Modeling Advantages

FEM



AEM



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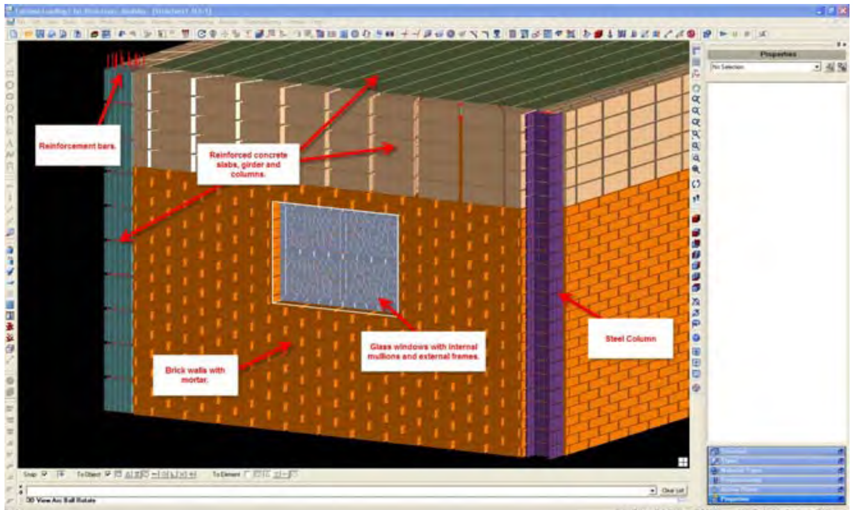
AEM vs. FEM: Modeling Advantages

Easy Element Connectivity



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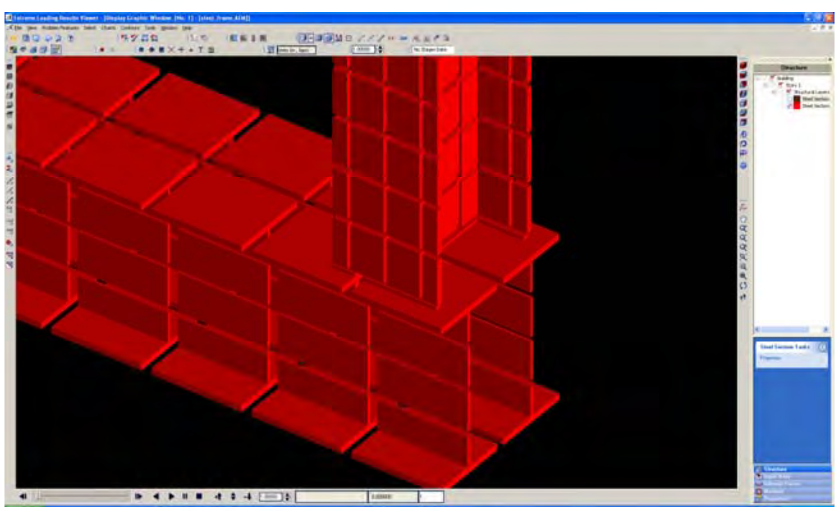
AEM vs. FEM: Modeling Advantages

Easy Modeling of Steel Structures



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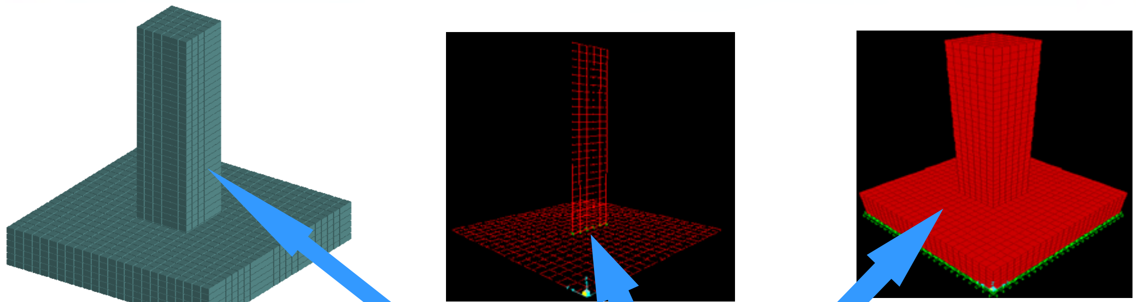
AEM vs. FEM: Modeling Advantages

No Need for Gap Elements



AEM | Shell Elements | Solid Elements

In Simplified FEM link elements should be **located and defined** in the beginning.

In AEM, link between a column and a footing is **automatically** defined at Springs

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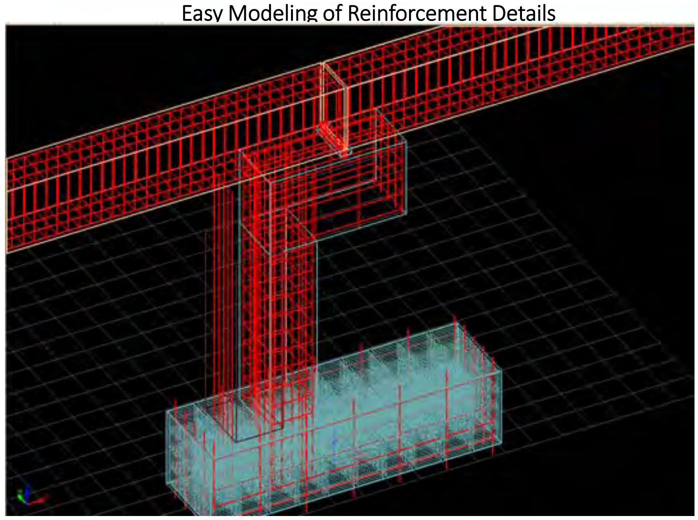
AEM vs. FEM: Modeling Advantages

Easy Modeling of Reinforcement Details



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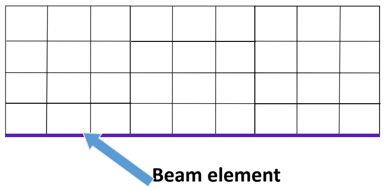
AEM vs. FEM: Modeling Advantages

Easy Modeling of Reinforcement Details

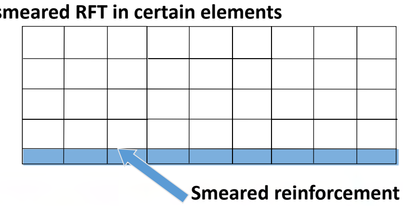
FEM

Complicated

1- Either simulated as beam elements compatible with elements



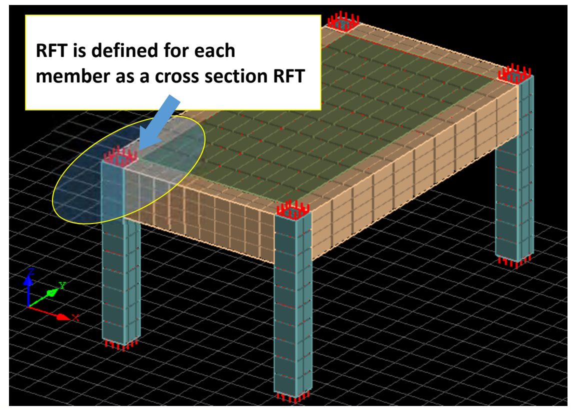
2- or as smeared RFT in certain elements



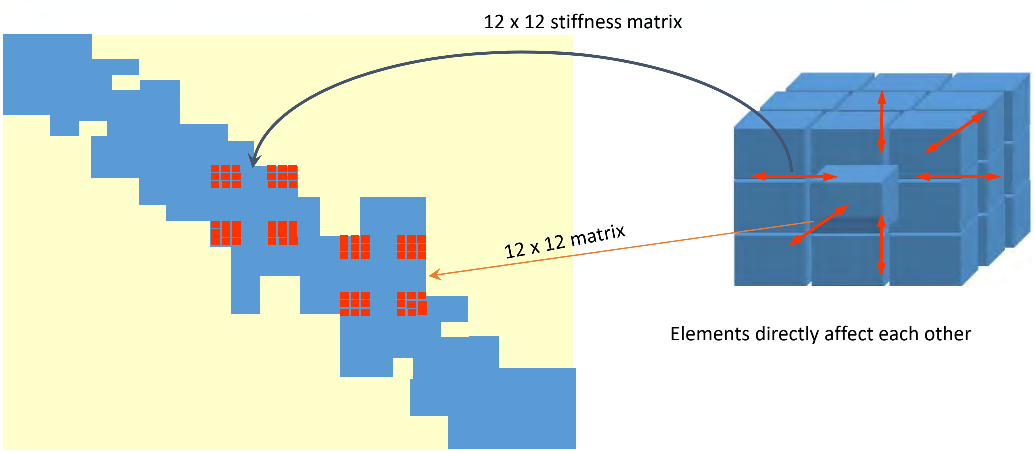
AEM

Simple

Reinforcing bars are easily introduced at their exact location



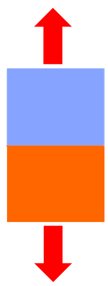
Assembly of Overall Stiffness Matrix

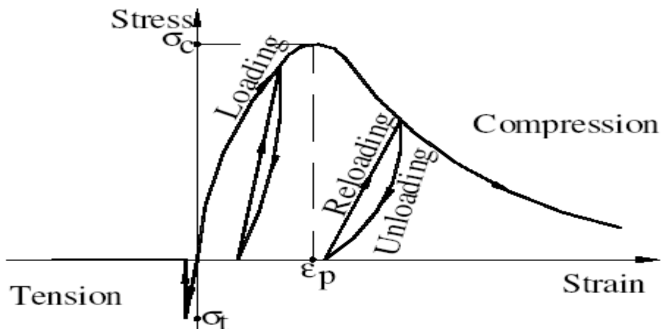


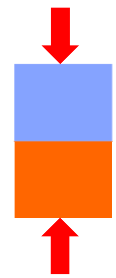
Overall Stiffness Matrix

Models for Constituent Materials

Concrete under axial stresses







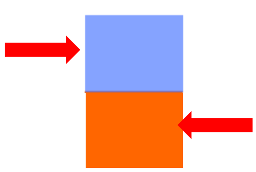
Tension

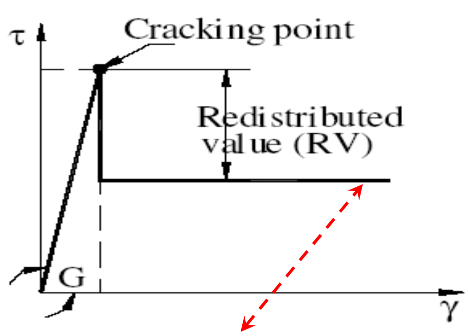
Fully path-dependent model for concrete
(Okamura and Maekawa, 1991)

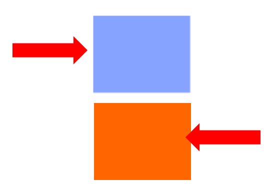
Compression

Models for Constituent Materials

Concrete under axial stresses

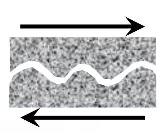






Before Cracking

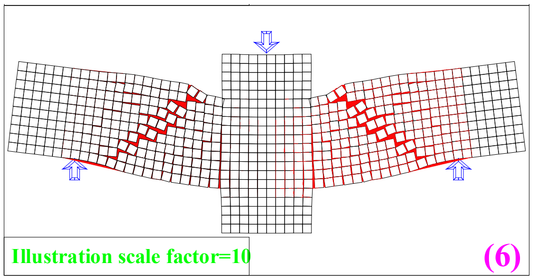
After Cracking

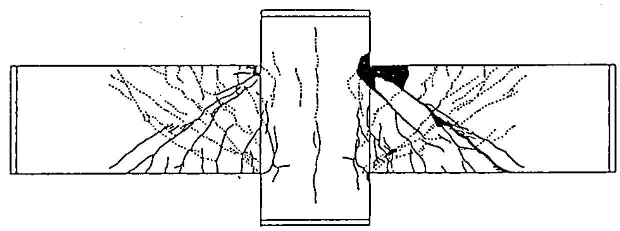


Friction and interlocking

Nonlinear Stage

Double Cantilever Subjected to Cyclic Loading





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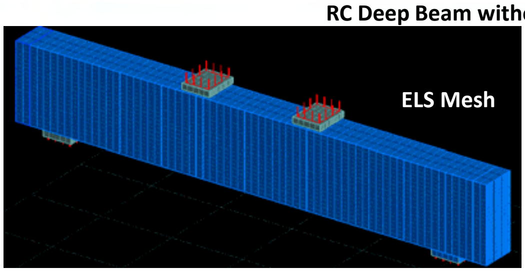
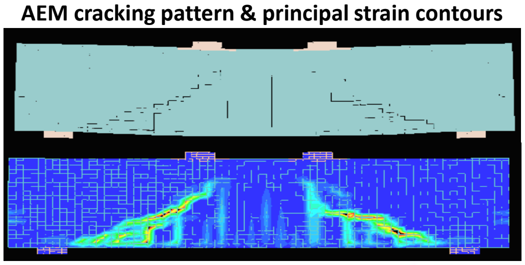
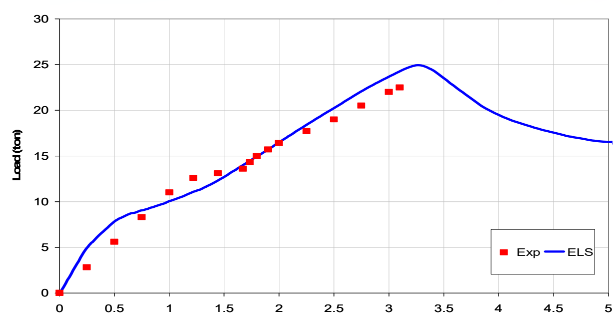
Nonlinear Stage

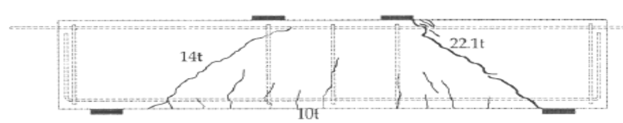
RC Deep Beam without Web RFT under Four-Points Loading



AEM cracking pattern & principal strain contours







Experimental cracking pattern

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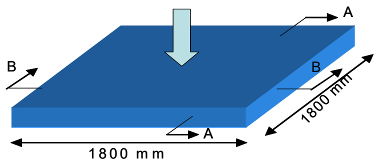
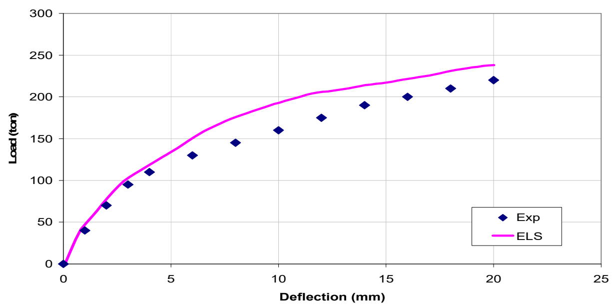
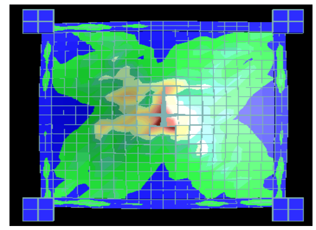
Nonlinear Stage

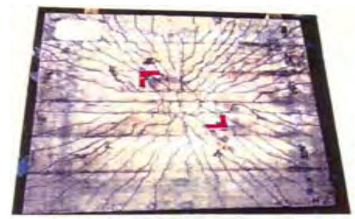
RC Slab under Point Load







ELS principal strain contours



Experimental cracking pattern

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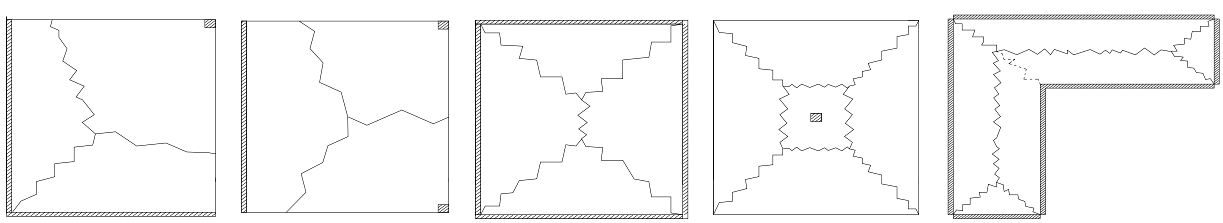
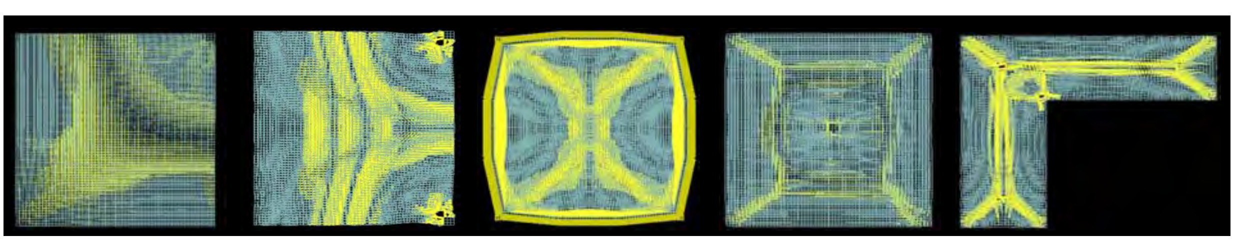
Nonlinear Stage

Fracture (Yield) Line of Slabs





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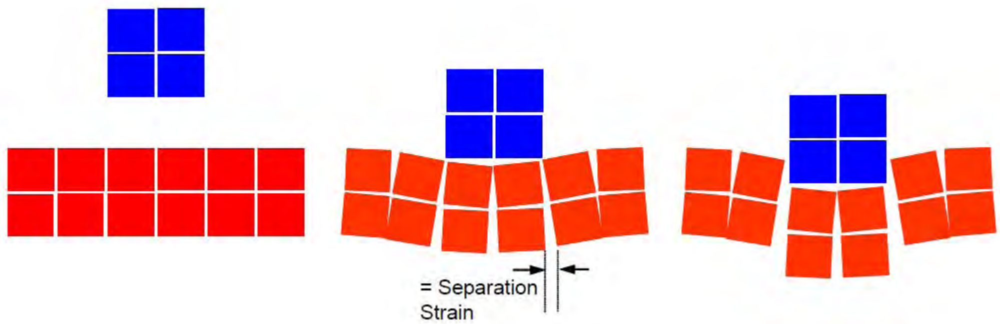
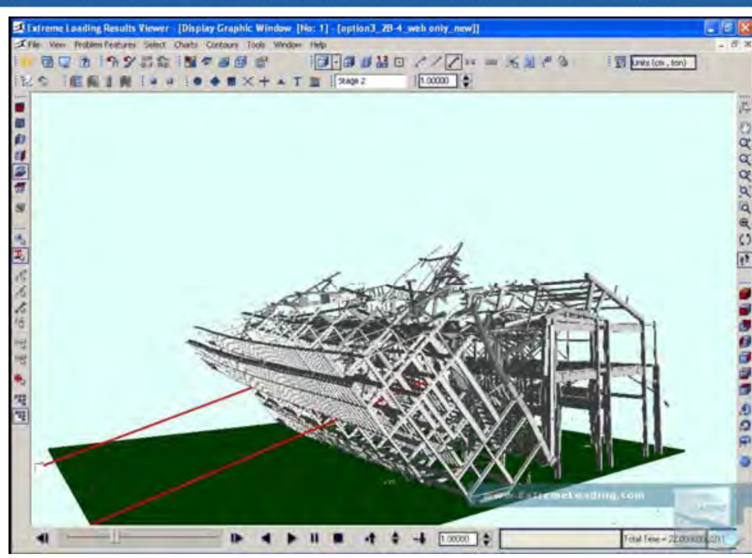
Automatic Separation



Automatic Separation & Collision



Smeared Crack Problems in Traditional FEM

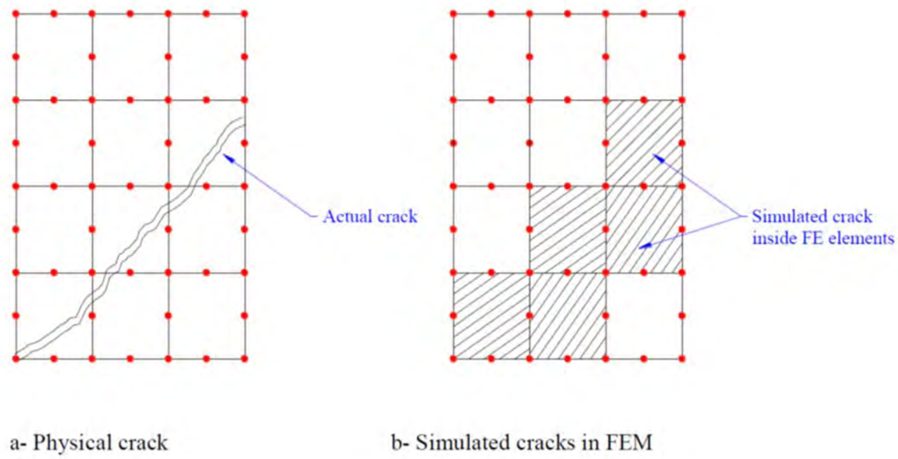
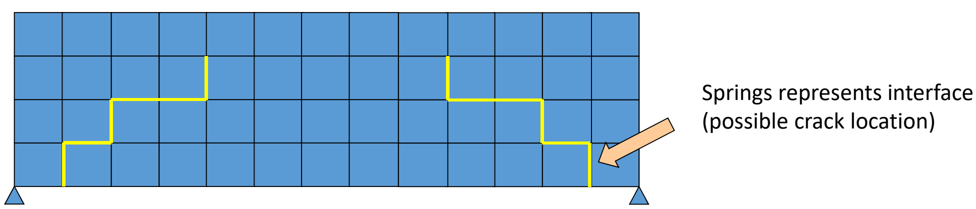


Figure 2-6 Smeared crack approach in FEM

Crack Modeling in AEM

Discrete Crack Approach

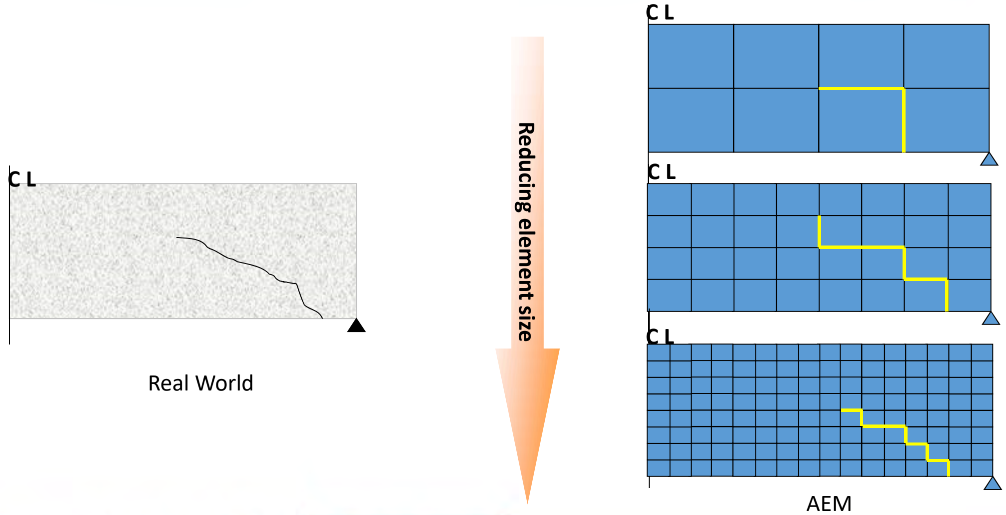


Explicit cracks are considered between elements

Cracks are continuous but mesh dependent, therefore, it is better to reduce mesh size

Crack Modeling in AEM

Cracks Mesh-Dependency



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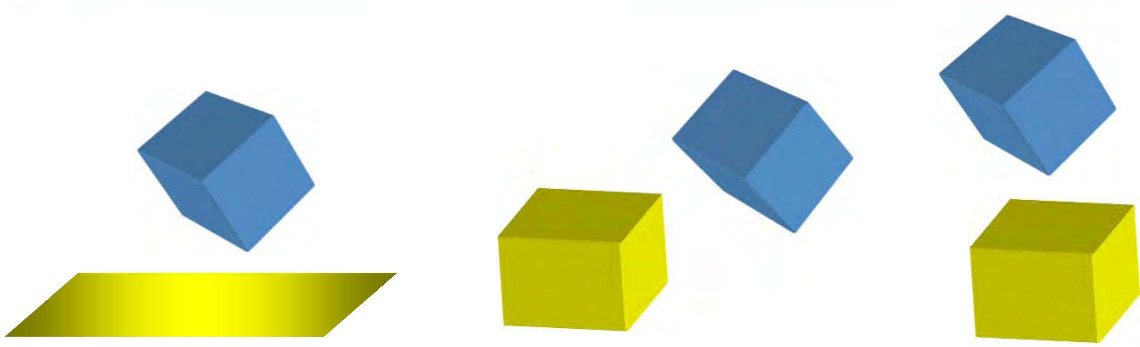
Types of Element Contact



Corner-Ground Type

Edge-Edge Type

Corner-Face Type

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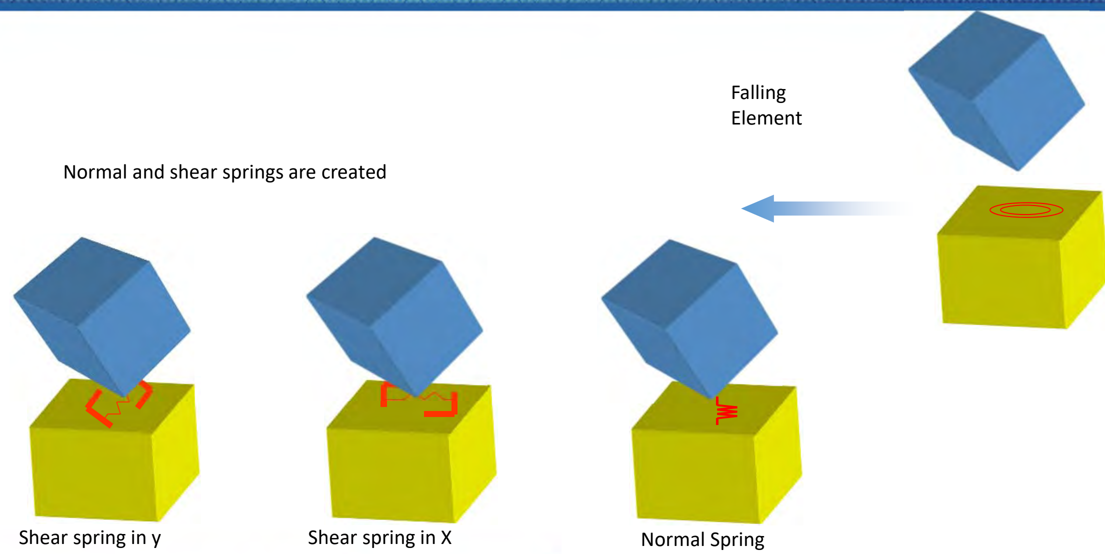
Element Contact



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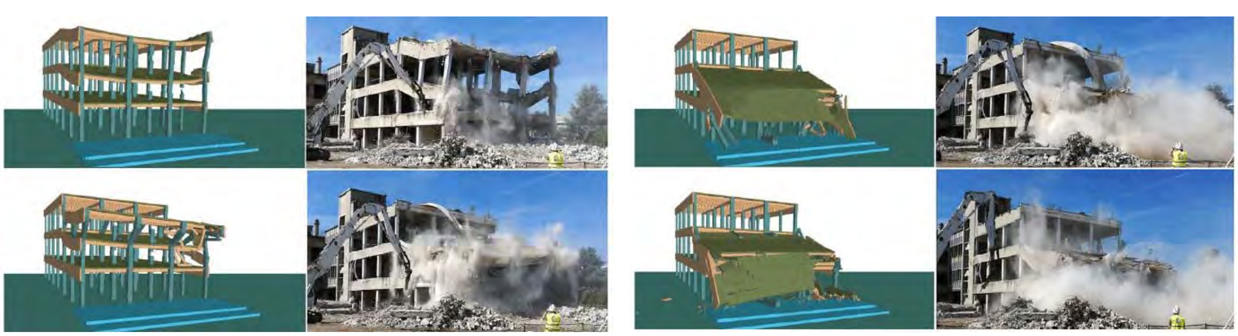
Debris Loading



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Relevant Published Research Using AEM

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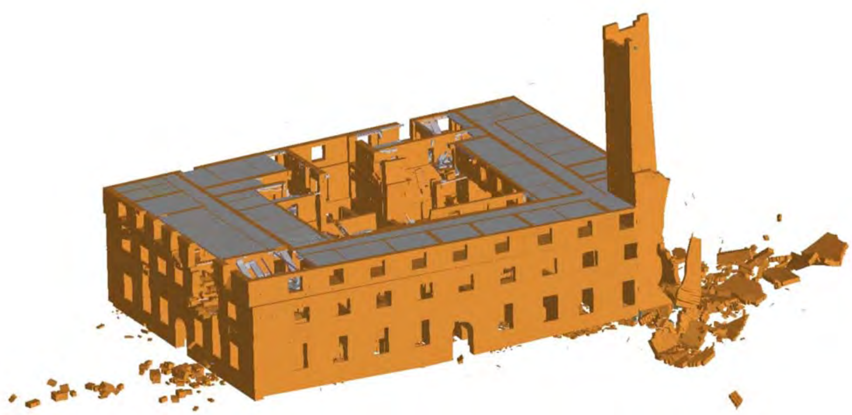
Historic Masonry Structures



Seismic Assessment of Damaged Margherita Palace
Using Applied Element Method (AEM)

UNIVERSITÀ DEGLI STUDI DE L'AQUILA

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Verification Case: Pyne Gould Building

February 2011 earthquake Christchurch, New Zealand
Drawings 1963 – 2009
Earthquake record

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Pyne Gould Collapse

On the 22nd February 2011 earthquake, the building's collapsed
18 fatalities occurred in this building alone
Well studied, well documented case

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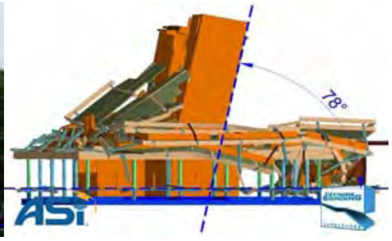
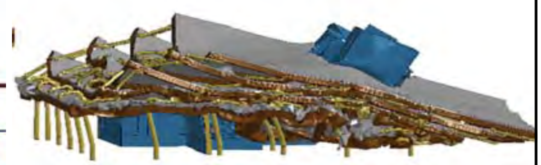
Comparison with Other Methods



Real Collapse
Pyne Gould Building

ELS by ASI

3.5 hours using 6 processors

LS DYNA by EMI

288 hours using 64 processors

ASI EUROPE.srl

Fraunhofer EMI

"Ratio 1:800"

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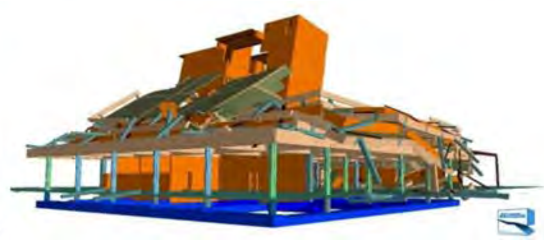
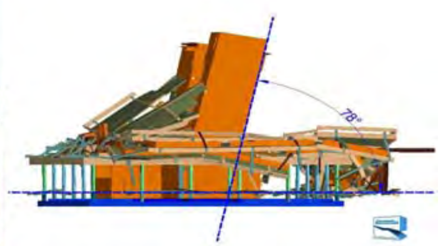
Comparison of Collapse Mode



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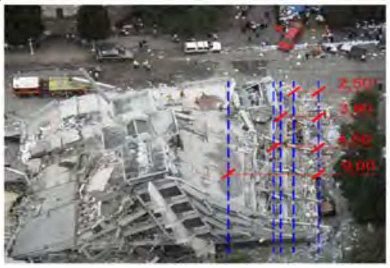
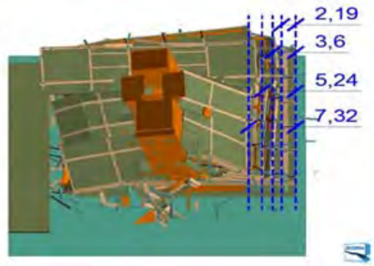
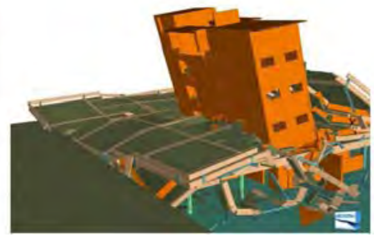
Comparison of Collapse Mode







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Research Using AEM

Home | Technology | Applications | Services & Support | Research

Home > Extreme Loading Technology > DTRA Report

DTRA Report

High Fidelity Modeling of Building Collapse with Realistic Visualization of Results

ASI was accepted for Phase I of the DTRA solicitation number 082-005, titled "High Fidelity Modeling of Building Collapse with Realistic Visualization of Resultin
of Phase I was to validate the Applied Element Method (AEM) as it's implemented in ASI's Extreme Loading® for Structures Software (ELS) for blast analysis an
below sumarizes the project or you can download the full report here.

The project started with collecting available information needed to show the theoretical background and developments of AEM from the time it originated in 199
report date in 2009. This was followed by gathering internal numerical tests that were performed with AEM for different materials including concrete, steel, bricl
under both static and dynamic loading conditions.Since AEM was relatively new compared to the Finite Element Method (FEM), it was important for ASI to run th
blind numerical tests in order to gain further acceptance of those who have been using FEM for decades. To accomplish this, a DTRA contractor was tasked to r
in order for ASI to perform the numerical analysis while retaining all output results until after the AEM analysis was completed. By following this approach, ASI a
parameters were adjusted to better match the FEM or test results. During the project ASI performed 22 blind numerical tests including 9 scenarios of walls unde
columns under blast loading, and 11 case studies of a 5-story structure under progressive collapse.

Walls under Blast Loading:

DISTRIBUTION STATEMENT A: Approved for Public Release.

High Fidelity Modeling of Building Collapse with Realistic Visualization of Resulting Damage and Debris Using the Applied Element Method

Prepared by:

Hatem Tagel-Din

Applied Science International, LLC

September 2009

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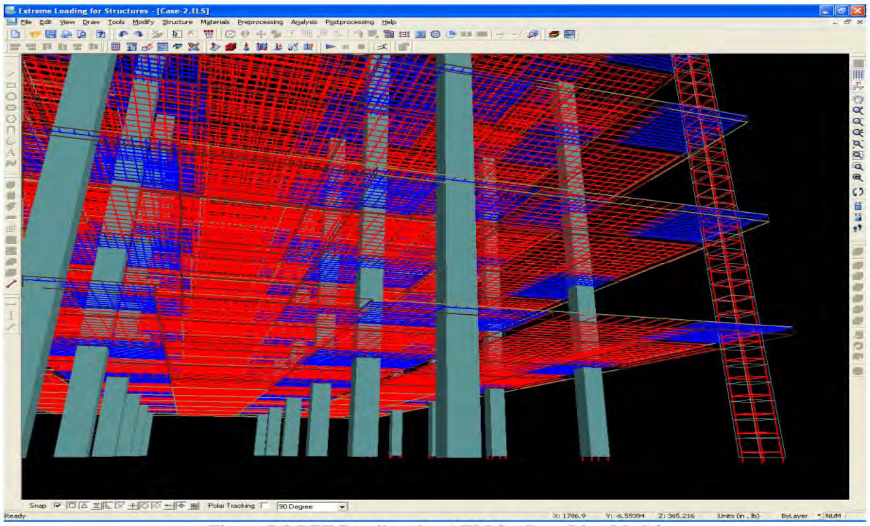

Figure 7-2 RFT Details using AEM for Base-Line Model

Research Using AEM

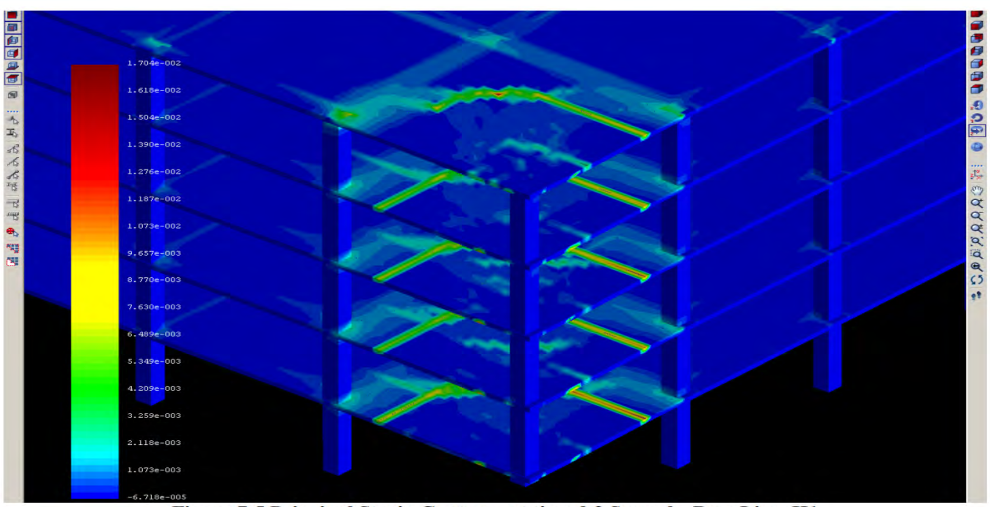

Figure 7-5 Principal Strain Contours at time 0.2 Seconds, Base Line, H1

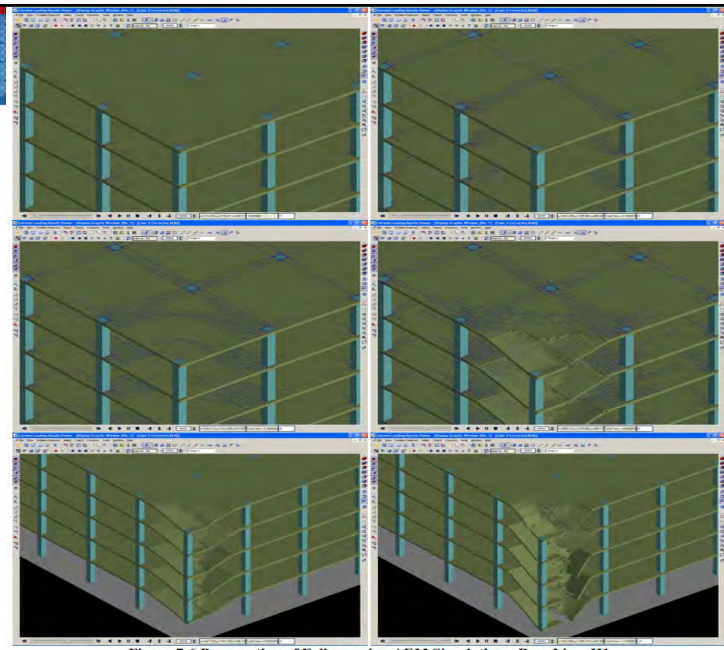

Figure 7-6 Propagation of Failure using AEM Simulations, Base Line, H1

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Research Using AEM

Structural Identification and Damage Characterization of a Masonry Infill Wall in a Full-Scale Building Subjected to Internal Blast Load

Timothy P. Kernicky, S.M.ASCE[1]; Matthew J. Whelan, A.M.ASCE[2]; David C. Weggel, P.E., A.M.ASCE[3]; and Corey D. Rice, S.M.ASCE[4]

Abstract: Structural identification continues to develop an expanding role within performance-based civil engineering by offering a means to construct high-fidelity analytical models of in-service structures calibrated to experimental field measurements. Although continued advances and case studies are needed to foster the transition of this technique from exploration to practice, potential applications are diverse and range from design validation, construction quality control, assessment of retrofit effectiveness, damage detection, and lifecycle assessment for long-term performance evaluation and structural health monitoring systems. Existing case studies have been primarily focused on large civil structures, specifically bridges, large buildings, and towers, and the limited studies exploring application to damaged structures have been primarily associated with seismic events or other conventional hazards. The current paper produces the first experimental application of structural identification to a component of a full-scale building structure with structural deterioration resulting from an internal blast load. Experimental modal analysis, nondestructive testing, and visual documentation of the structure was performed both prior to and after the internal blast, while a suite of blast overpressure transducers and shock accelerometers captured applied loads and structural response during the blast event. This paper presents an overview of the field testing and observed structural response followed by extensive treatment of the experimental characterization of structural damage in a masonry infill wall. Combined stochastic-deterministic system identification is applied to the acquired input-output data from the vibration testing to estimate the modal parameters of the infill wall for both the in-service state and in the postblast condition with damage characterized by interfacial cracking and permanent set deformation. Structural identification by global optimization of a modal parameter-based objective function using genetic algorithm is employed over two stages to produce calibrated finite-element models of the wall in the preblast and postblast conditions. Damage characterization is explored through changes in the structural properties of the calibrated models. Plausibility of the results are supported by observed cracking and spall documented in the experimental program and further reinforced through nonlinear applied element simulation of the response of the wall. **DOI: 10.1061/(ASCE)ST.1943-541X.0001158.** *© 2014 American Society of Civil Engineers.*

Author keywords: Structural identification; Blast loading; Vibration-based damage detection; Finite-element model updating; Applied element method.

- Applied Element Method

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Research Using AEM

Enhanced Modeling of Steel Structures for Progressive Collapse Analysis Using the Applied Element Method

Ahmed Amir Khalil[1]

Abstract: This paper studies performing progressive collapse analysis for steel structures using the requirements of recent codes released by the U.S. Department of Defense and the General Services Administration. Based on a review of the code requirements, the nonlinear dynamic progressive collapse analysis resulted in a more uniform factor of safety than the linear static analysis. The applied element method in the structural analysis is proposed as an efficient alternative for performing progressive collapse analysis. A case study is undertaken where the results of the progressive collapse analysis using traditional finite-element-method simplifications are compared with the results from the applied element method in the analysis of a moment-resisting steel frame. The case study shows that simplifications that are usually done in finite-element analysis when studying traditional load cases can be overconservative when performing progressive collapse analysis. The results show that the use of the nonlinear dynamic applied element method, while taking into account the effect of secondary members such as slabs and secondary beams, can lead to considerable savings in the total weight of the steel frame. **DOI: 10.1061/(ASCE)CF.1943-5509.0000267.** © *2012 American Society of Civil Engineers.*

CE Database subject headings: Progressive collapse; Steel structures; Structural failures; Decks.

Author keywords: Progressive; Disproportionate; Collapse; Steel; Deck; Applied element; AEM; Unified facilities criteria; Alternate path; Column removal.

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Engineering Structures 33 (2011) 3341–3350

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Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Toward an economic design of reinforced concrete structures against progressive collapse

H.M. Salem[a], A.K. El-Fouly[b,*], H.S. Tagel-Din[b]

[a] Department of Structural Engineering, Cairo University, Giza, Egypt
[b] Applied Science Int., 2012 TW Alexander Drive, Durham, NC 27709, USA

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Engineering Structures 30 (2008) 2478–2491

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Response of a reinforced concrete infilled-frame structure to removal of two adjacent columns

Mehrdad Sasani *

Northeastern University, 400 Snell Engineering Center, Boston, MA 02115, United States

Received 27 June 2007; received in revised form 26 December 2007; accepted 24 January 2008
Available online 19 March 2008

GSA contract No. GS09P06KTM0019

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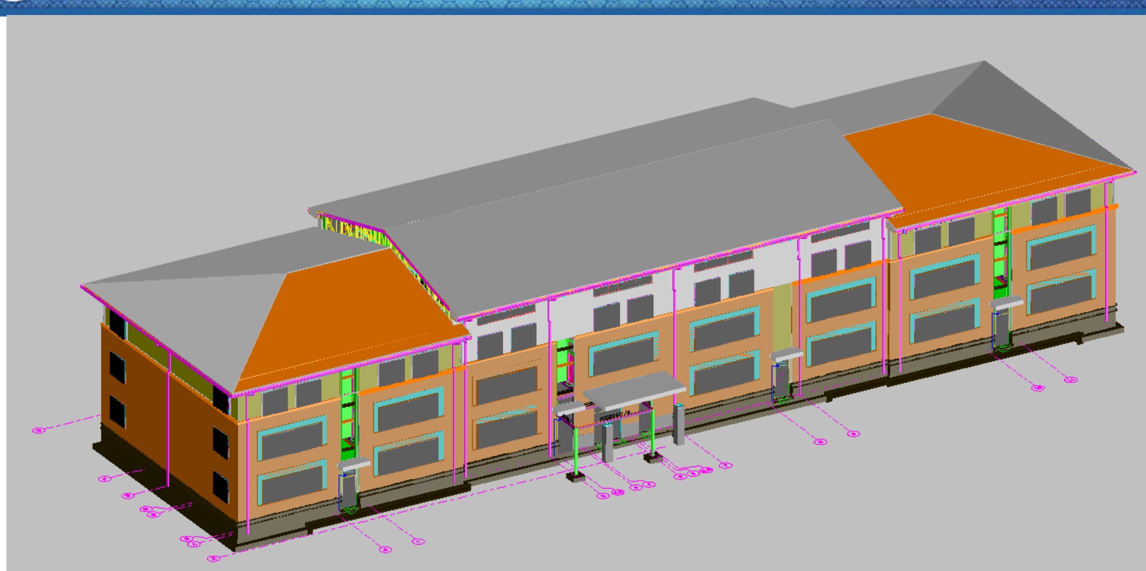
Cold Formed Steel Building



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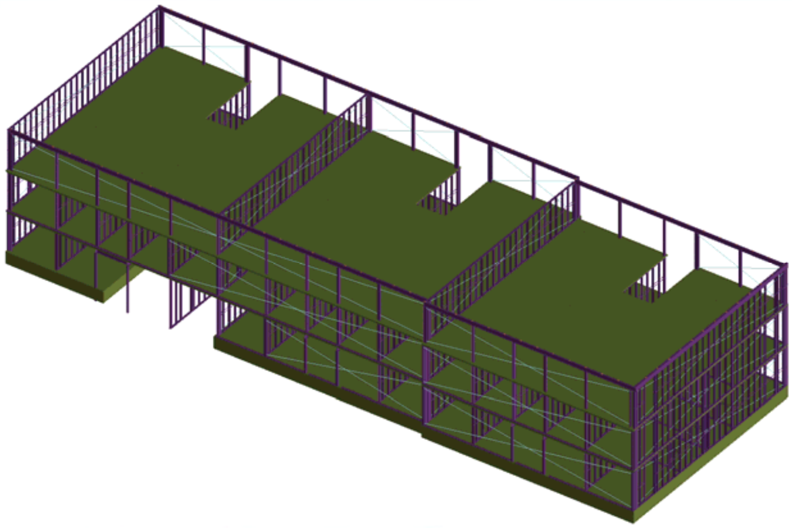
ELS Model



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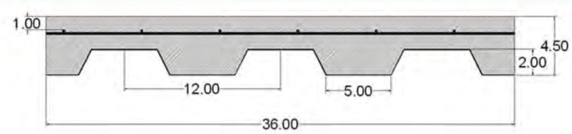
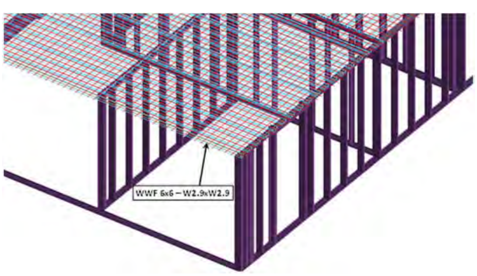
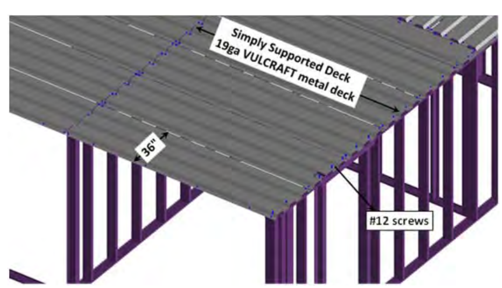
Composite Slab Deck







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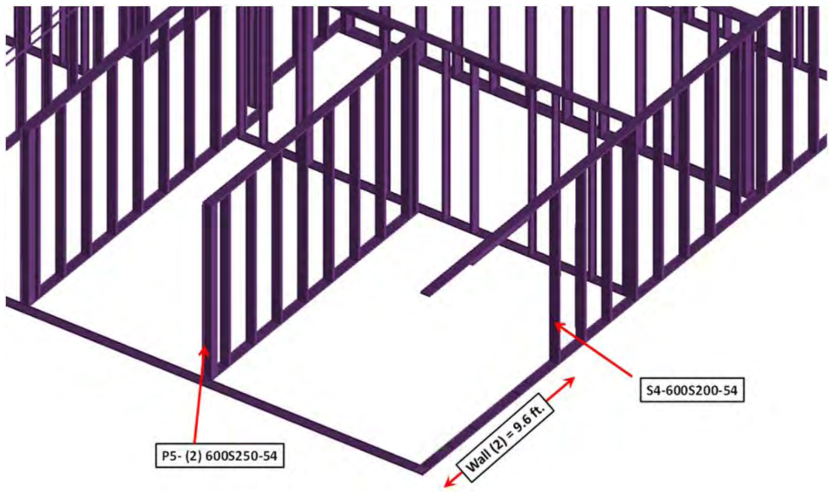
Progressive Collapse Analysis



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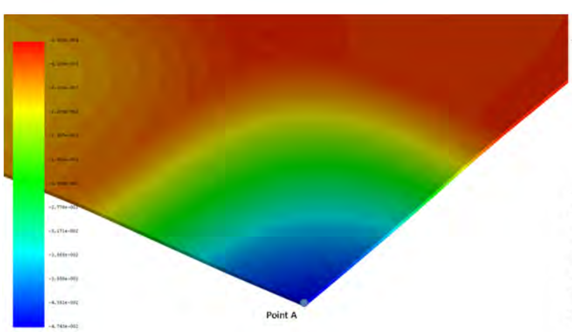
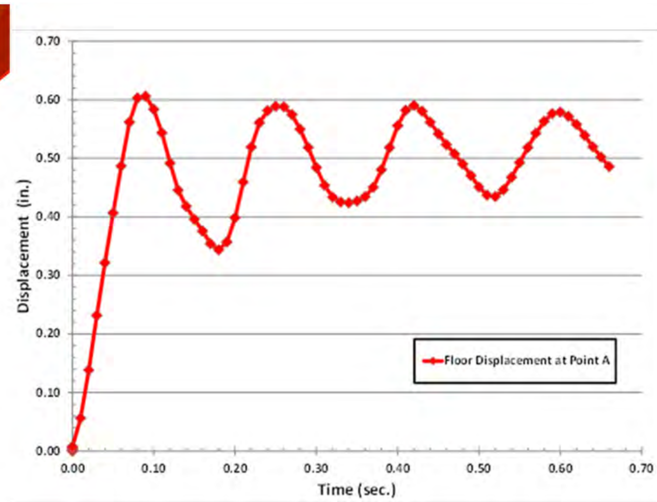
Results-Deformation Controlled Action

- **Displacement Contours for composite slab deck**





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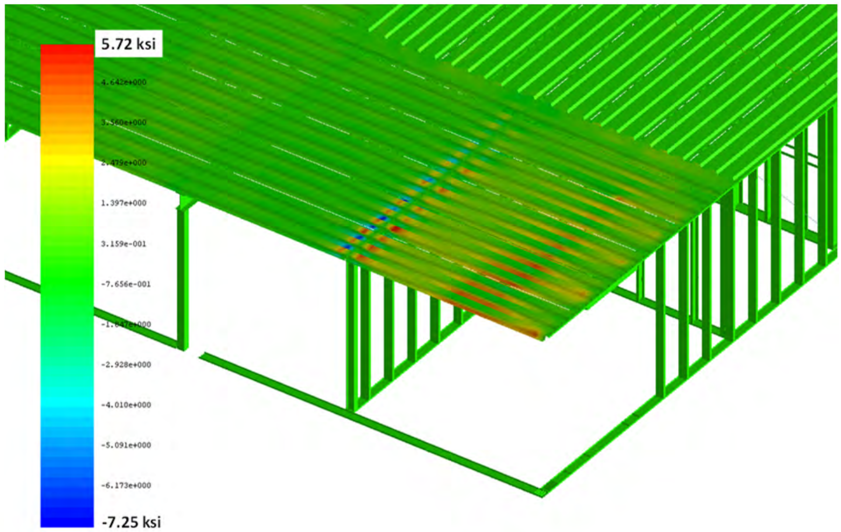
Results-Deformation Controlled Action

- **Maximum Stresses in Metal Deck**



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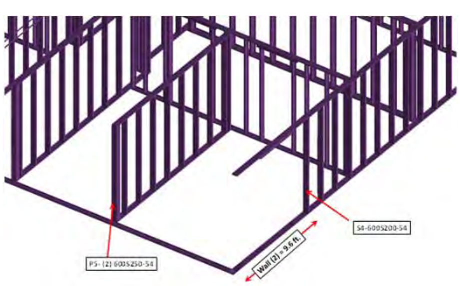
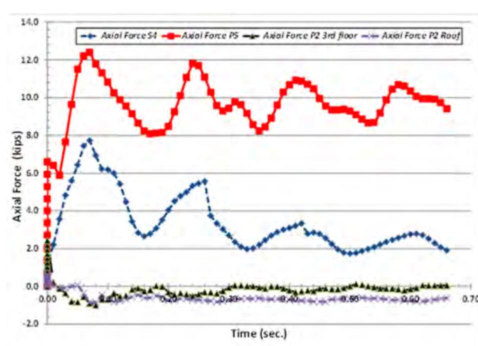
Results-Force Controlled Action





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Start Using Extreme Loading® for Structures
3-D Nonlinear Dynamic Analysis

- Free for 30 Days
- Easily model structures with no need for re-meshing or transition elements
- Process of crack initiation through progressive collapse is completely automated, no need for user intervention
- Wind, blast, impact, seismic load analyses are easily reached in a fraction of the time required by other methods
- Performance based design saves thousands on construction costs

Request Initial Project Consultation
The Engineers, Engineer

- 100% Free Service
- Reduce cost through performance based design, while meeting progressive collapse requirements
- Win more jobs by providing better information during the bid process
- Save hundreds of hours by avoiding over engineering and over building
- Accelerate construction schedules by reducing material requirements

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